Master thesis

The Floating Construction Method

‘a new method of constructing submerged and floating structures’

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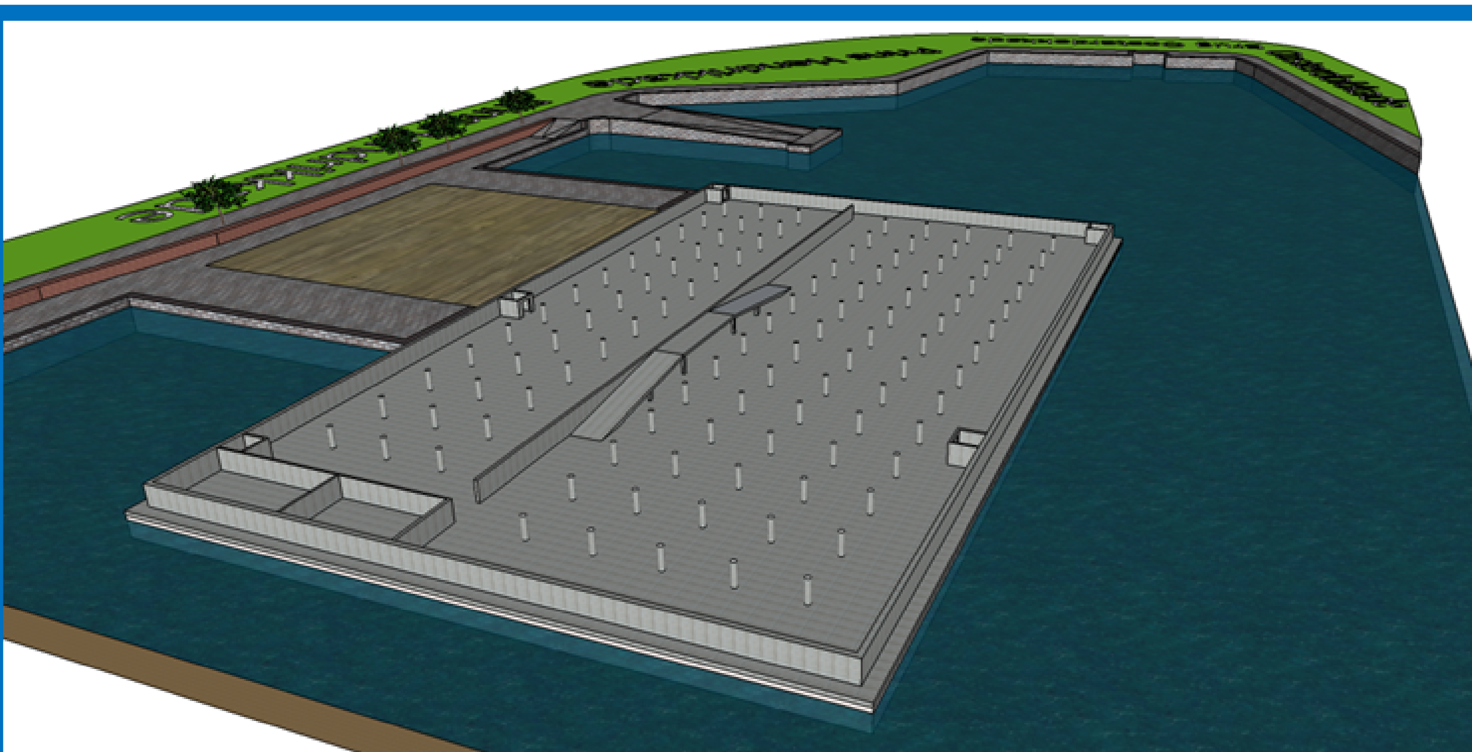
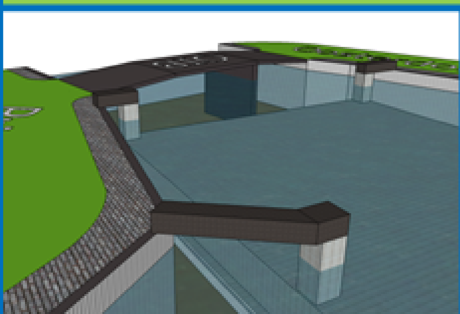
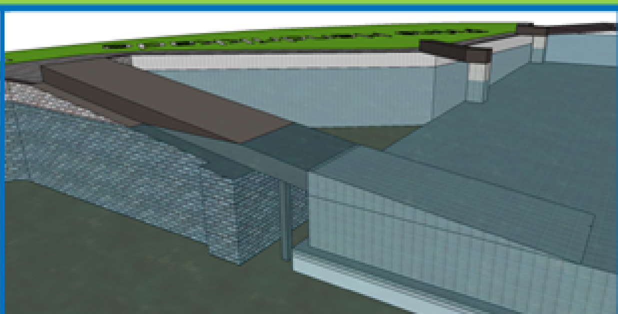
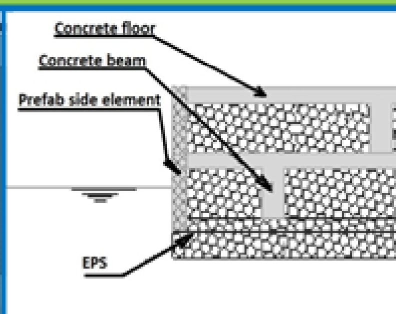


Rob Hendriksen







DURA VERMEER

TU Delft Delft University of Technology

MSc-Thesis

The Floating Construction Method

'a new method of constructing submerged and floating structures'

Rob Hendriksen

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Colophon

Delft University of Technology
Section of Hydraulic Engineering
Stevinweg 1
2628 CN Delft, the Netherlands

Dura Vermeer Beton- en Waterbouw BV
Taurusavenue 100
2132 LS Hoofddorp, the Netherlands

Supervisors Delft University of Technology

Prof.drs.ir. J.K. Vrijling
Delft University of Technology, Faculty of Civil Engineering and Geosciences,
Section Hydraulic Engineering

Ir. W.F. Molenaar
Delft University of Technology, Faculty of Civil Engineering and Geosciences,
Section Hydraulic Engineering

Ir. W.J.M Peperkamp
Delft University of Technology, Faculty of Civil Engineering and Geosciences,
Department Design and Construction, Structural and Building Engineering, Concrete Structures

Supervisor Dura Vermeer Beton- en Waterbouw

Ing. W.J. Schilder
Head Design departement, Dura Vermeer Beton- en Waterbouw BV

Ing. B.G. Olde Klieverik
Head Business office, Dura Vermeer Beton- en Waterbouw BV

Author

Ing. R. Hendriksen
Delft University of Technology, Faculty of Civil Engineering and Geosciences,
Master Hydraulic Engineering - Hydraulic Structures

Telephone: +316 24 999 257
Email: R.Hendriksen@student.tudelft.nl
Address: Oldenbarneveldenweg 16, 3772GG Barneveld, the Netherlands

Preface

Since the birth of cars, much has changed in de world of parking. Especially in cities. Ground level parking spaces are becoming scarce for a number of reasons. The first and most important reason is the lack of space in city centres. But big cities also begin to ban cars from their centres, the need for living space is increasing and citizens would rather have parks and squares than parking spaces. You could say that the citizens still want to use their cars, but don't want to see them when they are not being used. This often means that cars are parked just outside of the city centre or that cars are parked somewhere they can't be seen, like underground or underwater. We all know underground car parks, it's been well developed the last few decades and almost every significant city has one or more. When considering parking garages underwater, there is still a lot that could be done.

This MSc Thesis will be the final part of my master Hydraulic Engineering – Hydraulic Structures at the Delft University of Technology and focuses at a new construction method for parking garages underwater. The traditional construction methods for structures underwater are more or less the same as those for structures underground. One could wonder if this is the right approach. This study will focus at the existing construction methods and will research whether or not a new method could be feasible.

This study is a broad one and will review almost all the topics that are considered important during my master, namely concrete structures, construction methods, hydraulic and structural analysis and much more. This will be combined with a topic that I find important in a MSc Thesis, innovation. I hope you will enjoy reading this report.

Rob Hendriksen

August, 2011

Acknowledgements

I would like to thank the following people for their contributions to my MSc thesis:

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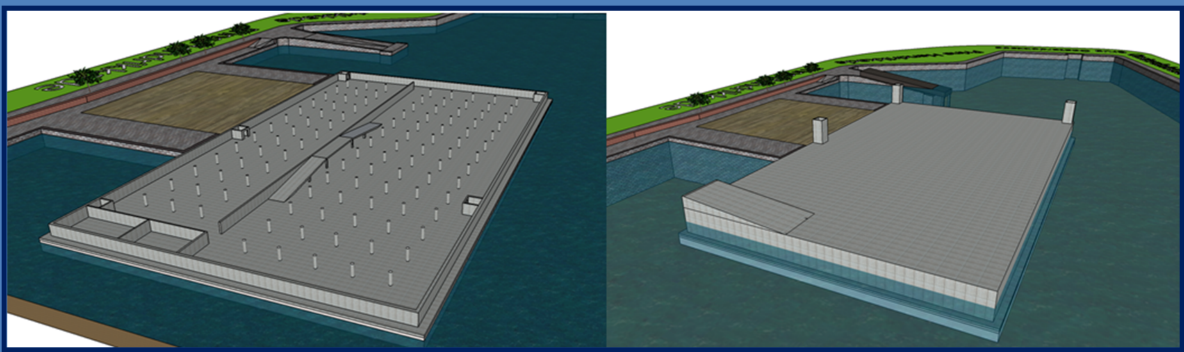
Further, I would like to thank J.W. Roel for sharing his knowledge on Flexbase and B. Sinnema for his help with the structural analysis.

Summary

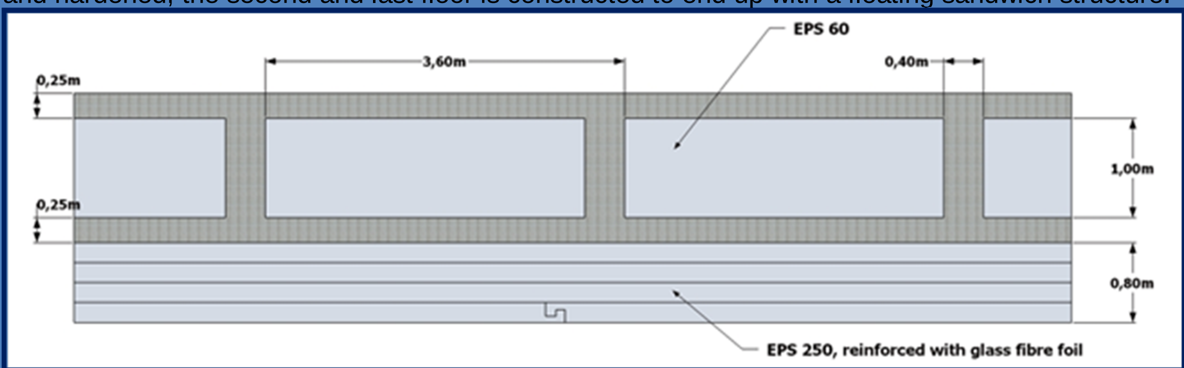
Parking problems in cities become more significant every day. Parking tariffs are going through the roof and parking spaces are limited. Therefore, cities are researching alternatives for parking on ground level and underground. One of these alternatives is parking underwater.

Amsterdam
One of these cities is Amsterdam. This city ranks number 5 on the list of most expensive cities to park in Europe. To create more parking spaces, the city of Amsterdam is researching the possibility of parking garages underwater at several locations. One of these locations is the Oosterdok near the central train station. A feasibility in 2009 study showed that a parking garage underwater with 350 parking spaces and constructed with the traditional construction method is not financially feasible. [Oosterdok, 2009]

The floating construction method
This situation is used to determine whether or not a new construction method could be the solution, namely the floating construction method. With this construction method, the structure is constructed directly on the water. The base consist out of a floor of EPS and concrete. This floor floats and makes it possible to construct the rest of the structure on top. This process is shown in the figure below. The advantage is the fact that an expensive temporary construction pit isn't needed, which normally forms 20 to 30% of the total construction costs!

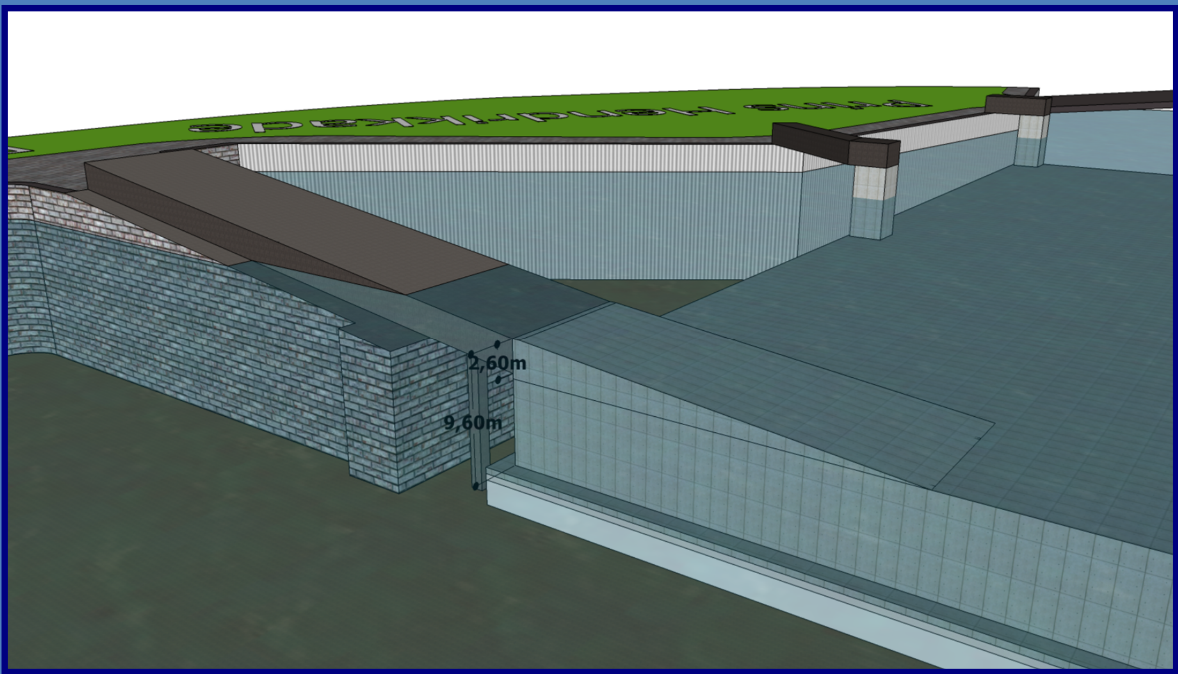


Flexbase floor
The floating floor consist of the patented Flexbase system and is shown in the figure below. It starts with constructing a 800mm thick EPS floor consisting of 4 layers of glass fibre reinforced EPS250. Directly on top of this EPS floor the concrete floor is poured. When this floor is hardened the formwork, consisting of EPS60 blocks, for the beam grid is placed. When the beam grid is poured and hardened, the second and last floor is constructed to end up with a floating sandwich structure.



The parking garage
The parking garage is constructed directly on top of the Flexbase floor. The parking garage has two floors and 750 parking spaces. It is 129 meters long, 76 meters wide and 6,5 meters high. This does not include the 2,05 meter thick Flexbase floor. When the garage is finished, it still floats and has a draught of 6,05 meters.

Immersing the parking garage
When the garage element is completed, it is immersed. Ballast tanks are placed inside the element and filled with water. When the element is at the desired depth and level, grout anchors are drilled through the floor to keep the element immersed. The ballast tanks are emptied and the garage element is coupled to the entrances for cars and pedestrians. The entrances are constructed separately.



Conclusions
The floating construction method is technically and financially feasible for this situation. The floating construction method is approximately **11,5%** cheaper than the traditional construction method.

When the benefits come exclusively from parking fees, the floating construction method becomes financially feasible for this situation with hourly parking tariffs of **€2,16**.

This research shows that the floating construction method could be a better and cheaper alternative for traditional construction methods.

Table of Contents

Part 0 Introduction

This report is divided into three parts. This is the first part, the introduction. This first part will explain why my research could be relevant, what the study objectives are and how I will try to accomplish them. The introduction will consist out of four chapters. A brief introduction to those chapters is given below.

Chapter 1: Parking in cities

The topics of this chapter are the history and the present of parking in cities, especially parking underground and underwater. Parking in cities is highlighted because the parking problems are most significant here. This chapter should make clear why there is a need for parking garages underwater.

Chapter 2: Construction methods

Next, chapter two will deal with the traditional construction methods for parking garages underwater. A link will be made between constructing underground and underwater. This chapter will also explain more about the different locations where these structures are constructed.

Chapter 3: Research scope

This chapter, together with chapter four, will form the guideline for the rest of the report. It will start with divining the problems concerning the traditional construction methods. This will result in a research opportunity. The final part of this chapter will review the study objectives for this MSc thesis.

Chapter 4: Report structure

Chapter four will clarify how the study objectives will be reached.

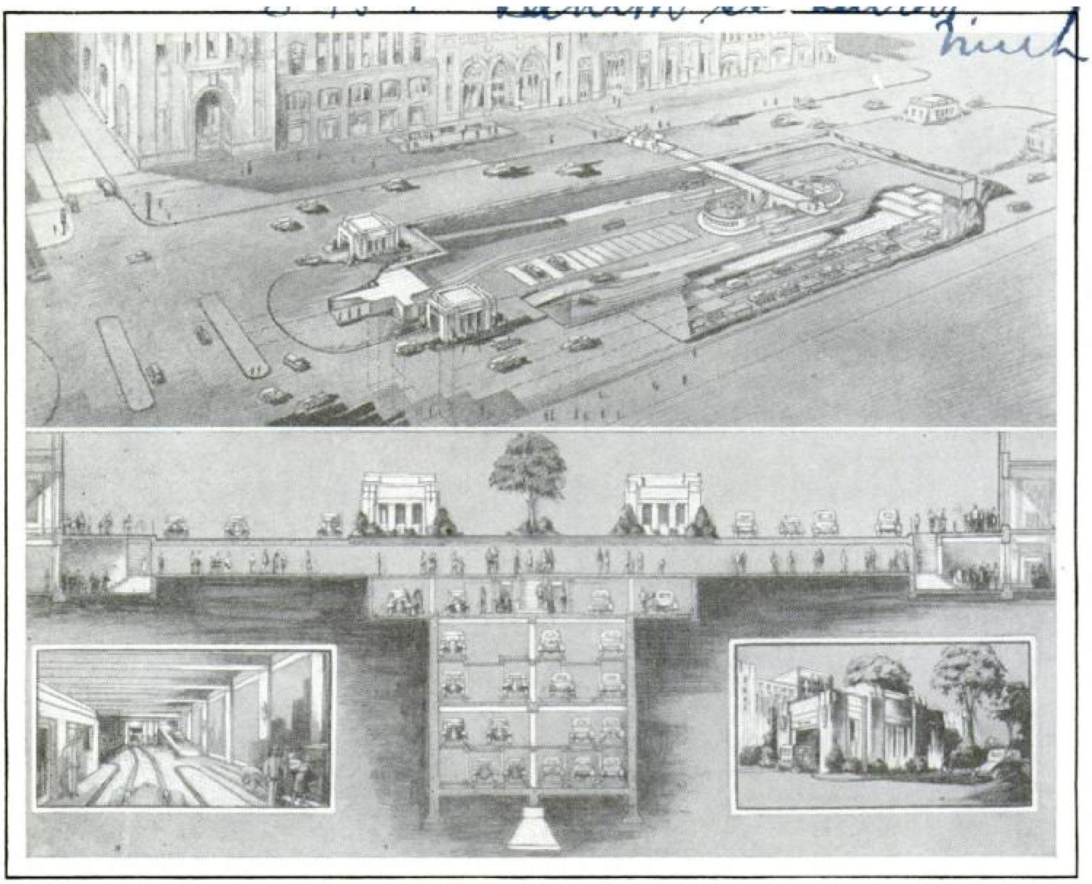
1. Parking in cities

When the first production car was introduced in 1885 by Karl Benz, there weren't much problems with parking your car. There was enough space and cars were parked on ground level. It stayed this way until World War II when the number of cars began to show an exponential growth. This, together with a fast growing population, meant that space was becoming scarce, especially in city centres. The intensity of traffic in cities increased and with that the need for parking spaces. This is why parking in cities is still becoming a bigger problem every day. Parking spaces are limited and parking tariffs are going through the roof. Especially parking spaces at ground level are expensive and in the centres of big cities there simply isn't any space for them anymore. The first step was to construct parking garages underground. But in a number of cities, also the space underground becomes scarce. This could be because of other underground structures (e.g. foundations, sewer systems, historical artefacts) or it could be that the local citizens are hesitant towards them (e.g. risks of settlement and collapsing buildings during construction). Another alternative is found underwater.

A relatively new market is the one of constructing parking garages underwater. In Alkmaar they already have a submerged parking garage. In a lot of other cities there are plans to do so (The Hague, Amsterdam etc.). This means that the market for submerged parking garages is expanding and there could be great potential for new (better) construction methods.

1.1 Underground parking

The first known idea for a parking garage underground is shown in figure 1. It is a sketch for an underground car park in the magazine "Popular Mechanics" in 1931. The headline said "Is underground parking going to solve the parking problems?". The figure shows a four story parking garage with lifts to move the cars up and down. However, the first underground parking garage wasn't constructed until 1942 in San Francisco. The parking garage had four stories and housed 1.700 cars. In that time is was also promoted as a bomb shelter, because of WW II.



Drawings Showing Details of Plan for Underground Garages to Solve Parking Problems in Congested Areas; Elevators and Electric Conveyors for Cars Are Included in Design

Figure 1: Sketch for an underground parking garage in 1931 [Popular Mechanics, 1931]

Since 1945, also the Netherlands started constructing underground parking garages. This first started in the big cities like Amsterdam and Rotterdam, but soon it spread all over the country. At first, underground parking garages were often constructed in combination with other structures like an office building or shopping centre as an extra facility for the users. Later on, when parking tariffs became higher and space in cities became more scarce, underground parking garages were also constructed as separate structures. These parking garages were then exploited by the government or private parties.

Currently, almost every significant city has multiple underground parking garages. In 1997 there was a large scale inventory among the counties (Dutch: gemeentes) in the Netherlands that have more than 30.000 citizens. They were asked whether or not they are investing in constructing underground. The conclusions were that since 1990 there is an upward trend in underground construction and that it is mostly used for transport and parking. Another conclusion was that fourteen counties are very interested in constructing underground and seven counties (e.g. Amsterdam, Rotterdam, The Hague) are using underground construction to solve the parking and transport problems within the city centres. [A.A. Balkema, 1997]

It might be evident that parking underground isn't a futuristic topic anymore and that it is already happening at a large scale. Almost every car owner is used to parking underground. One of the important topics now is enhancing the feeling of safety by creating a pleasant atmosphere underground. This, however, goes beyond the scope of this thesis. But there is another topic that came up during the last decade or two, namely parking underwater.

1.2 Submerged parking

In the last few decades, cities have found another alternative for parking in city centres, namely underwater. As said before, also the space underground becomes scarce and constructing a parking garage underground in the centre of a city can cause a lot of hindrance. This is why cities tend to look at spaces where hindrance is minimal for their citizens and the space underground is relatively free of obstructions. This is usually the case for city canals, harbours and rivers that run through the centre.

One of the best examples is the city of Amsterdam. They have struggled with parking problems for decades and still are. In an attempt to improve the public space above ground and to create more parking spaces, a number of feasibility studies have been published on parking garages underneath the city canals. An impression of such a parking garage is shown below. At the moment there are concrete plans to construct an underwater parking garage underneath the Oosterdok in Amsterdam, a harbour in the centre of the city. [Parkerenindestad.nl]

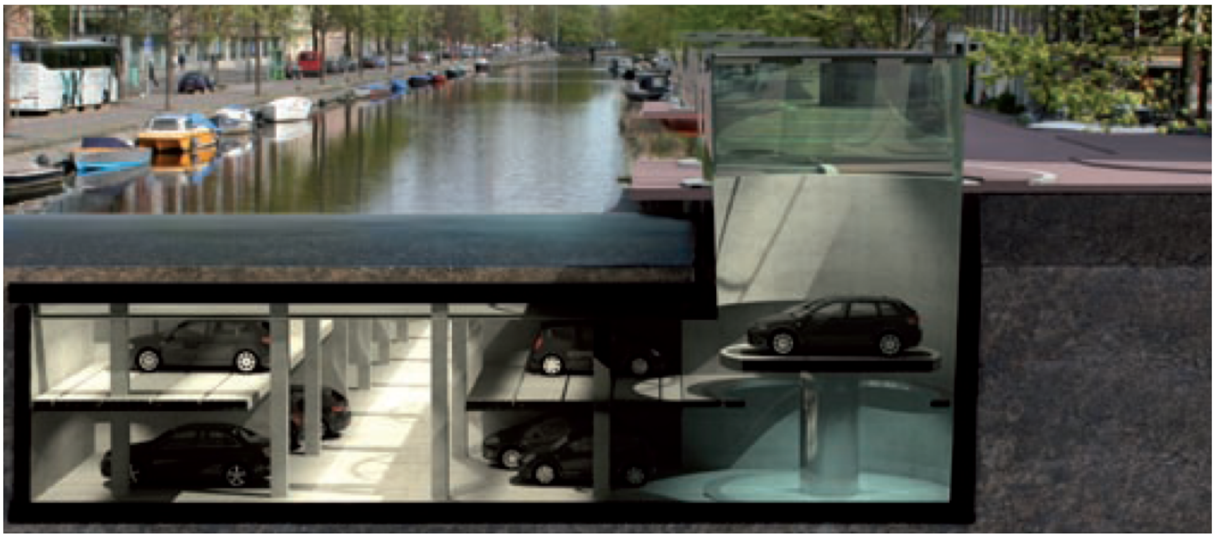


Figure 2: Impression of a parking garage underneath the Amsterdam city canals [De Ingenieur, 2007]

In Amsterdam they haven't constructed a parking garage underwater yet, but in Alkmaar they did. Underneath the "Singel", a canal that runs through the centre, a parking garage was constructed with a capacity of 400 cars. The big advantage here, is that the hindrance for traffic in the city is minimal. Construction happened within the borders of the canal and materials/equipment could be transported to the location by ships. [Bam.nl]

Other examples are found abroad. One of the best examples is the breakwater in Port Hercule in Monaco. This is a structure with multiple functions, namely a breakwater, a parking garage and a shopping centre. It is constructed as a caisson and is 352 meters long and 28 meters wide. It was completed in 2002.

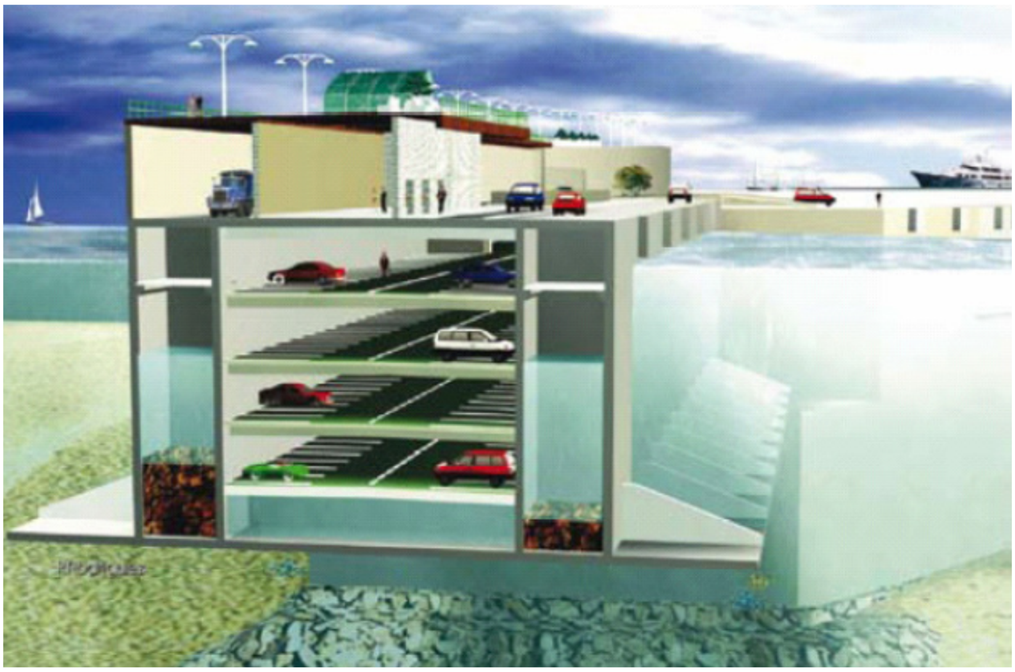


Figure 3: Breakwater / parking garage, Port Hercule Monaco [Fousert, 2006]

It is expected that there will be a growing demand for underwater parking garages in the future. Big cities all over the world are growing exponentially and space in the centres of these cities is getting scarce and therefore expensive. A lot of cities are looking at the possibility of using space underwater and are conducting feasibility studies. It is a matter of time before more and more cities start constructing parking garages under lakes, canals and rivers.

To conclude this chapter, the main advantages of parking garages underwater are:

i. Cars are parked out of sight;
ii. Relatively little hindrance during construction;
iii. Use of otherwise unused space in city centre;
iv. Relatively little obstructions underground (e.g. foundations, sewer systems).

Now that the need for underwater parking garages is made clear, the next chapter will explain how these structures are currently constructed.

2. Traditional construction methods

This chapter will give an overview of the most common construction methods for parking garages underwater. Focussing on garages in city centres. Before writing this chapter, an extensive study was done on all the possible construction methods. This chapter describes the two methods that are most common. The focus is not on thoroughly describing every aspect of the construction method, but on the applicability and the pros and cons of the methods.

2.1 Dominant locations

To be able to discuss the applied construction methods, one has to know certain aspects and boundary conditions of the location. Of course there isn't one specific location where submerged parking garages are, or will be, constructed. However, similarities could be found between these locations and these similarities could be used as the starting point.

A feasible location for a submerged parking garage must have at least some of the following aspects:

i. Parking spaces are scarce and needed, significant parking problem;
ii. It is not feasible to construct a parking garage anywhere else (underground or at ground level);
iii. Parking tariffs must be high enough for the financial feasibility of a submerged parking garage;
iv. Parking garages or cars in sight are not accepted;
v. Water must be present (e.g. waterway, lake, harbour, dock).

Locations that tick almost all of these boxes, some more than others, are large city centres like those of Amsterdam and Rotterdam in the Netherlands. But also cities abroad like Paris, London, New York and Barcelona. These cities are tourist attractions and renowned for their architecture. This means parking garages and cars in sight might not be wanted. In most of these cities, parking spaces are limited and the hourly tariffs are high. They also all have large water surfaces in the form of city canals, harbours, rivers or the sea.

For the location to be feasible the parking tariffs have to be high enough. A feasibility study done in 2008 by Maarten Faber shows that a parking garage under water becomes feasible with an hourly parking tariff above €1,86. In the large cities mentioned before, the tariffs are much higher, but in smaller cities, parking garages underwater might not yet be financially feasible. [Faber, 2008]

Below, the city centre of Amsterdam and it's parking tariffs per hour are shown. This city is comparable with most large cities that are situated near water (whether it's a river, a lake or the sea). The old centre is located near water with city canals running through it. Here the parking tariffs are the highest and the further you get from this centre, the lower the tariffs get. The main reasons are the limited amount of parking spaces and the parking policy of the city that wants to keep cars out of the centre.

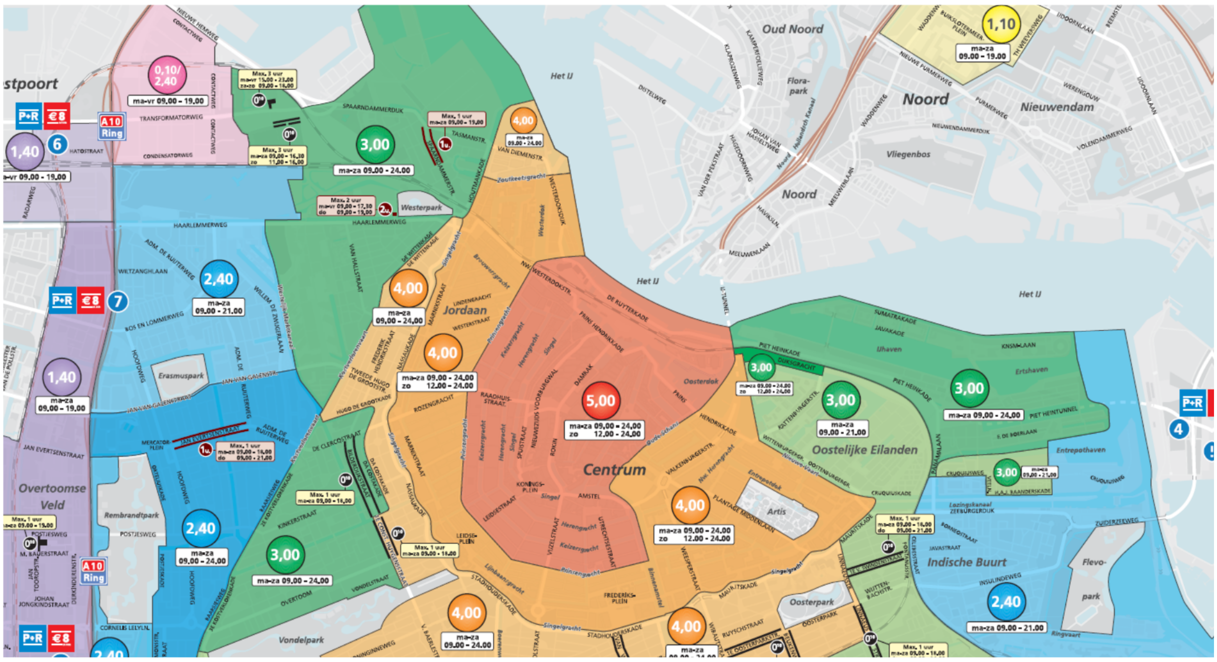


Figure 4: Hourly parking tariffs in Amsterdam [parkerenindestad.nl]

To recapitulate, these are some important aspects of a construction site in a city centre that have to be kept in mind:

i. High traffic intensity, as well on land as on water;
ii. Transport to construction site is difficult and unwanted (e.g. narrow streets or canals, traffic jams etc.);
iii. Space for a construction site is limited;
iv. Strict regulations for noise pollution;
v. Construction takes place close to other structures (risk of damage, settlement or collapsing buildings);
vi. There usually is a maze of sewer systems, gas pipes etc. underground (risk of damaging);
vii. Historical artefacts could be found underground (risk of delay or even annulment).

When constructing a parking garage underwater under these conditions, there are two construction methods that are most common, that is the “Cut and cover method” and the “Caisson method”. These two methods can have a lot of variations, however, this chapter will explain the most common method. This is done, because the pros and cons for the variations are more or less the same and will result into the same conclusions.

2.2 Cut and cover method

When constructing in-situ, a (temporary) construction pit has to be constructed. The purpose of this construction pit is to retain the water and soil as long as the parking garage is in the construction stage. This means it has to retain water and/or soil horizontally and vertically. There are a number of ways to do this and the different horizontal and vertical retentions can be combined to construct a watertight construction pit. The figure below displays a generalization, describing the main elements of a construction pit:

i. Vertical retention wall, retaining water, soil or water and soil;
ii. Horizontal retention, retaining water;
iii. Anchorage;
iv. Struts.

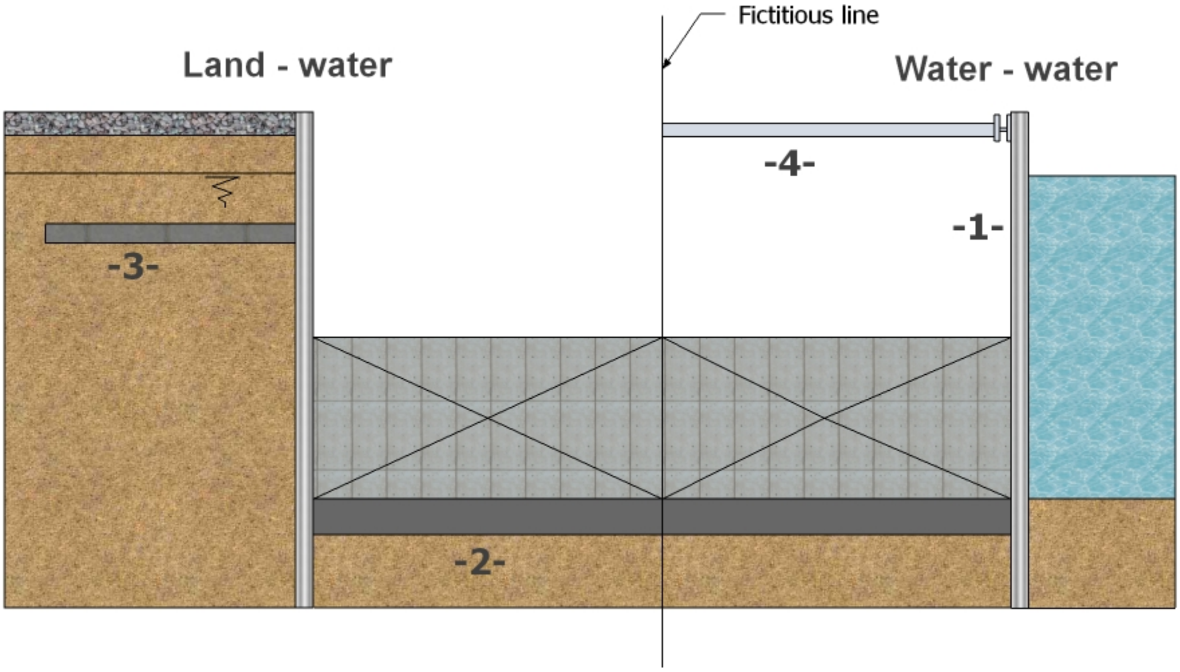


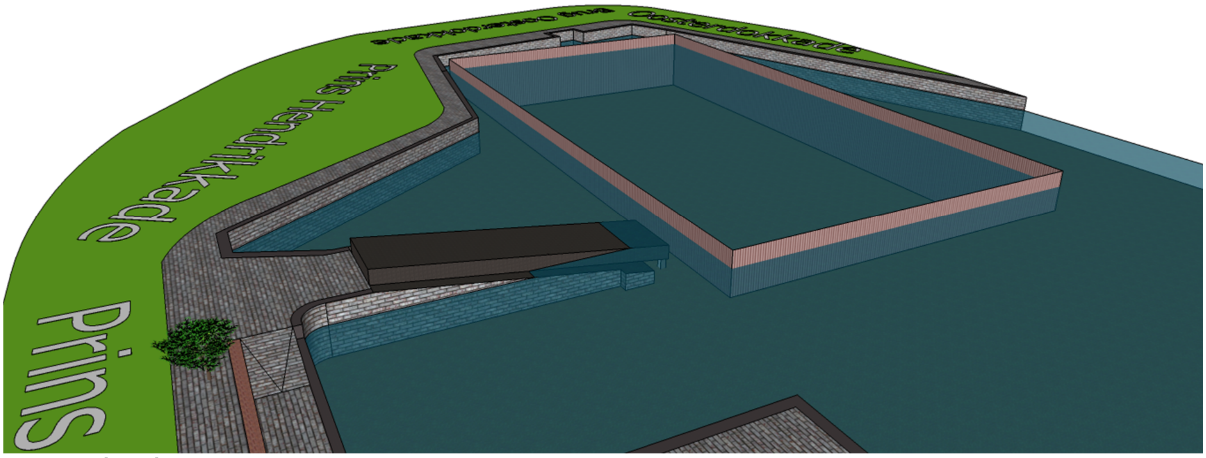
Figure 7: Sketch of a construction pit

Construction stages

Constructing a structure underground or underwater by using a (temporary) construction pit is the most common method. The various stages of this construction method will be explained. This is a generalization, but explains the main stages. This example uses sheet piling in combination with underwater concrete and tension piles. Also, one side of the construction pit is bordering water and the other side land. This could be both land or both water.

Step 1: Vertical retention

The first step is to create the vertical retention wall. This could be standard sheet piles or a variation like a combi-wall, concrete wall, secant piles or even an earth wall, depending on the depth of the construction pit and the available space. It could be that there already is a wall in places in the form of a quay wall. In that case, the wall has to be evaluated and it has to be determined whether or not it is sufficient for the construction pit.



Construction pit, 100 x 140 meters

Step 2: Driving tension piles

After the vertical retention is placed, the tension piles are placed. These tension piles will have to resist the vertical water force that acts on the floor of underwater concrete. The length and spacing of the piles depends on this water pressure and the subsoil. There are other ways to apply this horizontal retention, but this is the most common method.

Step 3: Pouring underwater concrete

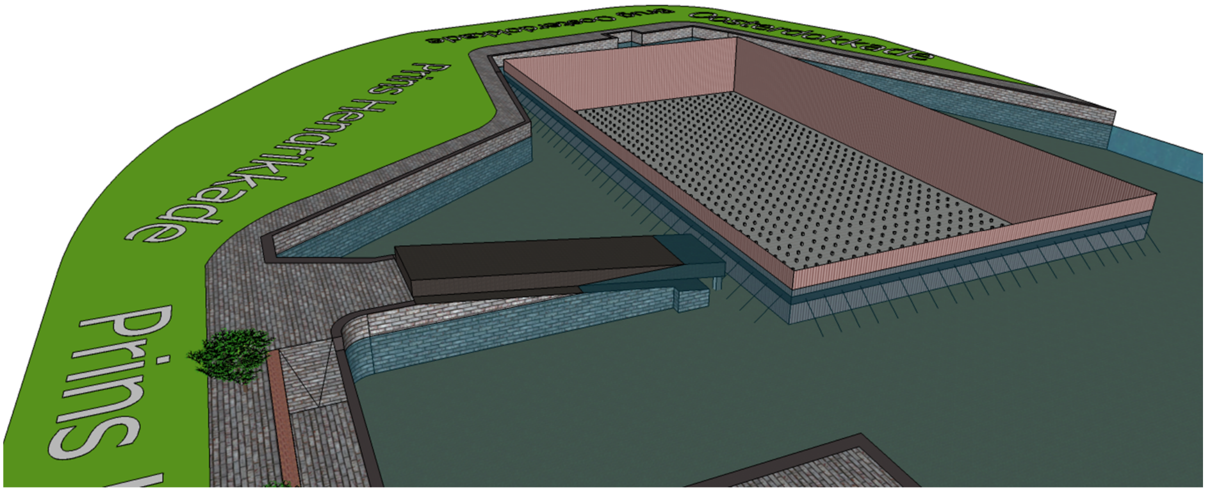
After the tension piles and sheet piling are cleaned for a better bond, the underwater concrete is poured. Now the concrete floor, together with the tension piles, forms the horizontal retention. This retention should be watertight. The quality of this floor depends for a large part on the (lack of) mixing of the underwater concrete with the soil and the bond between the concrete and the tension piles and sheet piling.

Step 4: Placing horizontal support

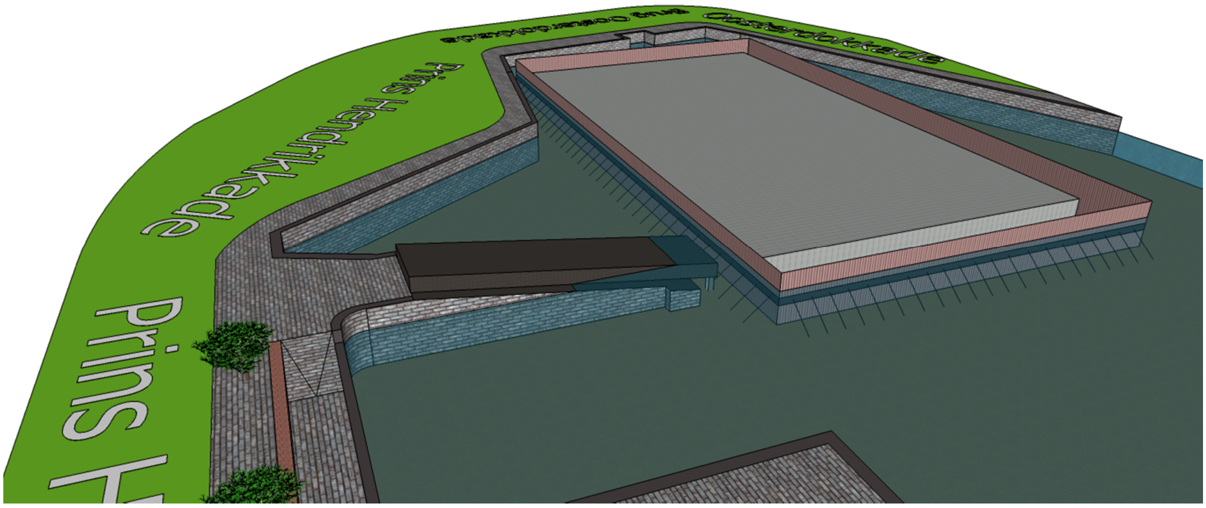
To resist the horizontal water and soil pressure, the sheet piling needs extra support. This can be done by struts or by anchorage. The figure on the next page shows anchors. Struts are available in different shapes and sizes, depending on the force they have to resist. The problem with struts is that the distance between the walls cannot be too large (+/- 25 meters). When the distance is larger, diagonal struts or anchorage should be used.

Step 5: Pump water out of construction pit

The vertical and horizontal retentions are in place and the construction pit should be watertight. If it isn't, there should be pumps in place to keep the pit dry. The water can be pumped out and the construction pit is ready for use. When the water is pumped out, the floor will be cleaned and, if the tension piles aren't part of the permanent structure, the part of the piles that stick out of the concrete floor are chopped off. To smoothen out the floor, an extra concrete floor is poured on top of the underwater concrete.



Step 6: Construct permanent structure
The permanent structure is constructed in the pit. Everything and everyone that is necessary for construction, has to be transported by land or water and has to be lowered down into the construction pit. This could be done by crane, lift or stairs. Temporary buildings/offices and storage for materials are placed in- or outside the construction pit, depending on factors like the availability of space.



Step 7: Remove temporary works
When the permanent structure is complete, all the temporary works can be removed. Possibly a part of the horizontal and vertical retention is part of the permanent structure. When the temporary structures are removed, the permanent structure (partially) disappears under water.

As said before, this is the most common way of constructing structures underground or underwater. There are a lot of variations to this method, but the overall method stays practically the same. The biggest risks of this method are settlement or collapsing structures near the construction pit, flooding of the construction pit and collision with other structures (e.g. foundations, sewer systems, historical artefacts) while driving the tension piles or sheet piling.

The last few years, there have been a couple of malfunctions of construction pits, which resulted in settlement and collapsing buildings and even human casualties. Therefore, regulations are getting stricter. At many places, further accidents are tried to be prevented by continuous monitoring, vibration free pile driving etc. However, there is still a lot of scepticism from the public towards deep construction pits near other structures. [A.A. Balkema, 1997], [van der Horst, 2009]

Pros	Cons
i. Takes a minimum of horizontal space, when using sheet pile walls; ii. Construction takes place right on site, no other construction site needed; iii. Temporary works could be incorporated into permanent structure.	i. When using sheet pile walls: piles penetrate the underground, risk of colliding with other structures; ii. Risk of settlement or collapsing structures in surrounding area; iii. Noise pollution in city centre; iv. Vibrations during pile driving, depending on driving method; v. Transport through busy city; vi. Temporary construction pit is usually a large part of the total costs (20-30%).

2.3 Submerged prefab elements

Another often used method is the immersion of prefab (tunnel) elements. This method is usually used for the construction of tunnels, but could also be used for the construction of underwater parking garages. The permanent structure is constructed in a dry dock at another location. The structure could be constructed out of any material, but the most common one in Europe is concrete. Using the figures bellow, the method will be explained.

<u>Step 1: Construction of tunnel elements</u>

The permanent structure is constructed in a dry dock. This could be an existing dock or one specially constructed for the job at hand. It is preferred to have the dock close to the permanent location. During construction the dock has to be kept dry, this is usually done by lowering the groundwater level with pumps. The figure below shows a temporary dry dock. This dock was used for the construction of the tunnel elements for the Piet Hein Tunnel.

Figure 8: Construction of tunnel elements in dry dock, Piet Hein Tunnel [www.bam.nl]

<u>Step 2: Transport of the elements</u>

When the element is constructed, the dry dock is flooded and the caisson begins to float. Important at this stage is the stability and water tightness of the tunnel element. Also, the transport route should be checked for obstacles like bridges and navigation locks. Weather conditions should also be taken into account when planning the transport, because wind and rain could change the flow velocity and direction. The element is transported to the final destination using tugboats. When the elements are transported, the dock is pumped dry and, when needed, prepared for the construction of the next elements.


Figure 9: Transport of tunnel element by tugboats, Piet Hein Tunnel [www.bam.nl]

Step 3: Immersion of tunnel elements
When the caisson element is at the final location, it has to be positioned. Positioning of the caisson is usually done when the flow velocity is low. The final positioning is done by tugboats, anchors and cables from floating equipment. When the tunnel element is at the final position, it can be immersed. This is done by filling water tanks inside the element. This should be done in a controlled manor, so the immersion happens gradually. Before the element is placed, preparations should be made. The bed has to be prepared (depending on the foundation type), and the water should be deep enough, if it wasn't already. It could be that the final structure is made out of more than one element, then they have to be connected with a watertight seal (e.g. a Gina profile).

Step 4: Securing the elements
The last step is to make sure the element doesn't float back to the surface. This could be done by covering the element with ballast or connecting it to tension piles. Also, in the element, an extra ballast floor could be poured. After the element is secured to the bottom, the water tanks are emptied. Now the underwater structure is ready for use. Next steps are the completion of the inner works and making a connection with the ground level (entrance/exit).

This method is used when the waterway has to be in use during construction and placing a construction pit in the water is not possible. Or when this construction method simply is cheaper. Another advantage is that the construction of the structure happens above ground and away from the city centre.

There is a lot more that could be said about this method, however, this part of the research focuses on the pros and cons of the method. [A.A. Balkema, 1997], [D. Chapman, 2010]

Pros	Cons
i. Construction above ground and away from the city centre; ii. Minimal obstruction of waterway at the final destination; iii. Elements could be re-used.	i. Dry-dock needed, extra costs; ii. Transport by water, could give extra problems; iii. Hardly any visual inspection during immersion; iv. Extra measures needed to keep caisson down; v. Size limitations could occur because of obstacles during transport.

3. Research scope

3.1 Problem definition

The methods described in the previous chapter haven't changed much in the last few decades and give multiple problems when they are applied on underwater parking garages in busy city centres. This chapter will discuss those problems. The main problems with the traditional construction methods for parking garages underwater will be mentioned. This is an elaboration of the "cons" that were mentioned in the previous chapter.

Cut and cover method

Need for an expensive construction pit

Constructing the temporary construction pit forms a large part of the total costs (usually +/- 20%). A large part of the budget usually goes to vertical retentions like sheet pile walls and horizontal retentions like underwater concrete with tension piles. Especially, with the ever rising steel prices, the use of sheet piling is a large part of the total costs.

Risk of settlement or collapsing structures

A big problem of construction pits in a city is that they change the soil structure and groundwater level in the surrounding area. This could be because of vibrations during pile driving and sheet piling or a construction pit that isn't watertight. This could cause settlement or even collapsing structures. Also, citizens could delay the project when they don't feel safe. This usually leads to higher construction costs.

Construction site in city centre

The complete structure is constructed in the centre of the city. This leads to hindrance for the citizens and the construction workers. During construction, roads can be blocked, extra transport goes through an already busy centre, machinery causes sound pollution etc. This could also harm tourism when the construction site is close to popular touristic destinations.

Immersion of prefab elements

Need for a dry-dock

When constructing the prefab elements there is need for a, generally expensive, dry-dock. This could be an existing dock or a dry-dock could be specifically made for the project at hand. When constructing in a city, this could give problems. In most cases there is no space available for a large dry-dock. This means that the dry-dock has to be constructed at a large distance from the actual site.

Need for transport to location

Furthermore there is need for transport to the location when using this construction method. This could be by land or by water. In both situations there have to be extra checks to make sure that the element is stable and strong enough during transport. This means extra risks and costs.

Restrictions in size

The dry-dock and transport could give restrictions to the size of the structure. When using an existing dry-dock the structure has to be adapted to the size of the dock. During transport by water the dimensions of the canal and locks could give restrictions. When transporting by road, the obstacles along the way (e.g. viaducts and tight corners) give restrictions to the size of the structure. The needed stability during transport could also give restrictions to the size and shape.

3.2 Research opportunity

The current construction methods for constructing parking garages underwater have disadvantages. It is believed that there could be an opportunity for a new construction method. One that could solve most of these problems. Together with Dura Vermeer Beton- en Waterbouw and Flexbase, an outline was developed for a new construction method, from which it is believed that it could solve these problems. In this chapter, the main idea will be explained. After that, the possible advantages and research goals will be reviewed.

The main idea

The new construction method will be the focus of my research, it is called the “floating construction method”. This method has already been applied partially at a small scale for floating structures (e.g. houses and greenhouses). My research will be focussing at the large scale use of this floating foundation. During this research, the focus will be on submerged parking garages. The main idea will be reviewed in the following paragraphs.

Step 1: Constructing the floating foundation

The first step of this construction method is creating a floating floor. This floor will be the work floor, but will also be the floor of the permanent structure. This step also shows the big difference with the other construction methods. Instead of constructing the parking garage on dry land, it will be constructed directly on water. The figure below shows the different steps of constructing the floating floor. Underneath it, the steps are explained.

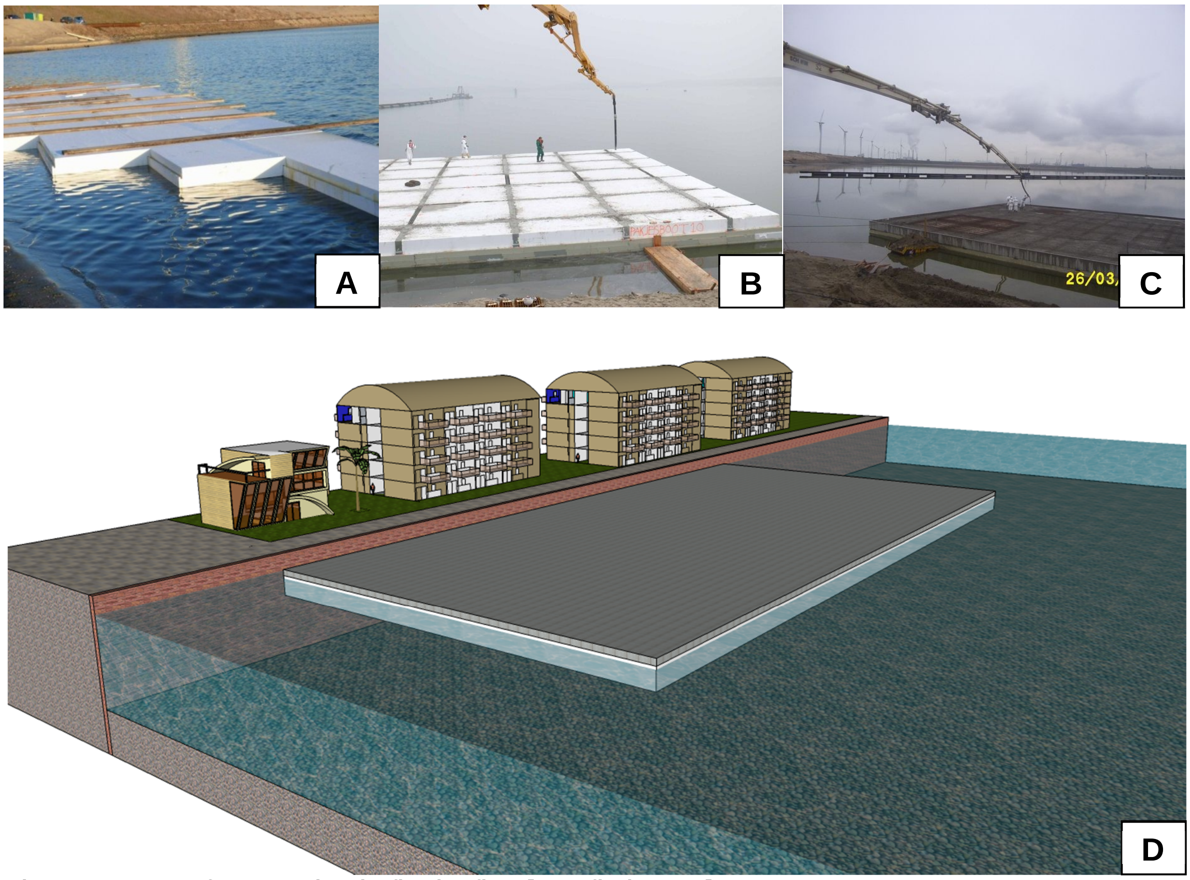


Figure 10: Stages of constructing the floating floor [www.flexbase.eu]

Figure 10a: Rectangular pieces of EPS are connected to make a floating surface in the desired shape.

Figure 10b: In pre-made recesses in the EPS, concrete beams are casted. Reinforcement cages are placed before casting the beams. The beams provide the needed rigidity to pour the floor.

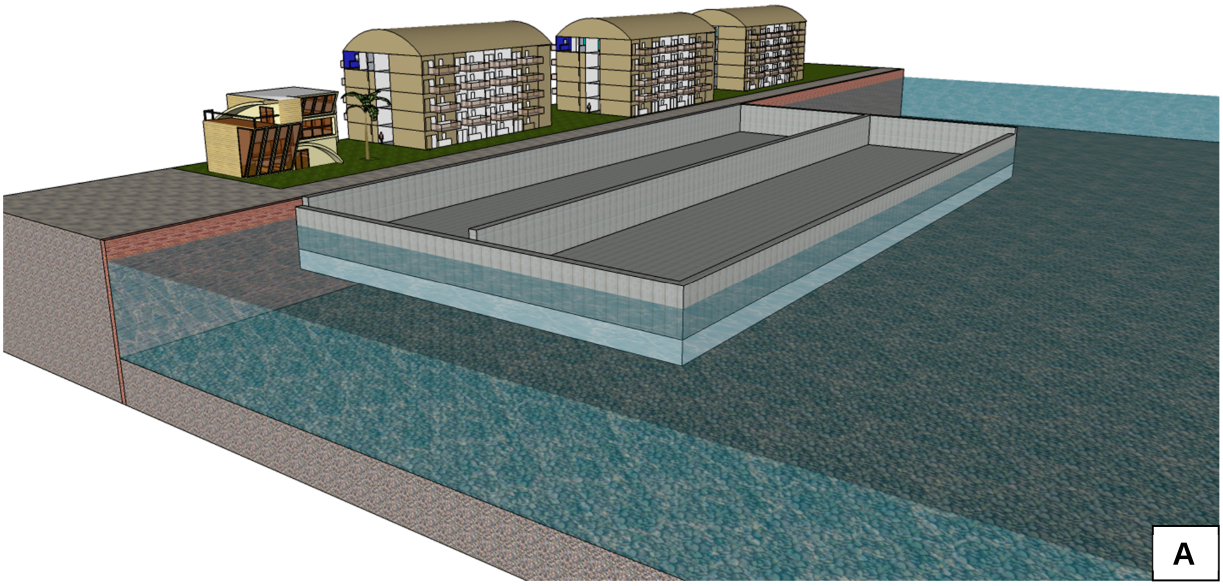
Figure 10c: After the beams are hardened, reinforcement is placed and the main floor is casted.

Figure 10d: The floating foundation is ready for the next construction stage.

Stage 2: Construction of the main structure

As was shown in figure 10, this method is already being used at a smaller scale. After stage one a house, a small office or a greenhouse is constructed directly on top of the floor. The floor forms the foundation and will stay afloat.

With the floating construction method however, the floating foundation will submerge during construction. The walls and columns of the parking garage will be constructed on top of the floating floor. As the structure begins to submerge, it is imported that the structure is watertight and stable. With the help of the figures on this page, this will be explained more thoroughly.



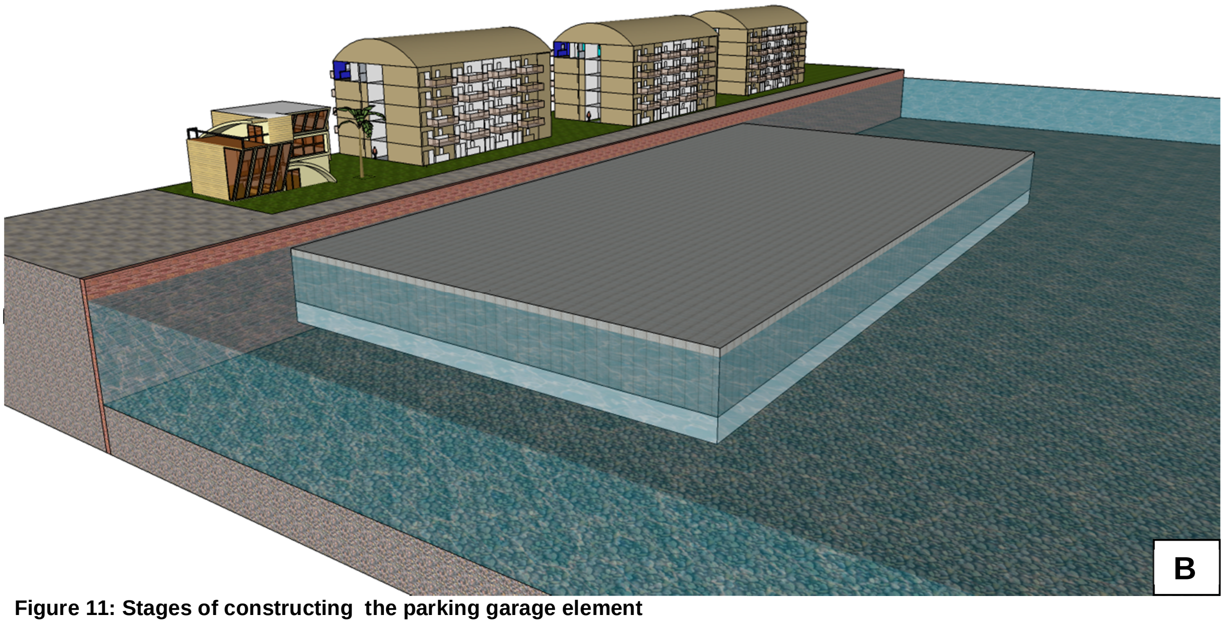


Figure 11: Stages of constructing the parking garage element

Figure 11a: The walls and columns of the parking garage will be constructed on top of the floating floor. In this phase it is important that the structure is watertight, because as the structure becomes heavier it begins to submerge. Stability is a key issue during this stage. The load, and with that the construction of the walls, has to be evenly distributed over the floor to keep the structure stable.

Figure 11b: When all the walls are constructed, the columns and roof can be constructed. At the end of this stage you end up with a floating parking garage.

Stage 3: Sink or swim

The floating garage element can be used for two types of structures, namely as a parking garage underwater or as a floating body for an office or apartments. When using it as a parking garage underwater, it will be immersed like it is done with the traditional method. But it could also be used as a floating structure. Choosing that option, the element will be used as a floating foundation and another structure could be constructed on top. The next few paragraphs will explain these two options, sink or swim.

Swim

The floating garage element could be used as a floating foundation for an office building, apartments or any other kind of structure. In this way the element serves multiple purposes. Extra ground space in the city is created and parking spaces are created for the users of the structure on top. Note that the structure in the figure below is only to demonstrate the construction method and that the structure could have any other shape or form.

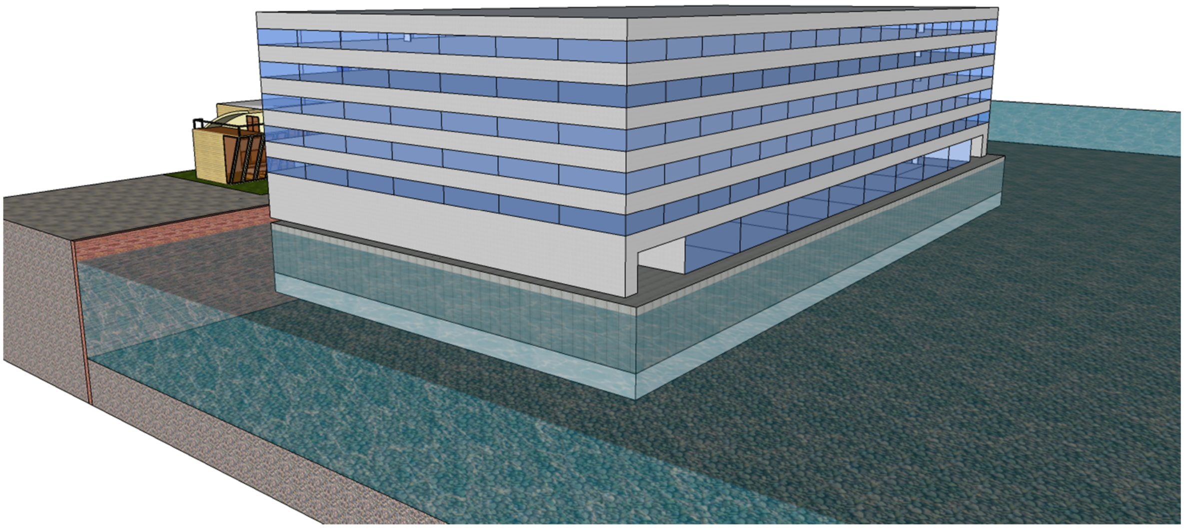


Figure 12: Parking garage element used as a floating body for an office building

Sink

When there is only a need for a parking garage under water, one can also use this construction method and submerge the garage element. This can be done by using the same technique as is used with the traditional method.

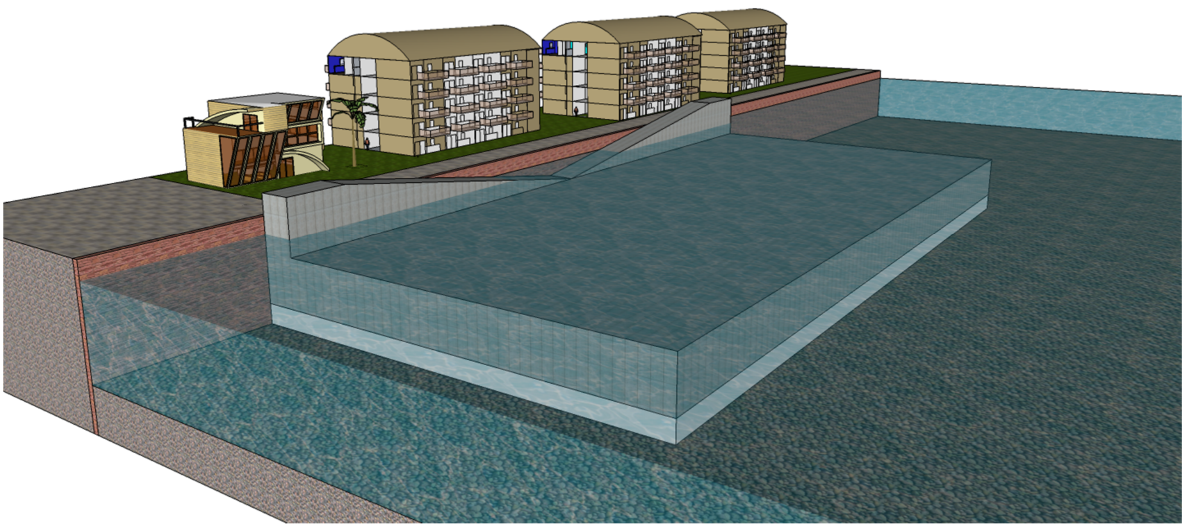


Figure 13: Parking garage underwater

Possible advantages

This MSc thesis will focus on the latter option, the parking garage underwater. Because the construction method is the same for the first option until the parking garage is submerged, it is believed that the construction method is both technically feasible as financially feasible for the first option if it is for the second. Because, with the second option, the construction method will have more stages like immersion of the garage element and the construction of a foundation, it will probably be more expensive.

To demonstrate the possible advantages, this method will be compared with the traditional construction method (in-situ with a construction pit). For both construction methods mentioned before, three significant disadvantages were mentioned. This paragraph will explain how the new construction method will solve these disadvantages.

<u>Floating construction method vs. cut-and-cover</u>

No need for construction pit

The floating construction method doesn't need the temporary works like sheet piles and underwater concrete to create a dry construction site. It only needs a floating floor of EPS that will also act as formwork for the concrete floor.

Minimal risk of settlement or collapsing structures

During construction, the floating construction method does not interfere with the surroundings. This minimizes the risk of settlement or collapsing structures.

Possibility of construction at a different location

There is still the option to construct the parking garage at another location. Is could not be favourable to construct the structure in the centre of the city. In that case, it could be constructed outside of the centre. When it is finished it is transported to the final location.

<u>Floating construction method vs. immersion of prefab element</u>

The floating construction method is in a few ways the same as this method, except for the fact that the parking garage is constructed directly on water instead of in a dry-dock. Of course, in a dry-dock, there are no stability issues during construction and the garage element doesn't have to be watertight until it is transported. Putting aside these issues for now, the advantages will be discussed.

No need for a dry-dock

When using the floating construction method there is no need for a dry-dock. This saves the costs for constructing or renting a dock.

No need for transport to location

Secondly there is no need for transportation to the location. This saves time and money and also minimizes risks.

No restrictions in size

Because there is no transportation to the final location, there are no restrictions to the size that could occur because of obstacles during transport.

Discussing these advantages, one can come to the conclusion that this construction method could not only safe a lot of money, but in some situations it could simply be the only desired option. One can think of the following situations;

i. There is no room for a dry-dock;
ii. Transport by water gives undesirable restrictions because of the dimensions of the waterway, bridges or locks;
iii. Large transport by land gives undesirable restrictions because of obstacles along the way (e.g. viaducts, tight corners);
iv. Construction in a construction pit gives problems because of an active waterway (traffic);
v. Construction in a construction pit is too expensive, because of the large depth;
vi. Construction in a construction pit could cause unfavourable settlement or other risks.

3.3 Study objectives

The previous chapters have made clear why there is a need for a new construction method and why it is believed that the floating construction method has significant advantages compared to the existing construction methods. This study will research whether or not these assumptions are right. The goal is to find out in which situations this construction method is feasible. The research question is:

“In which situation(s) is the floating construction method feasible?”

The objective is to answer this question. The next chapter describes how this goal will be reached and will be the guideline for the rest of the report.

4. Report structure

This chapter will describe the steps that are necessary to achieve the study objectives. It will not only describe what will be inside the scope of this research, but also what isn't. This chapter will be the guideline for the rest of the research and report.

The goal is to find out in which situation(s) the floating construction method is feasible. To reach this goal I will conduct a case study. This case study will be based on an existing project, namely the feasibility study on the Oosterdok parking garage in Amsterdam.

Case study: Parking underneath the Oosterdok

It must be made clear that the new construction method is the main focus of the MSc thesis and it is not the goal to make a detailed design for a parking garage. Therefore, an existing design is taken from the feasibility study conducted by the city of Amsterdam. The focus of this thesis will be the floating construction method and the technical and financial feasibility. The scheme on the next page shows the steps that will be taken to answer the research question. These steps are explained below.

<u>Chapter 5: Case study part: Parking underneath the Oosterdok</u>

This chapter will be the starting-point of the case study and will describe the geography, the requirements and the boundary conditions. As said before, the case study is based on the feasibility study conducted by the city of Amsterdam as it is believed that for this project the floating construction method could be a feasible solution. Note that when the floating construction method isn´t feasible for this situation, it doesn´t mean that the floating construction method is unfeasible for every situation. This case study is only used as a tool to investigate where this construction method could be feasible.

<u>Chapter 6: Preliminary design parking garage</u>

This chapter will deal with the preliminary design. This design is strongly based on the design that is preferred by the city of Amsterdam but will also take the recommendations of the feasibility study into consideration.

The dimensions of the floors, walls, columns and roof will still depend on estimations and rules of thumb, not on actual calculations. The actual dimensions will not be needed to develop the construction method in this stage.

<u>Chapter 7: Development of the floating construction method</u>

This chapter describes the separate steps of the floating construction method.

The construction method for the Flexbase floor will be the largest and most important part of this chapter since a Flexbase floor has never been constructed before on this scale. The construction method for the walls, floors and columns will play a smaller role. At the beginning of this study the choice has been made for the Flexbase floor, this means that other systems (e.g. concrete caisson, steel caissons) will not be reviewed.

<u>Chapter 8: Loads on structure</u>

This chapter will describe the various load combinations on the structure during construction and operation stage. This information will be the input for the structural calculations that will lead to the detailed design.

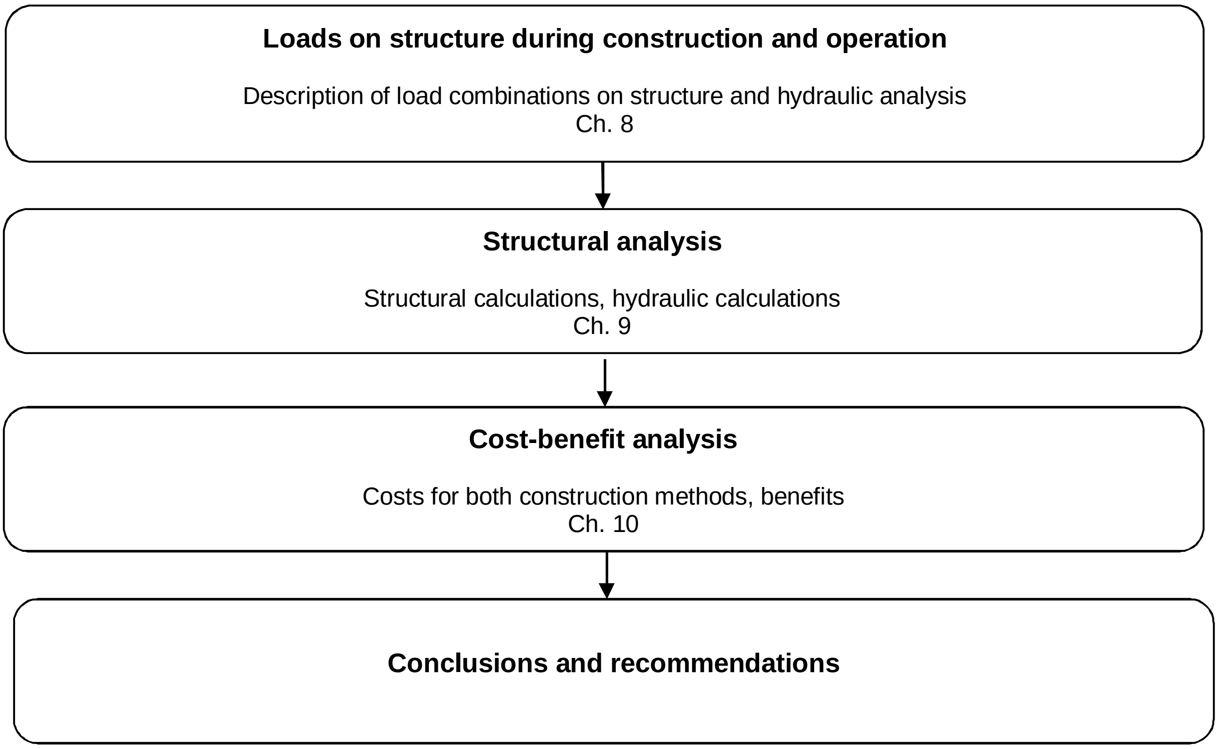
<u>Chapter 9: Structural analysis</u>

This chapter will show all the structural calculations that are needed to come to a detailed design for the parking garage and determine the technical feasibility. The complex and time consuming calculations will be done by using structural analysis and design software (SCIA) to save time. The rest of the calculations will be done by hand. Note that only the structural design is part of this case, it will not review the inner works of the parking garage (e.g. installations).

Chapter 10: Cost-Benefit analysis
This chapter will research the financial feasibility. It will first determine the construction costs of the final design. The second step is to determine whether or not the floating construction method gives an economic advantages, i.e. the construction costs are lower for this construction method than they are for the traditional method. But this is not necessarily the most significant benefit. There could also be other benefits that are not easy to express in direct costs like less hindrance for the surrounding area and a smaller risk of settlement.

The final step in determining the financial feasibility will be calculating the expected return over 50 years. This will show what the parking tariffs will have to be to make a profit in the end.

Report structure



Part three: Floating structures

In addition to the main case study there will be an extra case. This mini case will examine the possibility of using the parking garage as a floating body for apartments or an office building in a city centre. This option could add significant value to the structure and will most likely be easier to construct.

The mini case will not focus on structural calculations and a detailed design but on the possible (dis)advantages and research possibilities for further studies in the future.

Part I Case study: Parking underneath the Oosterdok

5. Introduction to the case study

An existing project is chosen for this case study, namely the Oosterdok parking garage in Amsterdam. At this moment, the city of Amsterdam is still investigating the feasibility of a parking garage underwater in the Oosterdok. The feasibility study, done in May 2009 by the city of Amsterdam, will be the starting-point of this case study. This chapter will first highlight the most important conclusions of the feasibility study. This will explain why there is a need for a parking garage and which designs and construction methods have been reviewed. To conclude, this chapter will explain why this case is ideal to use as the starting-point for the development of the floating construction method. [Stadeel Centrum, 2009]

5.1 Oosterdok parking garage

Because of multiple causes, the number of parking spaces in the surroundings of the Oosterdok island will decrease over the next few years. To compensate this decrease, the city of Amsterdam wants to construct a parking garage underneath the Oosterdok, i.e. underwater. They have also considered to construct the parking garage underneath the city canals, but after the feasibility study they have chosen for the location at the Oosterdok. The main reasons were: less disturbance during construction, lower costs, less risks during construction and better accessibility. The parking garage will mainly be used by inhabitants of the surrounding area with a parking licence (70%). With this new parking garage the city strives to keep the accessibility for the inhabitants and keep cars out of the city centre. [Oosterdok, 2009]

Geographic's

The figure below shows the future location of the Oosterdok parking garage. It will have a central location in between Amsterdam Central Station, the Oosterdok Island and the Wallen. The Oosterdok Island is developing rapidly at the moment and a large number of apartments and office buildings are being constructed. This is also one of the reasons why a large number of parking spaces is needed. The entrance of the future parking garage will be located at the Prins Hendrikkade, south of the Oosterdok. The Oosterdok has a direct connection with "Het IJ" which makes transportation of materials by boat possible. There is also a good connection with the A10 highway. The figure below shows an aerial photograph of the area in 2009. Now, in 2011, the Oosterdokkade has been replaced by the Oosterdok Island Bridge. This new situation will be reviewed in the next chapters.

Figure 14: Preferred location Oosterdok parking garage [Google Earth, 2009]

5.2 Reviewed construction methods and designs

Design Oosterdok parking garage

The Oosterdok parking garage needs at least 350 parking spaces to meet the current requirements. The feasibility study describes two designs for the parking garage, the first one has one floor and the second one has two floors. The design with two floors is chosen, mainly because of two reasons:

i. Less surface area means a smaller construction pit, i.e. less costs for temporary works (e.g. underwater concrete, sheet piles);
ii. Less surface area means less escape routes for pedestrians needed.

The second design aspect that is reviewed in this feasibility study is the entrance of the parking garage at the Prins Hendrikkade. A choice has to be made between a parallel and a perpendicular connection, which are shown in figure 15. The parallel connection is favoured, even though it would demand a longer tunnel, significant changes of the quay wall and will result in higher construction costs (approximately 4 million euro extra). The parallel connection fits better in the current situation and doesn't need the sacrifice of any water surface. Because of the significant extra costs for the parallel connection, extra research is recommended. [Oosterdok, 2009]

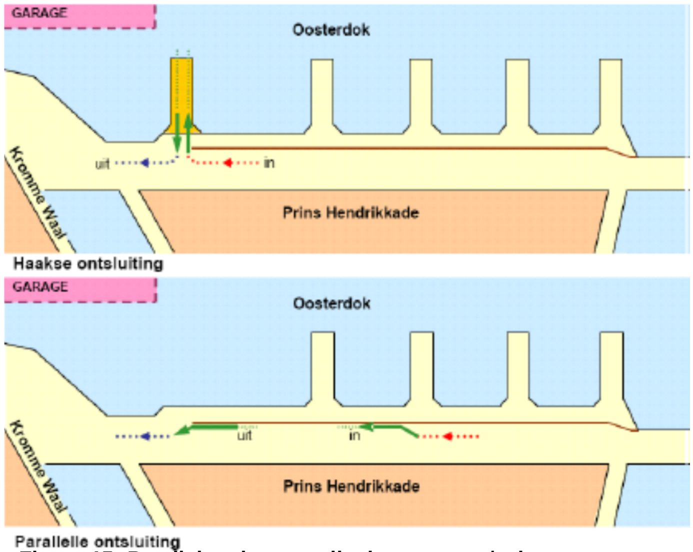


Figure 15: Parallel and perpendicular entrance/exit [Oosterdok, 2009]

Base alternative

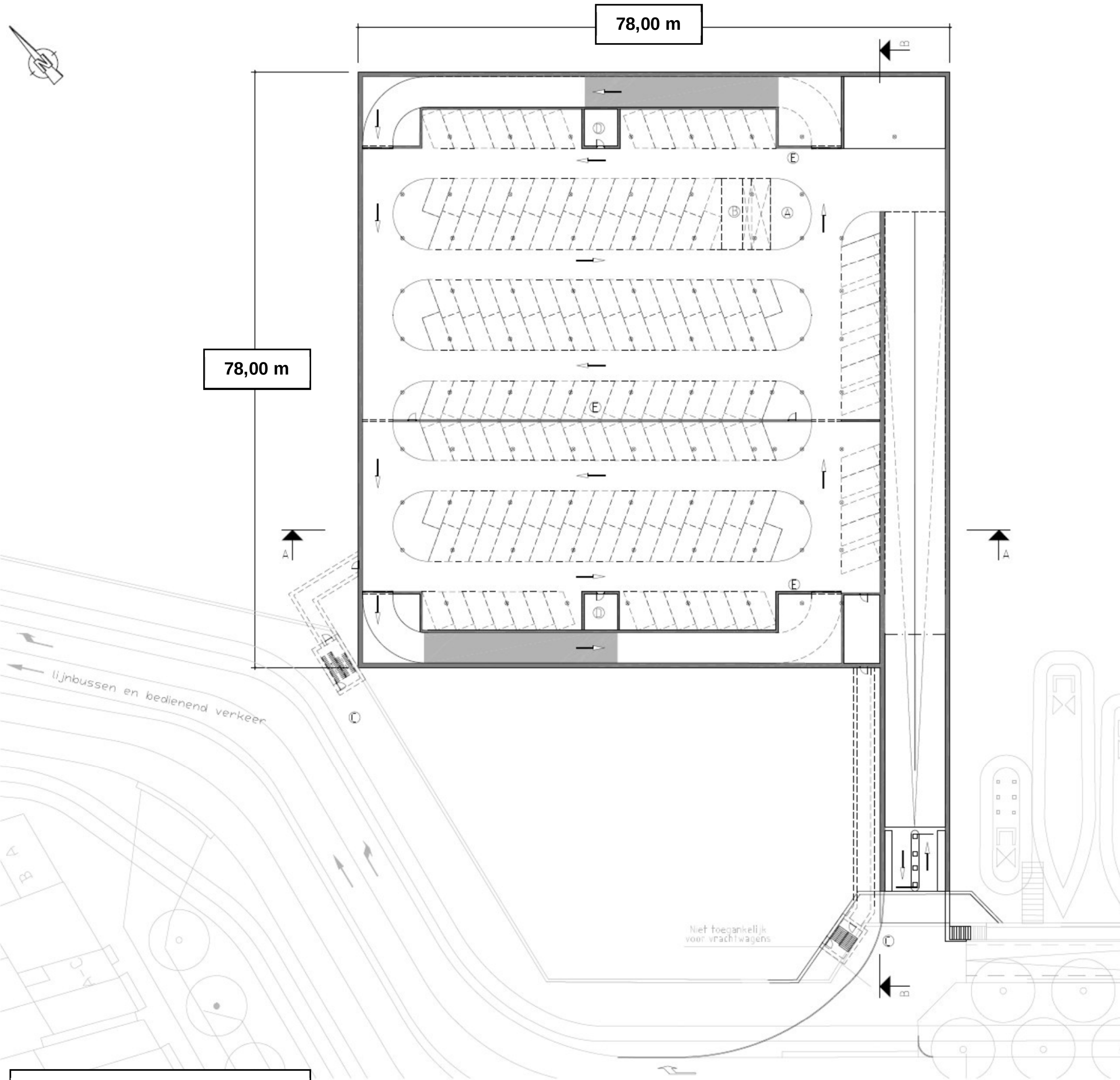
The study recommends further research in the future, but for now the favoured alternative is a parking garage with two floors that has an entrance with a parallel connection at the Prins Hendrikkade. Based on these requirements, a first design was made that is shown on the next page. Although a parallel connection is favoured, the base alternative has a perpendicular one. This seems strange but this is thought to be the design with the lowest costs. In this way it is easier to see what a parallel connection or another adaptation in the design would do to the costs.

The parking garage is 78 meters wide, 78 meters long and 7,70 meters high. The water depth above the garage is 4,60 meters. The floor and roof are only supported by walls on the outside and one on the inside of the garage. The rest of the support is created by columns. The columns are placed in a grid of approximately 7,20 by 8,25 meters. The absence of walls gives an open feel to the parking garage which gives users of the garage a greater feeling of safety.

The parking spaces are placed at an angle, this is called a fishbone configuration. This gives a few advantages. It is easier to park (which allows for a narrower road) and the parking unit (the width of two parking spaces and a road in the middle) becomes smaller. They have chosen for one-way traffic to make the parking unit even smaller. This ensures a smaller possible distance between the columns and a more optimal use of space.

The construction costs are determined for this base alternative. The next few paragraphs will review the construction methods and construction costs for this alternative. [Witteveen + Bos, 2009]

Base Alternative Oosterdok parking garage [Witteveen + Bos, 2009]



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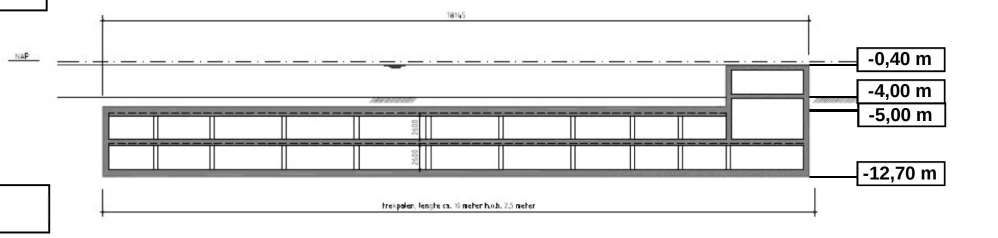
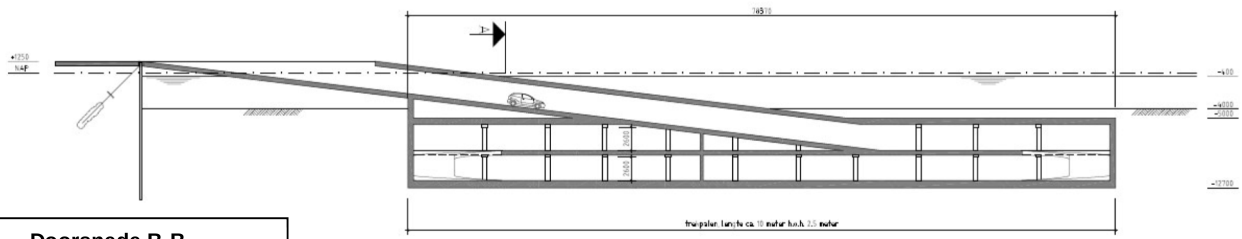


Doorsnede A-A



Doorsnede B-B

Construction methods

An estimation of the construction costs is made for the preliminary design that is shown on the previous page. Three construction methods were reviewed, namely:

i. Traditional construction method with a construction pit;
ii. Immersion of prefab elements;
iii. Pneumatic caisson method with prefab base elements.

Immersion of prefab elements

Because of the narrow waterways and bridges, the parking garage can't be made out of one element. The element would be too large and transport would be impossible. This means there will have to be multiple elements that have to be coupled at the final location. This would make the construction costs to high and make a parking garage with an open character impossible (because of multiple inner walls). Therefore it isn't considered as a feasible construction method.

The other two construction methods are thought to be feasible. The construction costs are calculated for these methods. The next few paragraphs will review these methods and the estimated construction costs.

Traditional with a construction pit

This method contains the following construction stages:

i. Construction of temporary construction pit with sheet piling and anchorage;
ii. Excavation of earth in the construction pit;
iii. Construction of work floor with tension piles and underwater concrete;
iv. Construction of permanent structure;
v. Filling the construction pit with earth and removal of the sheet piles and anchorage;
vi. Construction of the inner works and entrance/exit.

Because of the large distance between the walls of the construction pit (+/- 80 meters), struts can't be used. This means they have to be supported by anchorage. The use of anchorage is mentioned in the risk top-10 in the risk evaluation report. The anchors could not be sufficient to support the walls and the walls could cave in.

The sheet piles are also mentioned in the risk top-10. Driving the piles could cause settlement which could lead to damage to the surrounding structures. When the construction pit isn't watertight, the groundwater level could drop and this could also cause settlement. It could be that the sheet piling isn't watertight, but the floor of underwater concrete could also have this problem. Another risk is that the piles hit unknown obstacles underground which would result in higher costs.

Although this is a traditional and often used construction method, 8 out of the 10 risks in the risk top-10 are directly connected with the construction of the construction pit. These are risks that where already mentioned in chapter 3.

Pneumatic caisson method with prefab base element

This method contains the following construction stages:

i. Dredging the clay layer and applying a sand layer;
ii. Transport, couple and submerge the elements for the base of the caisson (this base is higher than the water depth so it rests on the bottom but the top stays above the waterline);
iii. Construction of the parking garage on top of the base;
iv. Immersion of the garage by excavating the soil underneath the caisson by divers;
v. Construction of the inner works and entrance/exit.

The advantage of this method is that it doesn't need a construction pit and it has relatively little influence on the surroundings (e.g. risk of settlement). The disadvantages are the high costs of excavation and the costs for the prefab base of the caisson. [Witteveen + Bos, 2009]

Construction costs and preferred method

The construction costs are calculated for both construction methods. The costs (construction costs / profit / risks / engineering etc.) for the base alternative, using the traditional construction method, are estimated at **€ 20.100.000,-.** When using the caisson method, the costs will be slightly higher, namely **€ 21.950.000,-.** This does not include the entrances, only the parking garage itself.

The temporary structures form a large part of the construction costs with the traditional construction method, i.e. the costs for the construction pit. These costs are:

i.	Sheet piles for the construction pit:	€ 825.000,-
ii.	Anchorage:	€ 605.000,-
iii.	Tension piles work floor:	€ 1.100.000,-
iv.	Underwater concrete:	€ 1.200.000,-

This forms a total of **€ 3.370.000,-**. This forms 30% of the direct construction costs. The caisson method is more expensive because of the prefab, extra concrete works (approximately € 3.000.000,- extra) and higher excavation costs.

The traditional construction method is preferred because this method is abundantly applied, therefore the risks are easier to control. And secondly the construction costs are lower.

Alternative location

The base alternative has an exit for pedestrians at the Oosterdok Island Bridge (OIB) and at the Prins Hendrikkade. This means that the pedestrians that exit at the Prins Hendrikkade have a larger walking distance to the centre. To prevent this, there is an alternative that has both exits at the location of the new OIB. This means that the location of the parking garage shifts to the west. This is the “West Alternative” that is shown as a blue square in the figure below. The grey square represents the base alternative. This figure shows the existing situation where the platform for busses at the Prins Hendrikkade is demolished and the OIB has replaced the Oosterdokkade. Although the west alternative is better for pedestrians, the entrance tunnel for cars has to be longer (77 meters). The costs for the west alternative are **€ 21.240.000,-.** The exit and entrance at the OIB are preferred, but because of the extra construction costs they conclude with giving a recommendation for further research on these alternatives. [Witteveen + Bos, 2009]

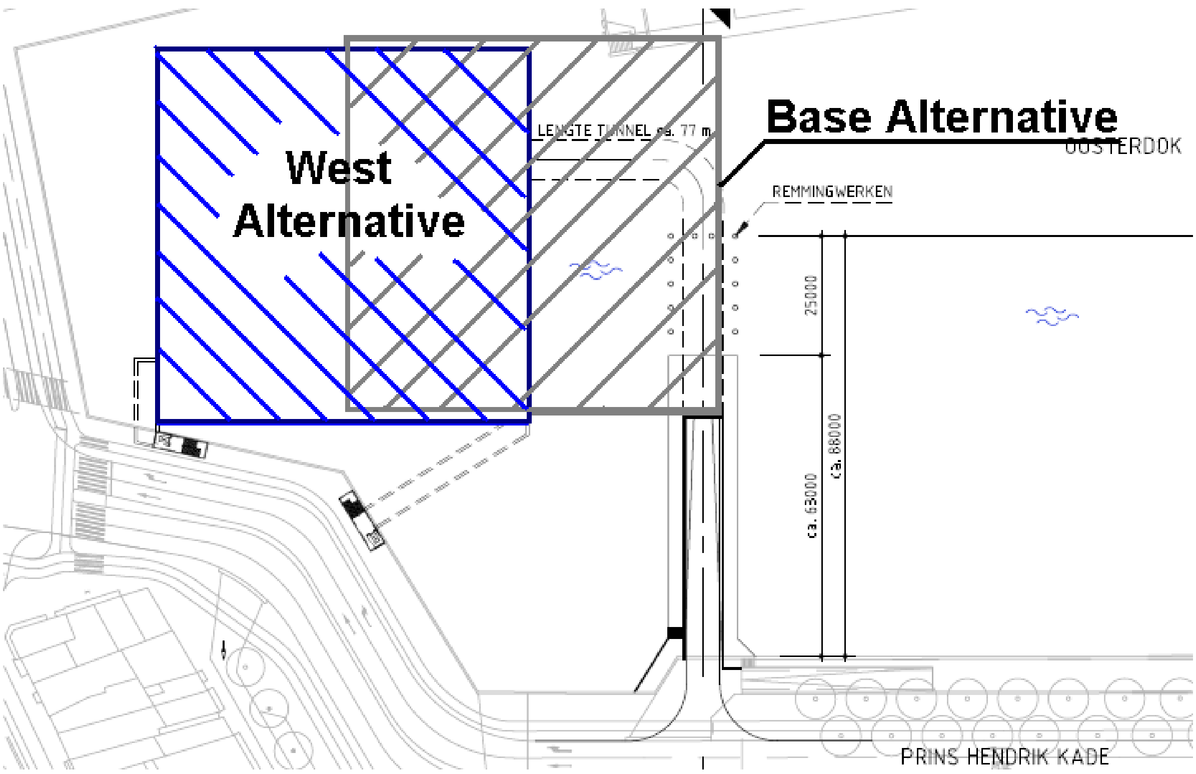


Figure 16: Location of the “West Alternative” and “Base Alternative” [Witteveen + Bos, 2009]

5.3 Opportunity for the floating construction method

This could be the perfect project for the floating construction method. Chapter 3.2 described the situation in which this construction method would most likely be feasible. The most important and relevant for this case study are reviewed.

Immersion of prefab elements is impossible because transport by water gives undesirable restrictions
This true for this location. The feasibility study concludes that immersion of elements is too expensive because of size restrictions caused by the waterway. With the floating construction method it is possible to immerse garage elements.

Construction in a construction pit is expensive
The costs for the construction pit form 30% of the direct construction costs. This is a very large part that could be saved when using the floating construction method. The floating construction method will have extra costs for the floating floor and measures to keep the caisson from floating back to the surface. For the floating construction method to be feasible, these costs will have to be lower than the costs of the construction pit.

Construction of the construction pit could cause unfavourable settlement
This is mentioned five times in the risk top-10. These risks are probably not present when using the floating construction method because it hardly affects the surroundings. [Witteveen + Bos, 2009]

These arguments are sufficient to believe that the floating construction method could be feasible for this situation. A preliminary design is worked out in the next chapter. This design will be based on the base alternative and the recommendations that are given in the feasibility study of the city Amsterdam.

Note that the floating construction method will be feasible for this situation when the construction costs are lower or when it gives significant advantages. But the aim of this MSc thesis is much broader. This case study is only used as a tool to develop the construction method and to determine in which situations the floating construction method could lead to lower construction costs or give other significant advantages.

6. Preliminary design

Before the floating construction method can be developed, a preliminary design has to be made. This preliminary design will be based on the base alternative as shown in chapter 5. The new design will take the recommendations of the feasibility study into account.

6.1 Starting-point

To get a better idea of the design and surroundings of the future parking garage, the surroundings are drawn in a 3-D model. At this moment the OIB is ready and the Oosterdokkade and bus platform are demolished. The two piers at the left are new and the new quay walls are constructed out L-walls on a pile foundation. The bridge is made out of prefab beams and has one support in the middle. Picture 17 shows a top-view of the 3-D model.

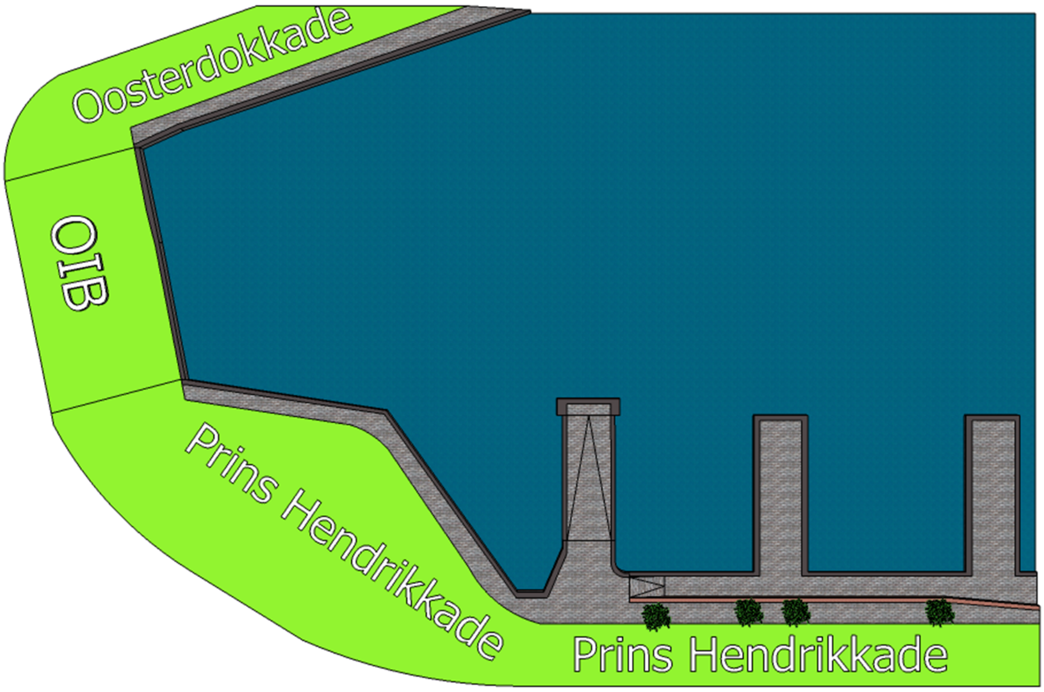


Figure 17: Top-view of the Oosterdok, Amsterdam [Google SketchUp 7]

As shown in figure 16, there are two alternatives for the location of the parking garage, the west and the east alternative. The east alternative makes optimal use of the already existing pier for the entrance/exit for cars. The west alternative has both entrances and exits for pedestrians at the crossing of the Prins Hendrikkade and the OIB. This makes the walking distance to the city centre smaller. However, the west alternative needs a long tunnel which will lead to high construction costs. The feasibility study recommends further investigation to find an optimal location.

Another recommendation is to make the parking garage larger. The base alternative has 350 parking spaces. With this number of spaces, the parking garage is barely feasible. To increase the revenues, it is recommended to investigate the feasibility of a parking garage with 500+ parking spaces. Because of the rapid development of the Oosterdok island, it is believed that there is enough demand for more parking spaces. It is estimated that the Oosterdok island will need over 1500 extra parking spaces.

With these two recommendations in mind, the location and rough dimensions for the parking garage are determined. The parking garage will become larger, to be exact: 78 by 130 meters instead of 78 by 78 meters. This means that the parking garage will make use of the already existing pier for the entrance/exit for cars and pedestrians (upper right corner) and will have three extra entrances and exits for pedestrians, two at the corner of the Prins Hendrikkade and the OIB and one at the side of the Oosterdok island. The latter entrance has to make sure that citizens and visitors of the Oosterdok island will make use of the parking garage. [Oosterdok, 2009]

The larger dimensions also mean that there will be more parking spaces. The next chapter will use this input and describe the preliminary design. Note that the figure doesn't show the final situation. The parking garage will eventually be underground and underwater. The figure is made this way to show the entire structure. The main structure will be constructed with the floating construction method (light gray in the figure). This part will also be the main focus of this study. The entrances for cars and pedestrians will be constructed in-situ.

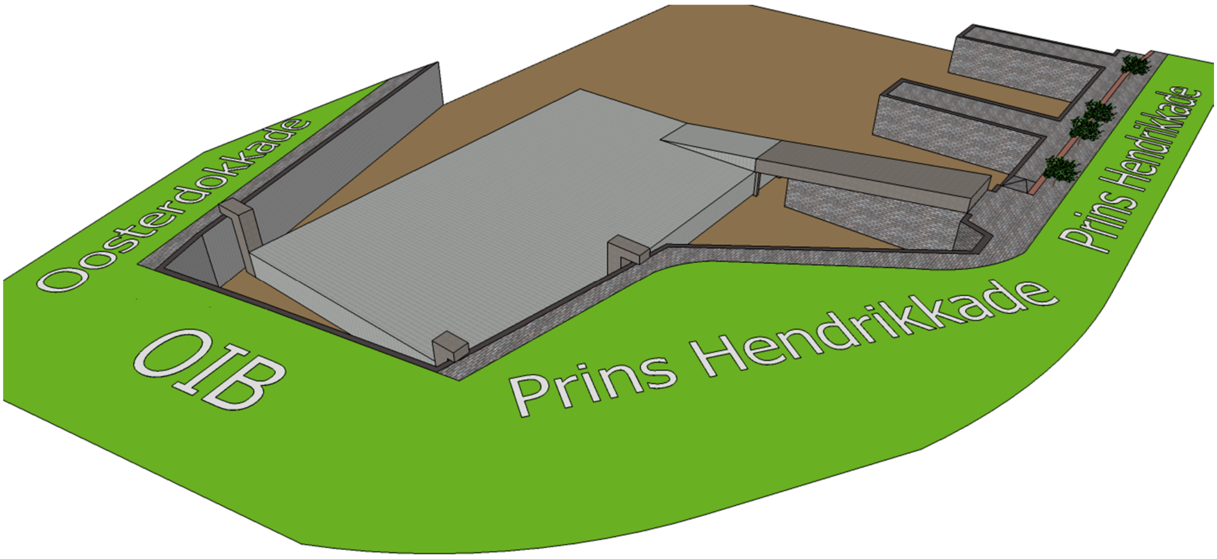


Figure 18: Location of the Oosterdok parking garage, preliminary design [Google SketchUp 7]

6.2 Preliminary design

The preliminary design will be based on the "Base Alternative". The two primary changes will be:

i. Larger length for more parking spaces, 130 meters instead of 78 meters;
ii. Four exits for pedestrians instead of two, needed to keep the escape routes short enough.

Requirements

The following requirements are used for the layout of the parking garage. These requirements comply with the NEN 2443 (Parking cars in parking garages). [Keypoint, 2010]

Capacity

The number of parking spaces will be 750.

Car accessibility

Number of entrances/exits	There will be one entrance/exit for cars. According to NEN 2443 this is enough for 350 cars an hour. This is more than enough for a parking garage of this size.
Width of roads	One-way roads will be 4,00 meters wide, two-way roads will be 6,00 meters wide.
Angle of ramps	<12,5%
Radius of curves	> 4,00 m
Traffic	One-way

Pedestrian and emergency exits

Number of exits	4
Distance nearest exit	< 40,00 m

Parking

Width parking space	2,40 m
Clearance columns	0,15 m
Parking angle	65°
Clearance height	2,60 m

Floor plan

Before the main structure can be designed, there has to be a floor plan. The floor plan determines the distance between the columns and walls. This is mainly determined by the parking unit width (length of two parking spaces plus a road). Using the requirements, this width is 14,40 meters. This means that a centre to centre distance of 7,20 meters between the columns would be ideal. The parking spaces are 2,40 meters wide and the parking spaces next to a wall or column are 2,55 meters wide. This means that the centre to centre distance of the columns will become 8,25 meters in the driving direction. This is shown in figure 19.

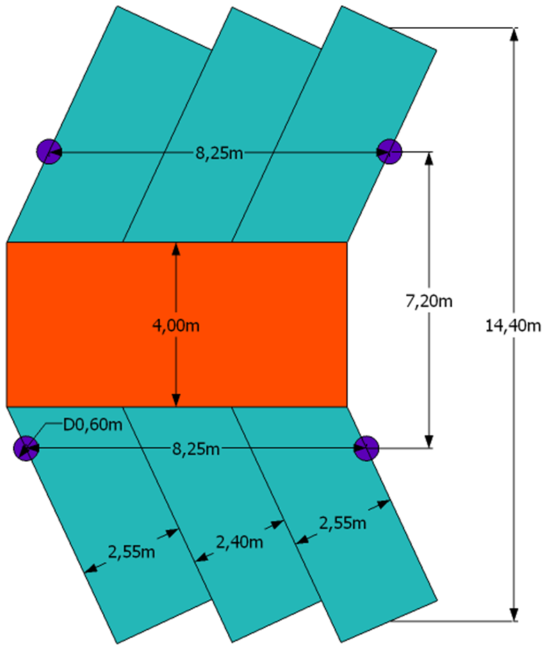


Figure 19: Dimensions parking spaces and columns

There will be one wall in the middle to compartmentalize the garage in case of a fire. There will also be rooms for the manager, technical equipment and storage.

Using these requirements, the figures on page 31 and 32 show the floor plans of the top and bottom floor. Both floors have 375 parking spaces. The parking garage will have a total of 750 parking spaces.

The main dimensions of the parking garage are:

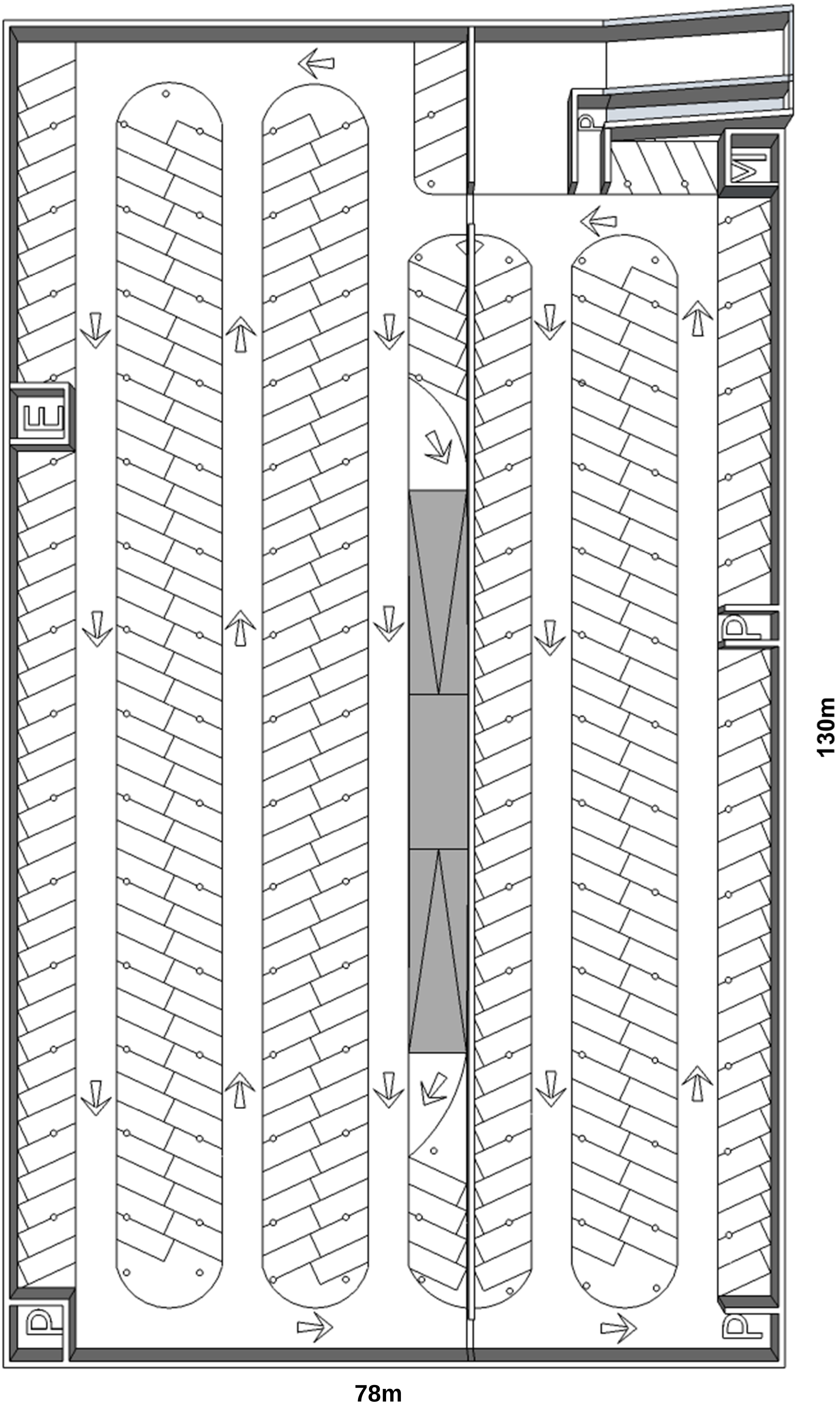
Width	78,00 m
Length	130,00 m
Height	7,70 m
Floor	800 mm*
Intermediate floor	600 mm
Roof	700 mm
Outer walls	600 mm
Inner wall	400 mm
Columns	Ø600mm, 8,25 m c.t.c. in drive direction and 7,20 m c.t.c. in perpendicular direction.
Free height above floor:	2,60 m

Symbols floor plan:

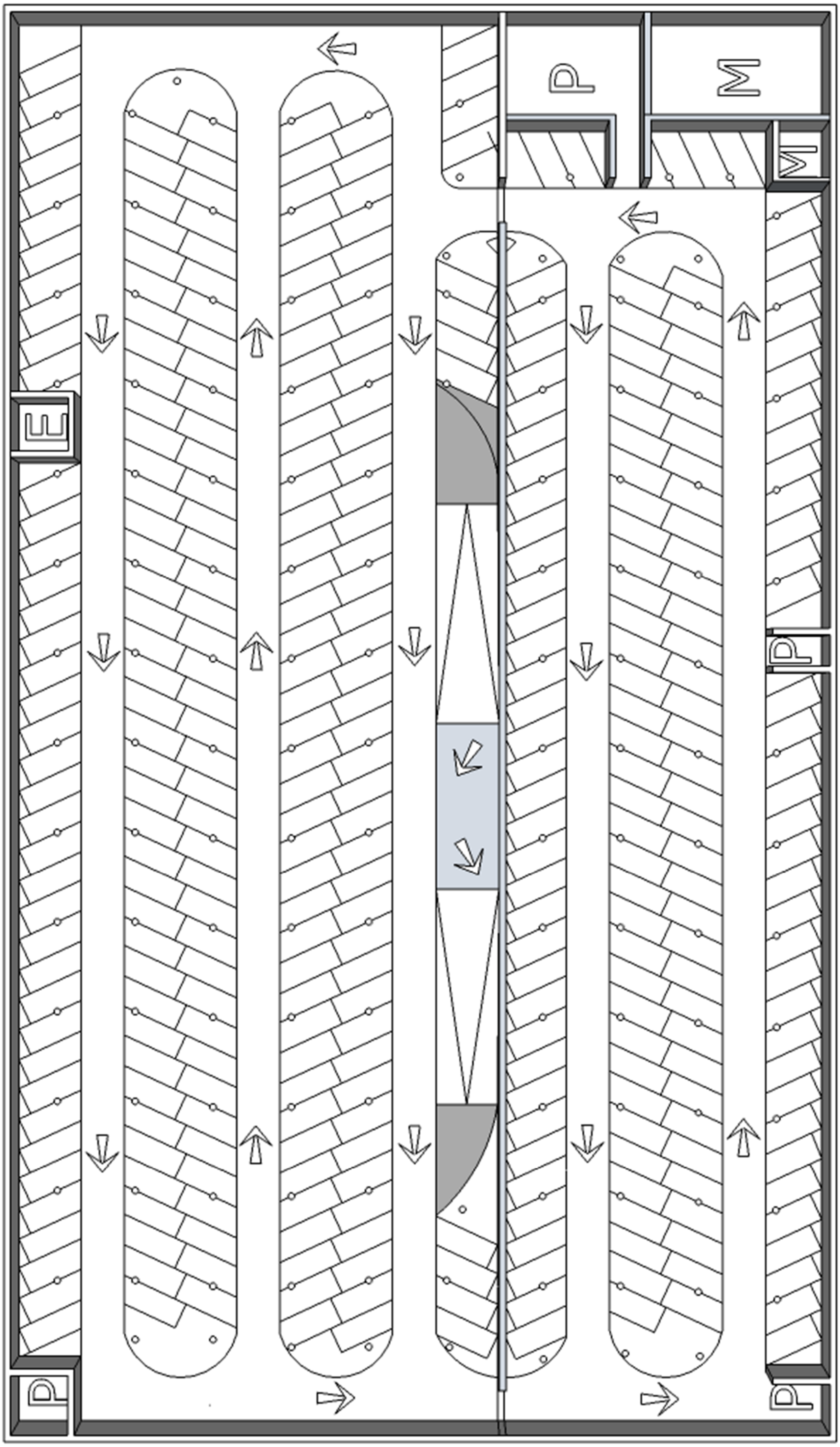
M	Maintenance/storage room
P	Entrance/exit pedestrians
E	Emergency exit

*Note that this is the thickness of the floor when constructing the parking garage with the traditional method. The Flexbase floor will become much thicker (in the order of a few meters).

Top floor



Bottom floor



7. Development of the floating construction method

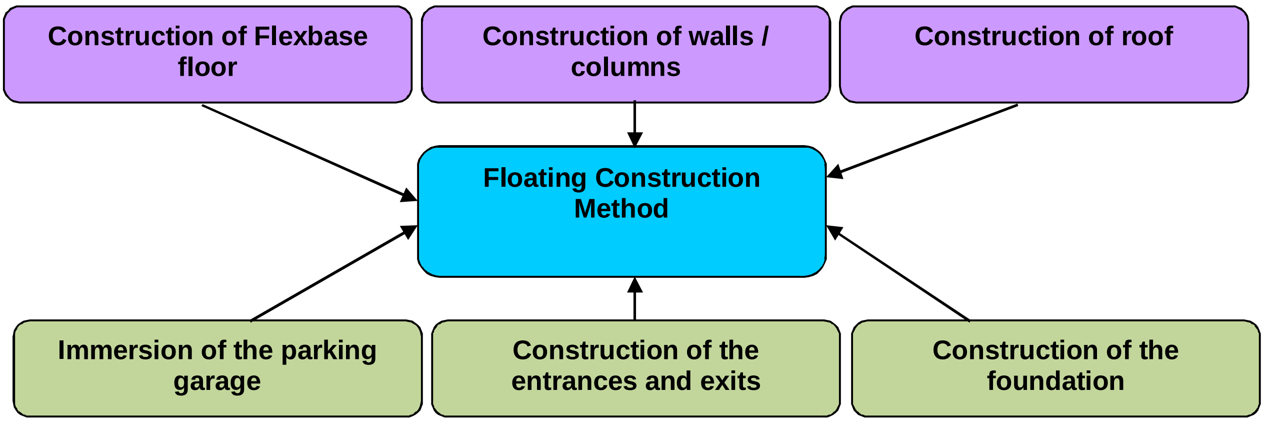
Constructing a parking garage on a floating Flexbase floor has never been done before. Therefore, this construction method has to be developed first. This chapter will describe this construction method chronologically. Meaning that it will start with the floor of the structure and will end with the finished structure. This chapter will be divided into the following sub-chapters:

i. Constructing the Flexbase floor;
ii. Constructing the walls and columns;
iii. Constructing the roof;
iv. Constructing the entrances/exits;
v. Constructing the foundation;
vi. Immersion of the structure.

Each chapter describes the construction stage chronologically. When possible, alternatives are reviewed. These alternatives could be other construction methods, different materials, equipment etc. Based on the positive and negative aspects of every alternative, a choice is made for a certain construction method, materials or equipment to be used. The input for these choices will not only be literature on the subject. Multiple interviews are held with experts on floating constructions, Flexbase floors, materials, equipment etc. This is done to incorporate the latest developments and experiences into this research.

Although it is described as if each construction stage is seen separately, the opposite is true. The figure below shows that all the construction stages have to work together to form an integral construction method.

Note that 'Immersion of the parking garage', 'Construction of the entrances and exits' and 'Construction of the foundation' have a different colour. After the first three construction stages (shown in purple) you end up with a floating parking garage. At this stage the choice could be made to immerse the parking garage onto a foundation or keep the garage afloat and use it as a floating foundation for another structure. This chapter will describe the first option and the extra case study will describe the second.



7.1 Constructing the Flexbase floor

Flexbase BV is a cooperation between Dura Vermeer and Unidek and develops floating foundations for construction projects on water. The Flexbase design is based on the combination of the buoyancy of EPS and the structural properties of reinforced concrete. This design will be the starting-point for the development of the floating construction method.

Combination of EPS and concrete

The combination of EPS and concrete was first introduced in the world of real estate in the 1980's. It was used for constructing houses on water and should save the costs of a dry dock or construction facility. [M. Koekoek, 2010] These systems relied on the EPS elements for its buoyancy and concrete for the structural strength. The great advantages of these elements is that they are practically unsinkable. These first systems used prefab concrete elements and where relatively small (10 x 10 meters). The Flexbase system also uses the EPS elements as formwork for the concrete beams and floor and is therefore an unique and patented construction method.

This combination needs less concrete than watertight concrete caissons. Concrete is only needed for the floor to give the body strength and it is not needed to create a watertight floating body, because the EPS is watertight itself. The biggest advantages of this EPS and concrete system are:

i. Unsinkable;
ii. Low maintenance;
iii. Construction on water possible;
iv. Any shape or form possible;
v. Low self-weight.

The disadvantage during construction is its flexibility.

Flexbase options

The Flexbase floor is available in three different versions: Flexbase light, Flexbase standard and Flexbase heavy. The three versions are shown in figure 20. The light version only consist out of an EPS base and a concrete floor. This floor is lightweight and could be constructed relatively fast. However, the structure is not very stiff and can only be used for lightweight structures (e.g. greenhouses).

The standard version has a rectangular beam grid to give the floor rigidity in two directions. The beams are poured first and will provide the needed rigidity to pour the floor. The beams and floor are reinforced with traditional reinforcement. This system can take higher loads and is used for the construction of middle weight structures like houses and small offices.

These first two options will probably not be strong enough for the construction of the parking garage. Therefore, the heavy version is needed. This version consists of an extra beam grid and floor which creates a sandwich structure. The dimensions of the beams and floors will depend on the needed rigidity and strength. The desired c.t.c. distance of the beams is approximately 4,00 meters. This is the standard size of the EPS elements which will fill up the space between the beams and act as formwork.

Research on other materials

Research was done for a large floating greenhouse (project 'Floating Roses') on using other materials than concrete to provide the structure with the necessary rigidity. The study researched alternative materials for the beams like steel, aluminium and composites. For the floor they considered bubbledeck floor systems and composites. The study concluded that these alternatives made the structure lighter, but much more expensive. Because the parking garage will be immersed, a lighter structure will not give an direct advantage. Therefore, other materials will not be reviewed at this moment. [Floating Roses, 2011]

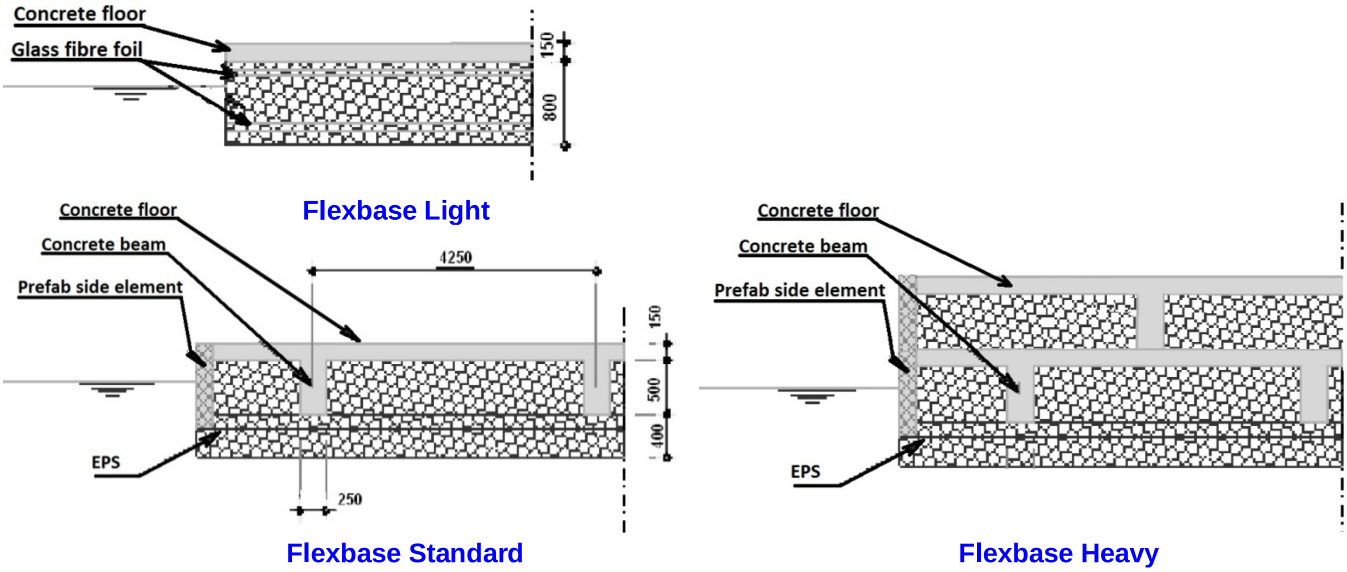


Figure 20: Standard Flexbase options

Note that only the standard Flexbase floor has been constructed before. Even that floor has only been constructed four times for an actual project. Flexbase Light and Flexbase Heavy are still concepts. At this moment there are no reasons to believe that these floors could not be constructed. Multiple feasibility studies on, and test with, these floors have been conducted. For these studies and tests see www.Flexbase.eu.

EPS

EPS (expanded polystyrene) consists for 98% out of air and is therefore much lighter than water (20 kg/m3). EPS 60 and EPS 250 are used for the Flexbase Heavy floor. The number stands for the short term compressive strength of the EPS at 10% distortion. This means that the EPS can respectively take 60 and 200 kPa of pressure for a short period of time. Because the structure will be immersed for a long period of time (life time 50 years), also the long term compressive strength will be used (approximately 30% of the short term strength). [www.unidek.nl]

See appendix H for EPS parameters

The Flexbase floor consist out of two types of EPS. The first layers will have to resist the water pressures and will be of a higher density. The layers between the two floors will only act as formwork and will be of lower density, namely EPS 60.

Appendix H shows that EPS distorts because of shrinkage, creep and the induced loads. The graphs in this appendix show the total distortion of the EPS in relation to the induced load. These graphs will be used to select the needed type of EPS. Two stages are divined, namely:

i. End of construction stage, floating;
ii. Operation stage.

Construction stage

Constructing the complete parking garage could take up to two years. Therefore the long-term compression strength is used (0,3 x short-term compression strength). When the garage element is finished, the draught is 6,05 meter as is shown in figure 21.

This means that the water pressure on the bottom EPS layer will be 60,5 kN/m^2. Also the sides of the second layer will have to withstand high pressures. When staying within the limits of the long-term compression strength, this means that EPS 250 is needed (60,5 / 0,3 = 201). The total distortion of the EPS will have a maximum of 2,5%.

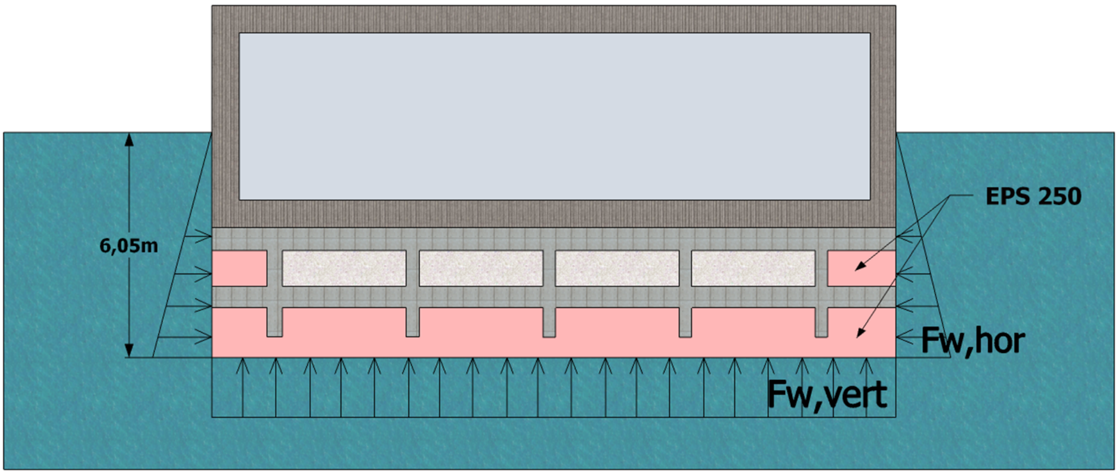


Figure 21: Maximum water pressure on EPS during construction stage

The graph in appendix H (page 122) also shows the distortion of the EPS when higher loads act on the EPS. However, the manufacturer cannot guaranty the distortion above the ‘0,3 x short-term load’ limit. In an email the engineer of Unidek stated that higher loads could cause uncontrolled distortions or even complete collapse of the EPS.

Operation stage
In the operation stage the water pressure on the bottom EPS layer is approximately 150 kN/m^2. This means that EPS 500 is needed. However, EPS 500 is more than twice as expensive as EPS 250 (approximately 100 euro per m^3 more). Considering that the layers of EPS below the first floor consist out of 15.500 m^3 of EPS, this means € 1.550.000,- extra costs to use EPS 500 instead of EPS 250.

It is therefore important to know whether or not it would be a problem if the EPS fails in the operation stage. In this case the worst case scenario is researched, namely complete collapse of the bottom layer. This layer is 800 mm thick. What happens with the structure depends on the method that is used to keep the garage element immersed, namely ballasting or grout anchors. This is described in the next few paragraphs, using figure 22 and 23.

Collapse of EPS: Ballast
The figure below shows the forces that act on the garage element in the operation stage. The force Fg represents the complete weight of the element, including the weight of the ballast. To keep the element immersed, this force will have to be larger than the buoyancy force ($Fg \approx 1{,}20 \times F_{buoyancy}$). In the worst case scenario (collapse of the EPS), the settlement of the element will be equal to the thickness of the collapsed layer, namely 800 mm! It is obvious that this would be catastrophic. To prevent this, EPS 500 is needed for the EPS layers below the first floor.

Conclusion: when ballasting the element, EPS 500 is needed.

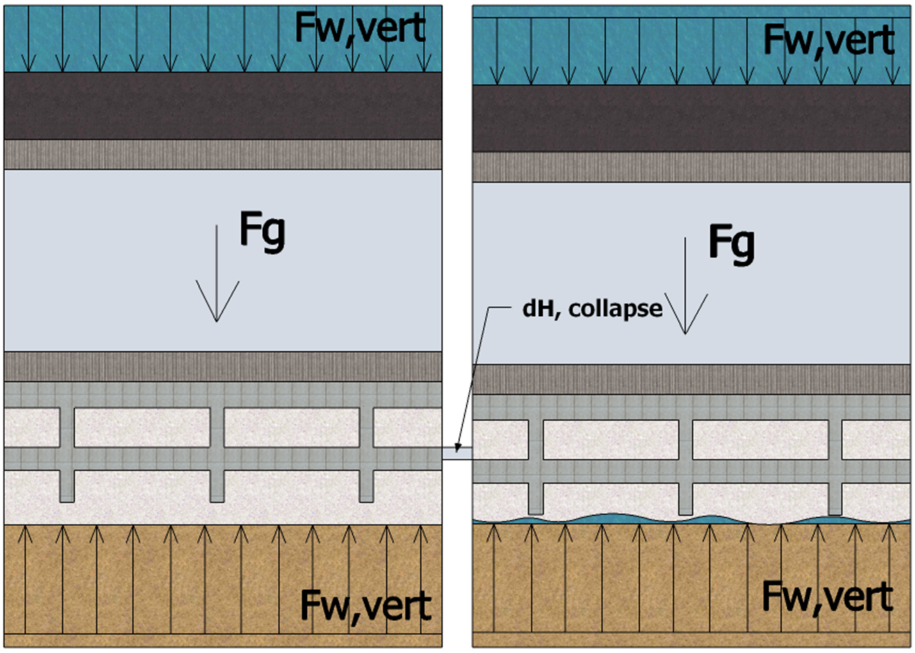


Figure 22: Settlement of element after collapse of bottom EPS layer, using ballast

<u>Collapse of EPS: Grout anchors</u>

This will not be the case when using grout anchors. Grout anchors are steel tension piles which are anchored by a grout body at the end. The steel piles lengthen or shorten when the load on the pile changes. When the EPS collapses, the buoyancy force gets less and the anchor pile shortens. This means that the total settlement does not equal the thickness of the collapsed layer but it equals the length the pile shortens. Which is:

$$E = \sigma/\varepsilon = (dF \cdot L\_0)/(A\_0 \cdot \Delta L)$$

In which:

E	=	Modulus of elasticity	[N/mm^2]
dF	=	Change in applied load	[N]
A0	=	Surface area steel pile	[mm^2]
ΔL	=	Change in length	[mm]
L0	=	Original length	[mm]

The piles go through the Flexbase floor and are approximately 15,00 meters long. For this calculation a GEWI grout anchor is considered with a maximum applied force of 771 kN. The surface area of the pile is 1.946 mm^2.

The c.t.c. distance of the piles is 3,6 x 4,12 meters. This means that the change in buoyancy force (collapse is 800mm) is 3,6 x 4,12 x 0,80 x 10 = 118,6 kN per pile. The modulus of elasticity of steel is 210.000 N/mm^2.

E	=	210.000	[N/mm^2]
dF	=	118,600	[N]
A0	=	1.946	[mm^2]
L0	=	15.000	[mm]

$$210.000 = (118.600 \cdot 15.000)/(1.946 \cdot \Delta L)$$

ΔL = 4,31 mm

The total settlement in the worst case scenario would be 4,31 mm which is almost negligible.

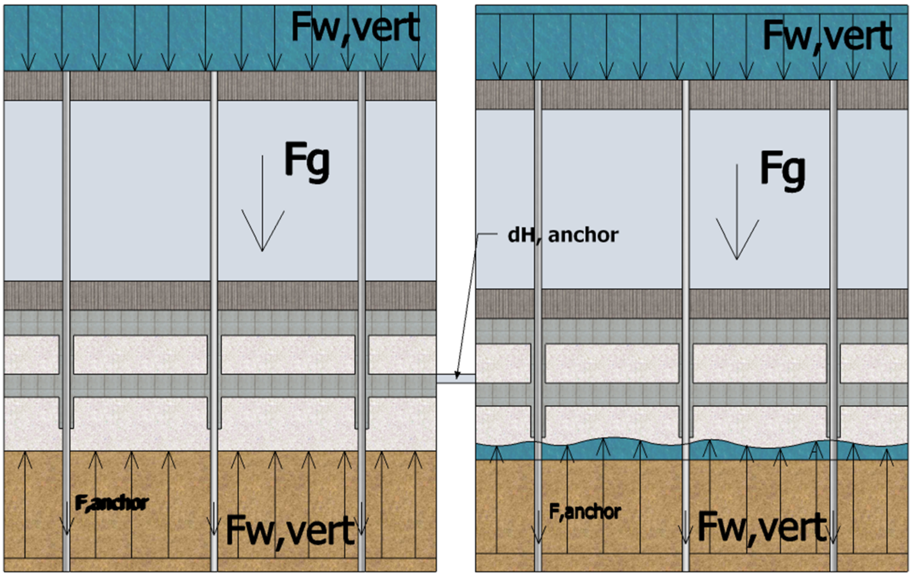


Figure 23: Settlement of element after collapse of bottom EPS layer, using grout anchors

Conclusion
EPS 250 could be used if grout anchors are used to keep the element immersed. It is important that the initial force on the grout anchors is more than 118,6 kN. In that case the anchor is still under tension after the EPS has collapsed. When the grout anchors are under pressure, this calculation becomes invalid because more factors have to be taken into account.

Note that this only happens in the worst case scenario. The chance that the EPS layer would completely collapse is very small. If the EPS only collapses locally the change in buoyancy force would also be divided over the anchors that are near that location, which means a smaller settlement.

See appendix I for the EPS choice

Normally the EPS has to be protected against oily materials and intrusion. The EPS gets coated on the outside. However, because the floor will be submerged, a coating will not be necessary. To recapitulate, the advantages of EPS are:

i. 100% recyclable;
ii. Watertight;
iii. Can take high pressure loads;
iv. Long life cycle (>50 years);
v. Relatively low costs;
vi. Easy to shape into any form.

Concrete
A choice has to be made for the type of concrete that is used for the beams and floors of the Flexbase floor. Concrete is available in many strength classes, however, concrete in higher strength classes is also more expensive. Here, the choice is also based on weight. Concrete with higher compressive and tensile strengths could make the structure lighter, but this would not necessarily be a benefit in this case. Therefore, concrete C28/35 is chosen, which is one of the most common types of concrete in construction and has been used for all the Flexbase floors up to now.

Tests with other types of concrete
Test have been conducted with self-compacting concrete and fibre reinforced concrete by Prinsen Waterbouw BV and Unidek. The self-compacting concrete has a low viscosity and isn´t ideal when pouring a floor on floating EPS. The concrete would flow to the lowest point and create a `pool´ of concrete. Therefore, concrete with a higher viscosity is recommended. The test with the fibre reinforced concrete showed that the concrete didn´t reach the suspected tension strength. It isn´t sure why, but experts think that the steel fibres do not sufficiently attach to the concrete because the formwork continually moves during hardening. This is another reason why traditional concrete and reinforcement are chosen. [Unidek, 2010]

Construction method
The next step is to develop the construction method, starting with the floor. There isn't much literature available on constructing a "Flexbase Heavy" floor, so to develop this method, interviews have been held with various experts in the field. These experts are:

i. Jan Willem Roël, General Manager at Flexbase;
ii. Björn Sinnema, Engineer at Advin BV;
iii. Jan Dirk Hartog, Director at Prinsen Waterbouw BV;
iv. Jasper Schilder, Head Designdepartement at Dura Vermeer Beton en Waterbouw BV.

To get sufficient input for the structural calculations and the cost estimation, not only the used materials will be reviewed, but also the planning and equipment will play a key role. The following paragraphs will describe the construction method.

Construction site
Before the construction of the floor can begin, there has to be a construction site. This site should have enough space to place a temporary office/cafeteria, park the cars of the workers, park trucks for unloading materials and store materials and equipment. Cars, trucks and boats should be able to reach the construction site to unload materials and/or equipment.

The figure below shows the location of the construction site. A temporary platform will be constructed in between two existing piers (shown as a brown square). The platform will be 50 meters wide and 85 meters long. This should create enough space for storage and the temporary office/cafeteria. At the quay wall, parallel to the Prins Hendrikkade, cars and trucks could be parked.

Constructing the parking garage will not occur at the final location, but at the location east of it (shown as a pink rectangle in figure 24). In this way, excavation of the bed, constructing the foundation and constructing the entrances and exits for cars and pedestrians can be done at the same time.

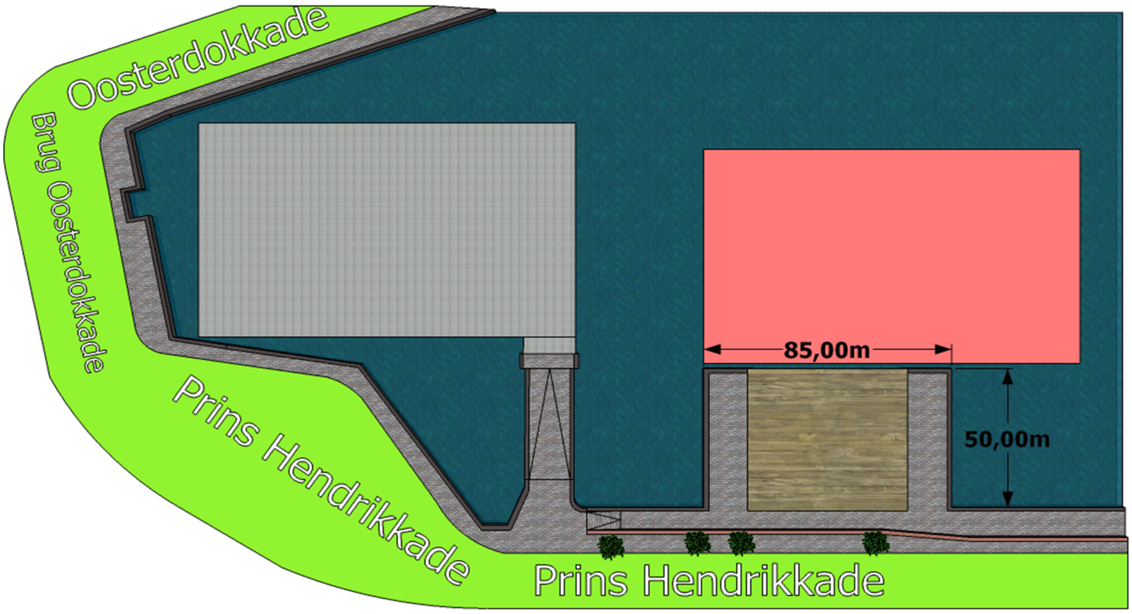


Figure 24: Location of the construction site

Constructing the EPS floor

The thickness of the EPS floor depends on the needed freeboard during construction and the combined weight of the concrete floors, beams and the outer walls of the parking garage. This is because of the fact that the freeboard of the floor should be sufficient until the outer walls of the parking garage are ready. After that, the construction is similar to a large concrete caisson and the freeboard will probably be sufficient. Calculations on the needed thickness of the EPS floor are shown in appendix B.

Figure 25 shows that the total thickness of the EPS Heavy floor is 2,55 meters (1,00m + 0,75m + 0,80m). This means a total of 26.520 m3 EPS (2,55m * 80m * 130m) is needed.

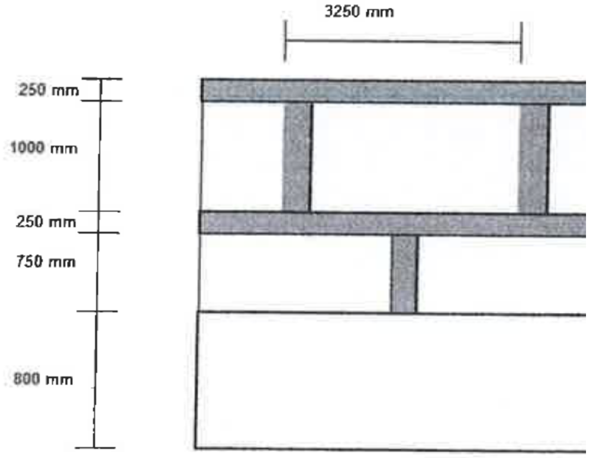


Figure 25: Dimensions Flexbase Heavy floor

<u>Constructing the first layer</u>

Constructing the Flexbase floor starts with the first layer of EPS. This layer consist out of EPS blocks that are four meters long, one meter wide and 20 centimetres high. The blocks are connected with a so called scarf joint (in Dutch: 'haaklas'). At the joints, the blocks are also glued together. During the construction of this first layer, workers walk on the EPS floor. To keep the joints from disconnecting, the workers walk on wooden planks.

The photos below show the process of constructing the first EPS layer.

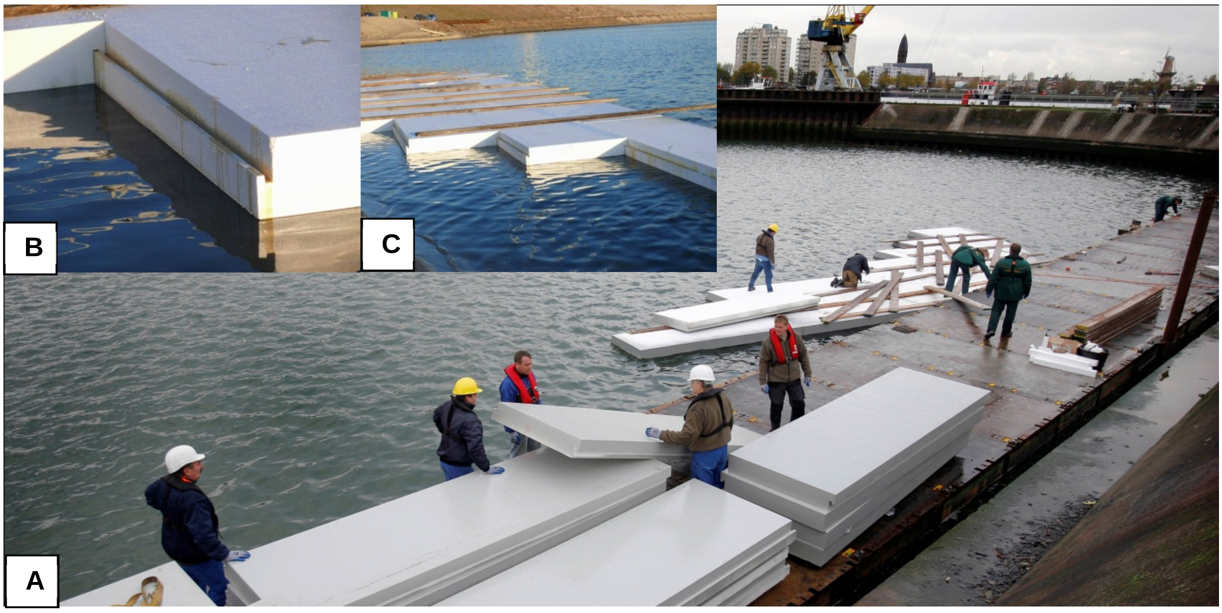


Figure 26: A: Constructing the first EPS layer B: The scarf joint C: Wooden planks on the joints [www.flexbase.eu]

<u>Constructing the next three layers</u>

The next three layers are 20 centimetres thick and are glued on top of the first layer. This glue only gives a perfect bond under ideal weather conditions and would not always be sufficient in the past. Therefore, a test has been conducted by Prinsen Waterbouw with a layer of glass fibre foil between the EPS blocks for extra rigidity and a better bond. The bond between glass fibre foil and EPS with glue is much better than the bond between EPS and EPS with glue. As an extra measure, the EPS blocks are held together with plastic pins. This test was done with a Flexbase floor of 16 square meters and showed improvement in the rigidity and strength of the floor. Therefore, this configuration is recommended for future projects. [Unidek, 2010]

Figure 27 shows the configuration of the first three layers with the glass fibre foil and the plastic pins. The plastic pins are 25 centimetres long and one is shown in this figure. These pins are drilled in with a simple hand drill and are placed 50 centimetres apart.

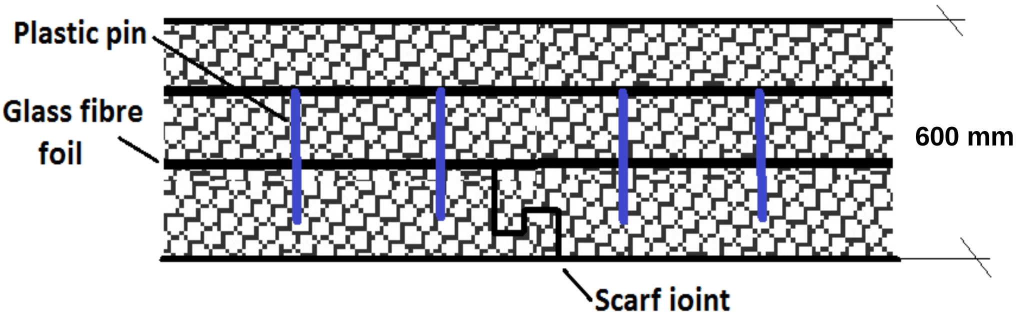


Figure 27: Left: Configuration of the first three EPS layers   Right: Plastic pin

Constructing the formwork

A layer of EPS blocks form the formwork for the first layer of beams and the first floor. This layer is 75 centimetres thick and consist out of one layer of EPS 60 blocks. Because the beams all have the same centre to centre distance, the blocks will all have the same size. Therefore, these blocks could easily be prefabricated. Because the blocks are glued together, the exact c.t.c. distance could differ somewhat from the preferred distance. Therefore, some on-site fabrication will always be needed. This layer will be glued to the third layer.

The c.t.c distance of the beams will be 4,12 meters in the “drive” direction and 3,60 meters in the other direction. These distances are half the c.t.c. distance of the columns in the parking garage. This done to create an optimal transmission of forces between the columns and the floor. This means that under every column, two beams will cross. The floor will have 23 beams in the drive direction and 33 in the other direction. Figure 28 shows one segment of the layout of the beams and EPS block. The eventual floor will consist out of 704 of these segments. Every segment needs four blocks of EPS. The dimensions of these blocks are 3,88 x 0,84 x 0,75 (l x w x h). These dimensions aren’t ideal, but it should be possible to keep the costs low because of the large quantity that is needed (704 x 4 = 2.816 blocks!).

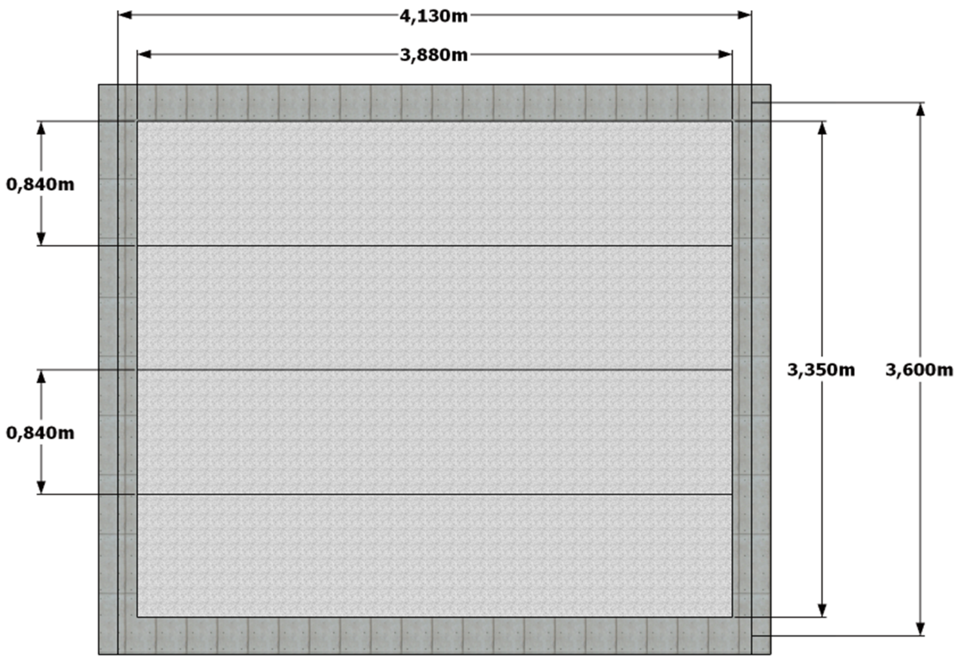


Figure 28: Layout of the beam grid

Figure 29: **A: Unloading of the EPS blocks** **B: Side-view of the EPS layers**
C: Top-view EPS formwork **D: Workers constructing the formwork**

The pictures above show the finished formwork of the Rotterdam Pavilion. For that project, concrete side elements acted as formwork on the sides of the platform. For this floor however, temporary formwork is used, so called ´omega profiles´.

Constructing the first beam grid

The first beam grid provides the initial rigidity that is needed to pour the first concrete floor. Because this rigidity isn't present when the beams are poured, the beams will be poured in two stages to minimize the load on the EPS floor. When the bottom half of the beams has hardened, the second half can be poured. Before the beams are casted, reinforcement is placed.

<u>Placing the reinforcement cages</u>

The reinforcement cages are constructed by steel fixers at the construction site. These cages can easily be placed and connected in the recesses. The finished beams will have protruding reinforcement bars that will be connected to the reinforcement of the floor. The reinforcement cages can be prefabricated.

<u>Pouring the concrete beams</u>

Pouring the beams isn't as straightforward as it sounds. Figure 30 shows why. When a uniform load is placed on top of the EPS floor, it will not distort. But when a concentrated load acts on the EPS floor it will. In this stage it means that when the beams begin to harden, the load has to be uniform over the entire floor or else the hardened beams will not be straight. When pouring the floor on top of the beams, these concentrated loads can bend the beams and even create large cracks.

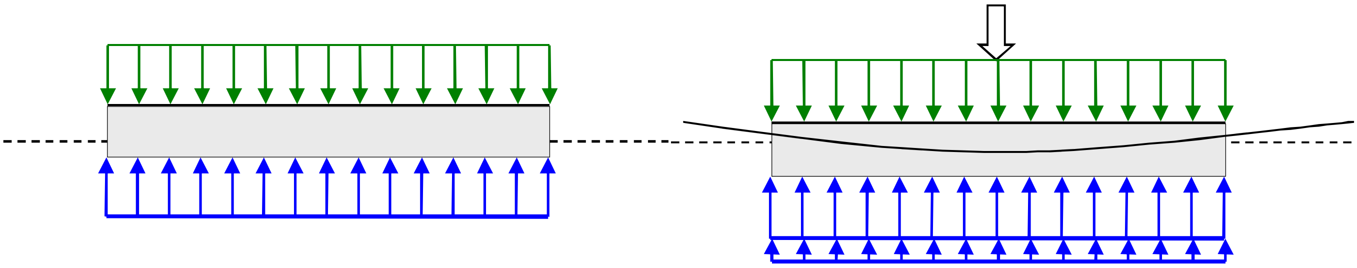


Figure 30: Loads acting on an EPS floor

A significant and important part of engineering Flexbase floors is creating pouring schemes. In this stage it is important that the load is uniform when the bottom half of the beams is poured. This also means that the concrete can't begin to harden before all the beams are poured! Therefore, the hardening process will have to be slowed down with concrete retarders. These retarders are usually made from phosphates and decelerate the chemical reaction between cement and water. This gives the concrete workers 8-12 hours to pour all the beams. A side effect of these retarders is that it usually makes the concrete stronger [www.cementenbeton.nl]. The entire beam grid consist out of approximately 1.000 m^3 of concrete. This means that they will have to pour 500 m^3 in this period of time. [Stubeco, 2005]

When we take a look at the distribution of the downward forces in the final situation, when the beams are poured, we see another problem. This is shown in figure 31 which shows the displacements of the Flexbase floor on water. The downward force from the concrete beams is larger in the middle and is the smallest at the corners. Because there are no beams at the sides of the platform, there is relatively more concrete per square metre at the centre of the floating floor. This means that the corners and the sides will curve up and the beams will not harden straight. This problem is solved with a simple solution. Before the beams are poured, extra weight is placed at the sides and corners. This weight keeps the sides and corners from curving up in the final situation. How heavy these weights will have to be, will be calculated and will be part of the pouring scheme. This method was also used during the construction of the 'Paviljoen' in Rotterdam. Note that these measures only reduce the distortions for 50% in the best case scenario.

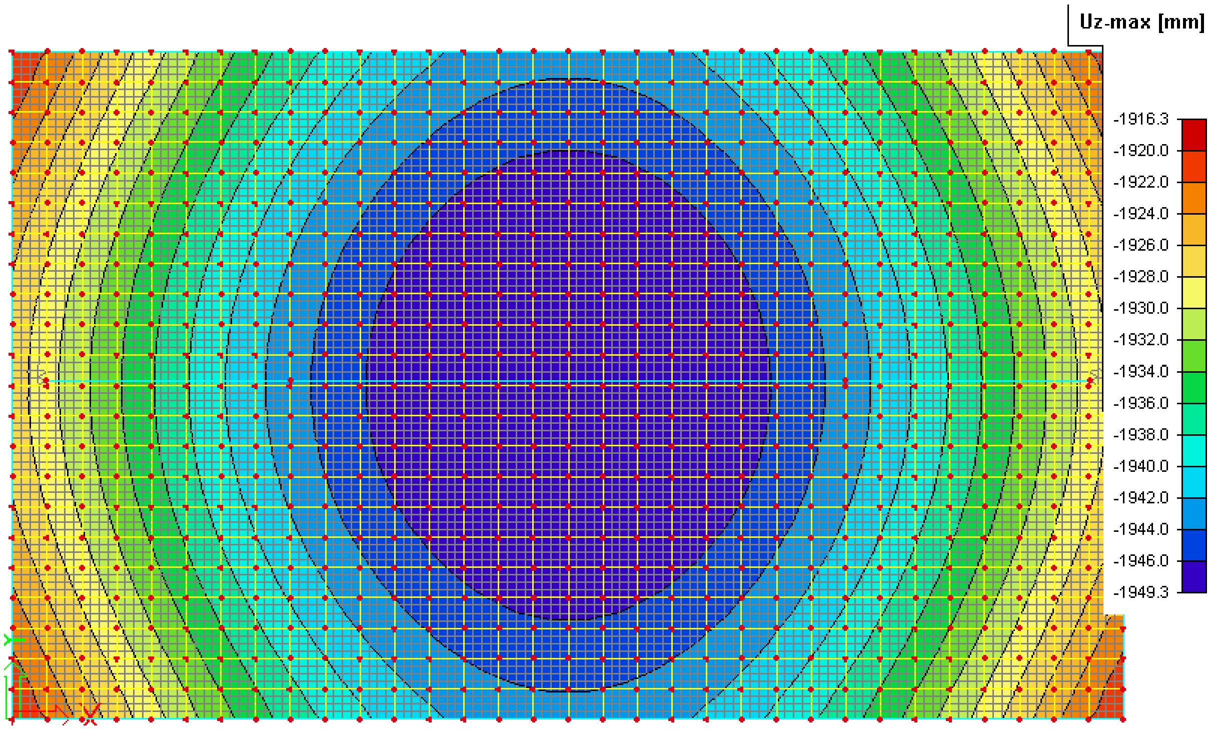


Figure 31: Displacements of Flexbase floor when finished

When the bottom half of the beam grid is hardened (after 28 days) the top half can be poured. The first half provides the needed rigidity. This pouring scheme will have to be followed carefully to prevent distortion of the beam grid.

Figure 32: Concrete pump pouring the beam grid

Figure 32 shows the beam grid while it is being poured. After this stage the floor can be poured. This picture shows that the beams are poured using a concrete pump. These pumps can maximally reach 40 meters. This would be enough if the floating floor was surrounded by land, but it is not. It is only adjoining land at one side. Therefore, one pump will stand on a pontoon at the other side. Concrete will have to be pumped to this pontoon through pipes.

Constructing the first floor

The beam grid provides enough rigidity to pour the concrete floor. The reinforcement of the floor is connected to the reinforcement of the beams and the floor can be poured. It is still very important to check the pouring scheme on beforehand. The beams should be checked on distortion, forming of cracks etc. The floor will be poured in one go and it consists of approximately 3.000 m3 concrete. Figure 33 shows workers that are pouring the concrete floor.

Figure 33: Pouring the concrete floor

Constructing the second beam grid and floor

The second beam grid and floor are constructed in the same way as the first. The only difference is that the beams can now be poured in one go because the first beam grid and floor provide enough rigidity and strength to pour without an elaborate pouring scheme. The pouring scheme only needs to prevent the floor from tilting too much.

The top floor will be connected to the walls and columns and will therefore have protruding reinforcement bars. At the end of the construction stage an extra floor will be poured to create a smooth finish. This floor will be the final floor for the parking garage and will have inclinations and gutters to get rid of water.

7.2 Constructing the walls and columns

The outer walls of the bottom floor of the parking garage will be constructed first. These walls are 2,60 meters high. The Flexbase floor will have to sustain the needed freeboard until the outer wall is finished. When it is finished, the freeboard will most likely be sufficient until the end of the construction stage.

Figure 34 shows a detail of the connection between the Flexbase floor and the outer wall. The wall is placed 1,50 meters from the edge of the floor. This is done to create space for the workers when they are constructing the wall. It is also necessary to place the formwork.

The order in which the walls are poured is very important in this stage. The walls are all placed on the edge of the floor and therefore the floor will bend. Calculations will have to show what the dimension of the floor will have to be to cope with these forces.

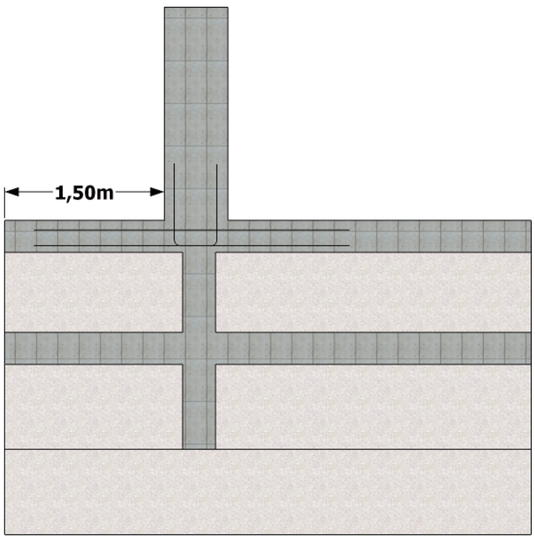


Figure 34: Detail of wall-floor connection

Figure 35: Panel formwork for column [PERI BV]

Figure 35 shows the formwork for the columns and figure 36 shows the parking garage after the construction of the inner and outer walls, the columns and the ramps. The next stage will be the construction of the intermediate floor.

The construction of the columns and walls of the second floor will be done in the same manner. The only difference is that the formwork of the walls of the second floor will have to be anchored to the walls of the first floor, because they can’t be placed on the edge of the Flexbase floor (the floor will already be submerged).

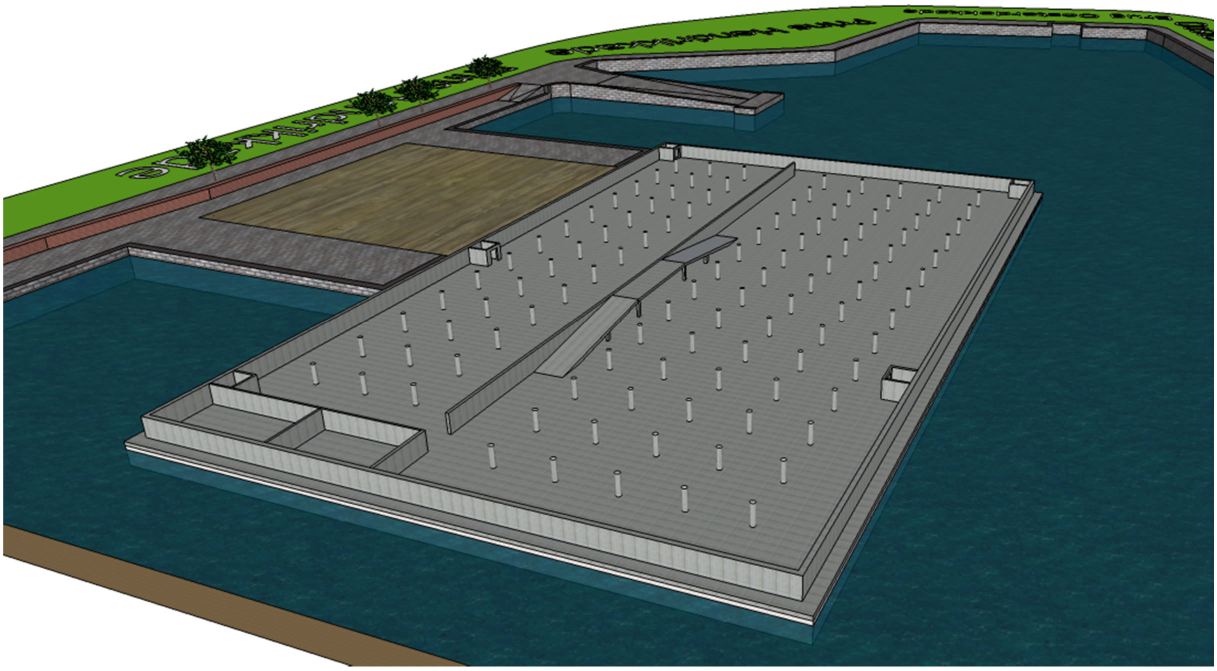


Figure 36: Structure after construction of walls and columns

7.3 Constructing the second floor and roof

The next step is to construct the intermediate floor. The roof and this floor will be constructed in situ, using panel formwork (see figure 37). The loads on the floor and roof will be transferred to the columns. Figure 38 shows four ways to connect a floor slab to the columns. The most popular is type "c" because of the ease of execution and low labour costs. However, when the loads on the floor and roof become too high or the crack widths in the roof become to large, one of the other types will have to be used. Type "d" (drop panel) increases the bearing capacity of the slab at the column, where the highest shear stresses occur. When this still isn't enough, type "a" or "b" could be chosen. These types transfer the forces from the floor to the beams, which transfer the loads to the columns. The formwork for these types is much more complicated and therefore more expensive.

Figure 37: Panel formwork for the roof/floor [PERI BV]

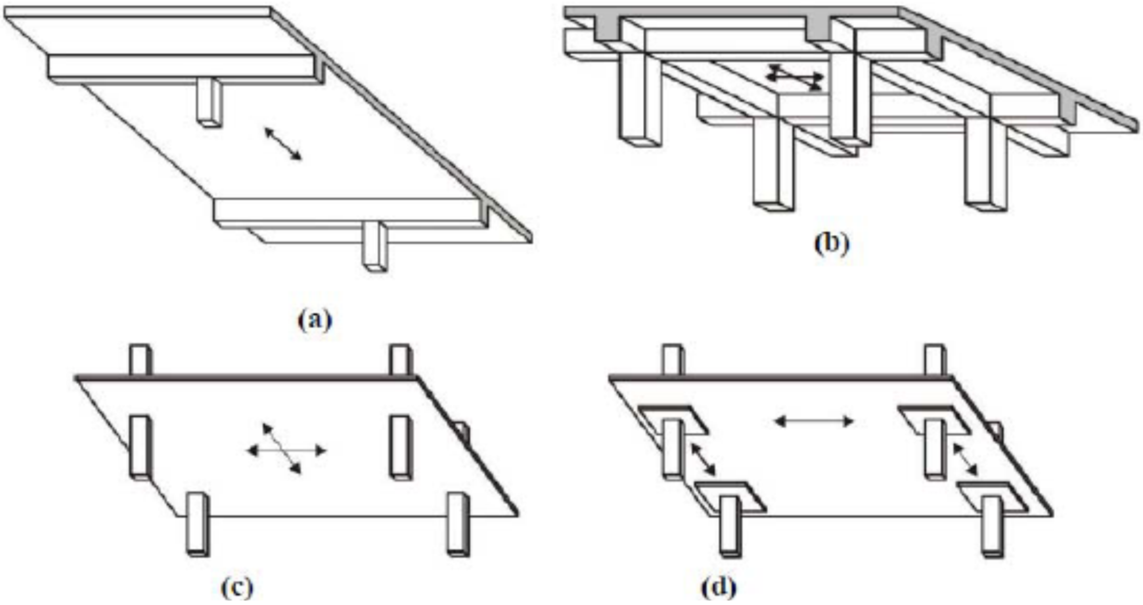


Figure 38: Connections between floor slab and column. A) Slab with supporting beams transferring load in one principal direction B) Slab with supporting beams transferring load in two principal directions C) Flat slab D) Drop Panel Flat Slab [CT3150, Concrete Structures 2]

7.4 Constructing the entrances/exits

The parking garage will have three entrances for pedestrians and one for cars and pedestrians. These entrances will be constructed separately. Figure 39 shows the finished garage element as it is constructed on top of the Flexbase floor. The entrance for cars and pedestrians is shown at the bottom left corner and the three entrance shafts for pedestrians are constructed on top of the roof. When the garage element is submerged, the top of the entrance shafts will stay above the water surface. The entrance for cars and pedestrians will submerge and be coupled to the rest of the entrance tunnel under water. The garage element would be too heavy on one side (and therefore incline too much during construction) if the entrance for cars would also be constructed in a way that it could be coupled above water. The construction of these entrances will be explained in the next few paragraphs.

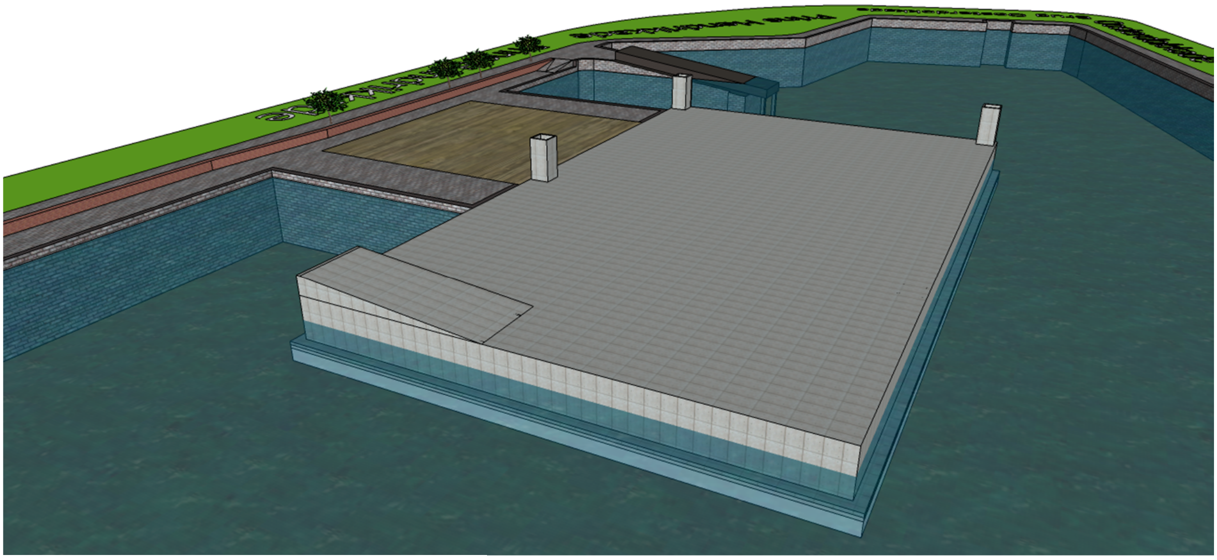


Figure 39: Completed garage element

Entrance cars and pedestrians

The entrance for cars and pedestrians makes use of the already existing pier at the Prins Hendrikkade. Figure 41 shows the connection between this entrance and the garage element (note that in the final situation the parking garage will be underground). The tunnel (dark grey part in the figure) will be constructed beforehand. This will be done by creating a construction pit with sheet piles and underwater concrete around the pier. Because only a small part of the tunnel is underwater, the construction pit will also be relatively small (approximately 20 by 40 meters). The tunnel will be closed off by a temporary bulkhead, this will also be done with the garage element.

When the garage element is immersed, the two closed-off ends are joined together. The alignment of the two parts is very important for the water tightness of the entrance tunnel. The garage element will have to be checked on settlement because the soil under the element will be unloaded (due to excavation) and loaded (by the garage element). This could cause unwanted settlement.

The entrance tunnel and garage element will be joined and sealed by a Gina Gasket (named after the Italian actress Gina Lollobrigida). This profile is shown in figure 40. The initial seal is created by the compression of the rubber gasket. Jacks will pull the element and tunnel together during immersion. After that, the area between the two elements is drained and the temporary bulkhead is removed and the connection can be made permanent from the inside.

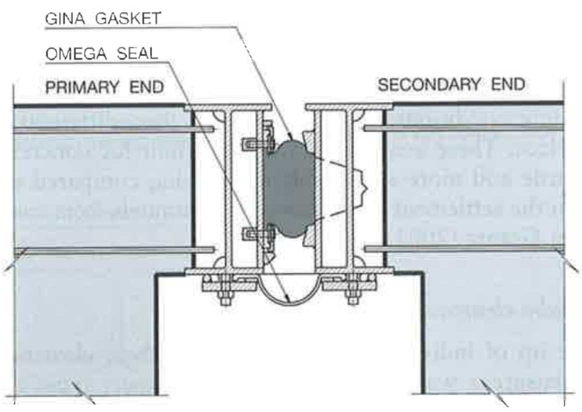


Figure 40: Gina Gasket

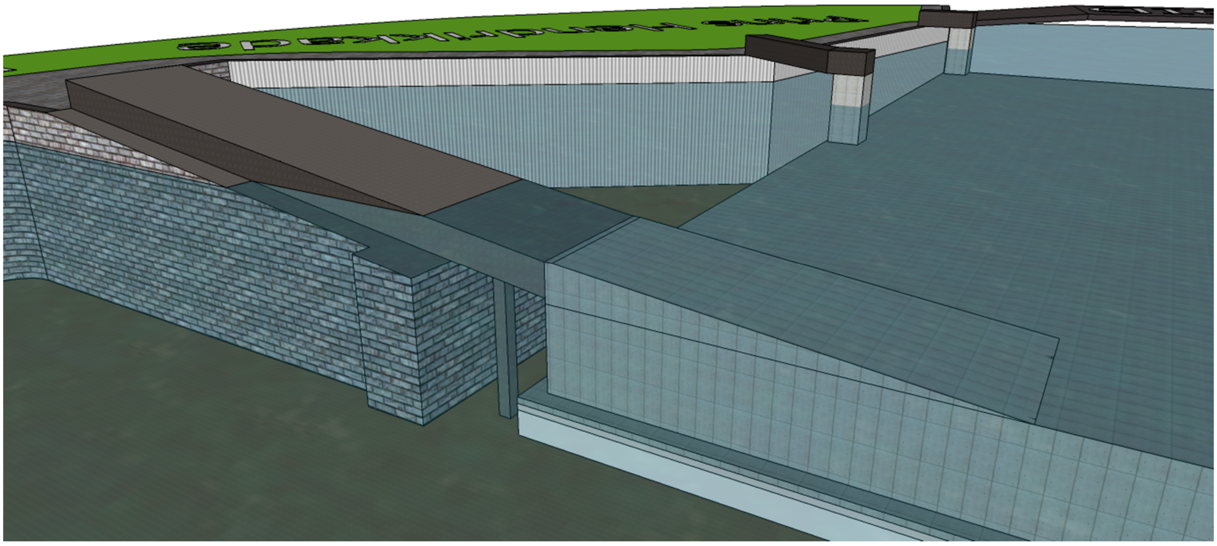


Figure 41: Connection between the entrance tunnel and garage element

Entrances for pedestrians

The entrance shafts protrude the water level when the garage element is immersed. This is done to avoid connections under water. The extra weight shouldn't cause problems to the stability of the element because the shafts are divided over the element and the weight is relatively small.

Figure 42 shows the three entrances. This figure also clearly shows the Oosterdok Island Bridge (OIB). The dark grey elements are constructed after the garage element is immersed. The dimensions of these elements are approximately 12,00 x 3,00 x 3,00 meters and could therefore easily be prefabricated.

The advantages of this method are:

i. No connections under water;
ii. No need for a construction pit (extra costs);
iii. No construction needed near quay wall (risks of settlement);
iv. Relatively fast construction method (relatively little hindrance for surrounding area).

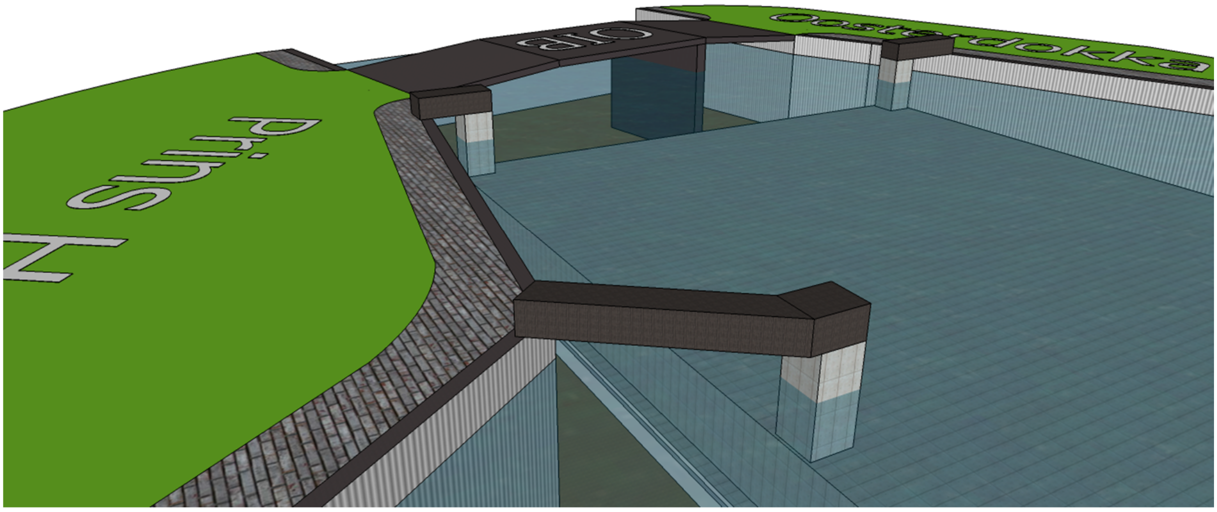


Figure 42: Entrances for pedestrians

7.5 Constructing the foundation

The foundation for the garage element will have to be constructed before it is submerged. Figure 43 shows the garage element in the final stage. It shows that the parking garage will be underwater and underground. The soil, a total of 137.750m3, will have to be excavated first. Figure 42 shows that the parking garage is situated close to the quay wall near the entrances for pedestrians. Before the construction of the Oosterdok Island Bridge began, an anchored wall was constructed consisting out of 13,00 meter long AZ 18 sheet piles. As well on the side of the Prins Hendrikkade as on the side of the Oosterdok Island. In front of this wall, the permanent quay wall is constructed. This wall consists of a L-wall (bottom at 0,00m NAP) on prefab piles (350 x 350 mm, bottom at -20,00m NAP). The stability of the quay wall during excavation is examined in appendix G.

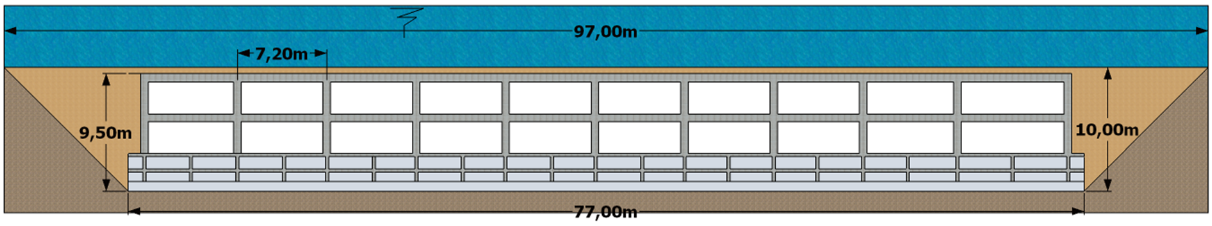


Figure 43: Cross-section of parking garage, operation stage

Soil layers

The bottom of the parking garage is at approximately -15,00m NAP. The soil at this depth consists of sand. This first sand layer starts at -12,00m NAP and ends at -16,00m NAP. The soil layer from -4,00m NAP to -12,00m NAP, that will be excavated, consists of clay layers and layers of silty clay. The second sand layer starts at -18,00m NAP and ends at -30,00m NAP. The layer between these two layers consists of clay and sand layers. The soil beneath -30,00m NAP consists of clay.

See appendix A for soil-drilling tests and soil parameters

Foundation

When the garage element is submerged, it will be placed on foundation pads. These pads will rest directly on the sand layer under the parking garage. The concrete pads are prefabricated and are 1,00 meter thick, 2,00 meters long and 2,00 meters wide. This is a traditional foundation method for tunnel elements. These pads are placed in a grid of approximately 10,00 by 10,00 meters. This means that the garage element will rest on 104 pads.

The foundation could also be constructed out of a levelled gravel bed. However, because the parking garage has to be perfectly level, a foundation of concrete pads is chosen. This is far more accurate than a gravel bed.

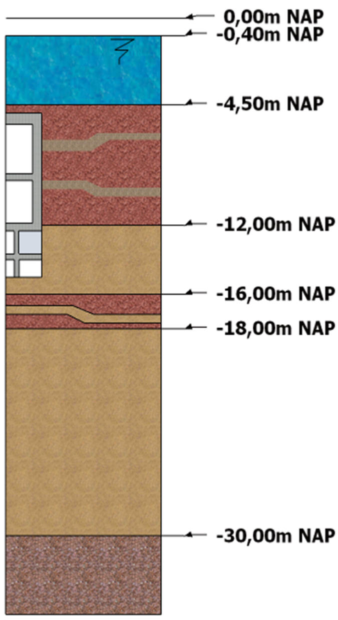


Figure 43: Soil composition

7.6 Immersing the parking garage

When the garage element, the foundation and the entrance tunnel are constructed, the garage element will be immersed. This chapter describe two stages, namely immersing and positioning the garage element and securing the element to the bed.

Immersing and positioning

The garage element will first be transported to the final location. In this case the element will only travel 200 meters. This could be done by using tugboats. The garage element will be connected to cables from cranes at the final location. These cranes can be positioned at the quay walls surrounding the Oosterdok. These cranes will position the element.

Water tanks will be placed inside the garage element. When the element is position above the final location, these water tanks will be filled. Approximately 27.500 m3 of water will be needed to immerse the garage element (see appendix B). This means that 66% of the floor space will be needed to place water tanks (two meters high). The Oosterdok is an ideal location to immerse the element, because flow velocities and wave action are minimal and there is no tide.

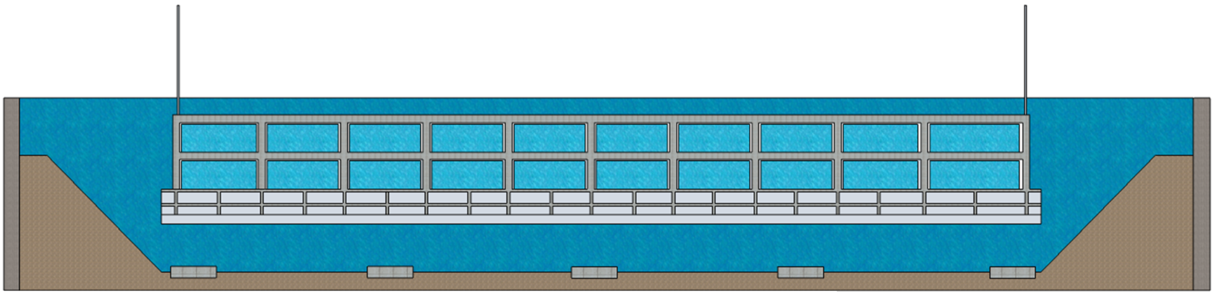


Figure 44: Immersion of the garage element by cables

Ballasting the element

To keep the element at the trench bottom, the water in the element will have to be replaced by ballast. There are a few options:

i. Ballast (concrete) on the roof and/or floors;
ii. Ballast (sand) on the Flexbase floor;
iii. Grout Anchors.

It is also possible to use a combination of these options. Chapter 7.1 and appendix I showed that grout anchors are the best choice for this situations, because it could save up to €1.550.000,- in costs for EPS.

<u>Grout anchors</u>

Grout anchors for tunnel elements and garage elements have already been reviewed in past studies. One of the solutions was to place the anchors through the columns of the element and secure them to the element on the roof. This method is reviewed in appendix J.

Appendix J shows the calculations on grout anchors The tension capacity of a single pile in a group for this situation is 612 kN. There are 234 columns and therefore 234 anchors could be placed through the these columns. The total tension capacity would be:

234 * 621 = **143.208 kN** < 249.523 kN (buoyancy force in operation stage)

An extra 106.315 kN of downward force would be needed to keep the element immersed.

There are a few options to create more downward force:

i. Add mass, extra concrete;
ii. Add more grout anchors;
iii. Use anchors with a larger capacity.

The first option would result in ballast concrete on the roof or inside the garage element. Beside adding mass, it would also add volume. This means that the net extra downward force would be 15kN/m^3. Keeping in mind that ballast concrete costs approximately 80-90 €/m^3 it would cost 6 €/kN.

The second option is to add extra grout anchors, but there aren't more columns to lead anchors through. This means that the anchors should be placed inside the garage element. This option is discussed below.

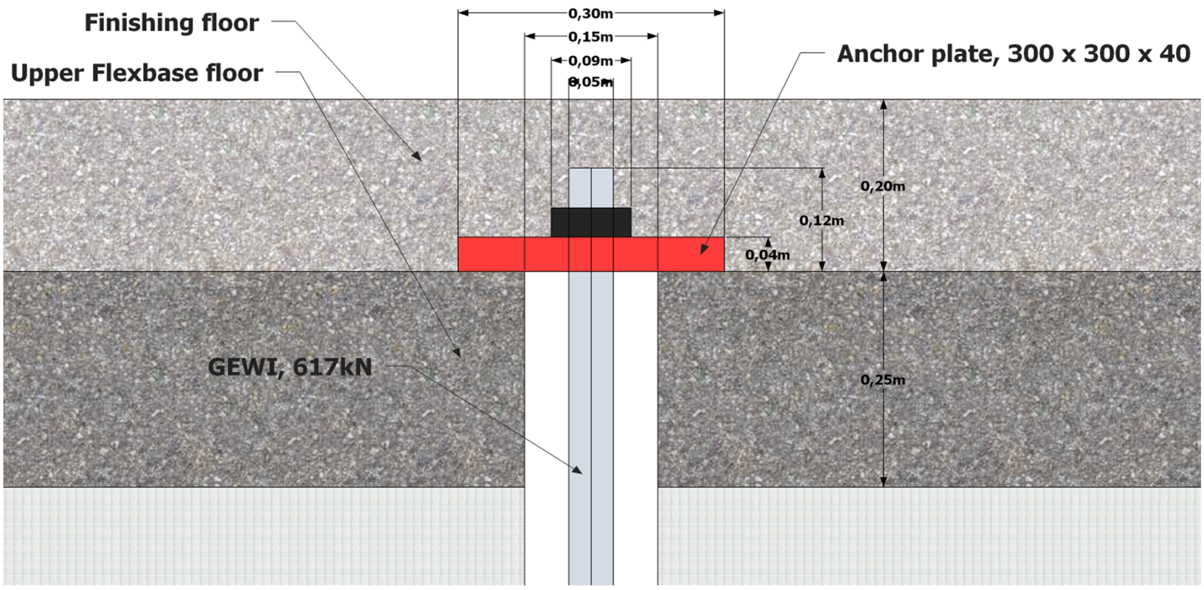
Drilling the anchors from inside the garage element

When the garage element is immersed and at the bottom of the Oosterdok, small drilling equipment could drill holes in the Flexbase floor and drill the anchors into the soil. This option is possible using a cuff but labour-intensive. Because of the restricted height, the anchors consist out of multiple pieces of 1,5 meters long. The advantage is that the anchors can be shorter because they are connected to the floor and not the roof. This saves approximately 10 meters per anchor.

To decide whether to drill anchors from the inside, add extra mass or use anchors with a larger capacity, the costs per alternative will be discussed. A company that delivers and places anchors has provided the needed information. Appendix J shows the costs per 100 anchors, including materials and labour.

Anchors of 25m^1 with a capacity of 1325kN are compared with 15m^1 long anchors with a capacity of 617kN that are drilled from inside the garage element. This comparison shows that the costs are predominately determined by the length and diameter of the anchor. Therefore, the anchors through the columns are more expensive per kN than the shorter anchors that are drilled from inside the element.

The 15m^1 long anchors cost **2,17 €/kN** and are less expensive than using ballast concrete or leading the anchors through the columns. The picture below shows a detail of the anchor head.



Detail of anchor head

Conclusion

All the anchors will be drilled from inside the garage element when the element is immersed. This does mean that during the immersion process, the ballast tanks must leave enough space to manoeuvre the drilling equipment.

The anchors will be placed next to the intersections of the beams, as close as possible to ensure the most favourable transmission of forces. The beam grid consist out of 31 x 21 beams. This means a total of 641 anchors and a total capacity of 395.497 kN.

8. Loads on structure

Now that all the steps of the construction stage are reviewed, the loads that act on the structure can be reviewed. The loads are determined using the building codes NEN 6702 and EC 1991-1. This will help to form the input for the structural analysis.

8.1 Load factors

The Eurocode distinguishes two types of limit states, namely:

1. ULS, Ultimate limit state;
2. SLS; Serviceability limit state.

The Eurocode recommends load factors of 1,0 in the SLS and 1,2 for unwanted loads and 0,9 for favourable loads in the ULS. Like Maarten Koekoek mentioned in his thesis on floating structures [M. Koekoek, 2010], this leads to a very high needed freeboard when applying the ULS load factors. Therefore, the SLS load factors will be used when calculating the freeboard and draught. The ULS load factors will only be used for failure of the structure and buoyancy calculations in the operation stage.

8.2 Weight and imposed loads

Under these loads, self-weight and imposed loads are considered. The dead weight causes a load on the structure and should be classified as a permanent fixed action. This load is also important during the construction stage, because it is directly linked with the draught and freeboard of the structure.

The imposed loads will only play a major role in the operation stage. The imposed loads are traffic, parked cars, pedestrians etc. The Eurocode specifies several categories. Parking garages for small vehicles are specified as category F. The characteristic values are shown in table 1. In this table qk corresponds with the uniform load on the floor and Qk with the load per axle. Note that qk is used for general effects and Qk for local effects.

Categories of traffic areas	q_k [kN/m^2]	Q_k [kN]
Category F Gross vehicle weight: ≤ 30 kN	q_k	Q_k
Category G 30 kN < gross vehicle weight ≤ 160 kN	5,0	Q_k

NOTE 1 For category F. q_k may be selected within the range 1,5 to <u>2,5</u> kN/m^2 and Q_k may be selected within the range 10 to <u>20</u> kN.

NOTE 2 For category G, Q_k may be selected within the range 40 to <u>90</u> kN.

NOTE 3 Where a range of values are given in Notes 1 & 2, the value may be set by the National annex.
The recommended values are underlined.

Table 1: Imposed loads on garages [EC 1991]

The barriers/walls in the parking garage should also be designed to withstand an impact force caused by a car. The horizontal characteristic force *F* (in kN), normal to and uniformly distributed over any length of 1,5 m of a barrier for a car park, required to withstand the impact of a vehicle is given by:

$$F = 0{,}5mv2 / (\delta c + \delta b)$$

Where :

m =	gross mass of the vehicle	[kN]
v =	velocity of the vehicle normal to the barrier	[m/s]
δc =	deformations of the vehicle	[m]
δ*b* =	deformations of the barrier	[m]

According to the Eurocode 1991, the following parameters should be used:

m = 1500 kg
v = 4,5 m/s
δc = 100 mm

Since the wall is rigid, δb is zero and this gives F is 150 kN.

8.3 Wind

Wind loads on the structure could lead to unwanted tilt of the floating structure. Wind loads will only act on the structure during the construction stage and will be relatively low because of the sheltered construction site and relatively low walls. The wind load on the structure will be calculated according to the Eurocode 1991-1. Appendix K3 shows that tilting due to wind forces isn´t significant (1:1721).

Wind will also induce wind waves. This will be reviewed in the next chapter.

See appendix C for wind force calculations

8.4 Water

The loads caused by water are divided into the following categories:

i. Hydrostatic water pressure;
ii. Waves;
iii. Current and drag.

Hydrostatic water pressure

When calculating the draught, buoyancy and freeboard of the structure, use is made of the hydrostatic water pressure principle. This principle can only be used in still standing water. Because the water is relatively calm in every stage, this principle can be used.

The hydrostatic pressure in any given point under water follows from the formula:

$$p = \rho_w \, g \, h$$

in which:

p =	water pressure	[kN/m^2]
h =	pressure head	[m]
g =	acceleration due to gravity	[m/s^2]
ρ_w =	density of water	[kN/m^3]

<u>Freeboard / draught / buoyancy</u>

The freeboard, draught and buoyancy follow directly from this principle. The freeboard, draught and buoyancy force have been calculated for several stages in appendix B. Without going further into detail, the figure and formulas below describe the principle.

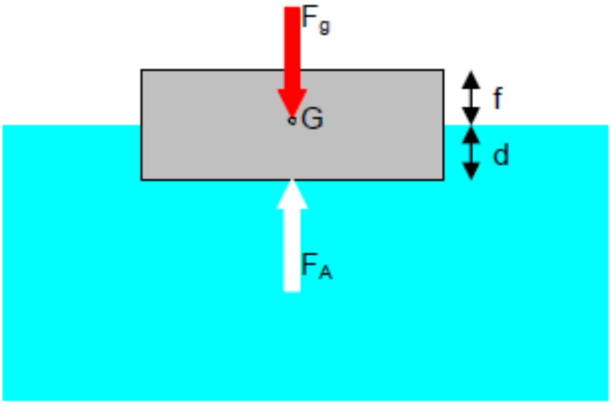


Figure 45: Principle of buoyancy

Fg = Fa
Fg = Gravity force [kN]
Fa = Buoyancy force [kN]

G = Centre of gravity

d = Draught [m]
d = Fg / (10 * A)
A = Surface area bottom [m^2]

Fa = 10 * V
V = Immersed volume

f = Freeboard [m]
f = h – d

Tilt

Because the entire structure is constructed on water, tilt is a very important factor during the construction stage. If an eccentric vertical or horizontal load acts on the floating structure, it will cause tilting. This could, for instance, cause unwanted flow of concrete during pouring. This means that all the loads should be evenly distributed over the structure during construction. Therefore, a pouring scheme will be made to make sure the tilt stays within certain limits.

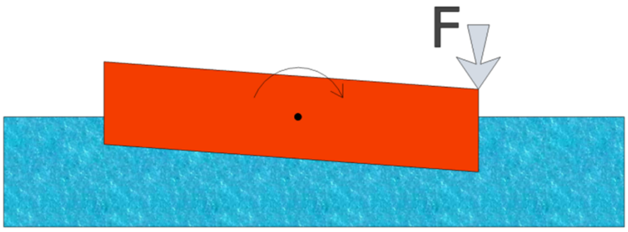


Figure 46: Principle of tilt

Tilt also means that the draught will increase, and the freeboard will decrease, on one side of the structure. The influence of tilt on the draught will have to be calculated. This will also be part of the pouring scheme. This scheme will be made using the engineering program SCIA. Because the structure is only supported by water in the construction stage, this will be modelled by supporting the structure with a continuous elastic foundation.

Static stability

For large floating element like the parking garage, the static stability will have to be calculated. When the element is unstable, tilting can be initiated during transport by external forces like mooring forces, wave motion etc. This rotation, caused by the external forces, should be corrected by a righting moment that will return the element to its original position.

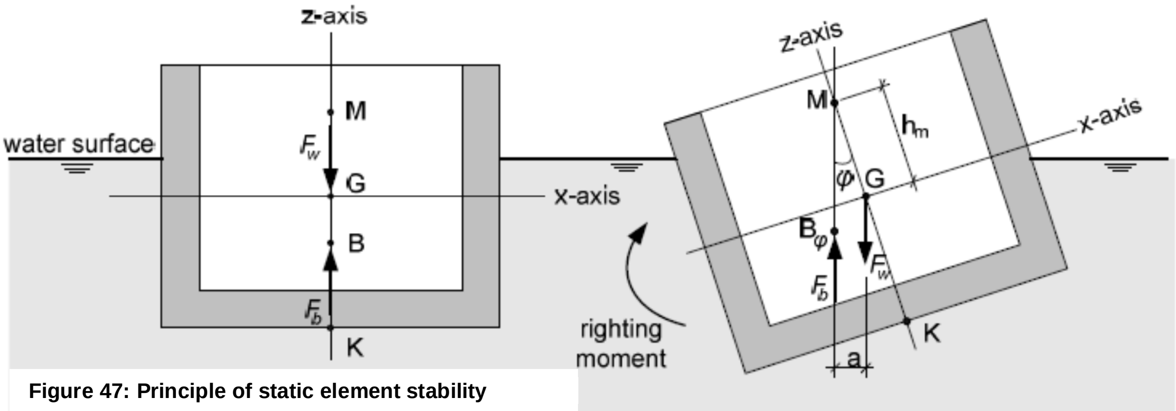


Figure 47: Principle of static element stability

Figure 47 shows this principle, where the left side shows the element in its original position and the right side the element in a tilted position. Further notations are:

B =	Centre of buoyancy	[m]
M =	Meta centre	[m]
G =	Centre of gravity of the element	[m]

The stability of a floating structure is determined by the distance between M and G. A larger distance means a structure that is more stable. From this we can also conclude that a low centre of gravity is favourable. Also, a relatively large width of the structure, with respect to the height, makes the structure more stable. The static stability is calculated in appendix D. This shows that the element is statically stable.

See appendix D for static stability calculations

Waves

During the construction stage, horizontal and vertical wave loads will act on the structure. Waves will create bending moments as is shown in figure 48. This will not create large problems when the structure is finished, but during construction it does. Especially when pouring the first beam grid or floor this forms a large risk! Waves could cause the concrete to deform and crack. Two types of waves can be distinguished in the Oosterdok, namely wind waves and ship waves. Other wave types, like tidal waves, are not present.

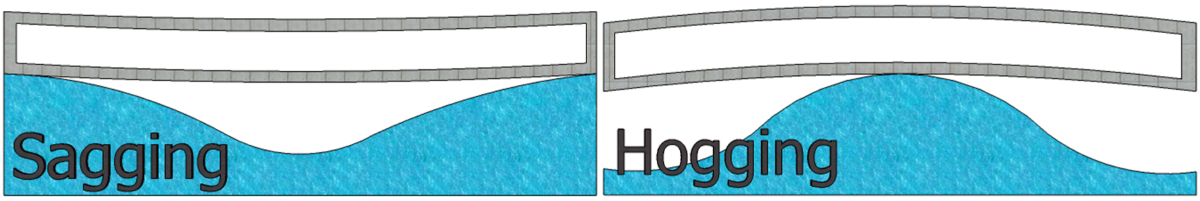


Figure 48: Bending moments in structure because of sagging and hogging

<u>Wind waves</u>

The height of the wind induced waves depends on four parameters, namely:

i. Depth of the water;
ii. Wind velocity;
iii. Durance of wind/storm;
iv. Fetch length.

To calculate the wave loads and dynamic stability of the structure, the significant wave height, wave period and wave length have to be known. These parameters can be calculated using Bretschneider. These parameters and the resulting wave loads are calculated in appendix E.

Calculations show that waves of 0,98 meters high could be expected. During the construction of the first EPS layers and concrete floor this is too high. Therefore it is recommended to avoid constructing the Flexbase floor during the storm season (October tot April) and use a breakwater to break the waves and make the fetch length smaller. This could simply be a barge or pontoon.

See appendix E for calculations on wind waves and wave loads

<u>Ship waves</u>

The second wave type that is present in the Oosterdok is ship waves. The waves are induced by the small boats that sail through the city canals. The wave height is directly linked to the size and speed of the ships. Because the boats are relatively small (length smaller than 10 meters) and the sail velocities are also relatively low (< 2 m/s) these waves aren't taken into account. Besides, boats will probably not pass the construction site, but be rerouted around the Oosterdok during construction.

<u>Dynamic stability</u>

In appendix D the static stability was checked, but also the dynamic stability will have to be checked when a floating element is subjected to waves. The element could start to sway and resonance could occur. The be sure this doesn't happen, the natural oscillation period of the element should be significantly larger than the period of the waves. The natural oscillation period of the element is given by:

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

In which:

T_0 =	natural oscillation period	[s]
h_m =	metacentric height	[m]
g =	gravitational constant	[m/s^2]
j =	polar inertia radius of the element	[m]

See appendix F for dynamic stability calculations

The dynamic stability of the element is calculated in appendix F. This shows that the eigenperiods of the element are 5,32 and 4,91 seconds, which is much larger than the period of the wind waves. Therefore, the element is dynamically stable. See appendix F for further conclusions.

Sway
When the dimensions of the element are smaller than the significant wave length, the element may tend to sway on the waves. The following rule of thumb is used:

$L_w < 0.7 \cdot \ell_e$ and $L_w < 0.7 \cdot b_e$ (dependent on the direction of the waves relative to the caisson)

where:

L_w =	wave length	[m]
ℓ_e =	length of the floating element	[m]
b_e =	width of the element	[m]

Since the wavelength is approximately 10 meters, the element will not tend to sway (see appendix E on wind waves)

Current and drag
When the element is subjected to flow, drag forces will act on it. Currents are considered non present in the Oosterdok, but during transport a drag force will act on the element. This force is proportional to the velocity head times the density of the fluid. The empirical formula for the for the drag force is:

$$F_d = 0{,}5\ \rho_w\ u^2\ (C_d + C_d')\ A$$

In which:

F_D =	drag force parallel to the flow direction	[kN]
C_d =	drag coefficient (static)	[-]
C_d' =	dynamic drag coefficient (time dependent)	[-]
A =	area facing flow, projected perpendicular to the flow direction	[m^2]
ρ_w =	density of water	[kN/m^3]

In this case we can assume the water is turbulent (Re>10^4), the shape is rectangular and the estimated velocity during transport is 0,5 m/s. The drag coefficients can be found in figure 6-1 and 6-5 of the CT3330 manual. The area facing flow is the draught times the width. This gives a total drag force of:

$$F_d = 0{,}5 * 10 * 0{,}25 * (\ 2 + 0{,}15\ ) * \ 5{,}30 * 77 = 1100 \text{ kN} = 2{,}68 \text{ kN/m}^2$$

This force does not only act as an uniform load on the wall of the element, but also as several concentrated loads where the towing cables are attached.

8.5 Ground

The subjects related to the subsoil are divided into the following categories:

i. Effective stress;
ii. Groundwater;
iii. Settlement.

Effective stress
The soil around the garage element in the operation stage induces two loads on the element, a vertical load and a horizontal load. The vertical load is induced by the layer of soil on top of the element. The horizontal load is induced by the soil layers next to the element. The element is considered to be rigid.

Vertical load
The vertical effective pressure on top of the element can be determined with the following formula:

$$\sigma'v = \sigma v - p \quad \text{i.e.} \quad \sigma'v = \sum_{i=1}^{n} y_{d,j}\ d_i + \sum_{j=1}^{m} y_{dn,j}\ d_i - p$$

In which:

σ'_v =	vertical intergranular stress	[kN/m2]
σ_v =	total vertical stress	[kPa = kN/m^2]
$y_{d,i}$ =	dry volumetric weight of soil layer i	[kN/m^3]
$y_{d,i}$=	wet volumetric weight of soil layer j	[kN/m^3]
σ =	volumetric mass of a soil layer	[kN/m^3]
d_i =	thickness of soil layer i above the considered plane	[m]
n =	number of dry layers above the considered plane	[-]
m =	number of wet layers above the considered plane	[-]
p =	water pressure in the considered plane	[kN/m^2]

There is only one soil layer on top of the element and this is a wet soil layer, therefore the formula can be simplified to:

$$\sigma'v = (y_{d,I} - \rho_w) * d$$

$$\sigma'v = (20{,}00 - 10{,}00) * 0{,}5 = 5{,}00 \text{ kN/m}^2$$

Horizontal load

Because the element is considered rigid, there will be no horizontal displacement. Therefore, the empirical formula of Jaky can be used to determine the neutral lateral effective stress:

$$\sigma_{h,n} = K_0 * \sigma_v \ with: \ K_0 = 1 - \sin(\text{Ø})$$

However, this formula cannot be used when:

i. The horizontal effective pressure on the wall of an element acts unfavourably;
ii. The bottom of the structure is wider than 15 m.

Both of these statements are true, therefore K_0 = 1 and σ'v = σ'h.

Groundwater

The element is completely surrounded by groundwater in the operation stage. The vertical and horizontal load on the element are calculated with the following formula:

$$p = \rho w\, g\, h$$

in which:

p =	water pressure	[Pa]
h =	pressure head	[m]
g =	acceleration due to gravity	[m/s^2]
ρ_w =	density of water	[kN/m^3]

Settlement

In the construction stage, approximately 10 meters of soil will be excavated. The element will be placed on the first sand layer and this will change the stresses in the underlying soil layers. This will cause settlement. This settlement is calculated using Koppejan's formula. Note that the settlement will be negative, meaning that the soil layers will expand like a continuous elastic foundation that is unloaded. The formula reads:

$$\varepsilon = \left(\frac{U}{C'_p} + \frac{1}{C'_s} \log(t) \right) \cdot \ln\left(\frac{\sigma'_{v;i} + \Delta\sigma'_v}{\sigma'_{v;i}} \right)$$

in which:

ε = relative compression	[-]
H = layer thickness	[m]
U = degree of consolidation	[-]
C'_p = primary compression coefficient	[-]
C'_s = secondary compression coefficient	[-]
C_p = primary compression coefficient	[-]
C_s = secondary compression coefficient	[-]
t = time in days	[d]

$\Delta\sigma_{v}'$ = increase of the vertical effective stress [kPa]
$\sigma'_{v;i}$ = initial vertical effective stress [kPa]

Settlement will not be taken into account. The total settlement will probably be very small because the element is placed on sand layers and the element will not induce a significant load onto the sand layer. Also, settlement of the soil layers will not lead to large settlements of the element because of the grout anchor foundation (see appendix J).

Soil stability

During excavation, the slopes and structures in the surroundings will have to be stable. This will probably not be a problem, except near the quay wall. Figure 49 shows the existing situation and the soil that will be excavated. This shows the anchored sheet pile wall that acts as a water screen and the L-wall on hollow steel piles with grout injection. This sheet pile wall and L-wall are placed in 2010 on the side of the Oosterdok island and the Prins Hendrikkade. Excavating the soil near the quay wall could cause settlements and/or failure of the sheet piles and L-wall.

To be safe, the soil between the piles, under the L-wall and near the sheet piles will not be excavated, but a sheet pile wall will be placed in front of the concrete L-wall to retain the soil. See appendix G for calculations on this sheet pile wall. This shows that a AZ 18 sheet pile wall with a length of 20,00 meters will be sufficient.

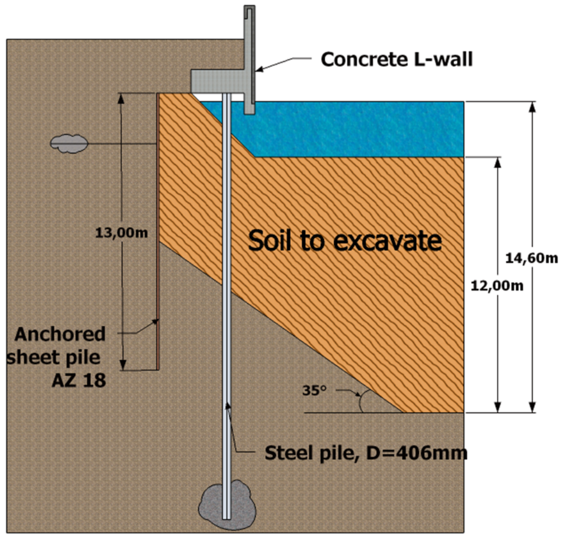


Figure 49: Existing quay wall

See appendix G for calculations on soil stability and sheet piling

8.6 Other loads

Loads during construction

During construction equipment and workers will induce forces on the structure. A significant load is caused by the needed formwork for the walls, columns and roof. These loads will be calculated according to Eurocode 1991-1.

Temperature

Fluctuations in temperature could lead to deformations and stresses in the structure. However, the temperatures underground are considered constant. Therefore, these loads will not be taken into account.

Fire

A fire in the parking garage (e.g. a burning car) could lead to failure of the structure. This will not be taken into account in the structural analysis and is given as a recommendation for further studies.

Forces on roof

The load on the roof could get larger over time because of a growing sediment layer. Ships could also lose part of their load or an anchor which would cause extra loads on the roof. An extra uniform load on the roof of 10 kN/m^2 will represent these loads.

8.7 Load combinations

To give a complete overview, this chapter shows the most important load combinations. Namely the combinations during the immersion stage and the operation stage.

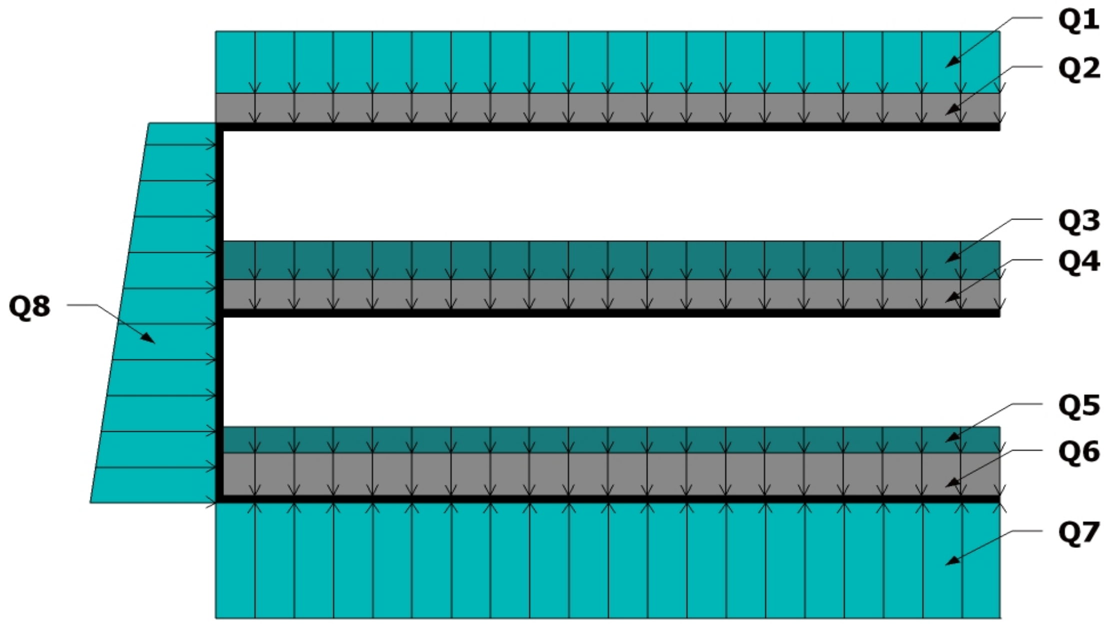
Immersion stage

During the immersion stage the garage element is filled with ballast tanks. There are more ballast tanks on the upper deck then on the lower deck. This is done to have enough space for the grout anchor equipment on the lower deck. The element reaches a depth of 13,1 meters. The load combination is shown on the next page.

Loads on the garage element during immersion (element at final depth):

Q1	Hydrostatic water pressure:	41,0 kN/m^2
Q2	Weight roof:	21,0 kN/m^2
Q3	Ballast tanks:	18,0 kN/m^2
Q4	Weight intermediate floor:	15,0 kN/m^2
Q5	Ballast tanks:	9,0 kN/m^2
Q6	Weight Flexbase floor:	18,2 kN/m^2
Q7	Hydrostatic water pressure:	131,0 kN/m^2
Q8	Hydrostatic water pressure:	41,0 - 131,0 kN/m^2

Loads on garage element, immersion stage

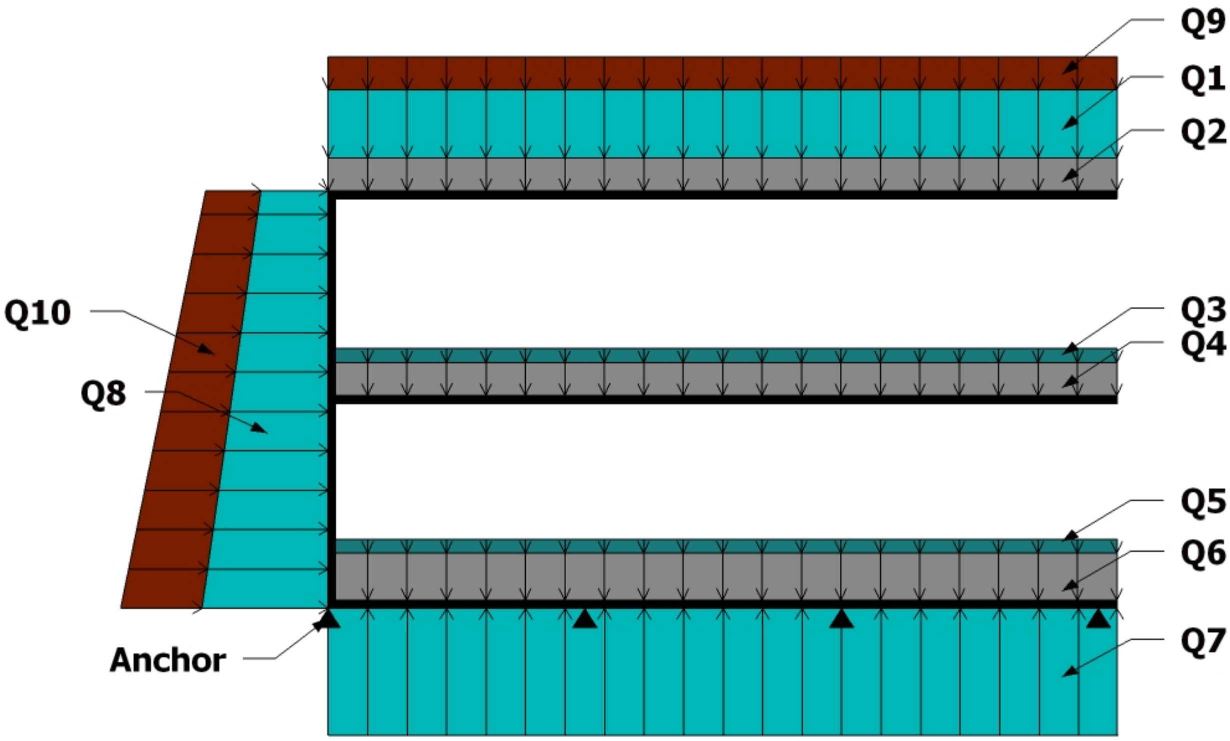


Operation stage
During the operation stage the garage element is situated underwater and underground. The ballast tanks are removed and the garage element is secured to the bed with grout anchors.

Loads on the garage element during the operation stage:

Q1	Hydrostatic water pressure:	41,0 kN/m^2
Q2	Weight roof:	21,0 kN/m^2
Q3	Maximum weight parked cars:	2,5 kN/m^2
Q4	Weight intermediate floor:	15,0 kN/m^2
Q5	Maximum weight parked cars:	2,5 kN/m^2
Q6	Weight Flexbase floor:	18,2 kN/m^2
Q7	Hydrostatic water pressure:	131,0 kN/m^2
Q8	Hydrostatic water pressure:	41,0 - 131,0 kN/m^2
Q9	Vertical soil pressure (incl. safety for sediment built-up):	18,0 kN/m^2
Q10	Vertical soil pressure:	18,0 - 108,0 kN/m^2

Loads on garage element, operation stage



9. Structural analysis

The structural analysis deals with the necessary calculations to prove that the floating construction method is technically feasible. This chapter will focus on the garage element only. The structural analysis is divided into two parts, namely:

i. Structural analysis of walls, columns and roof, i.e. the garage element;
ii. Structural analysis and pouring scheme of the Flexbase floor.

This chapter will focus on the second part, namely the Flexbase floor and will only briefly describe the first part. The reason for this is simple, the exact dimensions of the walls, roof and columns aren't the point of interest in this thesis. The needed dimensions for the Flexbase floor are.

9.1 Structural analysis garage element

The structural analysis of the garage element is divided into four parts, these are:

i. Columns;
ii. Roof;
iii. Intermediate floor;
iv. Walls.

The calculations will be reviewed briefly. For the complete calculations, check appendix L.

Columns

The c.t.c. distances of the columns are 7,20 and 8,25 meters, as shown in figure 50. The columns are modelled in 2D in SCIA Engineering to make sure the hand calculations are correct. The normal forces in the columns in the operation stage are calculated and this gives for the upper columns:

$$N'd = 5.197 \text{ kN}$$

And for the columns at the bottom floor:

$$N'd = 7.540 \text{ kN}$$

The moments at the top of the columns are close to zero. Therefore, it can be assumed that the columns are centrally loaded. The formula for the maximum load on the column is:

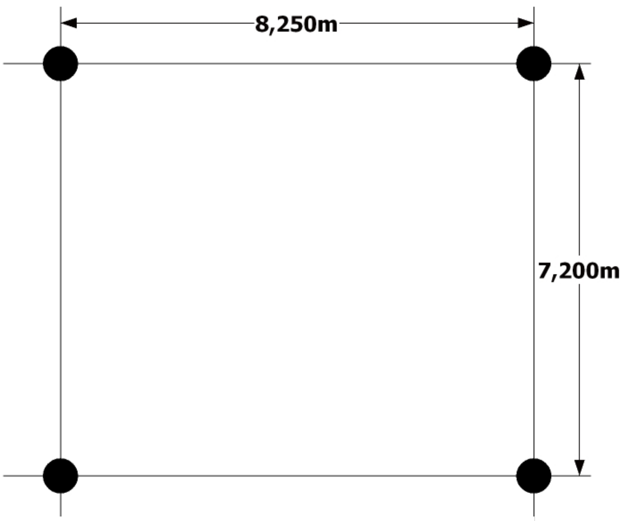


Figure 50: c.t.c. distances columns

$$N_u = A_c * f_{cd} + A_s * f_{yd}$$

In which:

A_c	=	Cross-section concrete	[mm^2]
A_s	=	Cross-section reinforcement steel	[mm^2]
f_{cd}	=	Pressure strength concrete	[N/mm^2]
f_{yd}	=	Pressure strength reinforcement steel	[N/mm^2]
A_c	=	300 * 300 * π = 282.743	[mm^2]
A_s	=	1% * A_c = 2.827	[mm^2]
f_{cd}	=	21 (C28/35)	[N/mm^2]
f_{yd}	=	425 (FeB500)	[N/mm^2]

This gives:

$$N_u = 282.743 * 21 + 2.827 * 425$$

$$N_u = 7.139 \text{ kN}$$

Upper columns: Unity check = 5.197 / 7.139 = 0,73 < 1,00 → OK

Bottom columns: Unity check = 7.540 / 7.139 = 1,06 > 1,00 → Not OK

Columns at the bottom floor will need a larger cross-section → D = 700 mm.

A_c	=	350 * 350 * π = 384.845	[mm^2]
A_s	=	1% * A_c = 3.848	[mm^2]
f_{cd}	=	21 (C28/35)	[N/mm^2]
f_{yd}	=	425 (FeB500)	[N/mm^2]

$$N_u = 384.845 * 21 + 3.848 * 425$$

$$N_u = 9.717 \text{ kN}$$

Bottom columns: Unity check = 7.540 / 9.717 = 0,78 < 1,00 → OK

Required dimensions

Bottom columns: D = 700mm

Upper columns: D = 600mm

Roof
The structural analysis of the roof consists of three parts, namely:

i. Needed reinforcement in roof slab (appendix L2);
ii. Punching shear check (appendix L4);
iii. Crack width control (appendix L2).

The needed reinforcement for punching shear and the crack width control are leading in this situation.

Punching shear
The punching shear reinforcement is calculated for the situation shown in figure 51. The following data applies to this situation:

Reinforcement: Ø16-150

Concrete cover: 35mm

V_{ed}: 5.261kN

The complete calculation is shown in appendix L4. The outcome is the following.

Punching shear perimeters:
190 mm – 320 mm – 800 mm – 1250 mm – 1750 mm

Reinforcement needed in perimeters:
Stirrups Ø16, 10 stirrups required per perimeter.

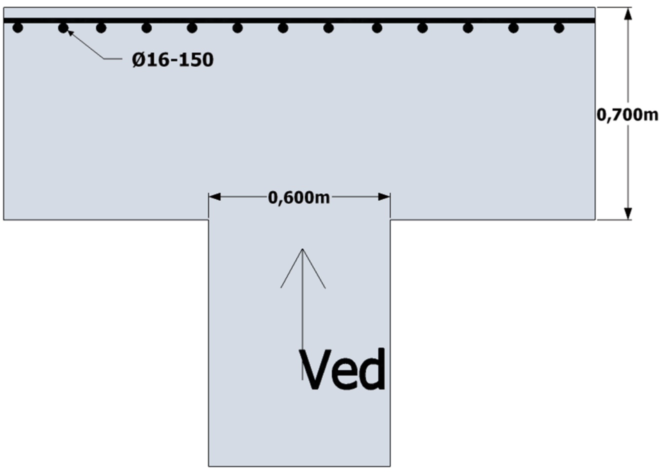


Figure 51: Reinforcement in roof above columns

Crack width control

Exactly above the columns the moments are extreme. For the crack width control the SLS is used. The moment above the columns is 840kNm.

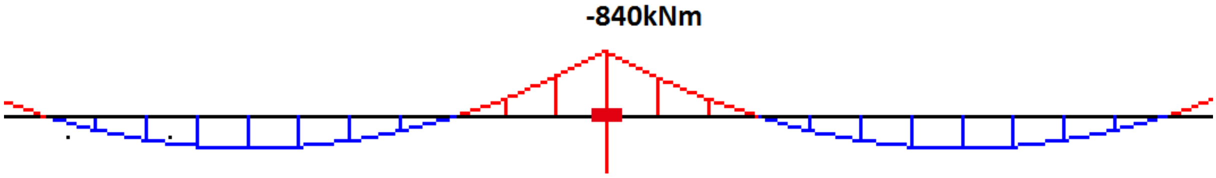


Figure 52: Maximum moment in roof above columns

Input

Concrete roof slab:	7,20 x 8,25	[m]
Concrete:	C28/35	
Height:	700	[mm]
w_{max} =	0,1	[mm]

The needed reinforcement was calculated for this situation in the operation stage. The outcome is shown in figure 53 and is the following:

Reinforcement: Ø 16 – 62,5, two rows

w_{max} = 0,103mm ≈ 0,1mm → OK

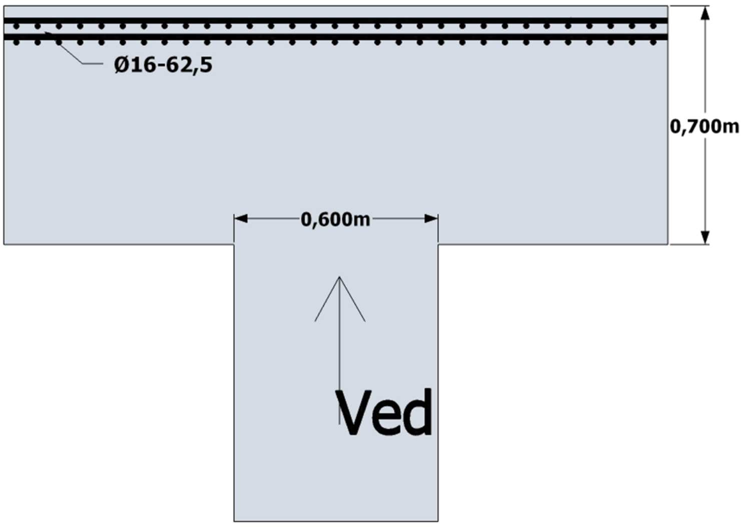


Figure 53: Needed reinforcement in roof for crack width requirements

Note: the reinforcement needed is more than used in the punching shear calculations. This means that the punching shear reinforcement stirrups can probably be less.

Intermediate floor

The structural analysis of the intermediate floor consists of two parts, namely:

i. Needed reinforcement in roof slab (appendix L3);
ii. Punching shear check (appendix L4).

The most important aspect of these calculations is the fact that they are not based on the operation stage but on the immersion stage. The reason simple, during the operation stage the maximum load on the floor is 2,5 kN/m^2 (cars, category F) and during the immersion stage this is 21,6 kN/m^2 (ballast tanks, see appendix L3).

Punching shear

The punching shear reinforcement was calculated for the situation shown in figure 54. The following data applies to this situation:

Reinforcement:	Ø20-125
Concrete cover:	35mm
V_{ed}:	2.343kN

The complete calculation is shown in appendix L4. The outcome is the following.

Punching shear perimeters:
165 mm – 275 mm – 690 mm

Reinforcement needed in perimeters:
Stirrups Ø20, 4 stirrups required per perimeter.

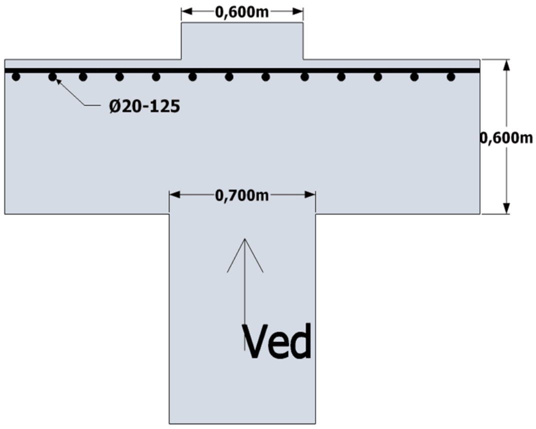


Figure 54: Reinforcement in intermediate floor above columns

See appendix L for the complete structural analysis of the columns, roof, intermediate floor and walls

9.2 Structural analysis Flexbase floor

The Flexbase floor forms the heart of the floating construction method and this thesis. The structural analysis of this floor is therefore one of the most important, if not the most important, part of this thesis. The structural analysis is divided into three stages, namely:

i. Construction stage (pouring scheme);
ii. Immersion stage;
iii. Operation stage.

Because of the complex schematization, these situations are modelled in SCIA Engineering. This chapter will start with the immersion stage. Why this stage is reviewed first, will be explained in the following paragraph.

Immersion stage

The immersion stage is reviewed first, because the moments and stresses in the floor cannot directly be changed like in the other two stages. In the construction stage, the stresses in the floor can be changed by changing the pouring scheme. In the operation stage the moments and stresses in the floor can be lowered by changing the grout anchor spacing.

The only way to lower the stresses in the floor during the immersion stage is by changing the actual dimensions of the floor. Therefore, this stage is reviewed first,

<u>SCIA Model</u>

During the operation stage the garage element is at its deepest point and the loads on the floor are at its maximum. However, the operation stage is probably not leading for the Flexbase floor. During the immersion stage the element also reaches the maximum depth, but the grout anchors aren't installed yet. This means that the loads are the same, but the leading floor span is that of the columns (8,25 and 7,2 meters) and not that of the grout anchors (4,12 and 3,6 meters). Therefore, the moments and stresses in the floor and beams will probably be larger in the immersion stage.

Only a small part of the floor is modelled in SCIA for this stage. Modelling the complete floor in SCIA isn't necessary during this stage and would make the program very slow and less accurate. The model is shown in figure 55 and 56.

The length and width of the model is two times the c.t.c distance of the columns. The edges are clamped to simulate the fact that it is a part of the entire floor. The columns are modelled on top of the floor and to simulate the immersion stage the bearings are placed at the top of these columns. To read more about the model, see appendix K7.

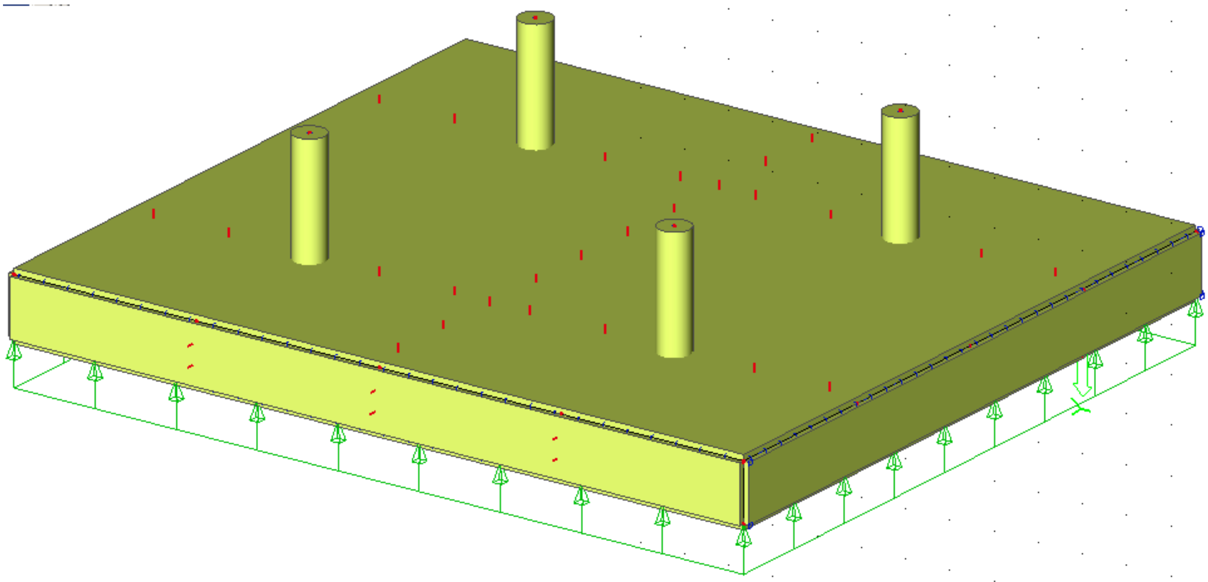


Figure 55: SCIA Model, part of Flexbase vloer

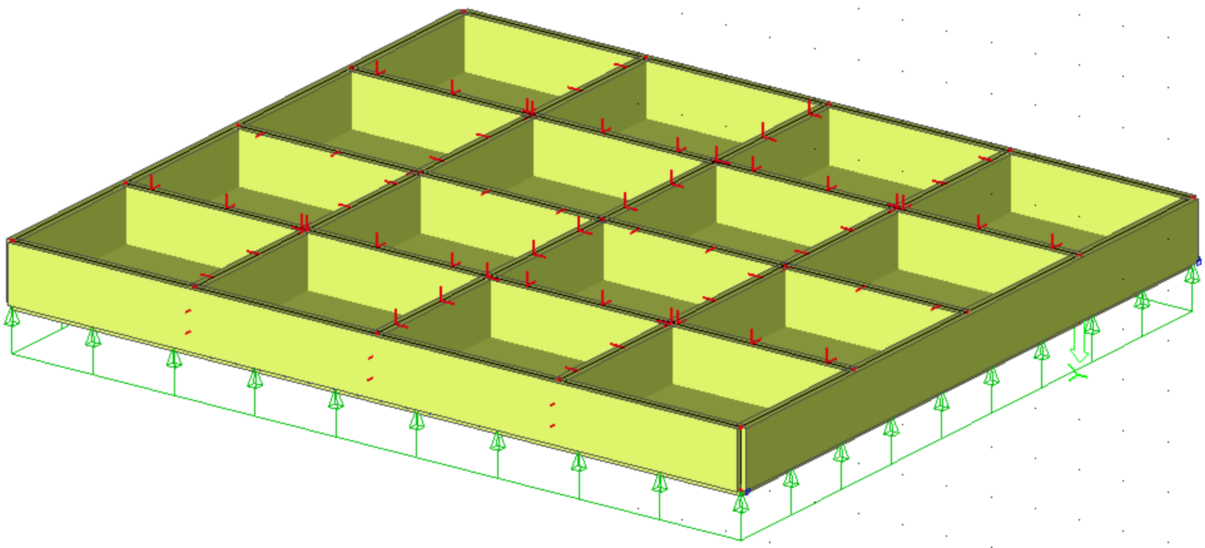


Figure 56: SCIA Model, part of Flexbase vloer, top floor left out

Note: the floor only has one beam grid instead of two (Flexbase Heavy). This will be explained on page 72.

Calculations

The Flexbase sandwich construction can be modelled as a HEA profile where the beams deal with the shear forces and the floor with the tension and compression. The following calculations will be reviewed:

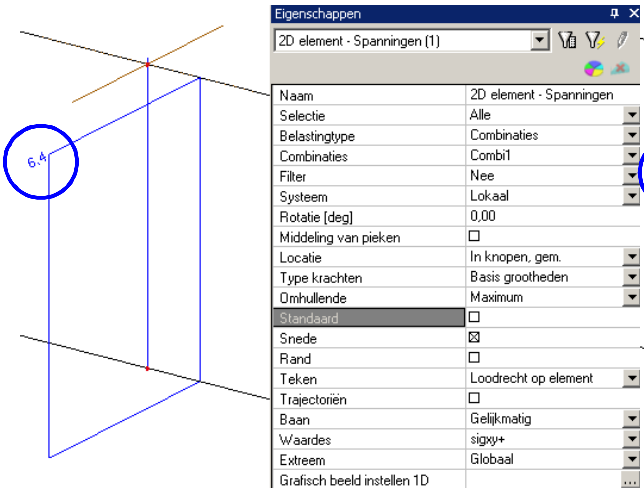
i. Calculation beams on schear force;
ii. Crack width control bottom floor (w_{max} 0,3mm);
iii. Crack width control top floor (w_{max} 0,1mm).

The difference between two and three is the fact that the bottom floor doesn't have to be watertigth, because this will not lead to water leaking into the garage. The bottom floor will only be checked on the maximum crack width for the highest exposure class (EC 1992-1-1).

Calculation beams on schear force

First, the averaged shear stress over the height of the beams is checked. This value can't be higher than 4,2N/mm^2 for C28/35. Figure 57 shows the averaged shear stress over the height of the beams at the cross section where the shear stress is at its maximum. This is shown for two dimensions, namely for a beam that is 250mm wide and for one that is 400mm wide. The averaged shear stress is 6,4N/mm^2 for the 250mm wide beam and 4,2N/mm^2 for the 400mm wide beam.

This shows that the minimum width of the beams in the beam grid will have to be 400mm. The beams are 1000mm high.



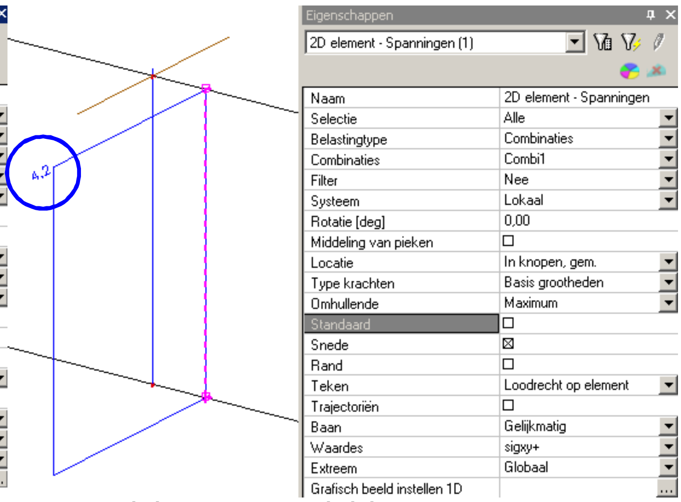


Figure 57: Averaged shear stress over height, 250mm **Averaged shear stress over height, 400mm**

The required shear reinforcement has been calculated for this situation. The complete calculation is shown in appendix K7. The needed reinforcement is stirrups Ø 20 – 125.

Crack width control bottom floor

The total load in SLS on the bottom of the floor is 138kN/m^2. Figure 59 shows the maximum tension in the bottom floor in the x-direction, which is 1.485kN/m. The crack width is checked for this situation.

The needed reinforcement is Ø 16 – 62,5, two rows. This is shown in figure 58.

See appendix K7 for complete calculations.

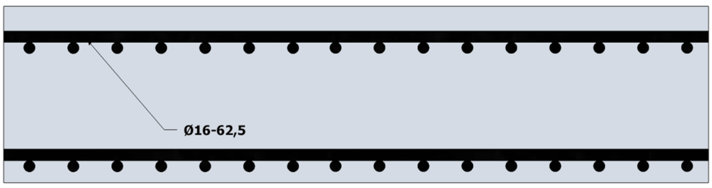


Figure 58: Needed reinforcement in bottom floor

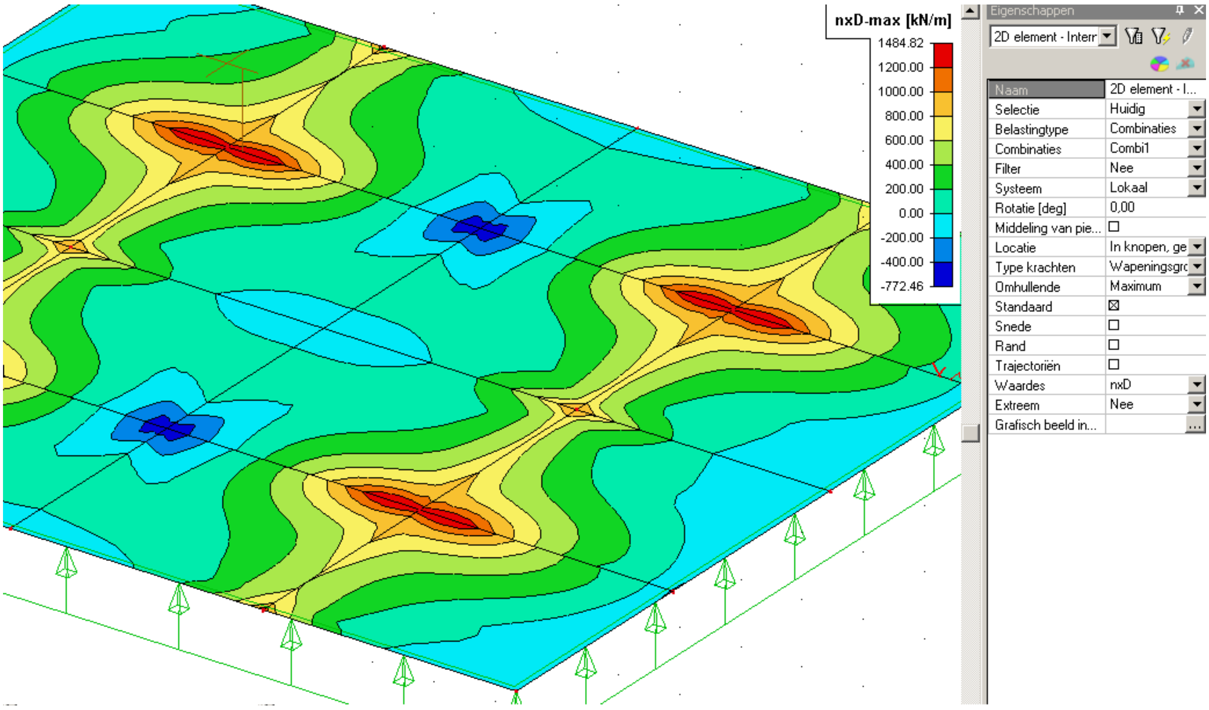


Figure 59: Stresses in bottom floor in immersion stage, x-direction

<u>Crack width control bottom floor</u>

The same was done for the top floor with a maximum crack width of 0,1mm. Figure 60 shows the maximum tension in the bottom floor in the y-direction, which is 1.000kN/m. The crack width is checked for this situation.

The needed reinforcement is Ø 16 – 83,3 , two rows.

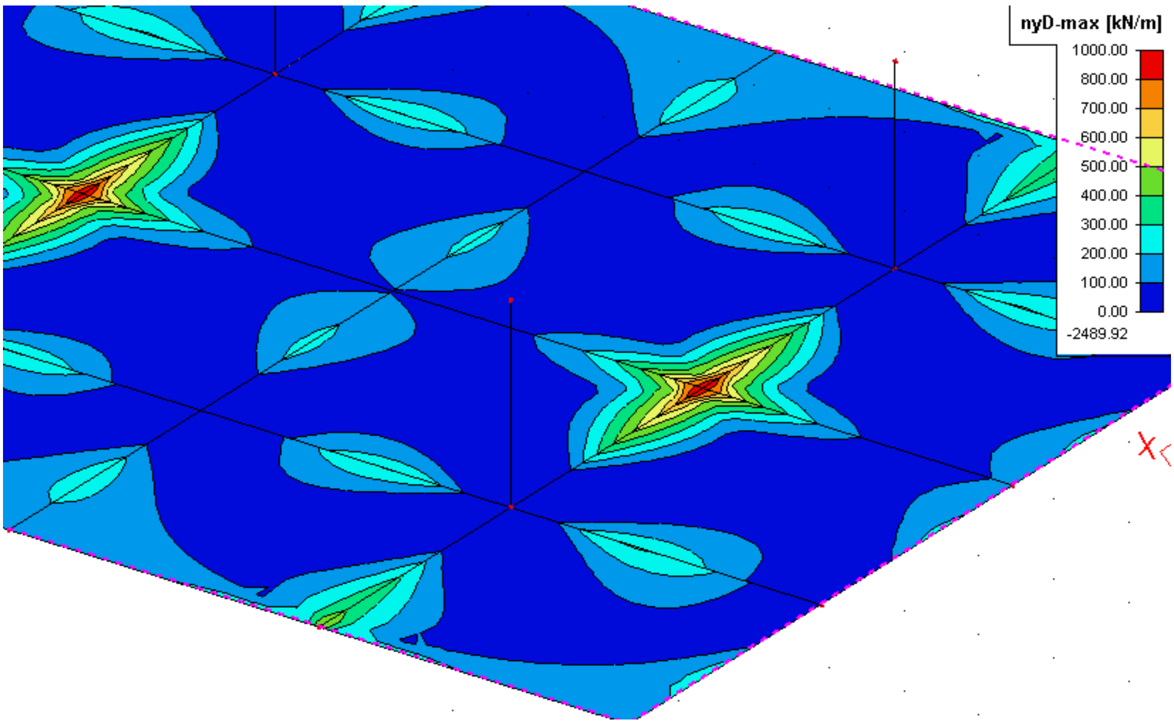


Figure 60: Stresses in top floor in immersion stage, y-direction

See appendix K7 for the complete structural analysis of the immersion stage

Operation stage

The same model will be used for the operation stage. The only difference is that the anchors are placed and the ULS will be checked. The anchors will be placed next to the intersections of the beams, as close as possible to ensure the most favourable transmission of forces.

The anchors are placed in a grid of 4,125 x 3,6 meters.

Modelling the anchors

The figure below shows a part of the bottom of the Flexbase floor. The blue squares represent the grout anchors. One anchor in the corner is missing, this will be explained later on.

Because the changes in anchor length are very small, the anchors are simulated by bearings of 20 x 20 cm that are restricted in the x-, y-, and z-direction.

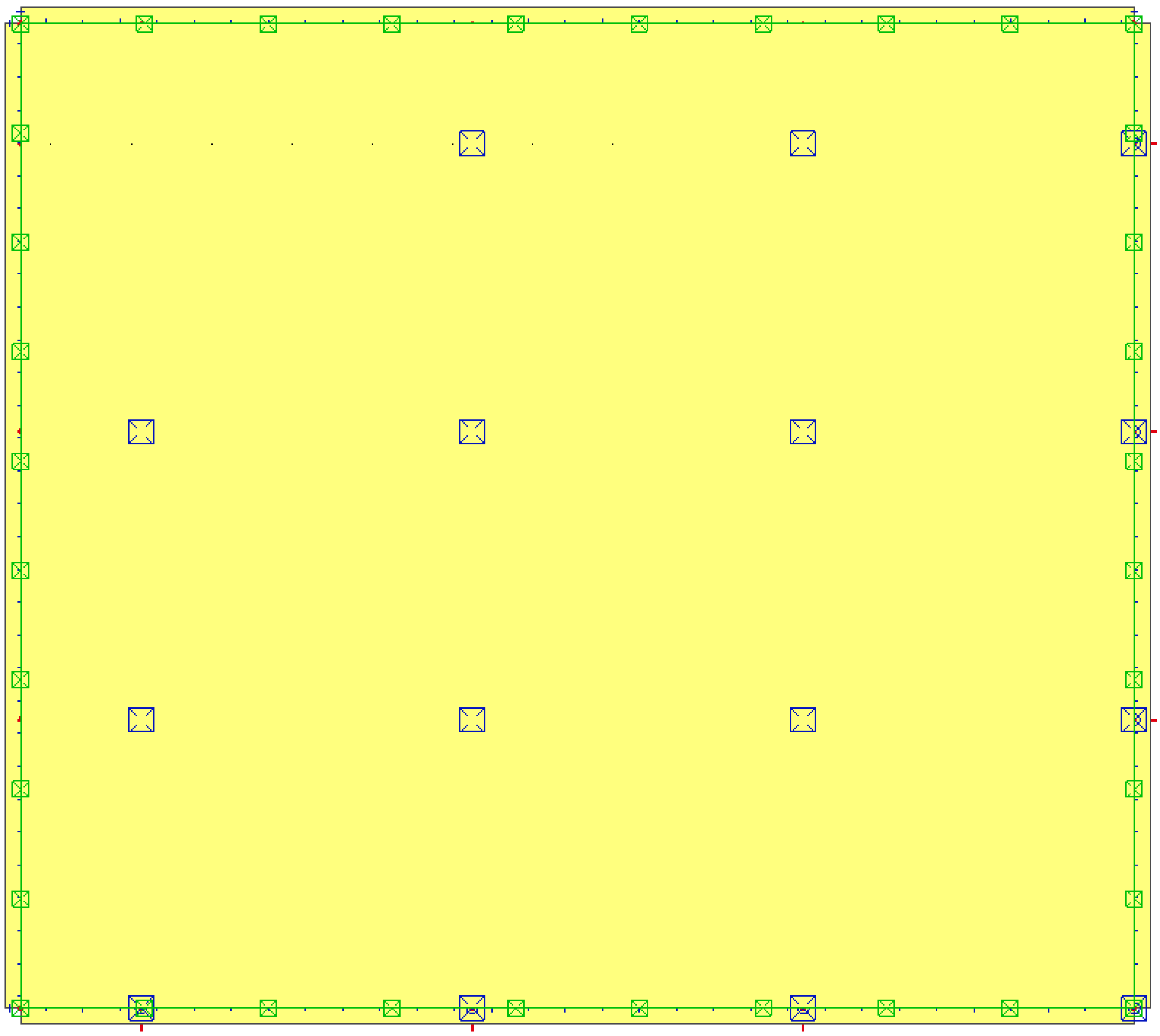


Figure 61: SCIA model, bottom view, anchors placed under the Flexbase floor

Forces in floor in operation stage

Two situations are modelled. One with all the anchors in place and one with one anchor missing. When one anchor is damaged or loses its function because of another reason, the anchors surrounding it will take its load. This creates an unwanted, but possible, situation and the floor has to resist these loads.

All the anchors in place
The figures below shows the tension stresses in the floor in the operation stage (water pressure against the floor). The maximum tension is 866kN/m, which is much less than in the immersion stage (1.486kN/m, see appendix K7). Therefore, this situation isn't governing.

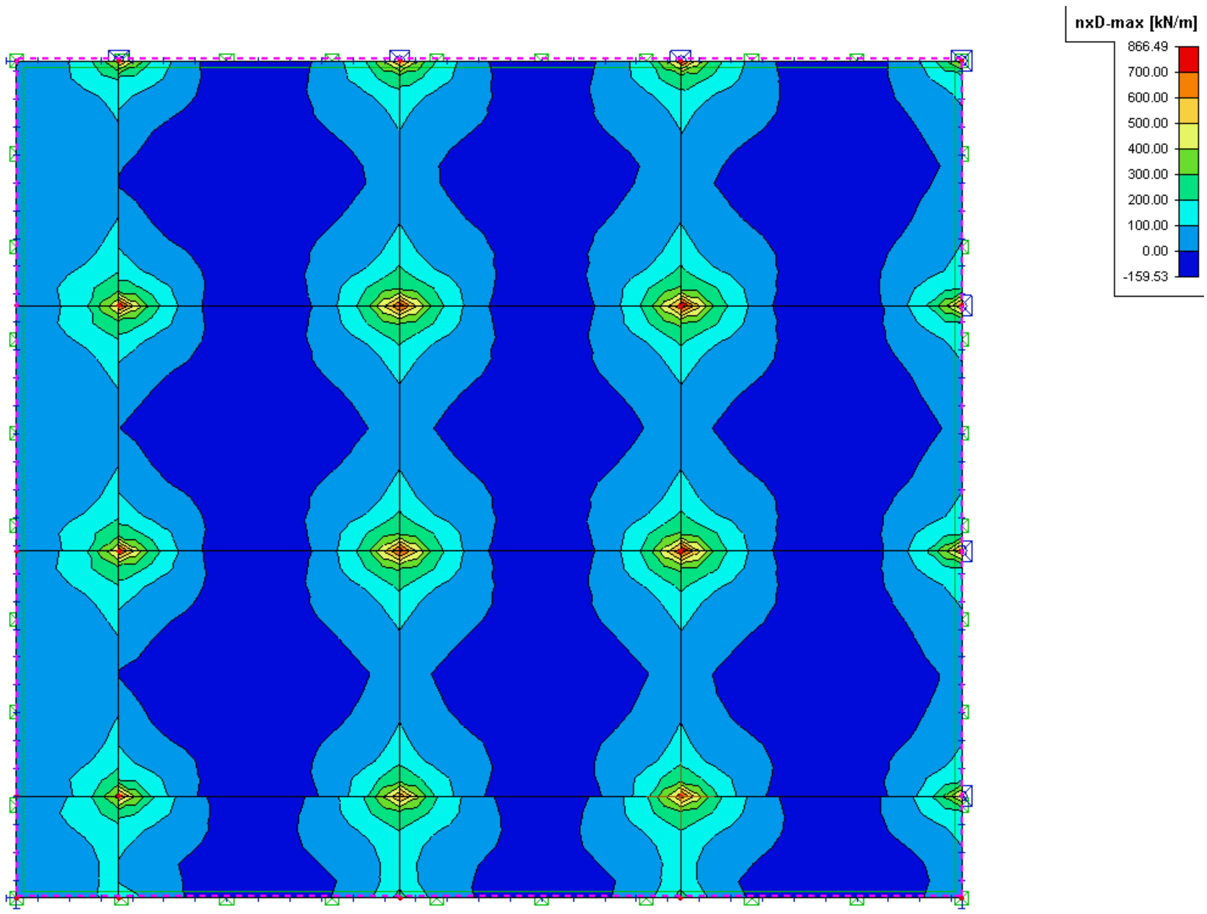


Figure 62: Tension stresses in bottom floor during operation stage, x-direction

One anchor missing
The most unfavourable situation takes place when the corner anchor loses its function, because the three surrounding anchors will have to take its load instead of eight anchors when an anchor in the middle of the floor loses its function.

The figure on the next page shows the tension stresses in the floor when the corner anchor is missing. This shows very high tension forces of 2.600kN/m around the anchor. It is however permitted to average the tension force over two times the effective height of the floor, which is 400mm. This is shown figure 64 on the next page. This shows that the high tension stresses are very local and that the averaged tension force over 2 * d is only 352 kN/m. Therefore, also this situation isn't leading and the leading situation is the immersion stage (see appendix K7).

See appendix J for the complete structural analysis of the operation stage

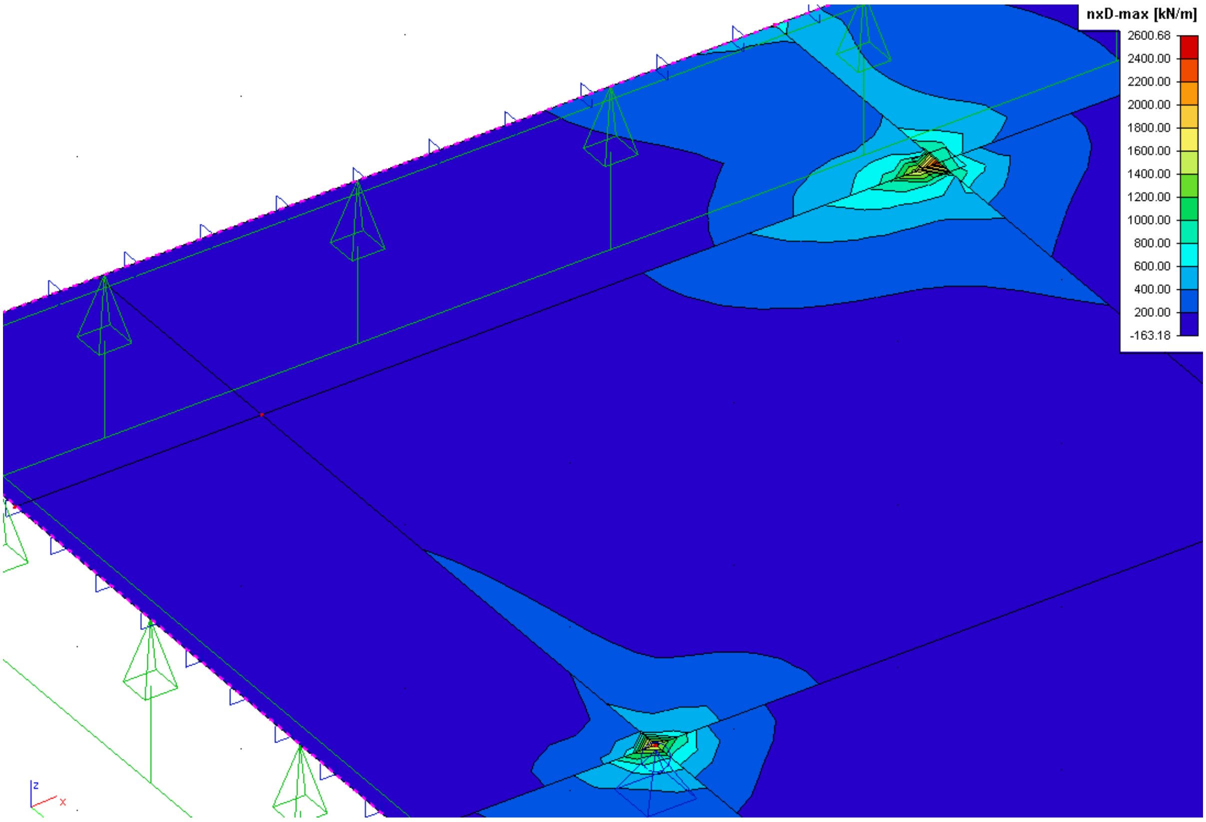


Figure 63: Tension stresses in bottom floor, one anchor missing in corner

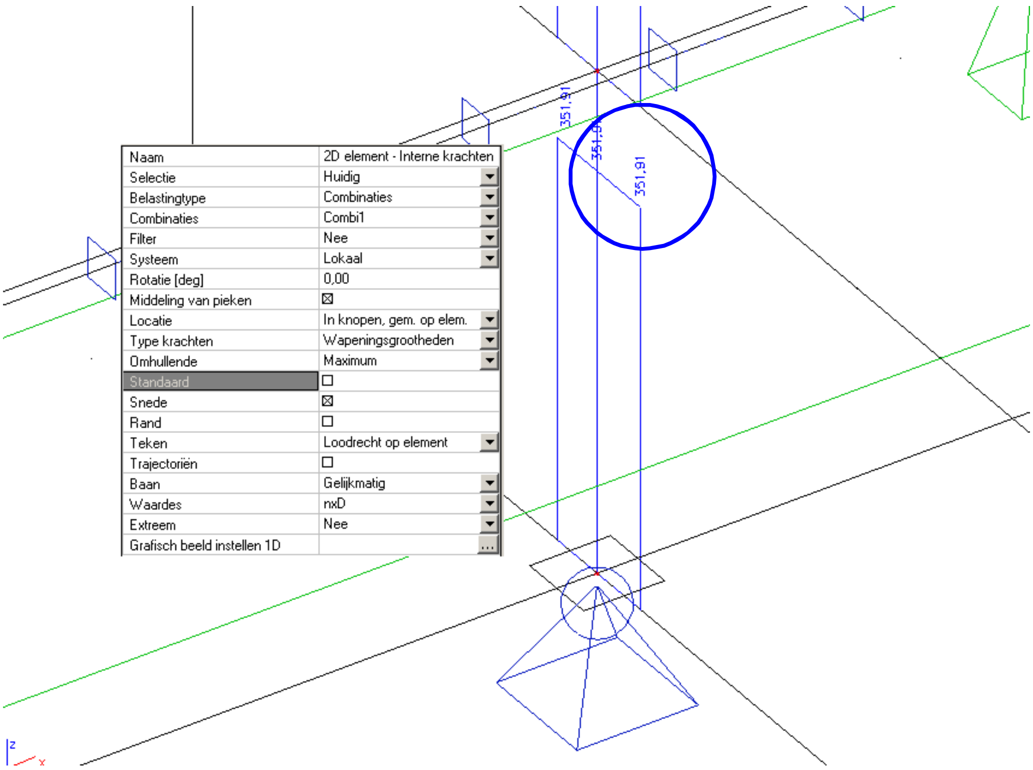


Figure 64: Tension stresses in bottom floor, averaged over 400mm, one anchor missing in corner

Construction stage

The structural analysis will deal with the construction of the Flexbase floor and the walls of the first floor. The outcome is a pouring scheme for the entire garage element. The entire garage element will be modelled in SCIA, in contrast with the other stages where only a part of the floor was modelled. It might be clear that a pouring scheme can only give accurate data when the entire floor and garage element are modelled.

The Flexbase floor

Before explaining how the floor is modelled in SCIA, it is important to know how the floor will be constructed. Figure 65 shows the floor as it will be constructed for the garage element. The Flexbase Standard and Flexbase Heavy floor have a beam grid under the bottom floor, this version hasn't got that beam grid.

The bottom EPS layers of the Flexbase Standard and Heavy where constructed and modelled with EPS 150 layers stuck together with glue. Up to now that is. This floor will use four layers of EPS 250, reinforced with glass fibre foil and plastic pins. In comparison with EPS 150, this 800mm thick EPS floor will be four to six times stiffer.

This means that the EPS floor can provide enough rigidity to pour the concrete floor directly on top and the beam grid isn't needed. At least, that is the idea.

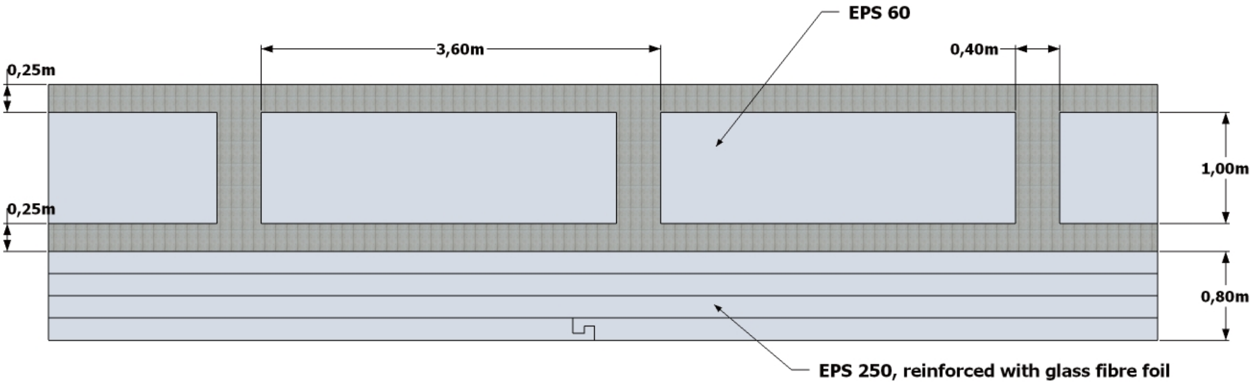


Figure 65: Dimensions of the Flexbase floor

The pouring scheme consists of three parts, namely:

i. Constructing the first floor on the EPS floor;
ii. Constructing the beam grid;
iii. Constructing the walls on top of the Flexbase floor.

The construction of the top floor isn't reviewed because it is believed, and it is known from experience, that the floor is rigid enough at this point to pour this floor. Secondly, only the construction of the outer walls on top of the Flexbase floor will be reviewed because this will most probably be the leading situation. Furthermore, this chapter gives a brief overview of the complete pouring scheme. For the complete pouring scheme, SCIA model and model control, see appendix K.

Constructing the first floor on the EPS floor

The EPS floor is model as a floor consisting out of a homogenous material with the characteristics of glass fibre reinforced EPS 250. The floor is 79,2 meters wide, 132 meters long and 800 millimetres thick. The floor is supported by a continuous elastic foundation to simulate water. The model is shown in figure 66.

The E-modulus of normal EPS 250 is 12.000 kPa. The E-modulus of glass fibre reinforced EPS 250 is two to three times larger. At the moment this can't be based on literature, but only on test done by the leading engineer of Unidek BV. At this moment, more tests are required. In the model an E-modulus of 30.000 kPa is used.

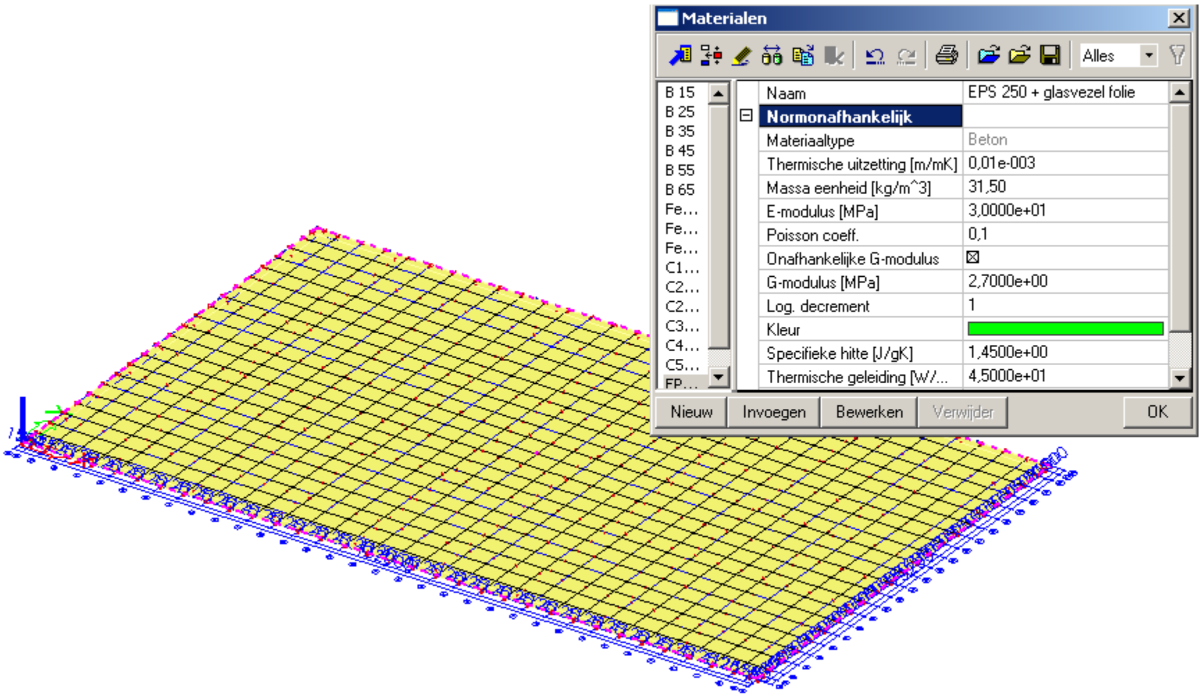


Figure 66: SCIA Model, Floor of glass fibre reinforced EPS 250

The pouring scheme simulates starting at the centre of the floor and working towards the outside. This is shown in appendix K5. Two things are checked, namely:

i. The maximum stress in the EPS floor;
ii. The maximum deformation of the EPS floor.

The maximum stress will have to be lower than the allowable stress in EPS 250 to make sure the EPS will not fail during construction. The maximum deformation will be needed to know which concrete consistency can be used. The next paragraph will review the conclusions drawn from the pouring scheme.

<u>Conclusions</u>

i. Pouring the concrete floor completely in one go would cause stresses and deformations that are too high. Therefore, the concrete floor will have to be poured in two layers. The first layer will not have to be hardened before pouring the second layer;
ii. The maximum deformation in the EPS floor is 1:42 (2,5%). This isn't significant when pouring concrete. To be on the safe side, concrete with consistency class 2 could be used;
iii. The maximum tension stress in the EPS floor is 0,158N/mm^2. The allowable tension stress in EPS250 is 0,350N/mm^2. This shows that the tension stresses in the EPS floor stay within the allowable limit.

See appendix K5 for the complete pouring scheme of the first floor

Constructing the beam grid

The pouring scheme for the beam grid is done in the same manor. The concrete floor is modelled in SCIA and the EPS floor is neglected. The floor is 250mm thick. Appendix K6 shows the several pouring stages. The beam grid will also be poured in two times. It could probably be done in one go, but to be safe it is better to do it in two stages.

The maximum moment in the floor during the construction of the beam grid is 57kNm. The reinforcement in this floor is two rows of Ø16-62,5 (appendix K7). A simple formula shows that this is more than enough. Therefore, the reinforcement meets the requirements.

See appendix K6 for the complete pouring scheme of the beam grid

Constructing the walls on top of the Flexbase floor

When the Flexbase floor is completed, the next step is to construct the walls. The figure below shows the Flexbase floor with the walls of the first floor finished. The walls are constructed in three stages, which are shown in appendix K3. Every stage consists of two separate stages, namely when the walls are just poured and when they are hardened. In the first stage they are modelled as loads in SCIA and in the second stage as walls. This means that in the second stage, they are part of the structure.

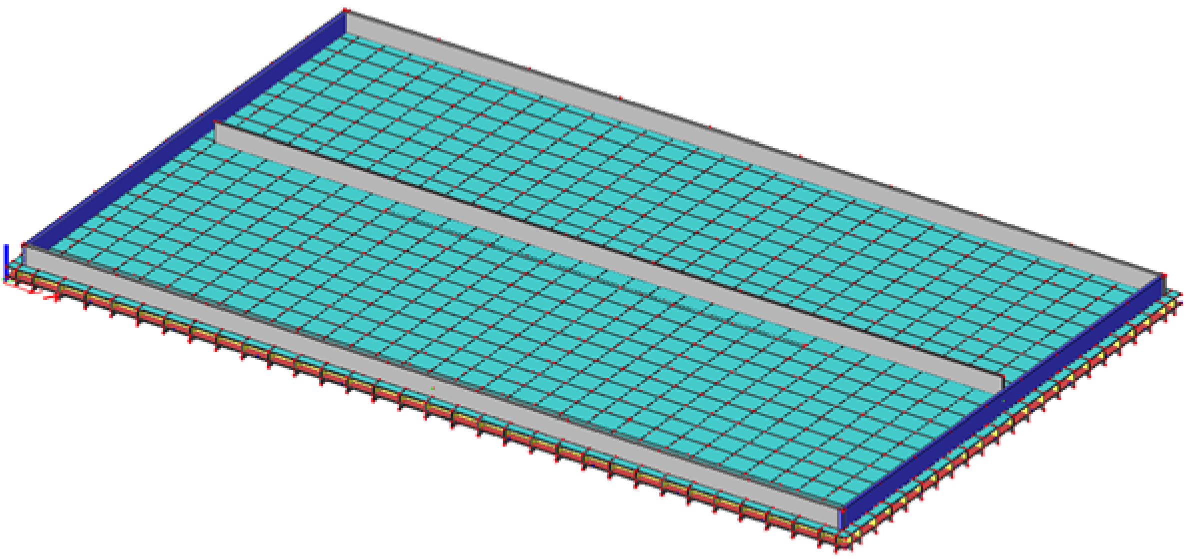


Figure 67: SCIA Model, Flexbase floor with walls of the first floor*

Appendix K4 evaluates all the separate stages and shows the maximum stresses in the Flexbase floor. Here, only the conclusions are reviewed.

Conclusions

i. Compared to the immersion stage the stresses in the Flexbase floor are very low. The maximum tension in the bottom and top floor is 475kN/m compared to 1.486kN/m in the immersion stage;
ii. During every construction stage, counterweights are placed to keep the deformations and stresses to a minimum. More research is needed to conclude if these counterweights are really needed;
iii. In this example the walls are poured in three stages. This is done to use the formwork more than ones. It is recommended to find an optimum between using the formwork as much as possible and constructing all the walls in one time.

* Note: the model used for the pouring scheme of the walls has two beam grids while the final design has one. Appendix K5 shows that leaving the bottom beam grid out hardly changes the results.

See appendix K3 and K4 for the complete pouring scheme of the walls

10. Financial analysis

This chapter will focus on the construction costs of the floating construction method and compare them to the costs of the traditional construction method (i.e. in a construction pit). Furthermore it will determine the benefits. That is, the expected return in the operation stage. Other benefits, like less risks, will not be mentioned here.

10.1 Costs

The direct construction costs will be determined for the floating construction method and the traditional method. The floating construction method was described in detail in chapter 7. For the traditional method this was briefly done in chapter 5. The design for both construction methods is exactly the same, except for the floor of course.

Floating construction method
The costs for the parking garage consist out of two parts, namely:

i. Investment costs (construction costs);
ii. Exploitation costs.

The exploitation costs are the same for both construction methods but the construction costs aren't. The construction costs for the floating construction method are determined in appendix M. These are the direct construction costs for the garage element only, so the entrances for cars and pedestrians not included. The reason for this is the fact that the construction costs for the entrances will almost be the same for both construction methods. Therefore it doesn't make a difference in the cost comparison.

<u>Investment costs</u>
The investment costs are the direct construction costs and all the other costs that will have to be made to complete the structure. These other costs will be determined by adding percentages of the construction costs. These costs are shown in appendix M. An overview of the total investment costs is shown in the table on the next page. The construction costs for the entrances are determined at €1.500.000,-.

<u>Exploitation costs</u>
The exploitation costs are all the costs that have to be made to keep the garage up and running during the operation stage. An overview of these costs are shown in the table below.

Exploitation costs per year	
Description	**Total**
Management en administration	€ 100.000,00
Surveillance	€ 288.000,00
Public facilities	€ 90.000,00
Daily maintenance	€ 96.000,00
Insurance and tax	€ 70.000,00
Reservations	€ 80.000,00
Other / unforeseen	€ 40.000,00
Total exploitation costs per year	**€ 764.000,00**

Investment costs Floating construction	
Description	**Total**
Preparation works	€ 10.000,00
Temporary works	€ 260.000,00
Ground works	€ 1.928.110,00
Foundation works	€ 1.072.600,00
Flexbase floor	€ 4.666.935,22
Concrete works	€ 7.498.030,00
Immersion of garage element	€ 85.000,00
Construction costs entrances	€ 1.500.000,00
Other	€ 60.000,00
Total construction costs	€ 17.080.675,22
To be specified later (10%)	€ 1.708.067,52
Total direct costs	**€ 18.788.742,74**
One-time costs / execution costs (10%)	€ 1.878.874,27
General costs / profit and risks (10%)	€ 1.878.874,27
Total indirect costs	€ 3.757.748,55
Unforeseen (10%)	€ 2.254.649,13
Engineering, overhead (20%)	€ 4.509.298,26
Total investment costs	**€ 29.310.438,68**

Traditional construction method

Investment costs

The same was done for the traditional construction method. The total investment costs are shown in the table on the next page. This immediately shows that the total investment costs are higher for the traditional construction for the same structure with the same amount of parking spaces. This is explained in chapter 10.3.

See appendix M for the complete calculations on costs

Investment costs Traditional construction	
Description	**Total**
Preparation works	€ 10.000,00
Temporary works	€ 1.974.400,00
Ground works	€ 1.394.097,50
Foundation works	€ 1.912.500,00
Concrete works	€ 12.427.980,00
Construction costs entrances	€ 1.500.000,00
Other	€ 60.000,00
Total construction costs	€ 19.278.977,50
To be specified later (10%)	€ 1.927.897,75
Total direct costs	**€ 21.206.875,25**
One-time costs / execution costs (10%)	€ 2.120.687,52
General costs / profit and risks (10%)	€ 2.120.687,52
Total indirect costs	€ 4.241.375,05
Unforeseen (10%)	€ 2.544.825,00
Engineering, overhead (20%)	€ 5.089.650,00
Total investment costs	**€ 33.082.725,30**

10.2 Benefits

The benefits consist out of the expected return over the entire exploitation stage. These benefits come from two sources, namely:

i. Parking subscriptions (citizens of Amsterdam and companies);
ii. Parking fees (visitors).

Parking subscriptions cost €30,- a month for citizens of Amsterdam and €48,- for the surrounding companies. The parking tariff in this area is €5,- an hour.

The city of Amsterdam has research the following situation for the Oosterdok parking garage:

i. 70% of the benefits come from subscriptions (67% citizens, 33% companies);
ii. 30% of the benefits come from parking fees.

The benefits from subscriptions are easy to calculate. It isn't that easy for the benefits from parking fees, because we simply don't know how many visitors will park in the Oosterdok parking garage. The city of Amsterdam has research several scenario's and came to the conclusion that the parking spaces will be occupied 5,3 hours a day on average. Of course, this is an educated guess and the future will tell whether or not this guess was a good one.

Double use
When calculating the benefits, 'double use' is a custom method. Visitors and companies will use the parking garage the most during office hours. Citizens of Amsterdam will use it most outside office hours. Therefore, not 225 parking spaces are used in the calculations for visitors (30% of 750) but 400.

The benefits per year are:

Subscriptions citizens (352 parking places):	€ 126.720,-
Subscriptions companies (173 parking spaces):	€ 99.648,-
Visitors (400 parking spaces):	€ 3.869.000,-

See appendix N for the complete calculations on benefits

10.3 Economic analysis

Now that the costs and benefits are known. The economic feasibility can be researched. First the costs and benefits will be reviewed.

Costs
Only one conclusion can be drawn from the data in chapter 10.1: there is a very good possibility that constructing a parking garage with the floating construction method costs less than with the traditional method. At least, when using the design that is used in this case study.

The total investment per parking space is (for the design with 750 parking places):

Traditional construction method:	€ 44.110,-
Floating construction method:	€ 39.080,-

This shows that the investment per parking space is 11,5% less with the floating construction method. Appendix M shows why the floating construction method is cheaper in this situation. A brief explanation is gives in the table on the next page. It first shows the costs that are saved when using the floating construction method instead of the traditional method. Then it shows the extra costs that will have to be made when using this construction method. The difference between these costs is positive and shows that the floating constructing method costs less. The most important expenses will be explained.

Construction pit
The largest part of the saved costs consist out of, as expected, the costs for the construction pit. These include the costs for the temporary sheet piling, the underwater concrete and the grout anchors. This is a total of € 6.124.960,-.

Floor of the parking garage
An important difference between the two construction methods is the floor of the parking garage. For the temporary construction method this is a simple concrete slab. For the floating construction method however, this is a complex sandwich construction and the costs are almost twice as high. It might still seem that the costs for the Flexbase floor are low, but because the needed EPS is also used as formwork, extra formwork for the complex beam grid isn't needed. This keeps the construction costs low.

Temporary sheet piling floating construction method
The sheet piling that is used for the floating construction method is used to keep the quay wall stable during construction. Note that these costs could not be necessary in another situation.

Extra costs concrete works
When using the floating construction method, the walls and columns are constructed on a floating floor. These means extra measures to make sure the walls and columns are level. Appendix M4

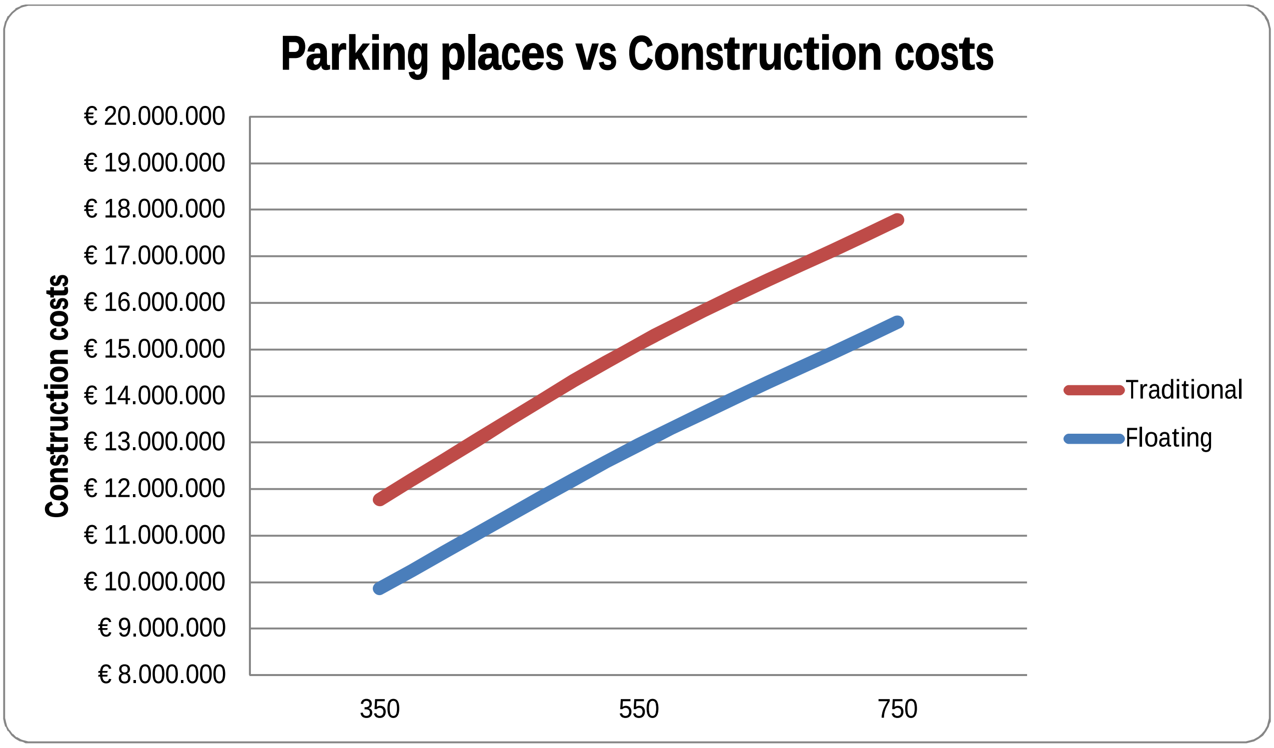
shows the detailed costs of constructing these walls. It also shows that 30% of the costs consists of labour costs. Based on this information, an extra team of two workers is added when pouring the walls. This adds €30,- per m^3 of concrete. These are the extra costs for constructing the walls on the Flexbase floor.

Costs saved with floating construction method	
Temporary works	
Temporary sheet piling, including anchors	€ 1.974.400,00
Foundation works	
Grout anchors floor	€ 1.912.500,00
Concrete works	
Underwater concrete	€ 2.238.060,00
Concrete floor parking garage	€ 2.745.050,00
Total	**€ 8.870.010,00**
Extra costs floating construction method	
Temporary works	
Temporary sheet piling, including anchors	€ 205.000,00
Breakwater	€ 30.000,00
Extra measures positioning Flexbase floor	€ 25.000,00
Ground works	
Extra costs ground works	€ 534.012,00
Foundation works	
GEWI-anchors	€ 872.600,00
Concrete tiles	€ 200.000,00
Flexbase floor	
Flexbase floor	€ 4.666.935,00
Concrete works	
Extra costs	€ 53.160,00
Immersion garage element	
Costs for immersing the element	€ 85.000,00
Total	**€ 6.671.708,00**
Difference	**€ 2.198.302,00**

Construction costs vs parking places

This shows that the floating construction method costs less for this design, with 750 parking places. The construction costs were also determined for a design with 350 and 550 parking places. The design with 350 parking places is the original design from the feasibility study conducted by the city of Amsterdam. The design with 550 parking places lays exactly in between these two designs. The graph on the next page shows the relation between the number of parking places and the construction costs. This shows that the floating construction method costs less in all situations.

The design with 350 parking places costs € 9.861.000,- when using the floating construction method and € 11.774.000,- when using the traditional construction method. After that, the construction costs increase linear with approximately € 14.300 per parking space for the floating construction method and € 15.010 per parking space for the traditional construction method. That is, in this situation.



Benefits

The conclusion that can be drawn from chapter 10.2 is that the benefits from parking fees are much larger than from subscriptions. This is probably the most significant reason why the parking garage in the Oosterdok wasn't seen as feasible by the city of Amsterdam.

Financial feasibility

This chapter will answer two question, namely;

i. Is the parking garage financially feasible for the situation in the case study;
ii. For which parking tariff is the floating construction method financially feasible.

Financial feasibility case study

The total investment for the city of Amsterdam is € 29.310.438,-. To be able to invest this sum of money, the city takes a credit loan with an interest rate of 10% and a lending term of 40 years. This amounts to yearly payments of € 2.720.000,-.

The benefits are € 4.095.368,- These benefits will be expected to grow with a rate of 2,5% a year. In the first year the occupation rate is believed to be 80%, the second year 90% and from then on 100%. Also the exploitation costs are calculated considering an annual inflation of 2,5%.

The installations will be replaced after 15 and 30 years. The reservations of € 85.000,- should be enough to account for these costs.

With this input the net present value is calculated in appendix N. The discount rate is 5%.

The calculations show that the net present value of the parking garage is **€ 20.540.348,82.** This means that the parking garage is profitable. While the feasibility study showed that a parking garage of 350 spaces, constructed with the traditional method, wasn't financially feasible. The reason for this is twofold:

i. The investment per parking space is less for the floating construction method;
ii. The investment per parking space is less because of the larger parking garage (750 spaces instead of 350).

Financial feasibility floating construction method

In the calculations to determine the financial feasibility of the construction method the entire investment is done during the construction stage.

All the benefits come from parking fees and the daily occupation rate is 5,3 hours per parking space. The benefits will be expected to grow with a rate of 2,5% a year. In the first year the occupation rate is believed to be 80%, the second year 90% and from then on 100%. This means that the total benefits are 750 (parking spaces) * 5,3 * 365 * parking tariff per hour.

The exploitation costs are calculated considering an annual inflation of 2,5%.

The installations will be replaced after 15 and 30 years. The reservations of € 85.000,- should be enough to account for these costs.

With this input the net present value is calculated in appendix N. The discount rate is 5%.

Conclusion

The calculations show that the floating construction method becomes profitable with an hourly parking tariff above **€ 1,46**. Note that this only counts for this situation and design (750 parking places).

When the complete investment has to be borrowed against an interest rate of 10% and a duration maturity of 40 years, the floating construction method becomes profitable with an hourly parking tariff above € 2,16.

Part II Conclusion and Recommendations

11. Conclusions

11.1 General

i. Based on the design in the case study, the floating construction method seems to be technically feasible;
ii. Based on the case study, the floating construction method appears to be a cheaper construction method than the traditional construction method (cut-and-cover);
iii. When all the benefits come from parking fees, the underwater parking garage in the case study, constructed with the floating construction method, is financially feasible for an hourly parking tariff of € 2,16;
iv. A parking tariff of €2,16 and higher is very common in larger cities. This means that the floating construction method could be a feasible construction method for a large number of cities. Note that this can only be said for the case when the boundary conditions are more or less similar to the ones in the case study.

11.2 Construction method

i. The construction method is cheaper than the traditional method because it doesn't need a temporary construction pit. It does need a more expensive (Flexbase) floor. But these costs are lower than the costs for a construction pit;
ii. It seems to be technically feasible to construct a Flexbase floor without the first beam grid. This means less costs for EPS, concrete and excavation of soil. This saves €1.244.131,- for this design (7,2% of total construction costs);
iii. When using grout anchors, it doesn't matter if the EPS distorts after the element is immersed. This means a lower grade of EPS can be used. In this design EPS250 instead of EPS500. This saves 50% in costs for EPS;
iv. It costs less to keep the element immersed with grout anchors (GEWI anchors) than with ballast concrete, 2,17€/kN compared to 6,00€/kN. These grout anchors are drilled through the Flexbase floor from inside the garage element when it is immersed;
v. The entrances for cars and pedestrians will have to be constructed separately;
vi. The Flexbase floor will have to be constructed when waves are low. In this case study waves of 0,98m could be expected. This is considered to be too high. Breakwaters in the form of pontoons or a barge would be necessary;
vii. In this case study, the location of the garage is close to the quay walls at some points. Temporary measures in the form of sheet piling are needed to keep the quay wall stable. The floating construction method could become financially unstable at locations where the garage is completely surrounded by unstable quay walls;
viii. The immersion stage is leading in the structural calculations for the Flexbase floor. The Flexbase floor is dimensioned for the loads it has to withstand during this stage;
ix. The draught during construction has a maximum of 6,05 meters. The water has to be deep enough for this draught. The conclusion can be drawn that the floating construction method becomes even more inexpensive in deep water, because excavation of soil, and measures to keep the surrounding soil stable will not be necessary (>10 meters);
x. The risks of settlement and damaging underground infrastructure because of driving sheet piles is almost completely eliminated with the floating construction method.

12. Recommendations

12.1 General

i. In general, further research is recommended overall. This construction method seems to be feasible and cheaper than existing methods;

ii. It would be interesting to continue the research with a floating parking garage or another large floating structure. This case study has reviewed an immersed parking garage, but this construction method could also be used for partially submerged structures. Part III of this thesis reviews this option further;

iii. It would be interesting to know what the effect is of a different surrounding to the technical and financial feasibility (e.g. depth of water, subsoil, quay walls);

iv. The immersion process is only reviewed briefly in this thesis. This should be researched more thoroughly;

v. This report mentioned the risks of traditional construction methods that are not present with the floating construction method. It is however necessary to know which risks are present when using the floating construction method. This needs further research.

13. Further research

During the thesis, certain parameters where unknown that could change the technical and financial feasibility of the construction method. Also, certain other aspects have come to the surface that should be researched to know more about the feasibility of this construction method. Not only in this particular case but also at other locations. These research possibilities are mentioned below.

Parameters reinforced EPS
Further research is needed on the glass fibre reinforced EPS to determine the exact E-modulus and tension strength. The distortions of the EPS floor are important during the construction of the concrete floor. When the EPS floor is rigid enough, a beam grid will not be needed. This saves time and money. To determine these parameters, test will have to be done.

Pouring concrete on EPS
It is recommended to conduct small scale tests with pouring the concrete floor directly on the reinforced EPS. It is important to know whether or not the displacement and inclination of the EPS floor is small enough to keep the concrete from flowing.

Deformation of EPS due to high loads
EPS has never been used for this purpose. Namely for heavy submerged structures. It is therefore important to know how EPS behaves when it has to withstand a high hydrostatic load for a long period of time (>50 years). This determines the type of EPS that has to be used. This is relevant for the floating concepts.

Effect of creep and shrinkage
The effect of creep and shrinkage of the floor on top of the beam grid should be researched. Shrinkage of the floor during hardening could cause cracks because the floors deformation is restrained by the beam grid. It is therefore important to know whether the floor could be poured in one go and how much reinforcement is needed.

Deformations during construction
More research is recommended on deformations during construction. In the calculations, the starting point of every stage is straight walls and floors. But when, for instance, a floor or wall hardened in a deformed state in the previous stage, this should be the starting point of the next stage. This is very labour intensive and wasn't reviewed during this research.

Allowable wave heights
Research is needed on the maximum allowable height of waves during construction. This determines for a large part where this construction method is applicable or whether or not a breakwater is needed.

Part III Research opportunity: Floating structures

14. Research opportunity

This research was focused on constructing an underwater parking garage with the floating construction method. But this method could also be used for floating or partially submerged structures.

Instead of immersing the garage element, it could be used as a floating platform to construct an office building, apartments or a parking garage on. The garage element, used in this thesis, has a freeboard of 3,00 meters and a buoyancy force of 270.000kN when finished. This is enough to construct a small office building or an extra parking layer on top. This is shown in figure 68 and 69.

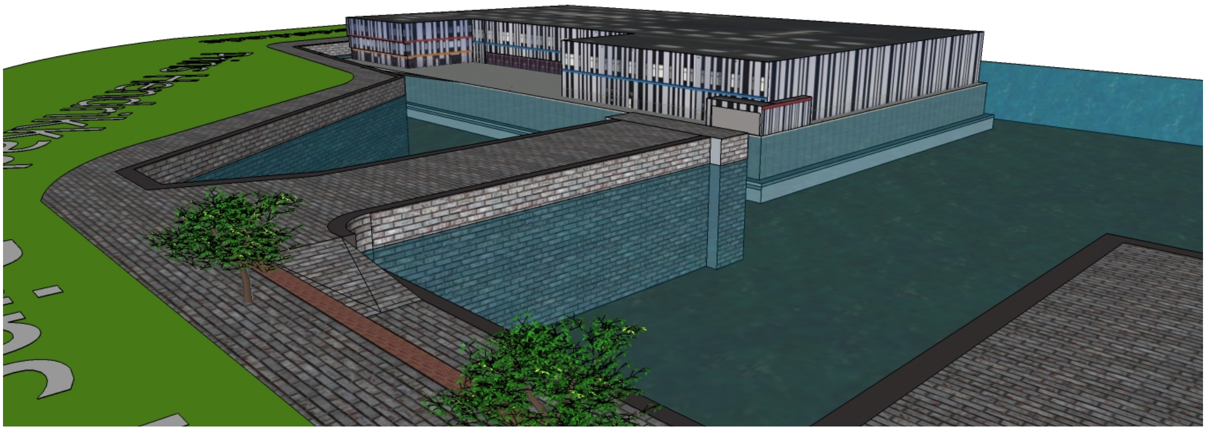


Figure 68: Floating office building on top of parking garage element

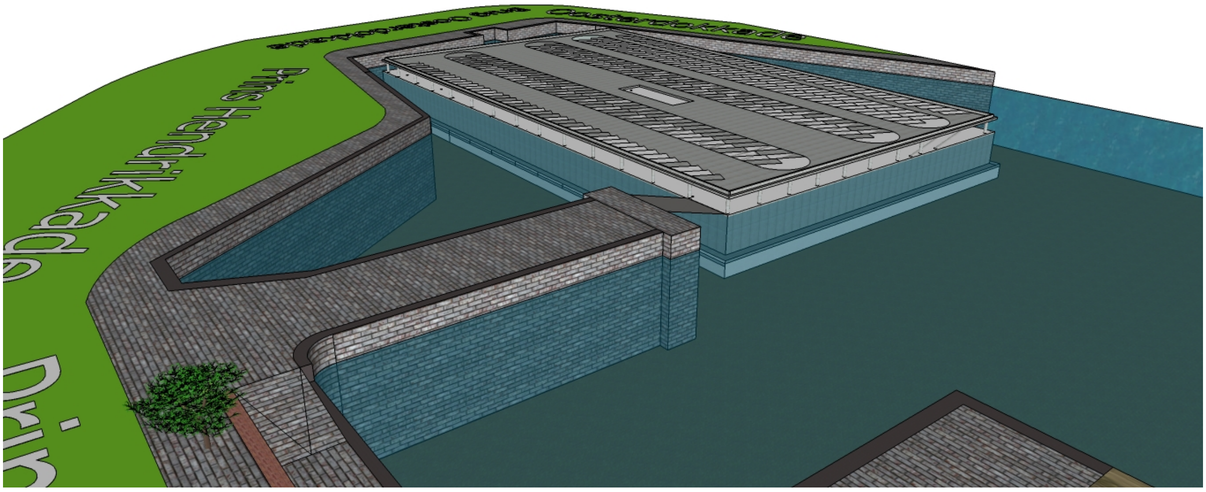


Figure 69: Floating parking garage, four floors

Depending on the depth of the water, the element could be designed to have a smaller or larger buoyancy force. The biggest advantages of this alternative are:

i. Adding extra value by creating construction space in the city centre;
ii. Immersion of element and excavation of soil is not needed.

Of course, the structure doesn't have to be a parking garage, it could be any other structure.

14.1 Costs

To get an idea of the financial feasibility of this method, a floating parking garage is considered. On top of the parking element, an extra floor is constructed. This is shown in figure 69. This doubles the total amount of parking spaces to 1.500. Therefore, the benefits should also double. The construction costs, however, are lower.

The reason is the fact that there are no costs for immersing the element, constructing a foundation and excavating soil (when the water is deep enough). The construction costs are shown in appendix O and an overview of the total investment is given in the table below. This shows that the total investment is €23.412.100,- or €15.608,- per parking space. This is very low. Of course, this is a rough estimation, but is shows great possibilities.

Investment costs	
Description	**Total**
Preparation works	€ 10.000,00
Temporary works	€ 55.000,00
Flexbase floor	€ 4.666.935,00
Concrete works	€ 8.911.460,00
Other	€ 50.000,00
Total construction costs	€ 13.643.415,00
To be specified later (10%)	€ 1.364.341,50
Total direct costs	**€ 15.007.756,50**
One-time costs / execution costs (10%)	€ 1.500.775,65
General costs / profit and risks (10%)	€ 1.500.775,65
Total indirect costs	€ 3.001.551,30
Unforeseen (10%)	€ 1.800.930,70
Engineering, overhead (20%)	€ 3.601.861,40
Total investment costs	**€ 23.412.099,90**

14.2 Recommendations

A first cost estimation shows great possibilities, but this idea needs more research before a conclusion could be drawn regarding the financial and technical feasibility. The following recommendations are given for furthers studies on this subject:

i. The dynamic behaviour of the floating structure will play a significant role. The structure is subjected to wind and wave loads during its complete life-cycle. It is especially important to conduct elaborated research regarding the dynamic behaviour of the structure during the operation stage. Oscillation of the element could make individuals sick and make working in the structure unbearable;
ii. More research is needed regarding the EPS floor. The loads on the EPS floor will be significant for the entire life-cycle. Large distortions of the EPS could be catastrophic;
iii. When the structure floats, it is important to know the effects of parked cars. Where do they park? How does this change the draught and inclination of the structure? Is a parking system needed to distribute the cars evenly of the structure etc.

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Appendices

A Soil-drilling tests and parameters

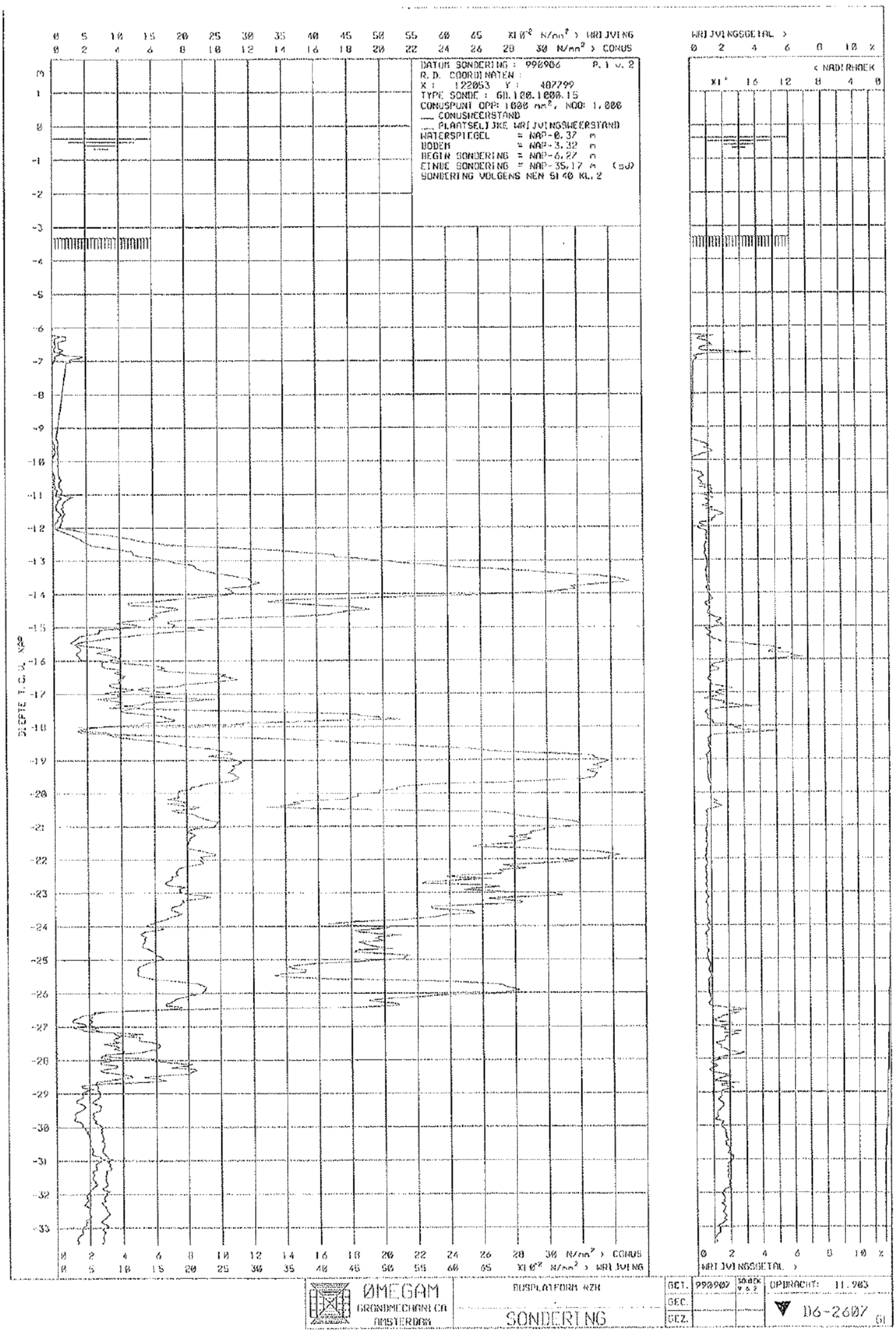
The Oosterdok Island Bridge is constructed directly near the location of the parking garage. This bridge was constructed in 2010 and soil-drilling tests were conducted for this project. These tests will not all be shown, but a representative drilling test is shown on the next page to get an idea of the soil parameters.

The complete report also mentions that, before construction began, an anchored sheet pile wall was constructed consisting out of 13,00 meter long AZ 18 sheet piles. On the side of the Prins Hendrikkade and on the side of the Oosterdok Island. In front of this wall, the permanent quay wall was constructed. This wall consists of a concrete L-wall (bottom at 0,00m NAP) on prefab piles (350 x 350 mm, bottom at -20,00m NAP).

Soil parameters

Soil type	Depth NAP (m)	Y_{sat} (kN/m^3)	q_c (Mpa)	φ (°)	C_p'	C_s'
Silty Clay	4-12	14,5	1	28	30	300
Sand	12-15	20	26	32	800	-
Silty Clay	15-16	18	8	28	30	300
Sand	16-18	19	12	34	800	-
Sand	18-27	19	24	34	800	-
Sand with clay	26-30	18	7	34	30	300
Clay	30-	16	4	30	40	240

Soil-drilling test Oosterdok



B Element buoyancy force

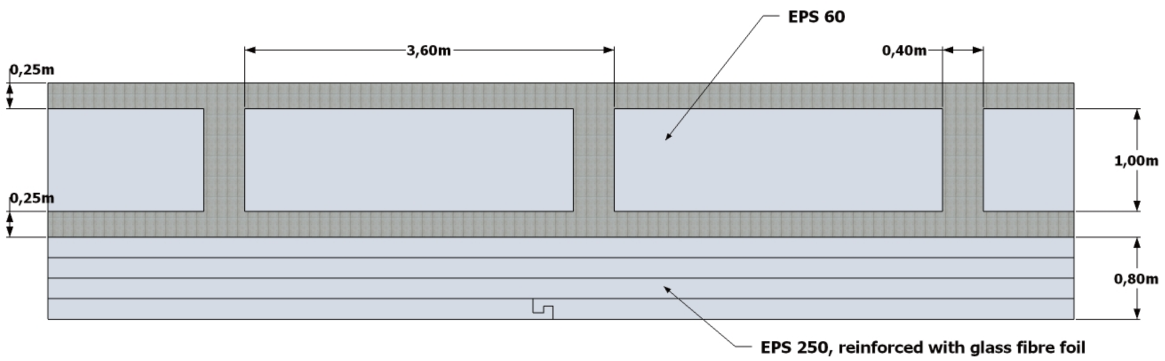
The buoyancy forces are calculated separately for the Flexbase floor and the rest of the structure. Used parameters are:

g = 10 m/s^2
ρ_w = 10 kN/m^3

Flexbase floor

Determination	Dimensions (m)	Weight density	Mass
First EPS layer	132 x 79,2 x 0,8	0,2 kN/m3	1.672 kN
Second EPS layer	119 x 70 x 1,00	0,2 kN/m3	1.666 kN
First concrete floor	132 x 79,2 x 0,25	25 kN/m3	65.340 kN
Beam grid	132 x 1,00 x 0,4 x 23 79,2 x 1,00 x 0,4 x 33	25 kN/m3 25 kN/m3	56.496 kN
Second concrete floor	132 x 79,2 x 0,25	25 kN/m3	65.340 kN
Total mass			**190.514 kN**

Volume of Flexbase floor = 24.045 m3 (l x w x h)
Buoyancy force Flexbase floor = (Volume x ρ_w) – Total mass
= (24.045 x 10kN/m3) – 190.514 = 49.937 kN
Immersion = 1,82 m
Freeboard = 0,48 m



Flexbase floor: dimensions

Garage element

Determination	Dimensions (m)	Weight density	Mass
Outer walls, first floor	130 x 2,60 x 0,6 x 2 75 x 2,60 x 0,6 x 2	25 kN/m3 25 kN/m3	10.140 kN 5.850 kN
Inner walls, first floor	128,8 x 2,60 x 0,25 105 x 2,60 x 0,25	25 kN/m3 25 kN/m3	2.093 kN 1.706 kN
Columns, first floor	104 x 0,35^2 x π x 2,60	25 kN/m3	2.601 kN
Finishing floor, first floor	128,8 x 73,8 x 0,2	25 kN/m3	47.527 kN
Ramps, first floor	57 x 4 x 0,30	25 kN/m3	1.710 kN
Intermediate floor	130 x 75 x 0,6	25 kN/m3	146.250 kN
Outer walls, second floor	130 x 2,60 x 0,6 x 2 75 x 2,60 x 0,6 x 2	25 kN/m3 25 kN/m3	10.140 kN 5.850 kN
Inner walls, second floor	128,8 x 2,60 x 0,25 105 x 2,60 x 0,25	25 kN/m3 25 kN/m3	2.093 kN 1.706 kN
Columns, second floor	104 x 0,3^2 x π x 2,60	25 kN/m3	2.434 kN

Ramp, second floor	22 x 2,60 x 0,6 22 x 10 x 0,3	25 kN/m3 25 kN/m3	429 kN 1.650 kN
Roof	130 x 75 x 0,7	25 kN/m3	170.625 kN
Total mass			**412.637 kN**

Volume of element = 63.375 m3 (l x w x h)
Buoyancy force garage element = (Volume x ρ_w) – Total mass
= (63.375 x 10kN/m3) – 412.637 = 221.113 kN

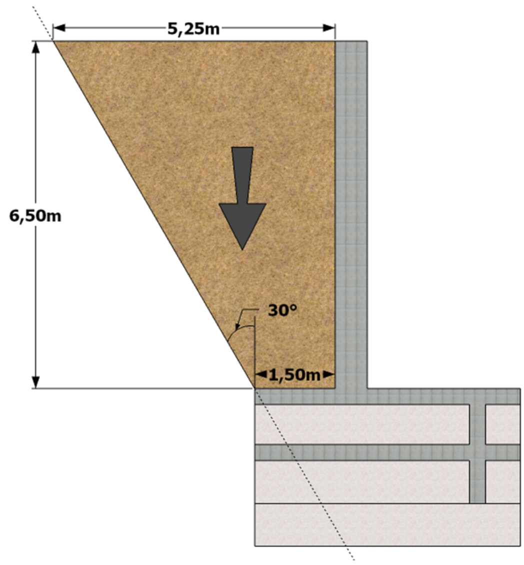
Immersion = 6,05 m
Freeboard = 2,75 m

Soil
A 0,50 meter thick layer of soil will rest on top of the garage element. However, for safety reasons, this downward load is not taken into account. The horizontal earth pressure on the Flexbase floor will be taken into account (1,50 m on the sides of the floor) and to be safe, only the soil directly on top of this floor. Y'= weight soil – weight water = 10,00 kN/m3.

To be safe, only the soil directly above the floor will be taken into account.

Volume * Y' =

((75 x 2 x 1,5 x 6,50) + (130 x 2 x 1,5 x 6,50)) x (19,00-10,00) = **35.977 kN**



Buoyancy when floating
This makes the total buoyancy of the element **271.050 kN**

The freeboard while floating is:

271.050 / (130 x 75 x 10) = **2,75 m**

Buoyancy in operation stage
The buoyancy in the operation stage is:

Buoyancy when floating – downward force of soil on Flexbase floor = **235.072 kN**

This is 24,11 kN/m2 (249.523 / (130 x 75))

Note that this calculation doesn't include safety factors! These safety factors are added in the anchor plan and immersion plan.

C Wind forces

The wind forces on the walls and roof of the garage element can cause unwanted tilt. The windforce will be calculated for the situation where the total load is the highest, namely when the outer wall has reached the highest point.

The walls will protrude 6,00 meters out of the water. Of course the long side of the wall is considered.

Because the height is smaller than 50 meters and h/b is smaller than 5, the wind load as a result of wind pressure, suction, friction and over- and underpressure is gives by the formula:

$$p_{rep} = C_{\dim} \cdot C_{index} \cdot p_w \ \ [\mathrm{kN/m^2}]$$

In which:

p_w =	extreme wind thrust	[kN/m^2]
C_{dim} =	factor for the dimensions of the structure	[-]
C_{index} =	wind type factor	[-]

Figure 8-2 shows that the Oosterdok falls in category II. Table 8-2 gives a value of 0,54kN/m^2 for p_w Table 8-3 gives a value of 0,85 for C_{dim}.. C_{index} is the wind type factor and is a factor for the wind pressure and suction. According to NEN 6702 this is 1,2. This gives a wind load of:

$$P_{rep} = 0{,}54 * 0{,}85 * 1{,}2 = 0{,}55 \text{ kN/m}^2$$

In this stage the roof hasn't been constructed yet, therefore the suction on the roof isn't taken into account. When the roof is constructed, the wall height is 4,00 meters. The wind force on the wall will become:

$$P_{rep} = 0{,}54 * 0{,}85 * 1{,}2 * 0{,}8 = 0{,}37 \text{ kN/m}^2$$

Figure 8-3 shows that the suction on the first half of the roof will become:

$$(0{,}55 / 1{,}2) * 0{,}8 = 0{,}37 \text{ kN/m}^2$$

This force will also tilt the structure. The force on the second half of the roof will counteract this force and is 0,18 kN/m2. For non-complex structures this could be done by hand, but because of the complexity of the structure this will be calculated using SCIA. Maarten Koekoek showed in his MSc thesis that SCIA gives good results. Therefore I will not try to proof this again. [M. Koekoek, 2010]

See appendix K3 for the results on tilting due to wind forces

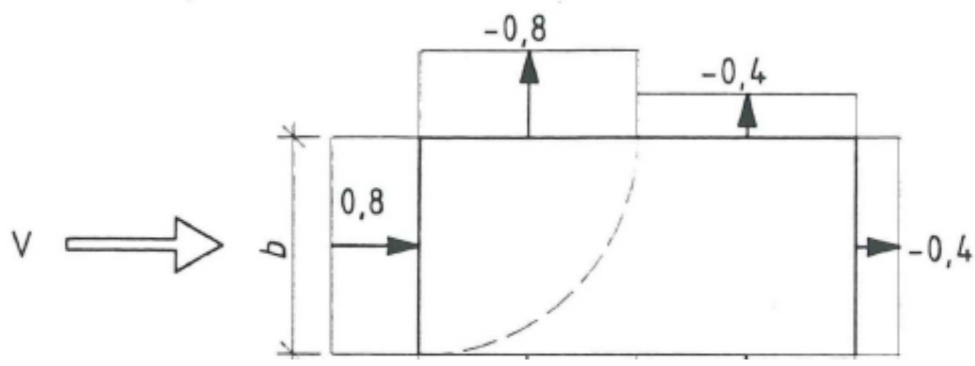


Figure 8-3 wind type factors on vertical walls

h m	p_w kN/m²					
	zone I		zone II		zone III	
	open	built-on	open	built-on	open	built-on
≤ 2	0,64	0,64	0,54	0,54	0,46	0,46
3	0,70	0,64	0,54	0,54	0,46	0,46
4	0,78	0,64	0,62	0,54	0,49	0,46
5	0,84	0,64	0,68	0,54	0,55	0,46
6	0,90	0,64	0,73	0,54	0,59	0,46

Table 8-2 extreme wind thrust values p_w in the Netherlands as a function of the height above the surrounding level *(from: NEN 6702)*

h m	b m							
	1	10	20	30	40	50	75	100
2	1,00	0,96	0,94	0,92	0,90	0,89	0,86	0,84
3	1,00	0,96	0,94	0,92	0,91	0,89	0,87	0,85
4	0,99	0,96	0,94	0,92	0,91	0,89	0,87	0,85
5	0,99	0,96	0,94	0,92	0,91	0,89	0,87	0,85
6	0,99	0,96	0,94	0,92	0,91	0,90	0,87	0,85

Table 8-3 factor for the dimensions of the structure C_{dim} *(from: NEN 6702)*

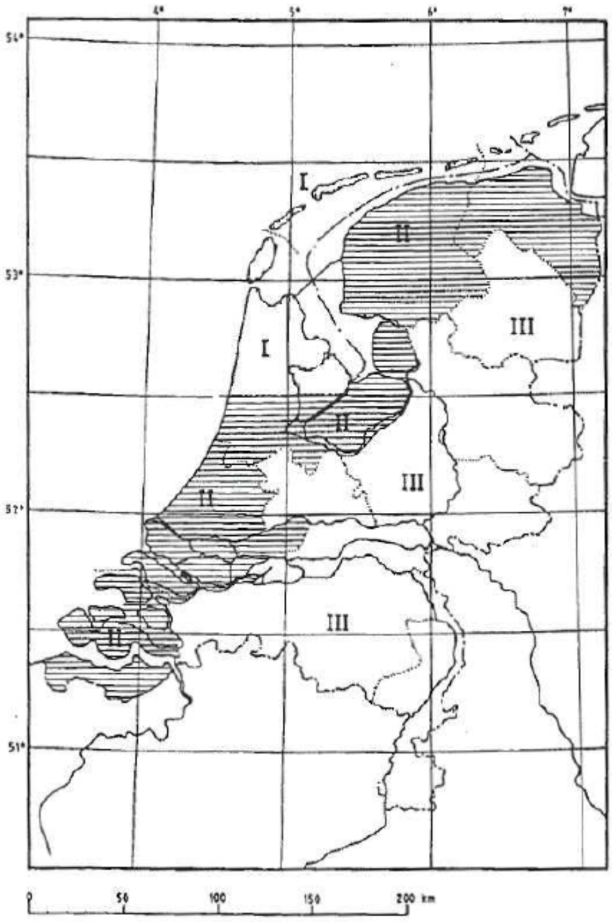


Figure 8-2 wind thrust zones in the Netherlands (from: NEN 6702)

D Static stability

For caisson elements, tilting around the length axis is most critical (surging) and will therefore be calculated in this appendix. The static stability will be calculated for the completed structure. The centre of gravity will be lower during construction and therefore the structure will be more stable than when it is finished.

For this calculation the element will be simplified. The columns and ramps will not be taken into account because of their relatively low weight compared to the rest of the structure (approximately 1,5%). The cross-section of the element is shown in figure D-1. The weight of the EPS will also not be taken into account. [CT3330, 2009]

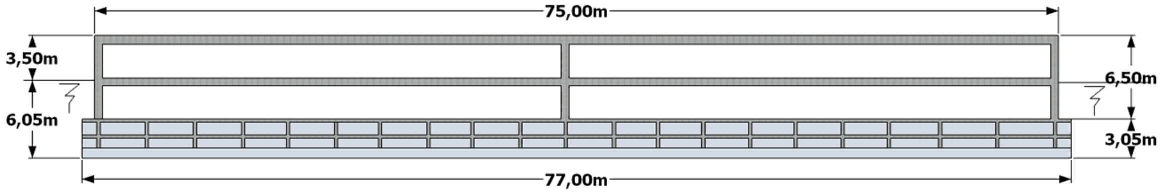


Figure D-1: Cross-section garage element (preliminary design)

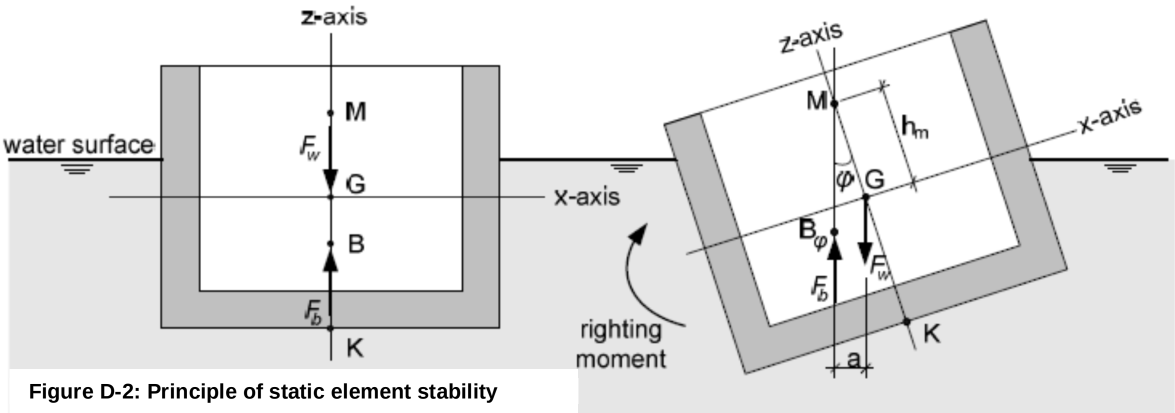


Figure D-2: Principle of static element stability

First the position of the centre of gravity is calculated. Therefore, the distance between K and G is calculated (see figure D-2).

KG = (Weight first beam grid * vertical distance from K + Weight first floor * Dist. K + Weight second beam grid * Dist. K + Weight second floor * Dist. K + Weight outer walls * Dist. K + Weight inner walls * Dist. K + Weight intermediate floor * Dist. K + Weight finishing floor * Dist. K + Weight Roof * Dist. K) / Total weight

KG = (26.620 * 1,175 + 64.838 * 1,675 + 35.494 * 2,3 + 64.838 * 2,925 + 31.980 * 5,95 + 4.186 * 5,95 + 146.250 * 5,95 + 47.527 * 3,15 + 170.625 * 9.15) / 592.358

KG = 5,41m

The centre of buoyancy is the distance between K and B. The distance KB is 0,5 * draught. The draught is 6,05m.

KB = 6,05 / 2

KB = 3,025m

The moment of inertia of the rectangular element at the surface is:

I = $1/12 * L * b^3$

I = $1/12 * 130 * 75^3$

I = $457 * 10^4$

The volume of displaced water is:

V = $(133 * 77 * 3{,}05) + (130 * 75 * 3{,}00) = 60.485 m^3$

BM = I / V

BM = $457 * 10^4 / 60.485$

BM = 75,56m

The metacentre height h_b is:

h_m = KB + BM – KG

h_m = 3,025 + 75,56 – 5,41

h_m = 73,18 > 0,5m → OK

This outcome was expected because of the ratio between the length and the height of the element.

Stability during immersion

As described in the Hydraulic structures reader [CT3330, 2009] the floating elements owes the stability to its large moment of inertia. When the element is immersed however, the element doesn't have a plane that intersects the waterline and the moment of inertia is zero. Stability is only achieved if the buoyancy point is above the centre of gravity. The remaining needed stability is achieved by lowering the element by cranes which position the element at the final location.

E Wind waves

The parameters of the wind waves are especially important for the dynamic stability of the element and the vertical load on the Flexbase floor (hogging and sagging). In a lesser extent for the horizontal load, because this load will probably be insignificantly small.

The wind waves are calculated using Bretschneider. The significant wave height and wave period can be estimated using the following formulas:

$$\tilde{H} = 0.283 \tanh(0.53\tilde{d}^{0.75}) \tanh\left(\frac{0.0125\tilde{F}^{0.42}}{\tanh(0.53\tilde{d}^{0.75})}\right)$$

$$\tilde{T} = 7.54 \tanh(0.833\tilde{d}^{0.375}) \tanh\left(\frac{0.077\tilde{F}^{0.25}}{\tanh(0.833\tilde{d}^{0.375})}\right)$$

In which:

$\tilde{H} = \dfrac{gH_s}{U^2}$

$\tilde{T} = \dfrac{gT_p}{U}$

$\tilde{F} = \dfrac{gF}{U^2}$

F = fetch

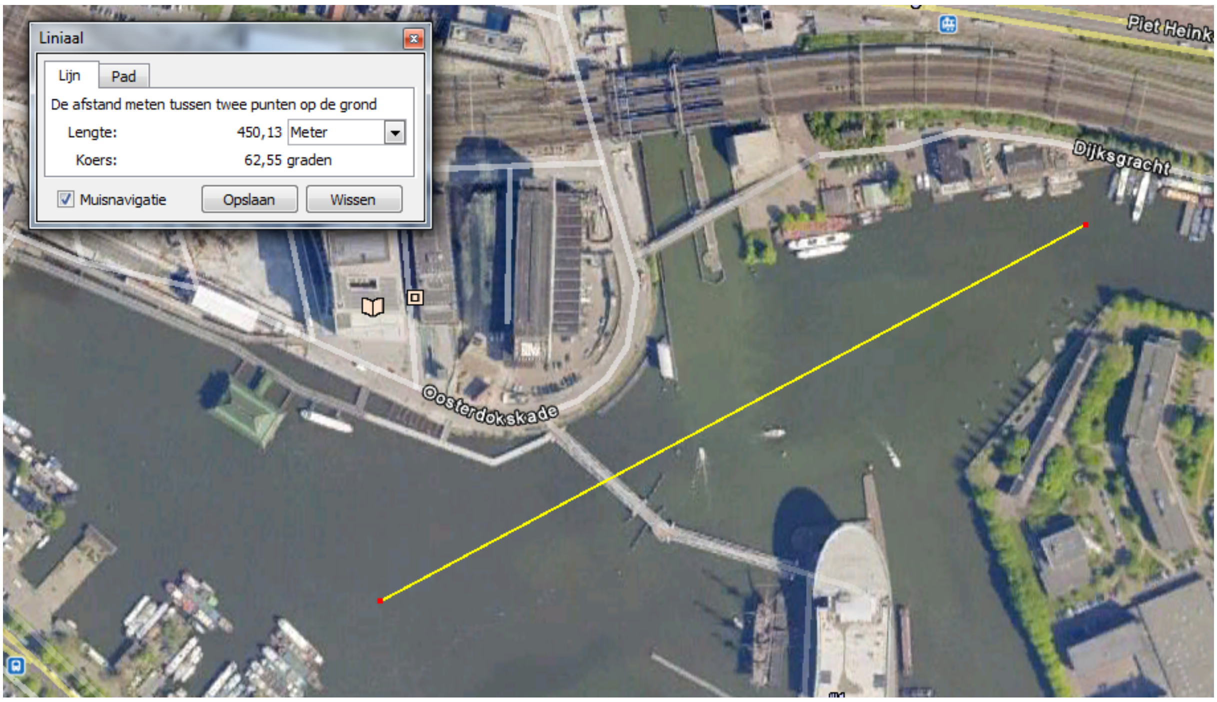
U = wind velocity at an altitude of 10 m

$\tilde{d} = \dfrac{gd}{U^2}$

d = water depth

T_p = peak wave period (= most common period)

The maximum fetch in the Oosterdok is 450 meters as shown in the figure. A maximum velocity of 25 m/s is assumed. This is the average wind speed over 10 minutes during a severe storm (wind force 10 on the Beaufort scale). This is a very high wind speed for this location but I want to be on the safe side. The depth of the Oosterdok is 4,50 meters.



The input is:

F = 450m
U = 25 m/s
d = 4,50m

This gives: **H_s = 0,45m** and T_s = 2,5s

H_s is the average wave height of the highest 1/3 of waves. This means that this wave occurs regularly. Therefore we need the design wave height which is H_d. Assuming that the waves are Rayleigh distributed, the probability that the design wave is exceeded during a storm with N waves is:

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

Assuming a wave period of 2,5s and a storm length of 2 hours we get N = 2880. Because the possibility that this storm will occur is low (once every five years) and the Flexbase floor will probably be constructed outside of the storm season (October - April), we allow an exceedance probability of 0,2.

This gives **H_d = 0,98m**

The wave length is found using the linear wave theory:

$$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi h}{L}\right)$$

After a few iterations it gives: L = 10m

To sum up:

H_d	=	0,98m
T_s	=	2,5s
L	=	10m

Loads on structure

It is important to know the vertical loads these waves induce. As mentioned before, the vertical load will cause hogging and sagging. In other words, the waves will carry the Flexbase floor. The wave tops and valleys are modelled as loads that are equal to the mass of these tops and valleys.

The loads are schematized as distributed loads as shown in the figure below. This load is equal to 0,955 * A * γ_w as was shown in the thesis of Maarten Kuiper.

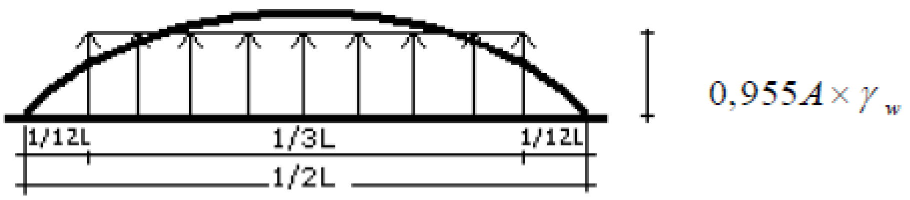


Figure E-1: Vertical wave load from sinus-load to distributed load [Maarten Kuiper, 2006]

We consider the highest wave, which is H_d with the corresponding wave period and wave length:

H_d	=	0,98m
T_d	=	4,0 s
L	=	21,5 m

The loads will alternate like is shown in figure E-2 and work over a length of 1/3 * 21,5 = 7,17m

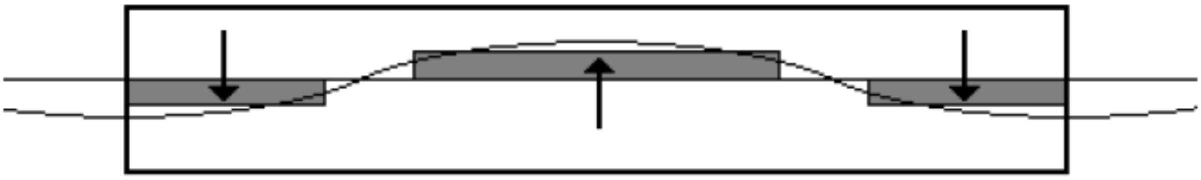


Figure E-2: Schematization of alternating vertical wave loads [Maarten Kuiper, 2006]

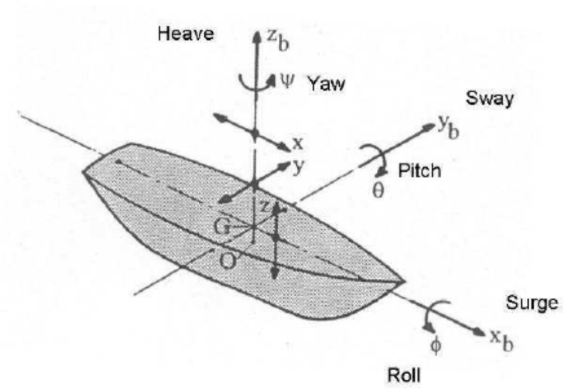
With A is 0,5 * 0,98 = 0,49m, the distributed load is:

0,955 * 0,49 * 10 = **4,68 kN/m^2**

This load works on the floor during construction and this will be modelled in SCIA. Extra measures will have to be taken if this load causes unwanted deformations or cracks (i.e. breakwaters).

F Dynamic stability

Because the element is placed in waves it can start to sway if the eigenperiod of the element is the same as the period of the waves (2,5 seconds). To prevent this from happening, the eigenperiod should be much larger. The figure below shows the six possible movements of the element, namely:



i. Yaw;
ii. Pitch;
iii. Roll;
iv. Heave;
v. Sway;
vi. Surge.

In practice the rule of thumb in chapter 8 is used for swaying. This showed that swaying will not be an issue.

Besides this check the eigenperiod of the element is calculated. As said before, this eigenperiod should be much larger than 2,5 seconds (T_s).

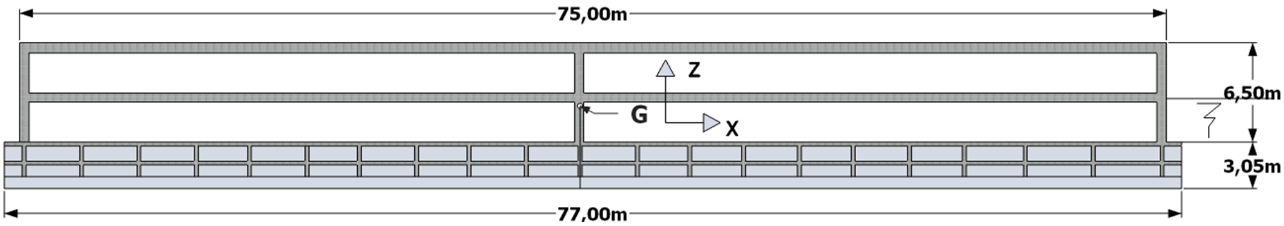


Figure F-1: Simplified cross-section dynamic stability

Note: for dimensions of the floor see page 99.

When the additional water mass and damping are ignored, the natural oscillation period of an element becomes:

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

In which:

$T_0 =$	natural oscillation period	[s]
$h_m =$	metacentric height	[m]
$g =$	gravitational constant	[m/s^2]
$j =$	polar inertia radius of the element	[m]

The polar inertia radius *j* is given by:

$$j = \sqrt{\frac{I_{polar}}{A}}$$

I_{polar} is $I_{xx} + I_{zz}$. I_{xx} is the polar moment of inertia around the z-axis and I_{zz} is the polar moment of inertia around the x-axis. Figure F-1 shows the simplified cross-section, the x-axis and the z-axis. I_{xx} and I_{zz} are both calculated in relation to the centre of gravity which is also shown in this figure. [CT3330, 2009]

The main dimensions of the parking garage are:

Width	75,00 m
Length	130,00 m
Intermediate floor	600 mm
Roof	700 mm
Outer walls	600 mm
Inner wall	400 mm
Free height above floor:	2,60 m

Using these dimension the outcome is the following:

I_{zz} = $1.556,21\ m^4$
I_{xx} = $85.712,45\ m^4$

I_{polar} = $87.268,66\ m^4$

A = $169,6\ m^2$

j = sqrt(87.268,66 / 169,6) = 22,68 m

For a precise calculation also the headwalls are taken into account.

$I_{zz,h}$ = $1.858,56\ m^4$
$I_{xx,h}$ = $228.515,60\ m^4$

I_{polar} = $230.374,16\ m^4$

A = $487,5\ m^2$

j_h = 21,74 m

The values of j and j_h will have to be averaged:

$j_{resulting}$ = (j (130 – 2 * thickness outer wall) + j_h (2 * outer wall)) / 130

$j_{resulting}$ = (22,68 (130 – 2 * 0.6) + 21,74 (2 * 0,6)) / 130

$j_{resulting}$ = 22,67 m

h_m = 73,18 m

T_o = (2 * π * 22,67) / sqrt(73,18 * 9,81)

T_0 = 5,32 s >> 2,5 → OK

The eigenperiod is 5,32 seconds which is much larger than the period of the wind waves in the Oosterdok. The period of the waves during a storm however is higher, approximately 4,0 seconds. This is still less than the eigenperiod of the element.

As shown in the thesis of Maarten Kuiper, T comes about 1,2 times bigger when adding the additional water mass. Also, the columns, part of the inner walls, the ramps and the EPS layers aren't taken into account. Therefore, in reality the eigenperiod of the element will even be higher. It can be concluded that the element is dynamically stable and the element will not be brought into natural oscillation.

The thesis of Maarten Kuiper also showed that the eigenperiod of the long side is smaller than the short side. The moment of inertia of the long side is larger, but the metacentre height is also much larger. Therefore the same calculation is done for the long side. In this case the element rotates around the x-axis and therefore $I_{polar} = I_{zz} + I_{yy}$.

For the long side of the element:

h_m = 227 m

A = 294 m^2

I_{zz} = 3.124 m^4

I_{yy} = 397.150 m^4

I_{polar} = 400.274 m^4

j = 36,88 m

Headwalls:

$I_{zz,h}$ = 3.221,51 m^4
$I_{yy,h}$ = 1.190.041,67 m^4

I_{polar} = 1.193.263,18 m^4

A = 676 m^2

j_h = 42,01 m

After averaging:

$j_{resulting}$ = 36,90 m

T_0 = 4,91 s >> 2,5 s → OK

This shows what was expected. The I_{polar} is indeed larger but the height of the metacentre also is. Therefore the eigenperiod is a bit smaller but still on the safe side.

G Soil deformations

Settlements

When a layer of soil is loaded or unloaded it will act as a spring, this is called settlement. During the construction stage the soil layers deeper than -14,00m NAP will be unloaded during excavation and loaded when the caisson is immersed. The corresponding settlement will be calculated using Koppejan. Settlement is important when the garage element is immersed, because this could cause unwanted stresses in the structure. The initial soil parameters are shown in the table below. In the excavated situation C_p' and C_s' should be multiplied by 3. [Maarten Faber, 2008]

Soil type	Depth NAP (m)	γ_{sat} (kN/m^3)	q_c (Mpa)	φ (°)	C_p'	C_s'
Silty Clay	4-12	14,5	1	28	30	300
Sand	12-14	20	26	32	800	-
Silty Clay	14-16	18	8	28	30	300
Sand	16-18	19	12	34	800	-
Sand	19-27	19	24	34	800	-
Sand with clay	26-30	18	7	34	30	300
Clay	30-	16	4	30	40	240

Koppejan distinguishes two types of settlement, namely w1 and w2. W1 is the initial direct settlement and w2 is the secondary settlement. The formulas to be used are:

$$w_1 = \sum_{j=0}^{j=n} \frac{1}{C'_{p;j}} h_j \times \ln \frac{\sigma'_{v;z;0} + \Delta\sigma'_{v;z}}{\sigma'_{v;z;0}}$$

In which:

w_1 =	Primary settlement	[m]
$C_{p;j}$' =	Primary compression coefficient of soil layer j	[-]
h_j =	Thickness of layer j	[m]
$\sigma'_{v;z;0}$ =	Vertical effective stress before loading for the bottom of a layer	[kPa]
$\Delta\sigma'_{v;z}$ =	Vertical effective stress increase for the middle of a layer	[kPa]

$$w_2 = \sum_{j=0}^{j=n} \frac{1}{C'_{s;j}} h_j \times \log \frac{t}{t_0} \times \ln \frac{\sigma'_{v;z;0} + \Delta\sigma'_{v;z}}{\sigma'_{v;z;0}}$$

In which:

w_2 =	Secundary settlement	[m]
$C_{s;j}$' =	Secundary compression coefficient of soil layer j	[-]
t =	Duration in days. For the final settlement t= 10.000 days	[days]
t_0 =	time unit; t_0 = 1 day	[days]

Because of the stable sand layers, the settlement will probably not be significant. Therefore, it will not be calculated in this thesis and further research is recommended.

Stability during excavation

The depth of the Oosterdok at the location of the parking garage is 3,60 meters and the Oosterdok should keep this depth. Therefore, the soil will have to be excavated to place the garage element at a depth of 13,65m beneath the water level. Figure G-1 shows the soil that will be excavated. This figure also shows the problem: at two places the excavation activities come dangerously close to the quay wall. Figure G-2 shows the outline of the excavation. In the green areas it is assumed that the stability is satisfactory. In the red areas however, it will probably not be.

The drawings of the Engineering Bureau Amsterdam show that the quay wall is a concrete L-wall on steel piles with a length of 20 meters. The c.t.c. distance between these piles is 2,50 meters. I would assume that there are also raking struts, but they are not on the drawings, so they will not be taken into account. [Drawing 1, 2010]

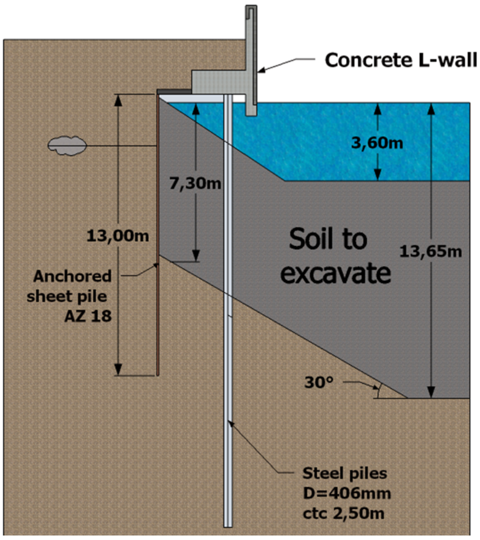


Figure G-1: Existing situation quay wall

Behind the quay wall it shows an anchored sheet pile wall of 13 meters long. This sheet pile wall is used as a water screen. The stability of these structures will have to be insured during excavation.

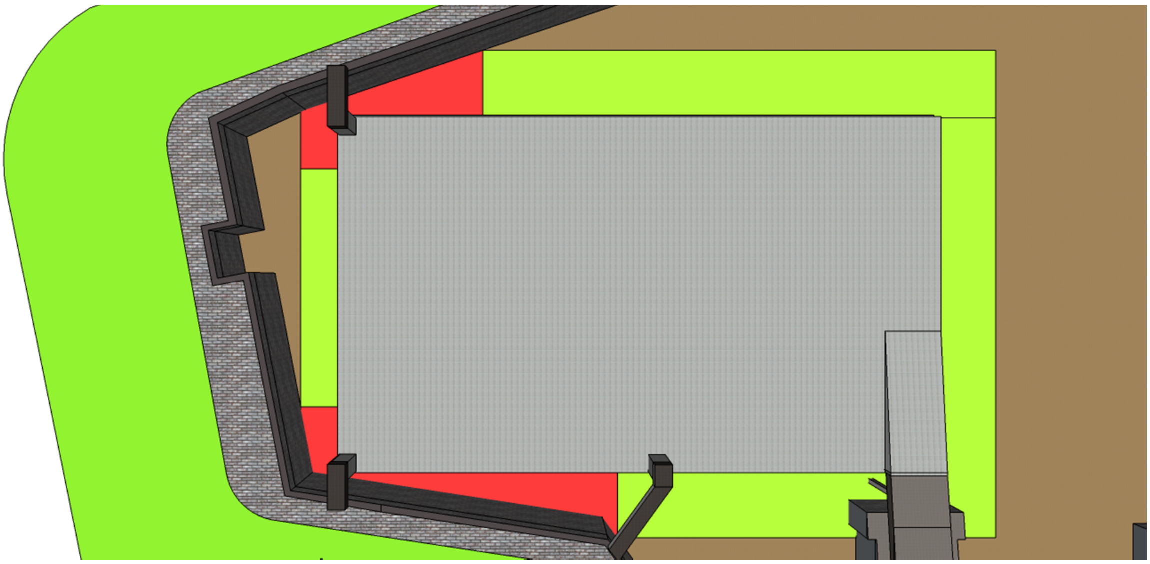


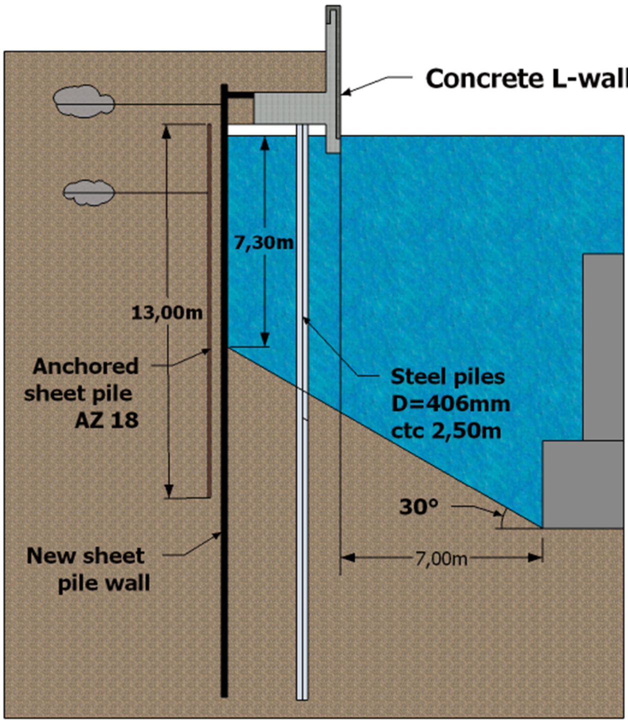
Figure G-2: Outline excavation

The solution is to place a second sheet pile wall between the quay wall and the existing screen pile wall as is shows in figure G-3. The top of the sheet pile could be coupled to the quay wall to keep it stable during excavation. The piles will lose some bearing capacity during and after excavation, but they are designed to have enough bearing capacity from only the second sand layer (-16,00m till -30,00m NAP) and this layer will not be excavated. [De Heus, 2008]

The following assumptions are made to model the situation:

i. Wave loads aren't taken into account;
ii. The existing sheet pile wall will have the same deflections as the new sheet pile wall;
iii. In the MSheet model, the existing sheet pile wall and quay wall are left out;
iv. The slope from -7,30m till -13,65m beneath the water level is replaced by a level bottom at -10,50m beneath the water level;
v. The maximum deflection of the sheet pile wall is 1/100 of the length with a maximum of 100mm.

In reality the existing sheet pile wall will help to retain the soil, therefore this is a conservative model. To keep the deflections at the top to a minimum, and with that the deflections of the quay wall, grout anchors are placed 1,00 meter from the top of the sheet pile wall. The spacing of these anchors is 2,00 meters. When taking a grout length of 5 meters and a cone resistance of 15 MPa (low value for sand), the anchor capacity will approximately be 400kN (conservative rule of thumb).



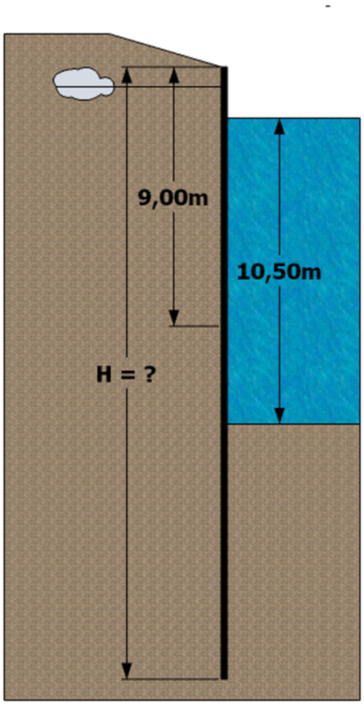


Figure G-3: A) Situation after constructing new sheet pile wall **B) Modelled situation after assumptions**

Msheet model
Msheet is a program that is usually used to calculate horizontal loaded sheet piling. The friction angles will have to be reduced as a safety measure. When working with curved slide plains, the reduction is:

$$\delta = \min(\phi - 2.5°; 27.5°) \text{ and for clay: } \delta = \phi/2$$

Msheet gives the maximal moments and deformations for the modelled situation. The maximal moments are different per type of sheet piling. The maximal deformation is 1/100 of the length with a maximum of 100mm for temporary sheet piling or sheet piling that isn't in the line of sight. For permanent sheet piling that is in the line of sight the maximal deformation is 1/200 of the length with a maximum of 50mm. [CUR 166, 2005]

Results
The situation described in the previous paragraphs is modelled in Msheet. This gives the following results:

Sheet piling	=	AZ18
Length sheet piling (H)	=	**20,00m**
Maximum deformation	=	80,4mm < 1/100 * length or 100mm → **OK**
Maximum moment	=	266,3 kNm < 432 kNm (maximum moment AZ18) → **OK**
Anchor force	=	247,7 kN < 400 kN → **OK**

The sheet piling wall will be of type AZ 18 and the planks will be 20,00m long. There will be two walls, each 60 meters long. One at the side of the Oosterdok Island and one on the side of the Prins Hendrikkade. The complete report is shown on the following pages.

1 Summary

1.1 Maxima per Stage

Stage no.	Stage name	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
1	New Stage	**80,4**	**266,3**	**-97,0**	**43,5**	**46,7**	---
Max		**80,4**	**266,3**	**-97,0**	**43,5**	**46,7**	---

1.2 Anchors and Struts

Stage name	Anchor/strut New Anchor	
	Force [kN]	State
New Stage	123,85	Elastic

2 Input Data for all Stages

2.1 General Input Data

Model	Sheet piling
Check vertical balance	Yes
Number of construction stages	1
Unit weight of water	9,81 kN/m³
Number of curves on spring characteristic	3
Unloading curve on spring characteristic	No

2.2 Sheet Piling Properties

Length	20,00 m
Level top side	0,00 m
Number of sections	1
Pr;max;point	0,00 MPa
Xi factor	0,75

Section name	From [m]	To [m]	Stiffness EI [kNm²/m']	Acting width [m]	Maximum moment [kNm/m']
AZ 18	-20,00	0,00	7,1820E+04	1,00	432,00

Section name	From [m]	To [m]	Red. factor EI [-]	Red. factor max. moment [-]	Note to reduction factor
AZ 18	-20,00	0,00	1,00	1,00	

Section name	From [m]	To [m]	Corrected stiffness EI [kNm²]	Corrected max. moment [kNm]
AZ 18	-20,00	0,00	7,1820E+04	432,00

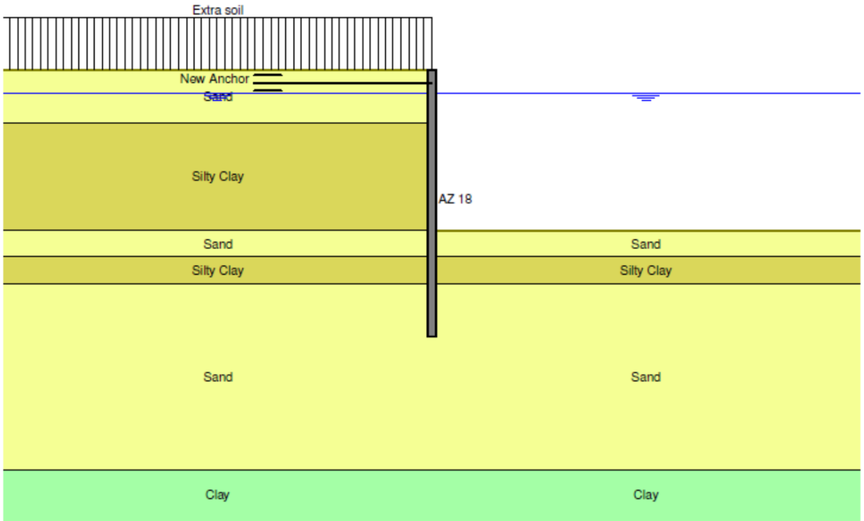
Section name	From [m]	To [m]	Height [mm]	Coating area [m²/m² wall]	Section area [cm²]
AZ 18	-20,00	0,00	380,00	1,35	150,00

2.3 Calculation Options

First stage represents initial situation	No
Calculation refinement	Coarse
Lambda recalculation	Automatic
Reduce delta(s) according to CUR	Yes

3 Construction Stage 1: New Stage

3.1 Outline



3.2 Input Data Left

3.2.1 Calculation Method

Calculation method: Ka, Ko, Kp

3.2.2 Water Level

Water level: -1,70 [m]

3.2.3 Surface

X [m]	Y [m]
0,00	0,00

3.2.4 Soil Layer Properties in Profile: Left

Layer name	Level [m]	Unit weight Unsat [kN/m³]	Unit weight Sat [kN/m³]	Cohesion [kN/m²]	Friction angle phi [deg]	Delta friction angle [deg]
Sand	0,00	17,00	19,00	0,00	32,00	2,50
Silty Clay	-4,00	12,00	14,50	10,00	28,00	2,50
Sand	-12,00	17,00	19,00	0,00	32,00	2,50
Silty Clay	-14,00	12,00	14,50	10,00	28,00	2,50
Sand	-16,00	17,00	19,00	0,00	32,00	2,50
Clay	-30,00	14,00	16,00	10,00	30,00	2,50

Layer name	Level [m]	Shell factor [-]	OCR [-]	Grain type
Sand	0,00	1,00	1,00	Fine
Silty Clay	-4,00	1,00	1,00	Fine
Sand	-12,00	1,00	1,00	Fine
Silty Clay	-14,00	1,00	1,00	Fine
Sand	-16,00	1,00	1,00	Fine
Clay	-30,00	1,00	1,00	Fine

Layer name	Level	Earth pressure coefficients			Additional pore pressure	
		Active	Neutral	Passive	Top	Bottom
	[m]	[-]	[-]	[-]	[kN/m²]	[kN/m²]
Sand	0,00	0,30	0,47	3,51	0,00	0,00
Silty Clay	-4,00	0,35	0,53	2,97	0,00	0,00
Sand	-12,00	0,30	0,47	3,51	0,00	0,00
Silty Clay	-14,00	0,35	0,53	2,97	0,00	0,00
Sand	-16,00	0,30	0,47	3,51	0,00	0,00
Clay	-30,00	0,33	0,50	3,23	0,00	0,00

3.2.5 Modulus of Subgrade Reaction (Secant)

Layer name	Level	Branch 1		Branch 2	
		Top	Bottom	Top	Bottom
	[m]	[kN/m³]	[kN/m³]	[kN/m³]	[kN/m³]
Sand	0,00	20000,00	20000,00	10000,00	10000,00
Silty Clay	-4,00	4000,00	4000,00	2000,00	2000,00
Sand	-12,00	20000,00	20000,00	10000,00	10000,00
Silty Clay	-14,00	4000,00	4000,00	2000,00	2000,00
Sand	-16,00	20000,00	20000,00	10000,00	10000,00
Clay	-30,00	6000,00	6000,00	4000,00	4000,00

Layer name	Level	Branch 3	
		Top	Bottom
	[m]	[kN/m³]	[kN/m³]
Sand	0,00	5000,00	5000,00
Silty Clay	-4,00	800,00	800,00
Sand	-12,00	5000,00	5000,00
Silty Clay	-14,00	800,00	800,00
Sand	-16,00	5000,00	5000,00
Clay	-30,00	2000,00	2000,00

3.2.6 Anchors

Name	Level [m]	E-Modulus [kN/m²]	Cross section [m²/m']	Length [m]	Angle [deg]	Yield force [kN/m']	Pre-tension. force [kN/m']
New Anchor	-1,00	2,100E+08	1,000E-03	5,00	0,00	200,00	n.a.

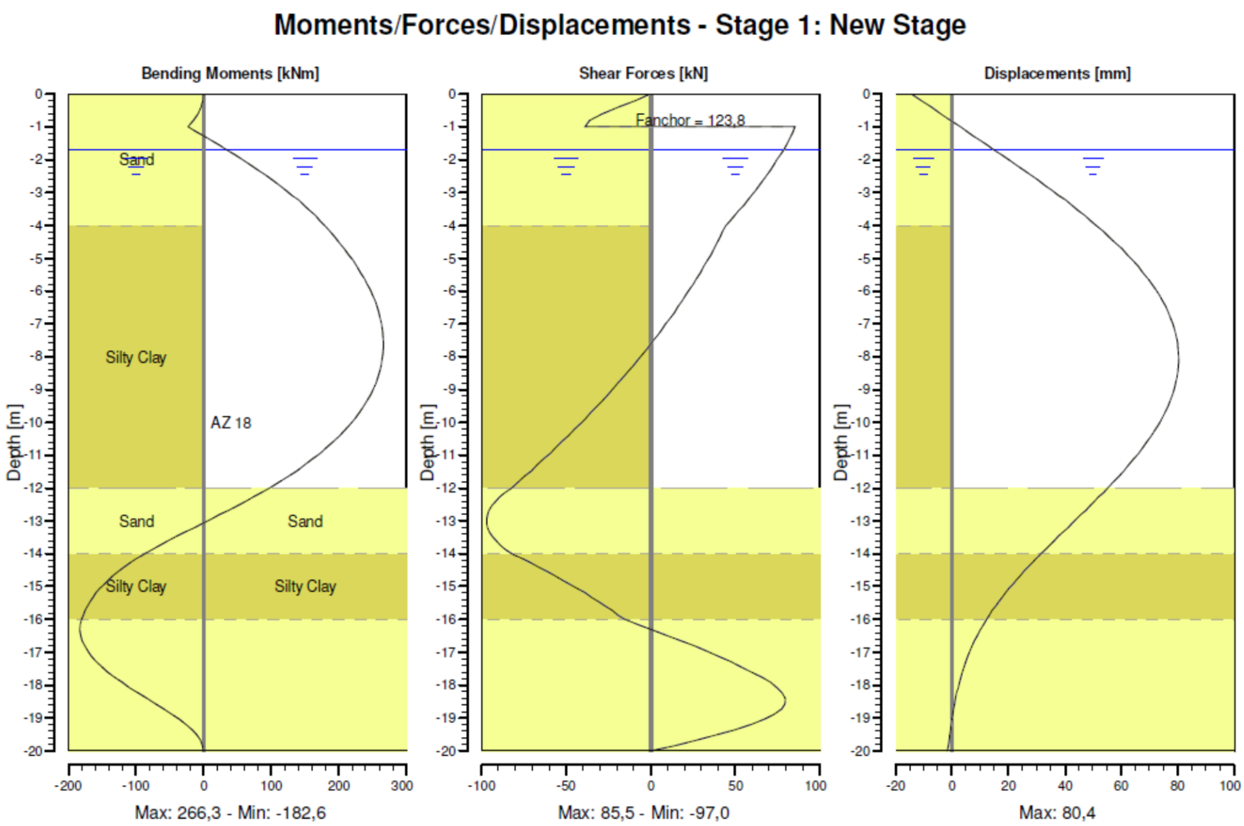
3.2.7 Uniform Loads

Name	Load [kN/m²]
Extra soil	10,00

3.4 Calculation Results

Number of iterations: 6

3.4.1 Charts of Moments, Forces and Displacements



H EPS parameters

EPS will not only provide the buoyancy force in the construction stage but it will also be part of the foundation during the operation stage. Therefore it is important to be aware of the behaviour of the material. This appendix will review the parameters of the material while appendix I will review the choice for the type(s) of EPS that will be used. This appendix is based on the technical documentation of Kemisol BV. [Kemisol, 2004]

Types of EPS

There are multiple types of EPS available. Here we use the European terms. EPS is available in the range EPS 30 up to EPS 500. The number stands for the short term compressive strength of the EPS at 10% distortion. This means that the EPS can respectively take 30 and 500 kPa of pressure for a short period of time. Because the structure will be immersed for a long period of time (life time 50 years), also the long term compressive strength will be used (approximately 30% of the short term strength).

The available types of EPS and corresponding masses are shown in the table below.

Europees type EPS	minimum volumemassa [kg/m³]
EPS S	
EPS 30	
EPS 50	
EPS 60	13.5
EPS 70	
EPS 80	
EPS 90	
EPS 100	18.0
EPS 120	
EPS 150	22.5
EPS 200	27.0
EPS 250	31.5
EPS 300	36.0
	40.5
EPS 400	45.0
	49.5
EPS 500	54.0

Tabel 2.5 : minimum volumemassa in functie van het type EPS

Shrinkage

Due to temperature changes after production, the EPS blocks will shrink. This is described below.

2.14 Krimp

Naast de reversibele lengteverandering, vindt bij geëxpandeerd polystyreen ook een temperatuurgekoppelde irreversibele lengtevariatie, namelijk krimp plaats.
De eerste 24 uur heeft men krimp te wijten aan het afkoelen van het juist geproduceerde blok isolatiemateriaal. Het productieproces, grondstof en densiteit zijn hier de belangrijkste parameters.

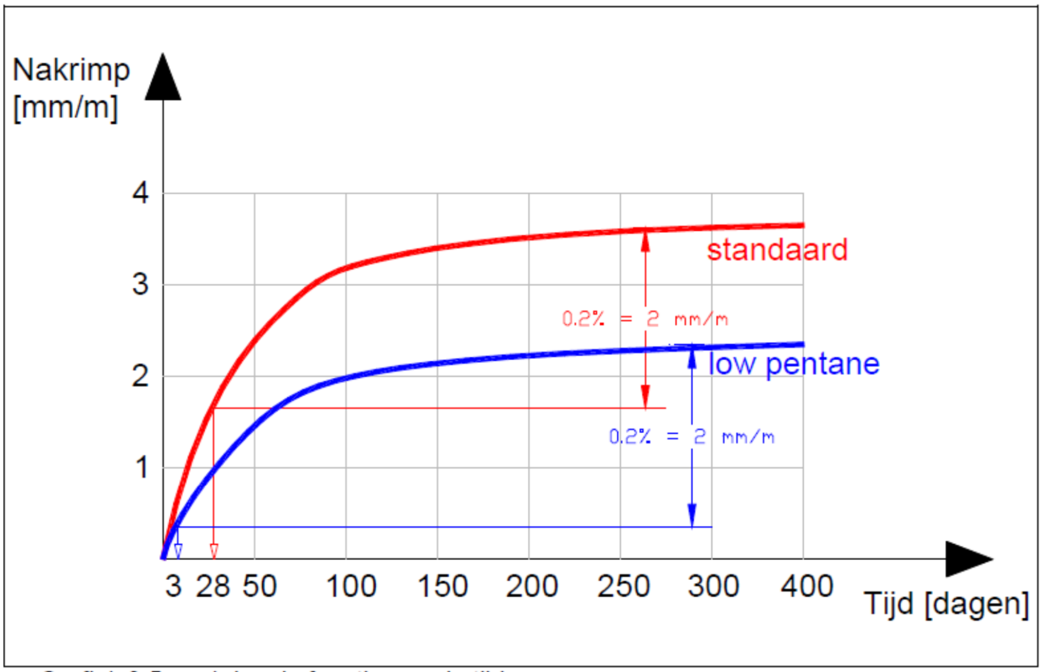
Met nakrimp wordt de kontraktie aangeduid van geëxpandeerd polystyreen dat meer dan 24 uur oud is. Deze nakrimp verloopt in het begin zeer snel waarna men een langzaam verloop krijgt tot een eindwaarde.
Afhankelijk van de volumemassa en het produktieproces bedraagt de nakrimp 0,3 à 0,5 %.

Het belangrijkste deel van de nakrimp wordt door stockeren in de fabriek reeds weggenomen (14 dagen).

De eindwaarde wordt bereikt na 150 dagen. De nakrimp bij het verlaten van de fabriek tot de eindwaarde bedraagt 1,5 à 2,0 mm/m.

Bij 'nieuwe' low-pentane grondstof is de nakrimptijd veel korter (stabilisatietijd 1 week i.p.v. 6 weken).

Bij gecacheerde toepassingen kan, afhankelijk van de bekleding, deze nakrimp verhinderd worden.



Grafiek 2.5 : nakrimp in functie van de tijd

Voor een aantal toepassingen is het nodig een maximum waarde aan te geven voor de niet omkeerbare (irreversibele) lengteveranderingen.

Water absorption
EPS consist for 95% out of a closed cell structure. Well it is immersed, the pores will absorb water and this will lead to a higher mass density. This is described below.

3.1 Het vochtgehalte

Het vochtgehalte wordt bij geëxpandeerd polystyreen weergegeven in volume % m.a.w. m^3 vocht per m^3 materiaal.

Het vochtgehalte kan variëren tussen 0 (= droog materiaal) en de verzadiging (= alle open poriën met water gevuld). Op deze kontinue schaal zijn er enkele belangrijke zones of punten van laag naar hoog.

De wateropname is afhankelijk van:

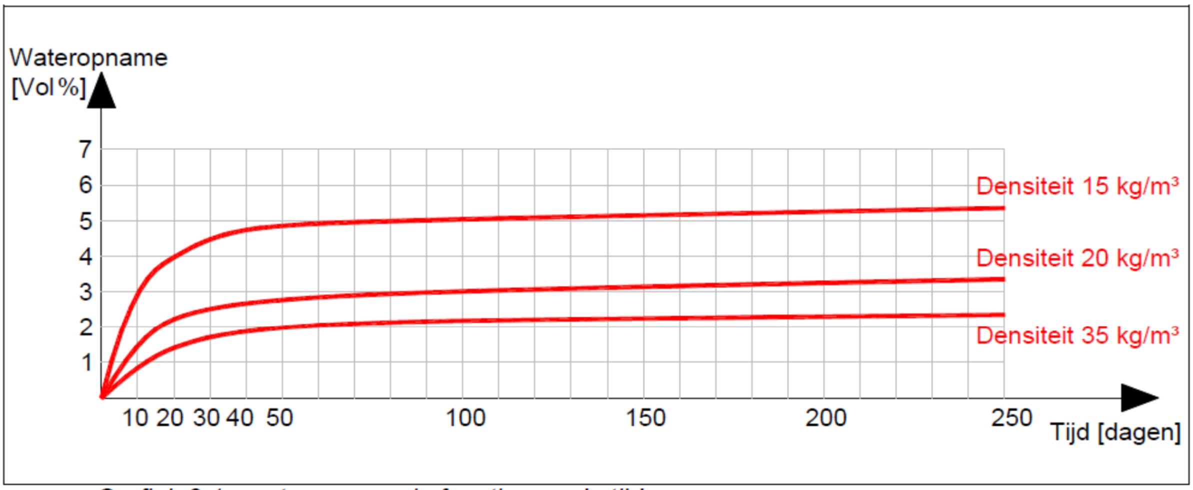
- de proefmethode
- de vorm van het proefstaal (verhouding oppervlak / volume)
- oppervlaktebehandeling (gesneden met mes, gloeidraad, ...)
- voorgeschiedenis van het materiaal; bij hogere drukbelasting breekt men de cellen en verhoogt men de vochtopname
- klimaatgegevens: bij vorst-dooi cycli verhoogt de vochtopname
- de tijdsduur van de waterbelasting
- de hydraulische druk (diepte bij onderdompeling)

Het aanwezige water bevindt zich in hoofdzaak in de ruimte (smalle kanaaltjes) tussen de parels. Naast de densiteit is dus nogmaals de fusie en het aandeel regeneraat uitermate belangrijk.

De vochtopname bij onderdompeling na 7 dagen en na 1 jaar in functie van de volumemassa wordt in tabel 3.2 en grafiek 3.1 weergegeven. (EN 12087) De wateropname zal nooit meer bedragen dan 5 volume %.

Europees type EPS	Belgisch type EPS	vochtgehalte bij onderdompeling (vol %)	
		na 7 dagen	na 1 jaar
EPS S		-	-
EPS 30			
EPS 50			
EPS 60	PS 15 & PS 15 SE	3,0	5,0
EPS 70		3,0	5,0
EPS 80		3,0	5,0
EPS 90		3,0	5,0
EPS 100	PS 20 & PS 20 SE	2,3	4,0
EPS 120		2,3	4,0
EPS 150	PS 25 & PS 25 SE	2,2	3,8
EPS 200	PS 30 & PS 30 SE	2,0	3,5
EPS 250	PS 35 & PS 35 SE	1,9	3,2
EPS 300	PS 40 & PS 40 SE	1,8	3,0
	PS 45 & PS 45 SE	1,7	2,8
EPS 400	PS 50 & PS 50 SE	1,6	2,6
	PS 55 & PS 55 SE	1,5	2,4
EPS 500	PS 60 & PS 60 SE	1,4	2,2

Tabel 3.2 : vochtgehalte in functie van het type EPS



Grafiek 3.1 : wateropname in functie van de tijd

Duidelijk is te zien dat het grootste gedeelte van de wateropname plaatsvindt tijdens de eerste twee maanden. Nadien is de toename gering.

EPS-blok onder water:

Een EPS blok dat zich onder water bevindt gedurende 9 jaar bevat tot 9 vol% vocht.

Mechanical parameters of EPS

High water pressures will act on the EPS blocks during construction and in the operation stage. It is therefore important to know how the EPS acts when it is imposed to high loads. This is described below.

4. MECHANISCHE EIGENSCHAPPEN

De mechanische eigenschappen worden weergegeven bij een temperatuur van ± 20 °C. Geëxpandeerd polystyreen is een thermoplastisch materiaal. Dit betekent dat de mechanische eigenschappen temperatuurgebonden zijn. Bij hogere temperaturen verminderen de mechanische eigenschappen.

4.1 Druksterkte

De druksterkte is één van de belangrijkste eigenschappen van geëxpandeerd polystyreen. Deze eigenschap is in hoofdzaak afhankelijk van de densiteit. Als andere bepalende parameters vermelden we de celstructuur, de temperatuur en de ouderdom van het materiaal. De parelgrootte of het type grondstof heeft weinig invloed op de druksterkte.

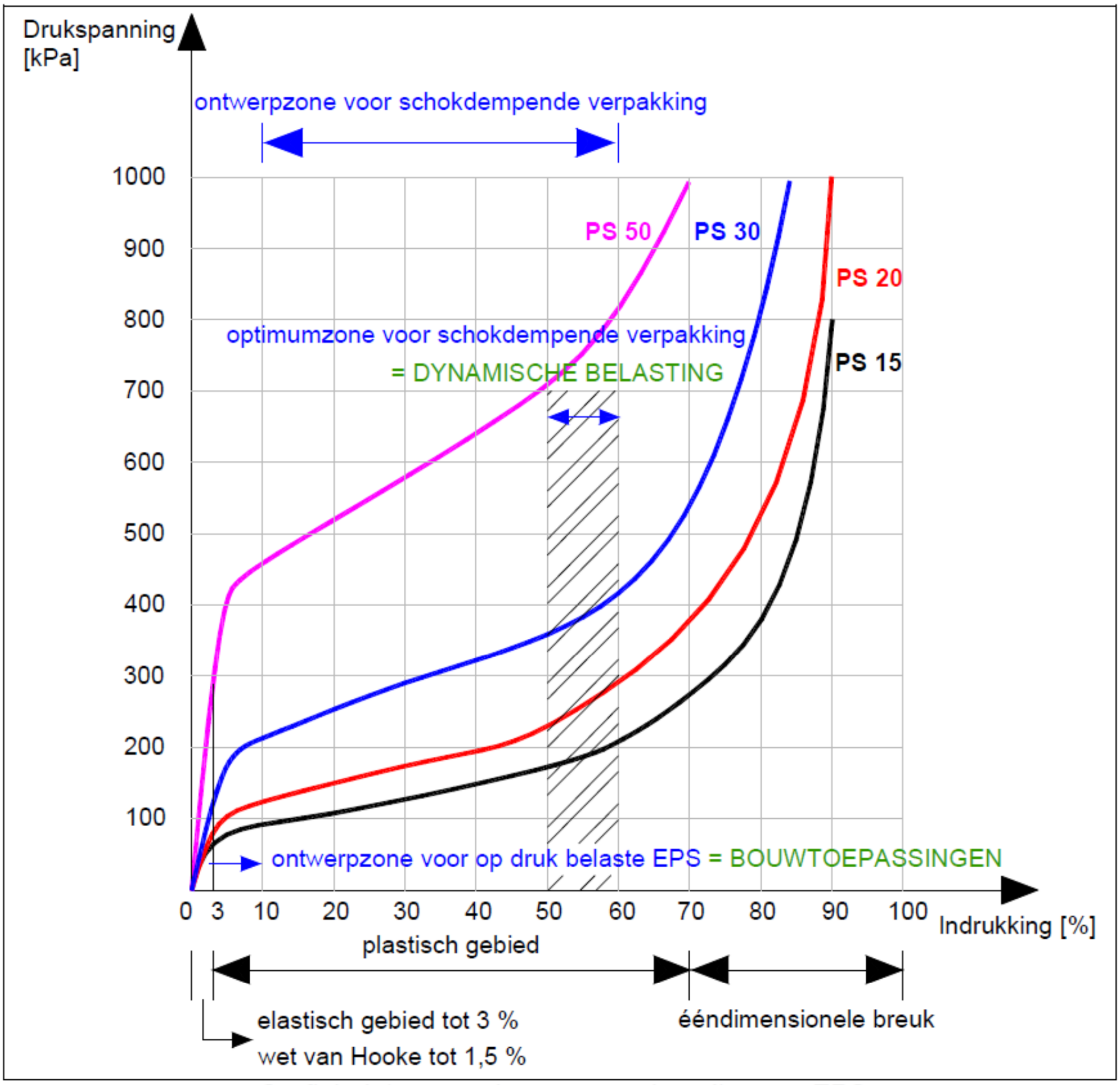
4.1.1 Spanning-vervormingsdiagram

Het is een klassieke proef die overgenomen werd uit de metaalindustrie en daardoor bij zijn toepassing bij kunststoffen een grondige kennis van de materialen vergt om op degelijke wijze geïnterpreteerd te worden. We moeten immers bij EPS rekening houden met de tijd en temperatuurfuncties en ook met de thermische voorgeschiedenis van het materiaal.

EPS heeft een visco-elasticiteits-curve die typisch is voor een bros/stijf materiaal.

EPS is een thermoplast en is dus temperatuur-gevoelig. Bij verhoging van de temperatuur nemen de sterkte en de elasticiteitsmodulus af en de breukrek toe.

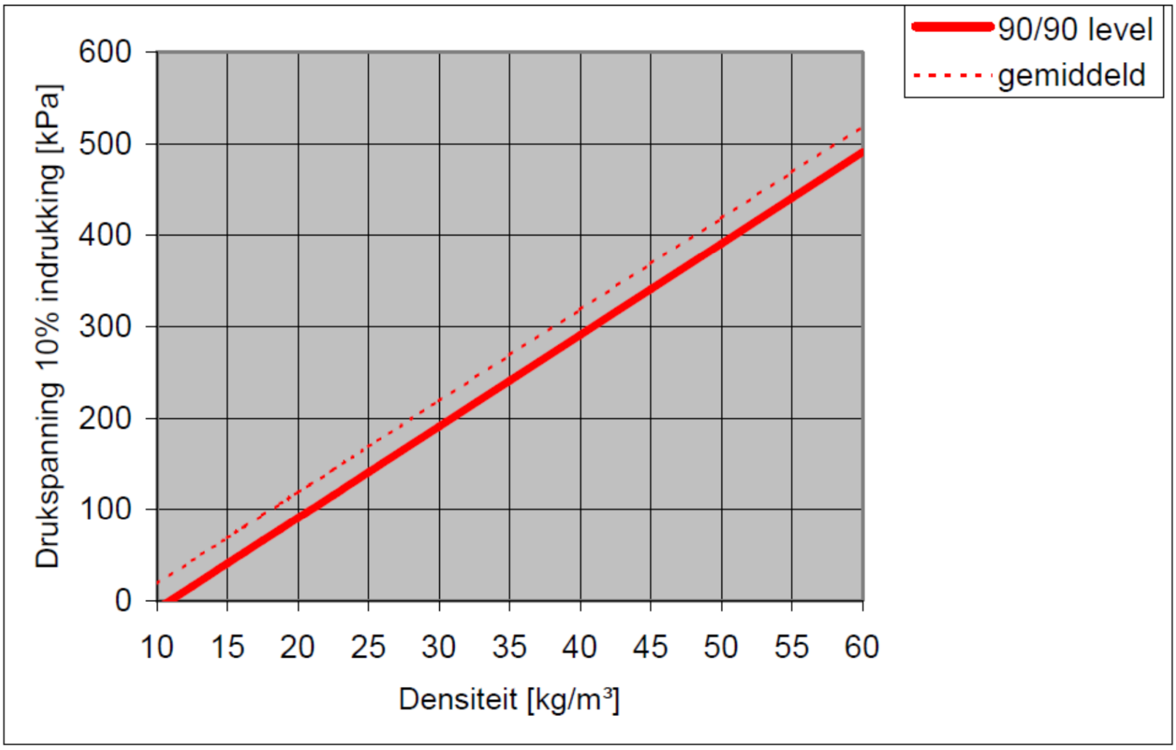
Een verandering in de vervormingssnelheid heeft nagenoeg dezelfde invloed als een temperatuurswijziging. Grote vervormingssnelheid verhoogt de modulus en de sterkte en geeft een lagere breukrek.



Grafiek 4.1 : spanning- vervormingsdiagram EPS

4.1.2 Drukspanning bij 10% indrukking (σ-10%)

Drukproef volgens EN 826 (Europese norm)
- afmetingen proefmonsters (mm): 50 x 50 x 50
- voorlast : 250 ± 10 Pa
- belastingsnelheid : [d(mm)/10] per minuut m.a.w. bij een dikte van 50 mm, 5mm/min



Grafiek 4.2 : drukspanning bij ogenblikkelijke indrukking van 10 % in functie van de densiteit

4.1.3 σ-2% en σ-3% lange termijn

Volgens EN 1606 "kruipgedrag lange duur drukbelasting":
De kruip bij lange duur drukbelasting ε_{ct} en de totale dikte reductie ε_t worden bepaald na minstens 120 dagen proefopstelling bij de gedeclareerde druksterkte σ_c in stappen van minimum 1 kPa. De resultaten worden 30x geëxtrapoleerd om het gedeclareerde 'level' te bereiken. De kruip bij lange duur wordt gedeclareerd in 'levels' i_2 en de totale dikte reductie in 'levels' i_1 in stappen van 0,5% bij de gedeclareerde druksterkte. Geen enkel testresultaat mag het gedeclareerde 'level' bij de gedeclareerde druksterkte overschrijden.

Geëxpandeerd polystyreen bereikt bij een indrukking van 2 à 3 % de proportionaliteitsgrens. (De uiterste gebruiksgrens toestand is 3% = de maximum toelaatbare ontwerpbelasting).

De drukspanning in functie van de volumemassa bij een temperatuur van 23 °C en een indrukking op lange termijn (= 50 jaar) van 2 % (= 0,25 $\sigma_{10\%}$), 2.5 % (= 0,3 $\sigma_{10\%}$) en 3 % (= 0,35 $\sigma_{10\%}$) zijn weergegeven in tabel 4.2. Dit zijn veilige rekenwaarden.

Europees type EPS	**type EPS**	$\sigma_{2\,\%}$	$\sigma_{2,5\,\%}$	$\sigma_{3\,\%}$
'Level' volgens EN 13163 CC(totale diktereductie/kruip/termijn)		**CC(2/1/50)**	**CC(2,5/1,5/50)**	**CC(3/2/50)**
EPS S		-	-	-
EPS 30		7,5 kPa	9 kPa	10,5 kPa
EPS 50		12,5 kPa	15 kPa	17,5 kPa
EPS 60	PS 15 & PS 15 SE	15 kPa	18 kPa	21 kPa
EPS 70		17,5 kPa	21 kPa	24,5 kPa
EPS 80		20 kPa	24 kPa	28 kPa
EPS 90		22,5 kPa	27 kPa	31,5 kPa
EPS 100	PS 20 & PS 20 SE	25 kPa	30 kPa	35 kPa
EPS 120		30 kPa	36 kPa	42 kPa
EPS 150	PS 25 & PS 25 SE	37,5 kPa	45 kPa	52,5 kPa
EPS 200	PS 30 & PS 30 SE	50 kPa	60 kPa	70 kPa
EPS 250	PS 35 & PS 35 SE	62,5 kPa	75 kPa	87,5 kPa
EPS 300	PS 40 & PS 40 SE	75 kPa	90 kPa	105 kPa
	PS 45 & PS 45 SE	87,5 kPa	105 kPa	122,5 kPa
EPS 400	PS 50 & PS 50 SE	100 kPa	120 kPa	140 kPa
	PS 55 & PS 55 SE	112,5 kPa	135 kPa	157,5 kPa
EPS 500	PS 60 & PS 60 SE	125 kPa	150 kPa	175 kPa

Tabel 4.2 : drukspanning op lange termijn bij respectievelijk 2%, 2,5% en 3% indrukking i.f.v. het type EPS

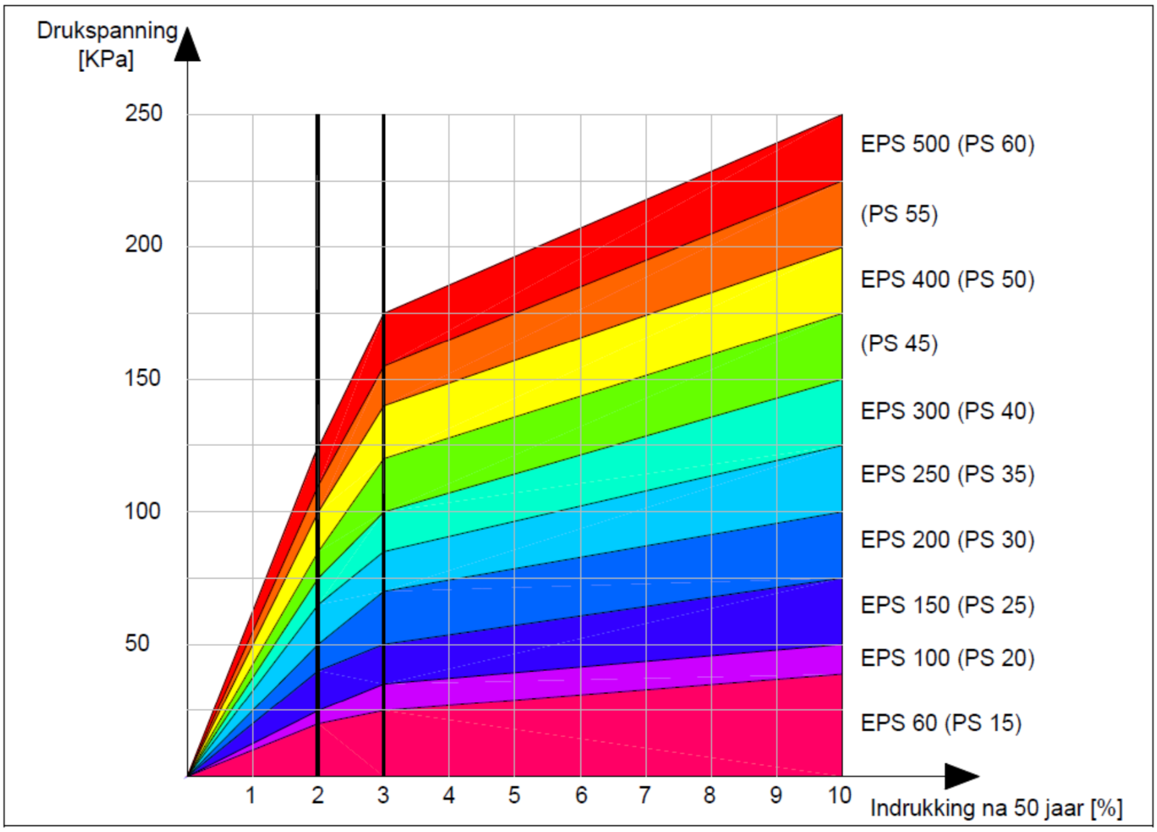
Kruip:

Indien de lange duur drukbelasting beperkt wordt tot 25% van de druksterkte bij 10% (= druksterkte lange termijn van 2% gedurende 50 jaar); dan is de vervorming maximaal 1% op korte termijn en blijft de kruip beperkt tot 0,2% waarvan de helft optreed gedurende de eerste 24 uur.

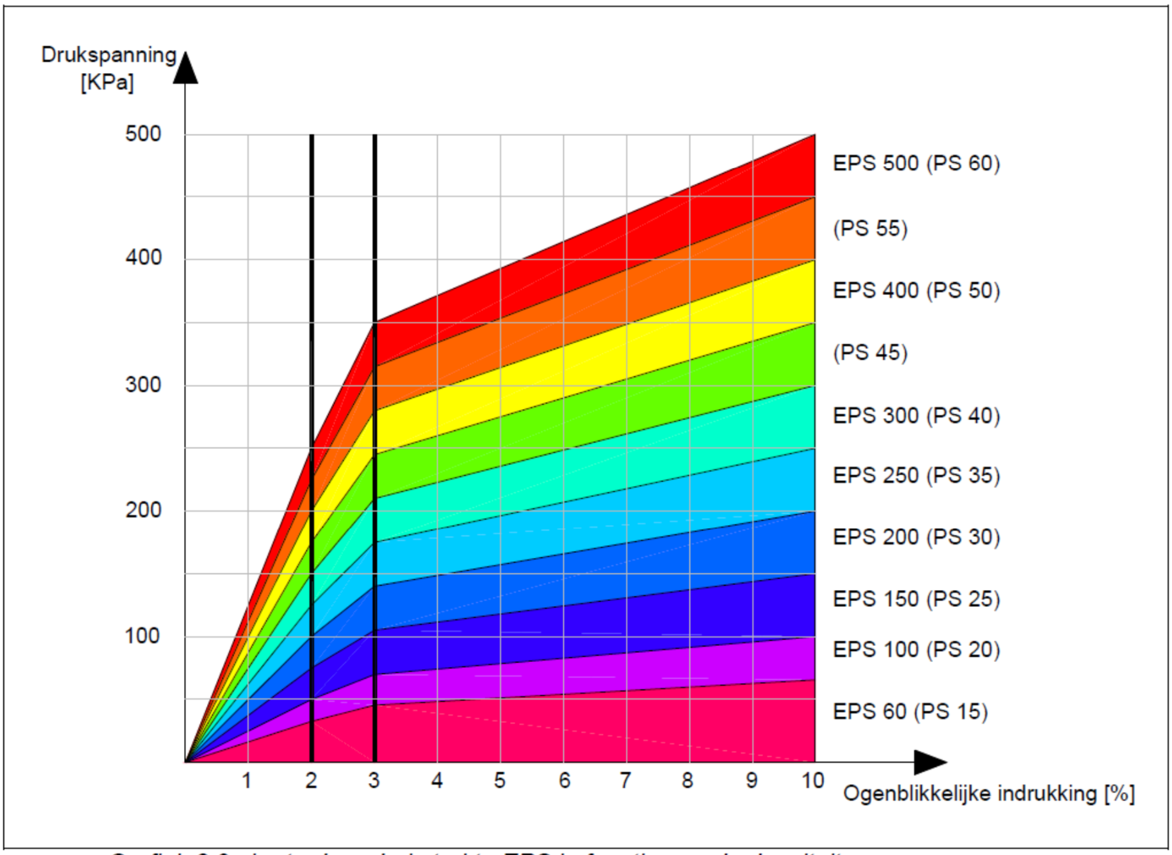
$\varepsilon_c = a.t^n$ = kruip

Het kruipgedrag op een logaritmische schaal is lineair te noemen.

$\varepsilon_t = \varepsilon_i + \varepsilon_c$ waarin ε_i = ogenblikkelijke indrukking en ε_t = de totale indrukking



Grafiek 3.8 : lange duur druksterkte EPS in functie van de densiteit



Grafiek 3.9 : korte duur druksterkte EPS in functie van de densiteit

I Choice for EPS type

The Flexbase floor consist out of two types of EPS. The first layers will have to resist the water pressures and will be of a higher density. The layers between the two floors will only act as formwork and will be of a lower density, namely EPS 60.

Appendix H shows that EPS distorts because of shrinkage, creep and the induced loads. The graphs on the previous page show the total distortion of the EPS in relation to the induced load. These graphs will be used to select the needed type of EPS. Two stages are divined, namely:

i. End of construction stage, floating;
ii. Operation stage.

Construction stage

Constructing the complete parking garage could take up to two years. Therefore the long-term compression strength is used (0,3 x short-term compression strength). When the garage element is finished, the draught is 6,05 meter as is shown in figure I-1.

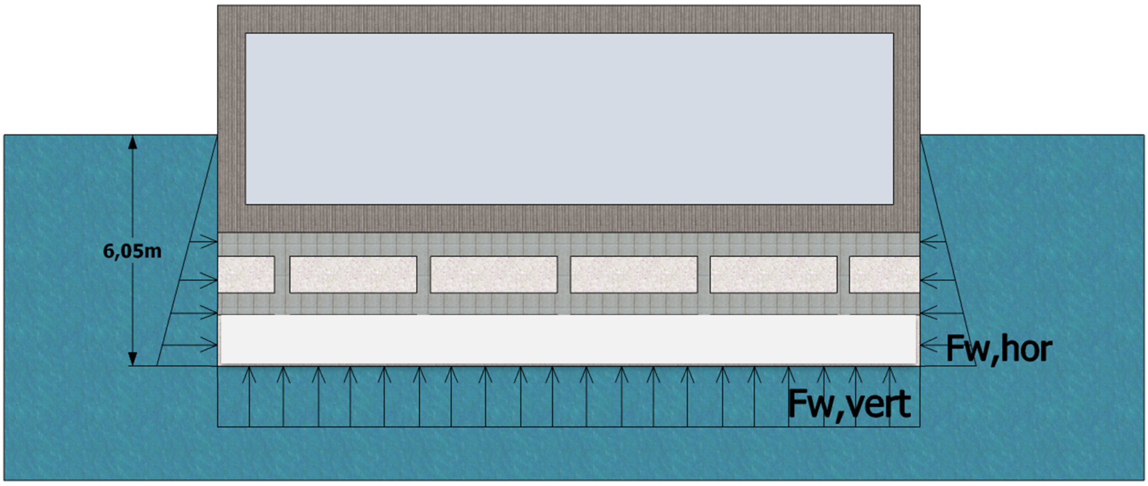


Figure I-1: Maximum water pressure on EPS during construction stage

This means that the water pressure on the bottom EPS layer will be 60,5 kN/m^2. When staying within the limits of the long-term compression strength, this means that EPS 250 is needed (60,5 / 0,3 = 201). The total distortion of the EPS will have a maximum of 2,5%.

The graph also shows the distortion of the EPS when higher loads act on the EPS. However, the manufacturer cannot guaranty the distortion above the '0,3 x short-term load' limit. In an email the engineer of Unidek stated that higher loads could cause uncontrolled distortion or even complete collapse of the EPS.

"Dit zijn belasting schema's gebaseerd op toepassingen boven de waterspiegel waarbij is uitgegaan van het deel van het last/vervormingsdiagram waarin een evenredige vervorming plaats zal vinden met opvoering van de belasting. Bij hogere belastingen vinden ongecontroleerde vervormingen plaats tot aan geheel onomkeerbare vervormingen en collaps.

Neem EPS 250 > maximaal toelaatbare belasting lange duur 0,3 x 250 kPa (waarbij aangegeven dat bij deze maximale belasting een maximale vervorming van 2,5% mag worden aangehouden)."

Eric Las, Kingspan Unidek B.V.

EPS below first floor: EPS 250
EPS between first and second floor: EPS 60

Operation stage
In the operation stage the water pressure on the bottom EPS layer is approximately 150 kN/m^2. This means that EPS 500 is needed. However, EPS 500 is more than twice as expensive as EPS 250 (approximately 100 euro per m^3 more). Considering that the layers of EPS below the first floor consist out of 15.500 m^3 of EPS, this means € 1.550.000,- extra costs to use EPS 500 instead of EPS 250.

It is therefore important to know whether or not it would be a problem if the EPS fails in the operation stage. In this case the worst case scenario is researched, namely complete collapse of the bottom layer. This layer is 800 mm thick. What happens with the structure depends on the method that is used to keep the garage element immersed, namely ballast or grout anchors. This is described in the next few paragraphs, using figure I-2 and I-3.

Collapse of EPS: Ballast
The figure below shows the loads that act on the garage element in the operation stage. Note that the figure shows the Flexbase Heavy variant. The definitive design only has one beam grid. The force Fg represents the complete weight of the element, including the weight of the ballast. To keep the element immersed, this force will have to be larger than the buoyancy force ($Fg \approx 1{,}30 \times F_{buoyancy}$). In the worst case scenario (collapse of the EPS), the settlement of the element will be equal to the thickness of the collapsed layer, namely 800 mm! It is obvious that this would be catastrophic. To prevent this, EPS 500 is needed for the EPS layers below the first floor.

Conclusion: when ballasting the element, EPS 500 is needed.

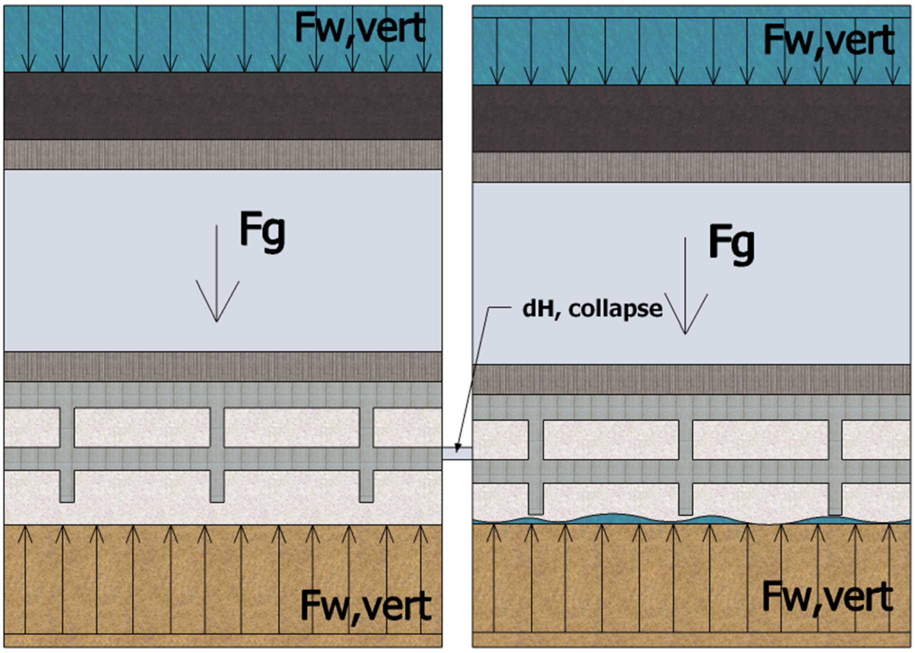


Figure I-2: Settlement of element after collapse of bottom EPS layer, using ballast

Collapse of EPS: Grout anchors
This will not be the case when using grout anchors. Grout anchors are steel tension piles which are anchored by a grout body at the end. The steel piles lengthen or shorten when the load on the pile changes. When the EPS collapses, the buoyancy force gets less and the anchor pile shortens. This means that the total settlement does not equal the thickness of the collapsed layer but it equals the length the pile shortens. Which is:

$$E = \sigma/\varepsilon = (dF \cdot L_0)/(A_0 \cdot \Delta L)$$

In which:

E	=	Modulus of elasticity	[N/mm^2]
dF	=	Change in applied load	[N]
A_0	=	Surface area steel pile	[mm^2]

ΔL	=	Change in length	[mm]
L_0	=	Original length	[mm]

The piles go through the Flexbase floor and are approximately 15,00 meters long. For this calculation a GEWI grout anchor is considered with a maximum applied force of 771 kN. The surface area of the pile is 1.946 mm^2.

The c.t.c. distance of the piles is 3,6 x 4,12 meters. This means that the change in buoyancy force (collapse is 800mm) is 3,6 x 4,12 x 0,80 x 10 = 118,6 kN per pile. The modulus of elasticity of steel is 210.000 N/mm^2.

E	=	210.000	[N/mm^2]
dF	=	118,600	[N]
A0	=	1.946	[mm^2]
L0	=	15.000	[mm]

$$210.000 = (118.600 \cdot 15.000)/(1.946 \cdot \Delta L)$$

ΔL = 4,31 mm

The total settlement in the worst case scenario would be 4,31 mm which is almost negligible.

Conclusion

EPS 250 could be used if grout anchors are used to keep the element immersed. It is important that the initial force on the grout anchors is more than 118 kN. In that case it is still under tension after the EPS has collapsed. When the grout anchors are under pressure, this calculation becomes invalid because more factors have to be taken into account.

Note that this only happens in the worst case scenario. The chance that the EPS layer would completely collapse is very small. If the EPS only collapses locally the change in buoyancy force would also be divided over the anchors that are near that location which means a smaller settlement.

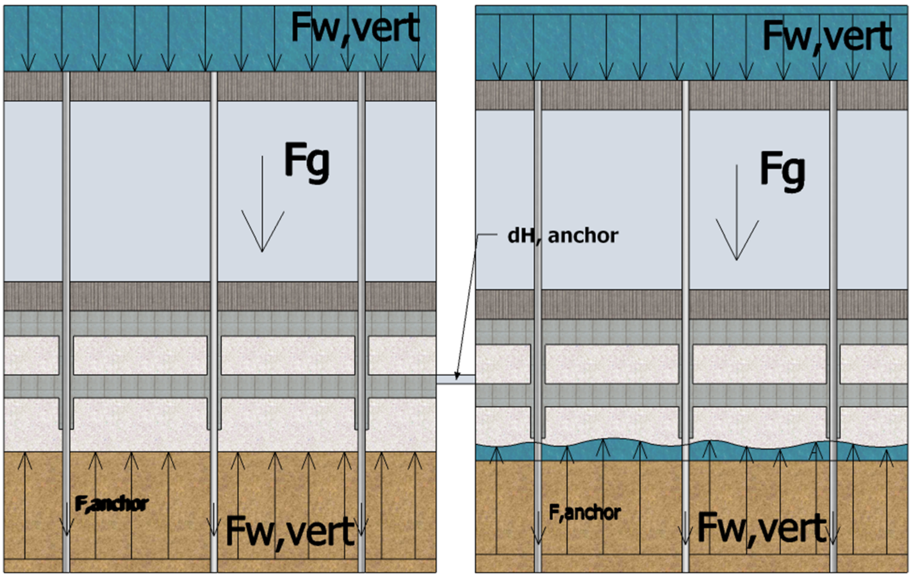


Figure I-3: Settlement of element after collapse of bottom EPS layer, using grout anchors

J Ballasting and grout anchors

The conclusion of appendix I made clear that it is wise to use grout anchors to keep the garage element immersed. In case the bottom EPS would collapse, it wouldn´t lead to failure of the structure. The grout anchors could be placed through the columns as is shown in figure J-1. Then there will be a total of 234 grout anchors (13 x 18). The c.t.c. distance is 7,20 meters and 8,25 meters.

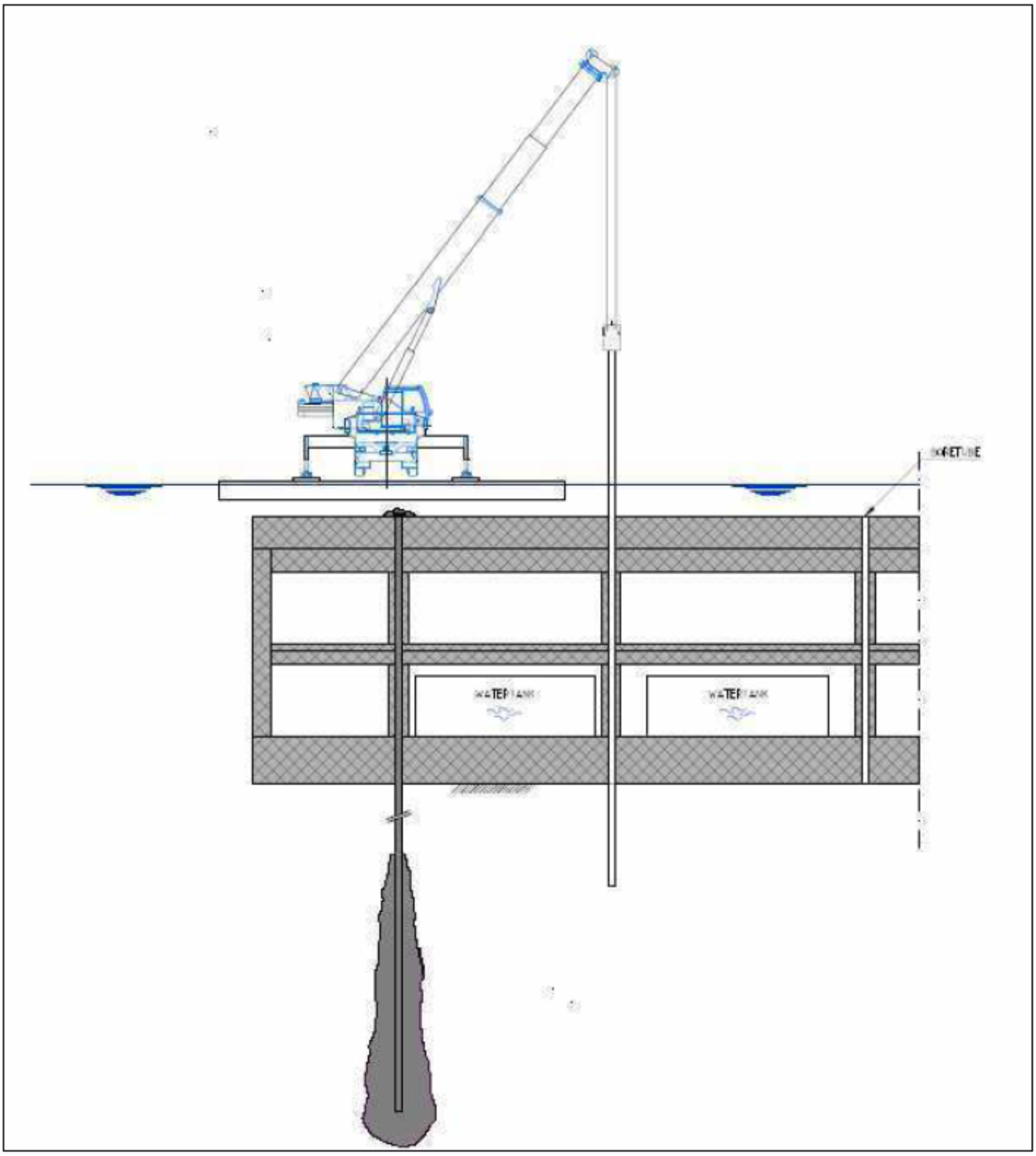


Figure J-1: Installation of grout anchors through columns. [Maarten Faber, 2008]

Tension capacity of a single pile

The tension capacity of a single pile is given by the formula [CUR 2001-4]:

$$Fr, tension, d \; = \int_0^L O_{p;gem} * P_{r;z;d} \, dz$$

In which:

$F_{r,tension,d}$	=	Tension capacity of single pile	[kN]
$O_{p;gem}$	=	Average perimeter of grout body	[m]
L	=	Length of grout body	[m]
Z	=	Depth	[m]
$P_{r;z;d}$	=	Shaft friction	[kPa]

$$P_{r;z;d} = \propto_t * \; q_{c;z;d}$$

In which:

α_t	=	Tension pile factor	[-]
$q_{c;z;d}$	=	Cone resistance	[kPa]

Figure J-2 shows the main dimensions of the grout anchor. The grout body will be 8,00 meters long and injected into the layer with the highest cone resistance. The cone resistance in this layer is 24 Mpa, but according to the CUR 2001-4 the, maximum value to be used in a calculation is 15 Mpa. The tension pile factor for grout anchors is 0,009. [CUR 2001-4]

$$P_{r;z;d} \; = \; 0{,}009 * \; 15.000$$

$$P_{r;z;d} \; = \; 135 \, kPa$$

For this calculation a Titan 103/78 grout anchor is used. The parameters are shown in table J-1. I couldn't find the radius of a grout body in literature, therefore I asked the engineers of Dura Vermeer Beton- en Waterbouw. They use the radius of the pile plus 4,00 centimetres in their calculations. I have also seen other calculations from Titan BV, other student etc. But they all use other diameters for the grout body. Maarten Faber even uses a diameter of 0,32 meters, which would almost double the tension capacity in this case.

$$Fr, tension, d \; = \; 8 * (0{,}1004 + 2 * 0{,}04)\pi * 135$$

$$Fr, tension, d \; = \; \mathbf{612 \; kN}$$

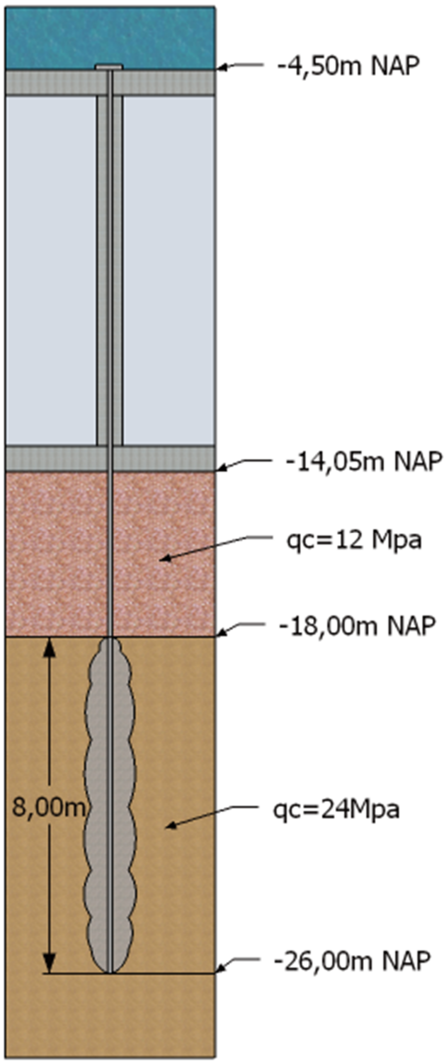


Figure J-2: Dimensions grout anchor

Tension capacity of a single pile in a group

When a pile is part of a group, the following aspects play a role:

i. The effect of compaction during installation;
ii. The effect of tensioning.

The factors to take these effect into account are respectively f1 and f2. Therefore the shaft friction becomes:

$$P_{r;z;d} \; = \; f1 * f2 * \propto_t * \; q_{c;z;d}$$

Because the distance between the piles is relatively large, at least 7,20 meters >> 6 * D_{pile}, these effects are close to zero and f1 and f2 approach 1,0. These effects will therefore not be taken into account.

Ankertype	Een-heid	Titan 30/16	Titan 30/11	Titan 40/20	Titan 40/16	Titan 52/26	Titan 73/53	Titan 73/56	Titan 73/45	Titan 73/35	Titan 103/78	Titan 103/51	Titan 127/111
Buitendiameter	mm	30	30	40	40	52	73	73	73	73	103	103	127
Buitendiameter voor stat. berekeningen	mm	27,2	26,2	36,4	37,1	48,8	69,9				100,4		
Binnendiameter	mm	16	11	20	16	26	53	56	45	35	78	51	111
Toelaatbare belasting	kN	125	182	292	377	530	734	596	894	1025	1325	1987	1325
Toelaatbare dwarskracht	kN	58	88	138	164	240	329				535		
Breukkracht	kN	220	320	539	660	929	1160				1950		
Gewicht	kg/m	3	3,5	5,6	6,9	10,5	12,8				24,7		
Kleinste dwarsdrsnd	mm2	362	446	726	879	1337	1631				3146		
Kracht aan de vloeigrens	kN	180	260	430	525	730	970	785	1180	1355	1800	2726	1810
Vloeispanning	N/mm2	470	580	590	590	550	590				500		
Traagheidsmoment	cm4	2,37	2,24	7,82	8,98	25,6	78,5				317		
Weerstandsmoment	cm3	1,79	1,71	4,31	4,84	10,5	22,4				63,2		
Plast. Weerstandsmoment	cm3	2,67	2,78	6,7	7,83	16,44	32,1				89,6		

Table J-1: Parameters Titan grout anchors [www.timmermansgww.nl]

Effects of excavation

The cone resistance of a sand layer becomes lower after excavation, as is the case. The cone resistance after excavation is:

$$q_{c;z;excavation} = q_{c;z} * \sqrt{\frac{\sigma_{v;z;excavation}}{\sigma_{v;z;0}}}$$

In which:

$q_{c;z;excavation}$	=	Corrected cone resistance	[kPa]
$\sigma_{v;z;excavation}$	=	Vertical stress in soil layer after excavation	[kN/m^2]
$\sigma_{v;z;0}$	=	Initial vertical stress in soil layer	[kN/m^2]

The corrected cone resistance is calculated for the depth of -22,00m NAP (middle of the grout body).

$$q_{c;z;excavation} = 24.000 * \sqrt{\frac{290}{390}}$$

$$q_{c;z;excavation} = 20.696\ kPa$$

$q_{c;z;excavation}$ = 20,70 Mpa > 15,00 Mpa → no correction needed

Conclusions grout anchor through columns

The tension capacity of a single pile in a group is 612 kN. There are 234 piles, therefore the total tension capacity is:

234 * 621 = **143.208 kN** < **235.072** kN (buoyancy force in operation stage)

Next step

There are a few options to create more down force:

iv. Add mass, ballast concrete;
v. Add more grout anchors;
vi. Use anchors with a larger capacity.

The first option would result in ballast concrete on the roof or inside the garage element. Besides adding mass, it would also add volume. This means that the net extra down force would be 15kN/m^3. Keeping in mind that ballast concrete costs approximately 80-90 €/m^3 it would cost 6 €/kN.

The second option is to add extra grout anchors, but there aren't more columns to lead anchors through. This means that the anchors should be placed inside the garage element. This option is discussed below.

Drilling the anchors from inside the garage element

When the garage element is immersed at the bottom of the Oosterdok, small drilling equipment could drill holes in the Flexbase floor and drill the anchors into the soil. This option is possible using a cuff but is more labour-intensive. Because of the restricted height, the anchors consist out of multiple pieces of 1,5 meters long. The advantage is that the anchors can be shorter because they are connected to the floor and not the roof. This saves approximately 7 meters per anchor.

To decide whether to drill anchors from the inside, add extra mass or use anchors with a larger capacity, the costs per alternative will be discussed. Timmermans BV, a company that delivers and places anchors has provided the needed information. The next page shows the costs per 100 anchors, including materials and labour.

Anchors of $25m^1$ with a capacity of 1325kN are compared with $15m^1$ long anchors with a capacity of 617kN that are drilled from inside the garage element. This comparison shows that the costs are predominately determined by the length and diameter of the anchor. Therefore the anchors through the columns are more expansive per kN than the shorter anchors that are drilled from inside the element.

The $15m^1$ long anchors are **2,17 €/kN** and are less expensive than using ballast concrete or leading the anchors through the columns (note that the costs for the anchors through the columns do not include the costs for a pontoon or extra materials needed in the columns!).

Conclusion

All the anchors will be drilled from the inside of the garage element when the element is immersed. This does mean that during the immersion process, the ballast tanks must leave enough space to manoeuvre the drilling equipment (shown in the figure below).

Figure J-3: Small drilling equipment

Globale prijsvergelijking voor 100 Gewi-ankers, lang 15 m1 (boren met beperkte werkhoogte)					617 kN		2,17 euro per kN		
			Arbeid		Materieel		Materiaal		Totaal
Omschrijving	Aantal	eenheid	prijs/eenheid	prijs	prijs/eenheid	prijs	prijs/eenheid	prijs	
Aan- en afvoerkosten	1	pm		€ -	2000	€ 2.000,00		€ -	€ 2.000,00
Constructieve berekening, werktekening, werkplan	40	uur	90	€ 3.600,00		€ -		€ -	€ 3.600,00
Vakman Wegenbouw	320	uur	30	€ 9.600,00		€ -		€ -	€ 9.600,00
Boorkolonne incl. machinist	160	uur	35	€ 5.600,00	95	€ 15.200,00		€ -	€ 20.800,00
Ankerstaaf (Gewi 50T - 617 kN) (incl. overlengte)	1600	m1		€ -		€ -	20	€ 32.000,00	€ 32.000,00
Cement	120000	kg		€ -		€ -	0,12	€ 14.400,00	€ 14.400,00
Ankerplaat	100	st		€ -		€ -	40	€ 4.000,00	€ 4.000,00
Ankermoer	100	st		€ -		€ -	15	€ 1.500,00	€ 1.500,00
Contramoer	100	st		€ -		€ -	6	€ 600,00	€ 600,00
Koppelmoffen	900	st		€ -		€ -	25	€ 22.500,00	€ 22.500,00
Cementpomp	20	dag		€ -	150	€ 3.000,00		€ -	€ 3.000,00
Stroom/water/container	20	dag		€ -	120	€ 2.400,00		€ -	€ 2.400,00
Minigraver incl. machinist	160	uur	32,5	€ 5.200,00	25	€ 4.000,00		€ -	€ 9.200,00
Afspanploeg incl. apparatuur	8	dag	280	€ 2.240,00	150	€ 1.200,00		€ -	€ 3.440,00
Boorbuizen	1	pm		€ -	5000	€ 5.000,00		€ -	€ 5.000,00
TOTAAL				**€ 26.240,00**		**€ 32.800,00**		**€ 75.000,00**	**€ 134.040,00**
Prijs per stuk									**€ 1.340,40**

Globale prijsvergelijking voor 100 titan ankers, lang 25 m1 (boren vanaf ponton)					1325 kN		2,75 euro per kN		
			Arbeid		Materieel		Materiaal		Totaal
Omschrijving	Aantal	eenheid	prijs/eenheid	prijs	prijs/eenheid	prijs	prijs/eenheid	prijs	
Aan- en afvoerkosten	1	pm		€ -	2000	€ 2.000,00		€ -	€ 2.000,00
Constructieve berekening, werktekening, werkplan	40	uur	90	€ 3.600,00		€ -		€ -	€ 3.600,00
Vakman Wegenbouw	250	uur	30	€ 7.500,00		€ -		€ -	€ 7.500,00
Boorkolonne incl. machinist	100	uur	35	€ 3.500,00	95	€ 9.500,00		€ -	€ 13.000,00
Ankerstaaf (Titan 103/78 - 1325kN) (incl. overlengte)	2600	m1		€ -		€ -	78,8	€ 204.880,00	€ 204.880,00
Cement	300000	kg		€ -		€ -	0,12	€ 36.000,00	€ 36.000,00
Ankerplaat (standaard kopplaat titan)	100	st		€ -		€ -	90	€ 9.000,00	€ 9.000,00
Ankermoer	100	st		€ -		€ -	48,8	€ 4.880,00	€ 4.880,00
Contramoer	100	st		€ -		€ -	48,8	€ 4.880,00	€ 4.880,00
Koppelmoffen	835	st		€ -		€ -	60,1	€ 50.183,50	€ 50.183,50
Cementpomp	20	dag		€ -	150	€ 3.000,00		€ -	€ 3.000,00
Stroom/water/container	20	dag		€ -	120	€ 2.400,00		€ -	€ 2.400,00
Kraan incl. machinist	100	uur	32,5	€ 3.250,00	40	€ 4.000,00		€ -	€ 7.250,00
Afspanploeg incl. apparatuur	8	dag	450	€ 3.600,00	250	€ 2.000,00		€ -	€ 5.600,00
Boorkroon (lehmbohrkroon)	100	st		€ -	94,3	€ 9.430,00		€ -	€ 9.430,00
TOTAAL				**€ 21.450,00**		**€ 32.330,00**		**€ 309.823,50**	**€ 363.603,50**
Prijs per stuk							**Exclusief huur ponton**		**€ 3.636,04**

Cost estimation grout anchors, 15m[1], 617kN drilled from inside the element vs. 25m[1], 1325kN through columns.

Anchor plan
The cost estimation showed that using anchors is the least expensive option. Therefore, all the anchors are drilled from the inside.

The anchors will be placed next to the intersections of the beams. The beam grid consist out of 31 x 21 beams (excluding the extra beams under the walls). This means a total of 641 intersections and anchors.

Placing the anchors in a grid of 4,125 x 3,6 meters instead of 8,25 x 7,2 meters also has another advantage. The distance between the points of force transmission becomes smaller, and with that the moments in the Flexbase floor.

Modelling the anchors
To simulate the anchors in SCIA, the same model is used as in appendix K7. The figure below shows a part of the bottom of the Flexbase floor. The blue squares represent the grout anchors. One anchor in the corner is missing, this will be explained later on.

Because I am not interested in the dynamic behaviour at this moment, the anchors are simulated by bearings of 20 x 20 cm that are restricted in the x-, y-, and z-direction.

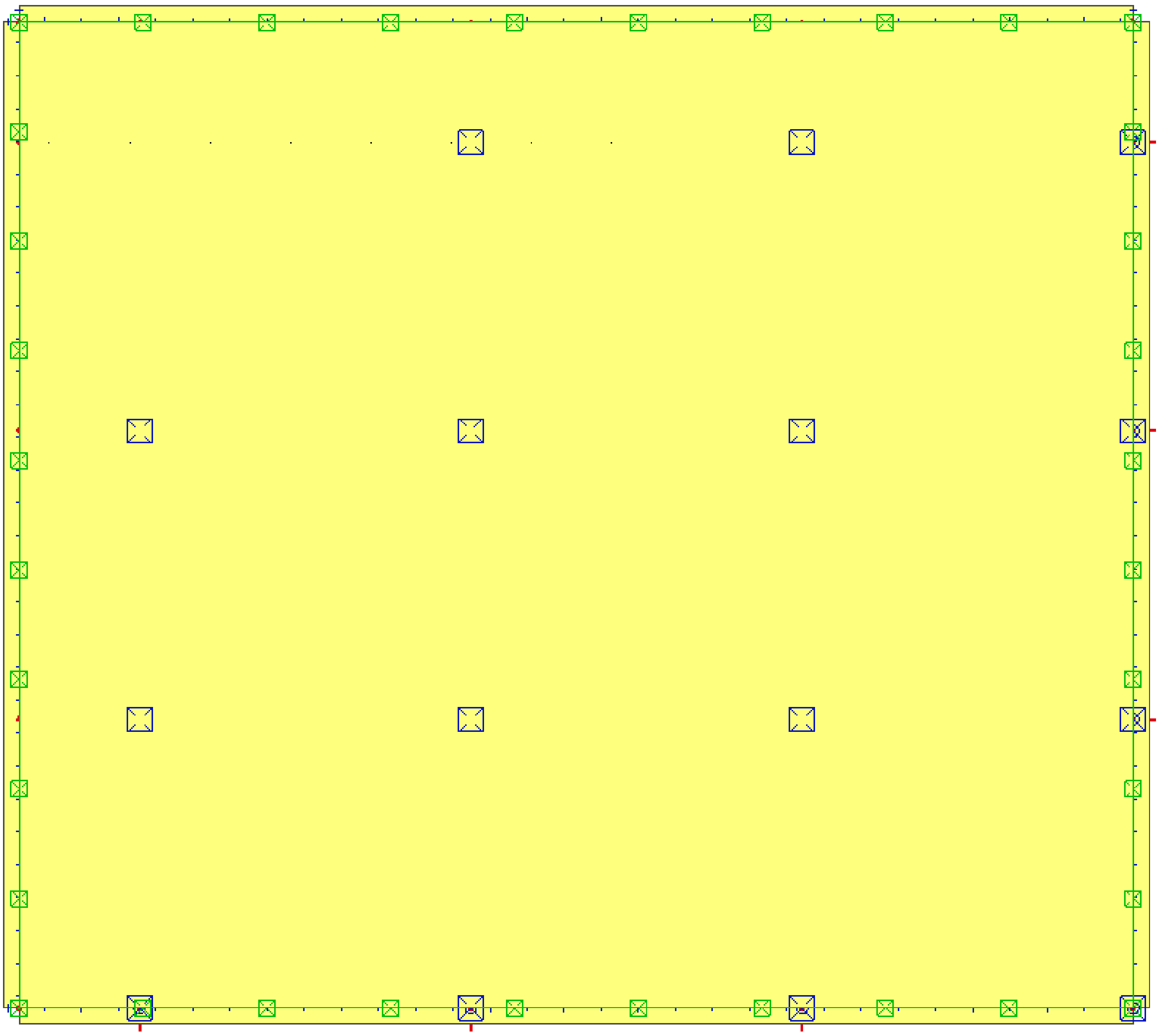


Figure J-4: SCIA model, anchors placed under the Flexbase floor

Forces in floor in operation stage

Two situations are modelled. One with all the anchors in place and one with one anchor missing. When one anchor is damaged or loses its function because of another reason, the anchors surrounding it will take its load. This creates an unwanted situation and the floor has to withstand these forces.

All the anchors in place

The figures below show the tension forces in the floor in the operation stage (water pressure against the floor). The maximum tension is 866kN/m. Which is much less than in the immersion stage (1.500kN/m, see appendix K7). Therefore, this situation isn't leading.

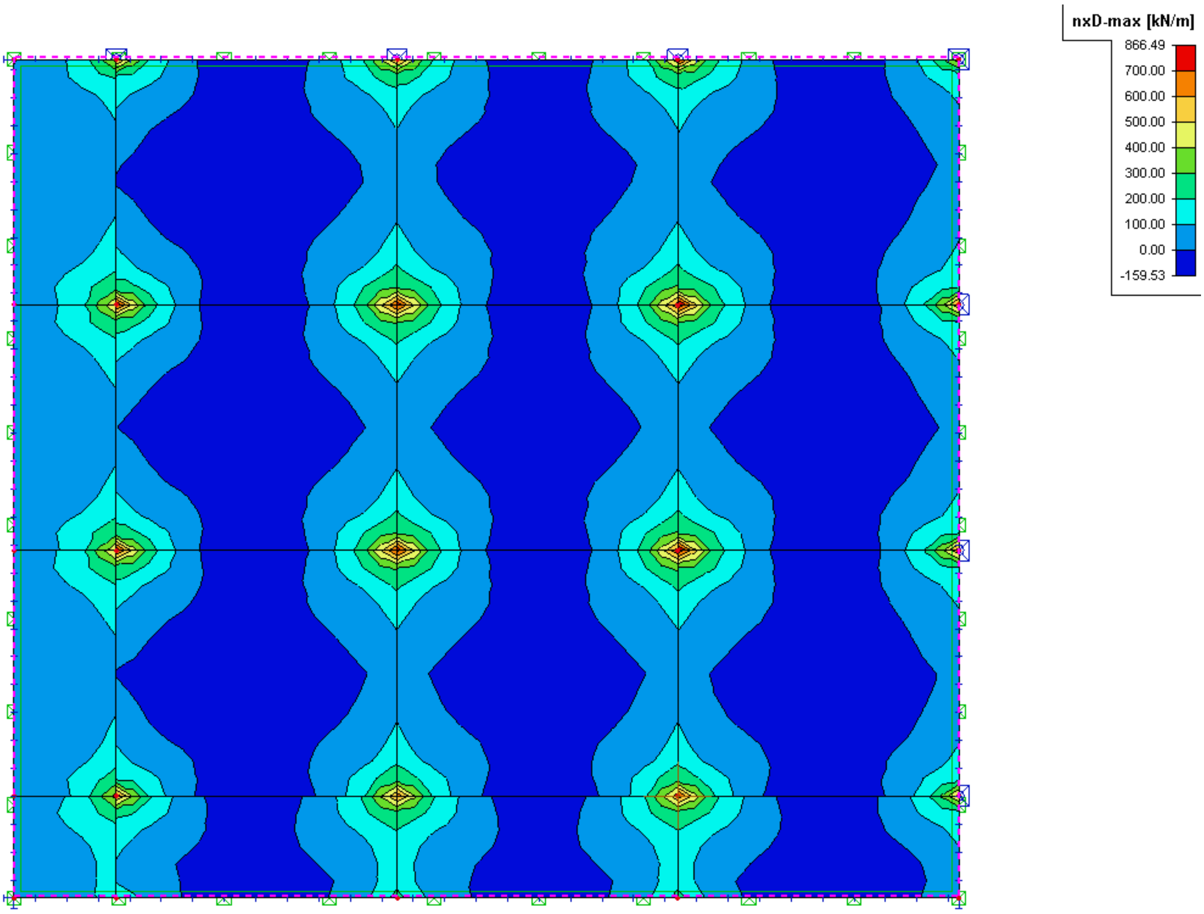


Figure J-5: Tension forces in top floor during operation stage, nxD

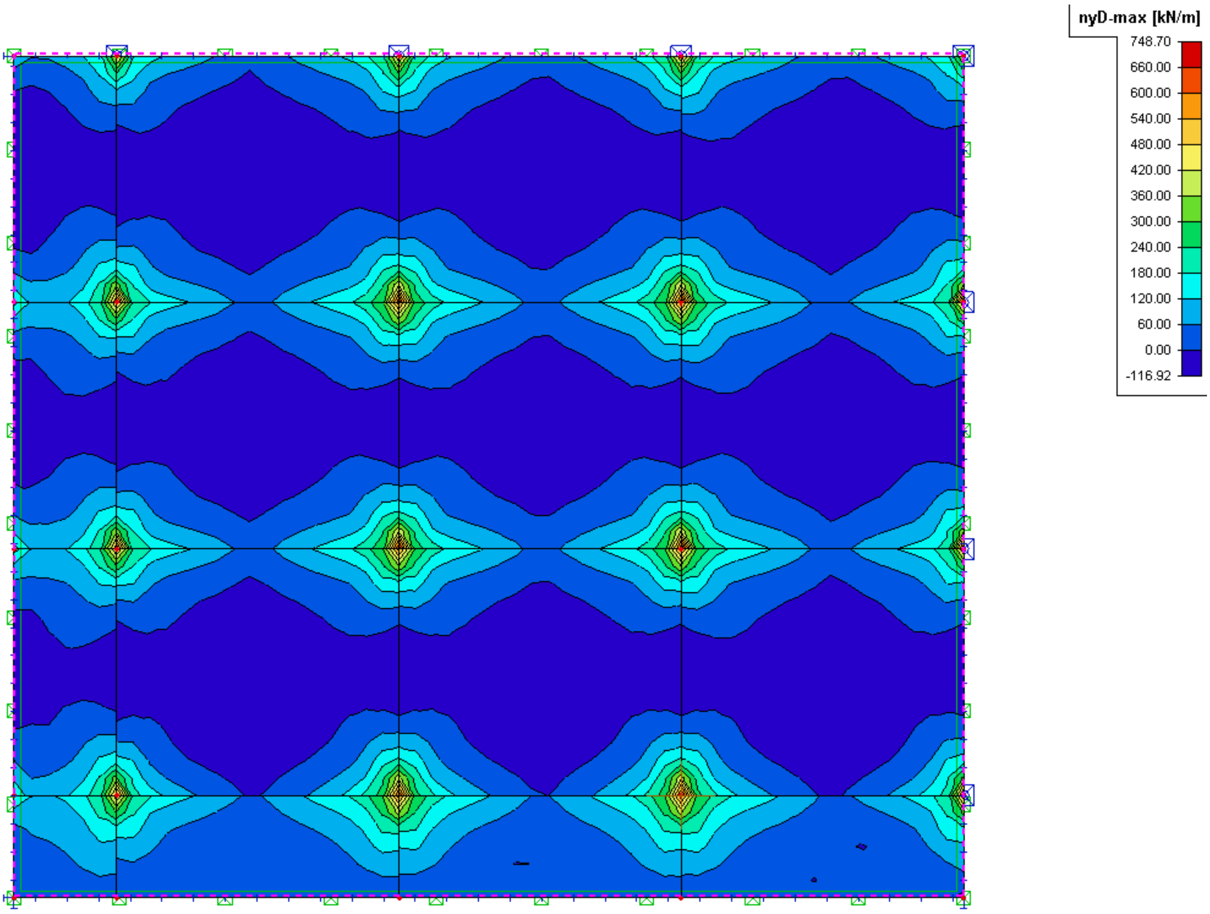


Figure J-6: Tension forces in top floor during operation stage, nyD

<u>One anchor missing</u>

The most unfavourable situation takes place when the corner anchor loses its function, because the three surrounding anchors will have to take its load instead of eight anchors when another anchor loses its function.

The figure on the next page shows the tension forces in the floor when the corner anchor is missing. The shows very high tension forces of 2.600kN/m around the anchor. It is however permitted to average the tension force over 2 * d, which is 400mm. This is shown in the second figure on the next page. This shows that the high tension forces are very local and that the averaged tension force over 2 * d is only 352 kN/m. Therefore, also this situation isn't leading and the leading situation is the immersion stage (see appendix K7).

Conclusions

i. The operation stage isn't leading, the immersion stage is;
ii. 651 anchors are used, 15m[1] long with a capacity of 617kN;
iii. The combined capacity is 651 * 617 = **401.667kN >> 235.072 kN;**
iv. The anchors are drilled from inside the garage element when immersed;
v. The costs will approximately be 651 * €1.340,40 = € 872.600,-.

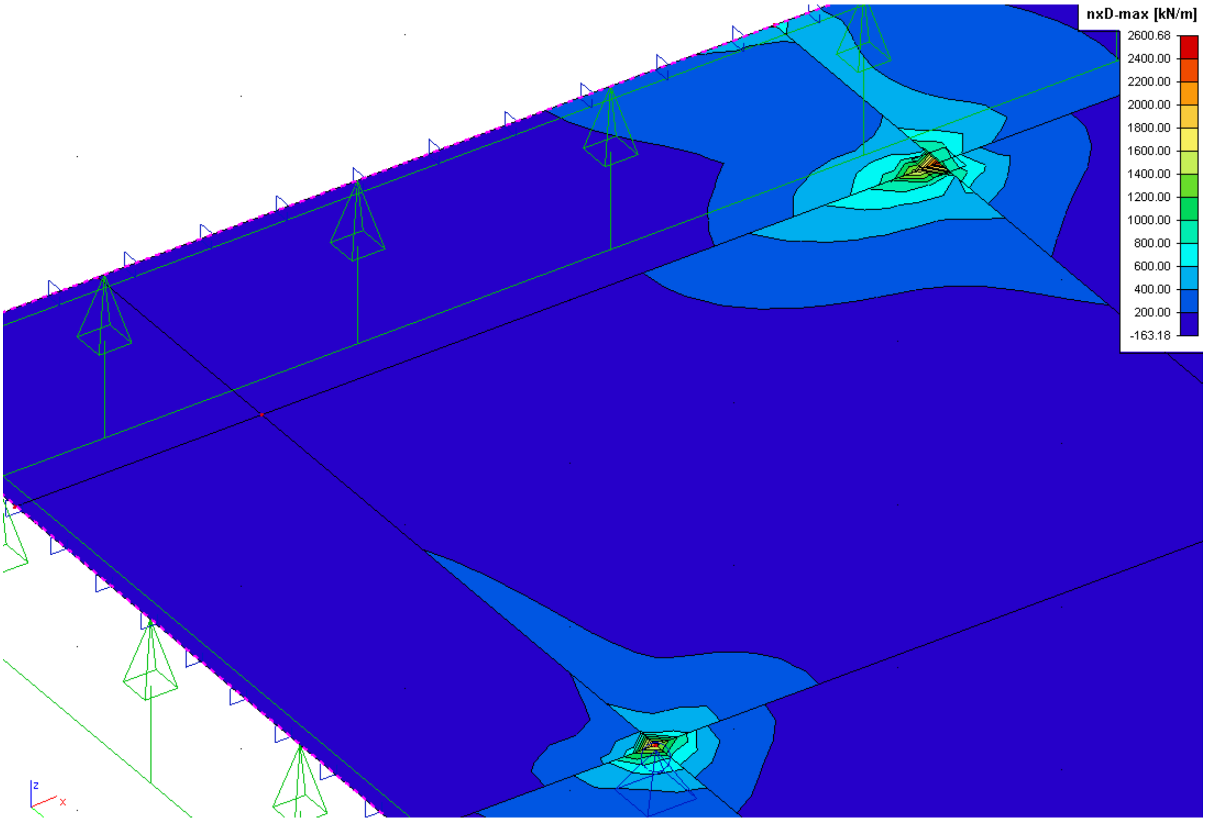


Figure J-7: Tension forces in bottom floor, one anchor missing in corner

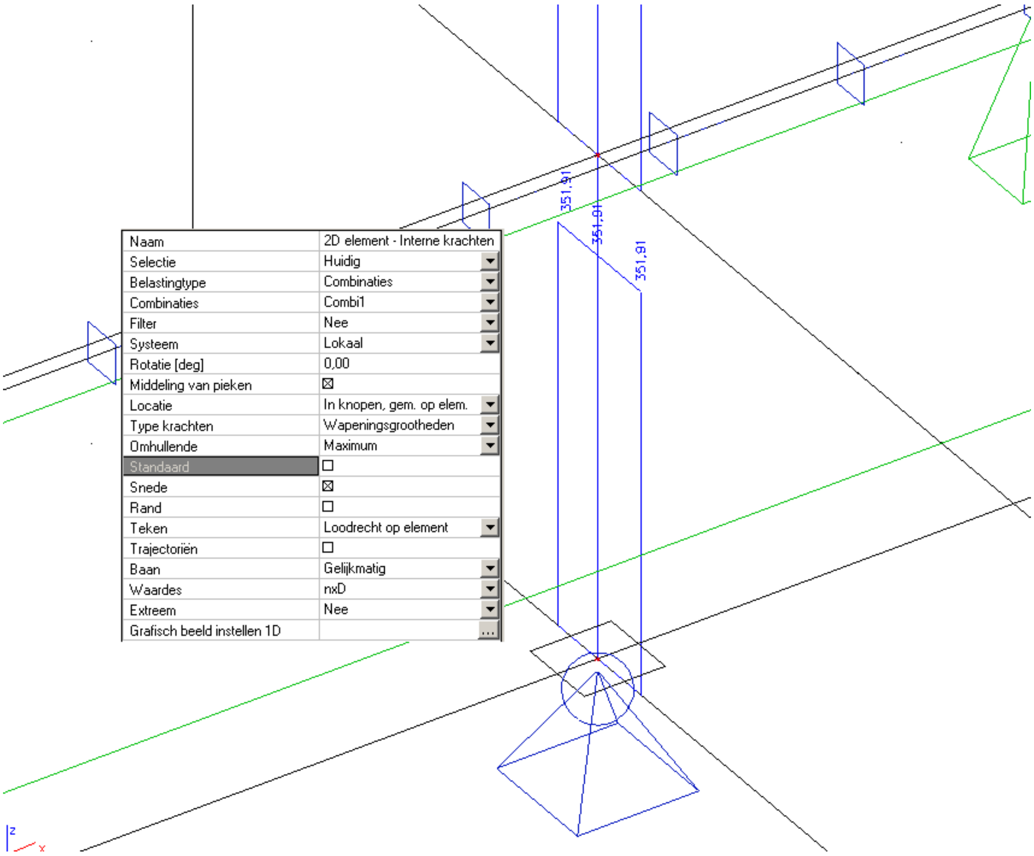


Figure J-8: Tension forces in bottom floor, averaged over 400mm, one anchor missing in corner

K1 SCIA Engineer: input

De complete constructie is gemodelleerd in het programma SCIA om zo de maatgevende spanningen en momenten te vinden. Deze appendices (K1 t/m K7) zullen het gehele proces beschrijven en beginnen met de input.

Gebruikte normen en richtlijnen

i.	NEN 6702	Belastingen en vervormingen;
ii.	NEN 6720	Voorschriften beton, Constructieve eisen en rekenmethoden;
iii.	NEN 6740	Zakkingen en rotaties geotechnische constructies;
iv.	EC 1991-1	Belastingen op constructies.

Gebruikte programmatuur

SCIA Engineering 2010.1

Technische uitgangspunten

Opbouw drijflichaam:	Vlakke betonnen vloeren en wanden	
Veiligheidsklasse:	3	
Belastingfactoren:	**Permanent**	
	Ongunstig UGT:	1,2
	Gunstig UGT:	0,9
	Gunstig/ongunstig BGT :	1,0
	Veranderlijk	
	Ongunstig UGT:	1,5
	Ongunstig UGT	1,2 (enkel bouwfase)
	Ongunstig BGT:	1,0
Referentieperiode:	50 jaar	
Minimale vrijboord:	0,50 cm (enkel bouwfase)	

Materialen

Beton:	Sterkteklasse:	C28/35
	Eigengewicht:	25kN/m^3
	$E_{ongescheurd}$:	31.000N/mm^2
	$E_{gescheurd}$:	10.000N/mm^2
Betonstaal:	FeB500	
Water:	Bedding:	k=10kN/m^3
	Eigengewicht:	10kN/m^3

Eisen rotatie

Tijdens de constructiefase zullen de rotaties en vervormingen gecheckt worden. Hiervoor worden de vervormingseisen uit de NEN 6740 gebruikt.

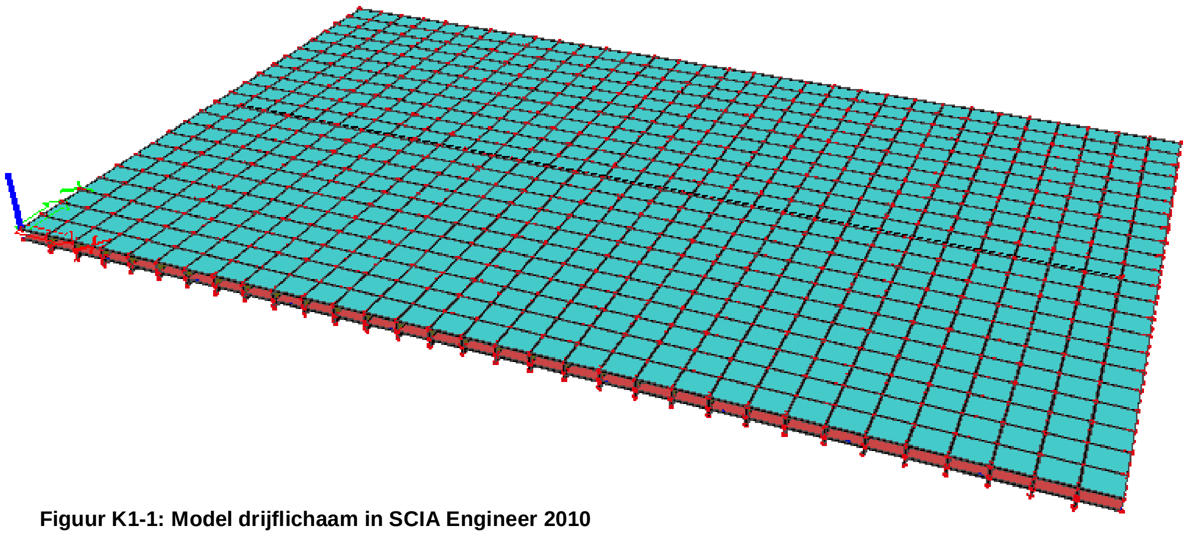
Voor de BGT geldt voor zowel de vervorming als de rotatie maximaal 1:300.

Voor de UGT geldt voor de rotatie maximaal 1:100.

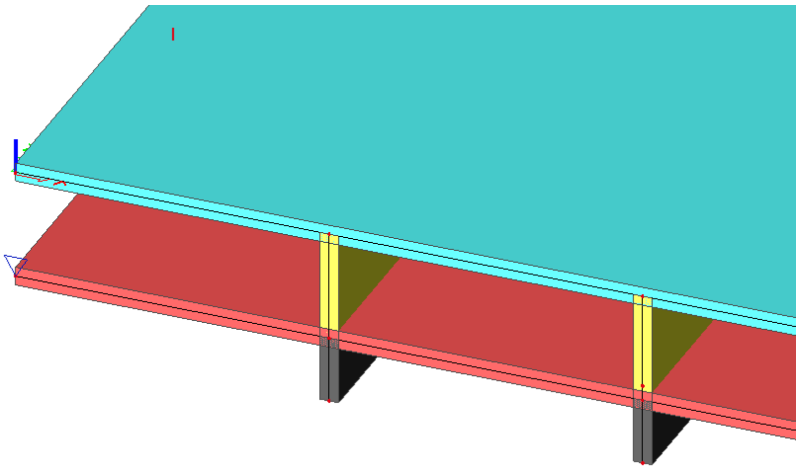
SCIA model: Drijflichaam (eerste versie)
Het drijflichaam uit het eerste ontwerp heeft een oppervlak van 132 bij 79,2 meter. Het drijflichaam is 3,05 meter hoog. Boven op het drijflichaam komt de constructie van de parkeer garage met een totale hoogte van 6,50 meter. In dit model bestaat de vloer uit de Flexbase Heavy variant, dus met twee balkenroosters. Later zal aangetoond worden dat het weglaten van het onderste balkenrooster geen grote veranderingen geeft in de uitkomsten. In het eerste deel van de structurele analyse is daarom deze variant gebruikt.

Het drijflichaam is ingevoerd als een elastisch ondersteunde sandwichconstructie bestaande uit wanden en vloeren. De hart op hart afstand van de wanden in lengte richting is 4,125 meter (31 balken) en in de breedte richting 3,60 meter (21 balken). De wanden zijn 250 mm dik. De onderste wanden zijn 750 mm hoog en de bovenste wanden zijn 1000 mm hoog. De vloeren zijn 250 mm dik.

Het EPS is buiten beschouwing gelaten bij de modellering van de constructie. Het drijflichaam is in de constructiefase elastisch ondersteund, de bedding is 10 kN/m^3. Om het drijflichaam in de x- en y-richting vast te zetten zijn twee ondersteuningen aangebracht in het midden. De ondersteuningen zijn vrij in de z-richting en vast in de x- en y-richting. Dit voorkomt onrealistische verplaatsing rondom de x- en y-as.



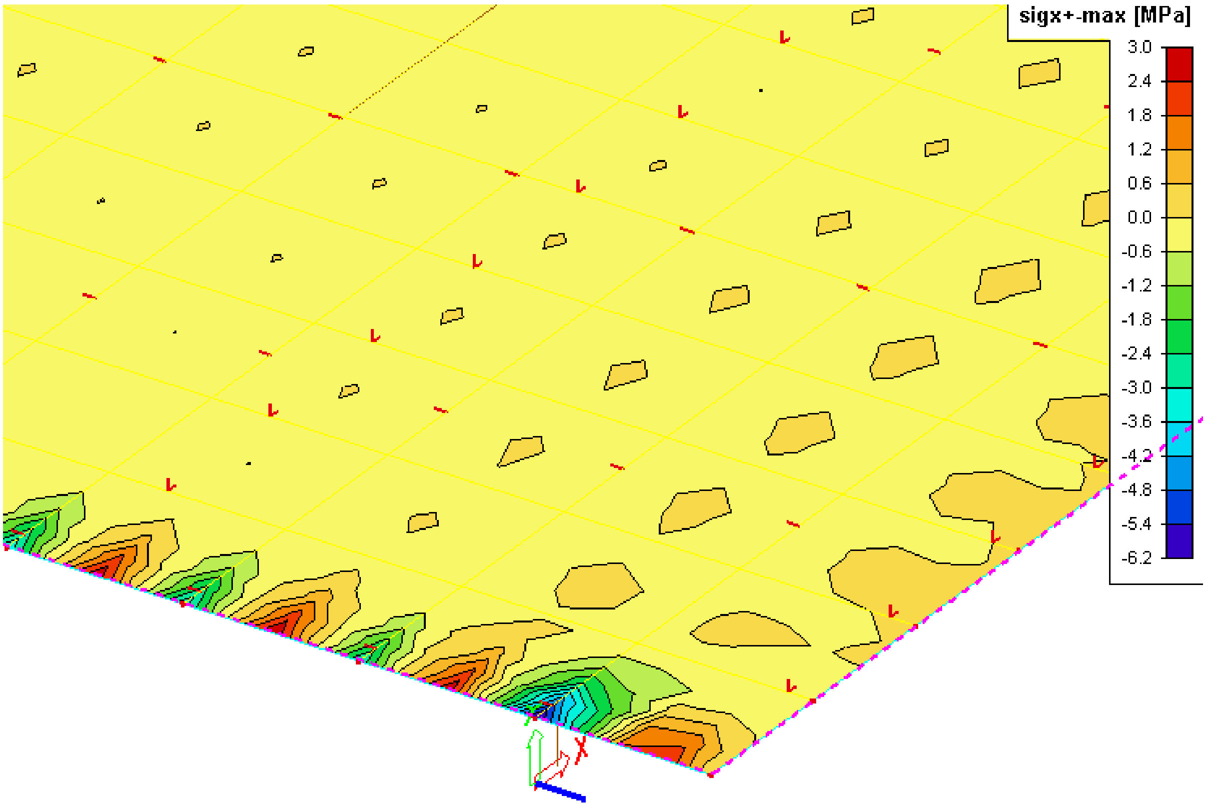
Figuur K1-1: Model drijflichaam in SCIA Engineer 2010



Figuur K1-2: Detail model drijflichaam in SCIA Engineer 2010

Problemen met eerste ontwerp drijflichaam

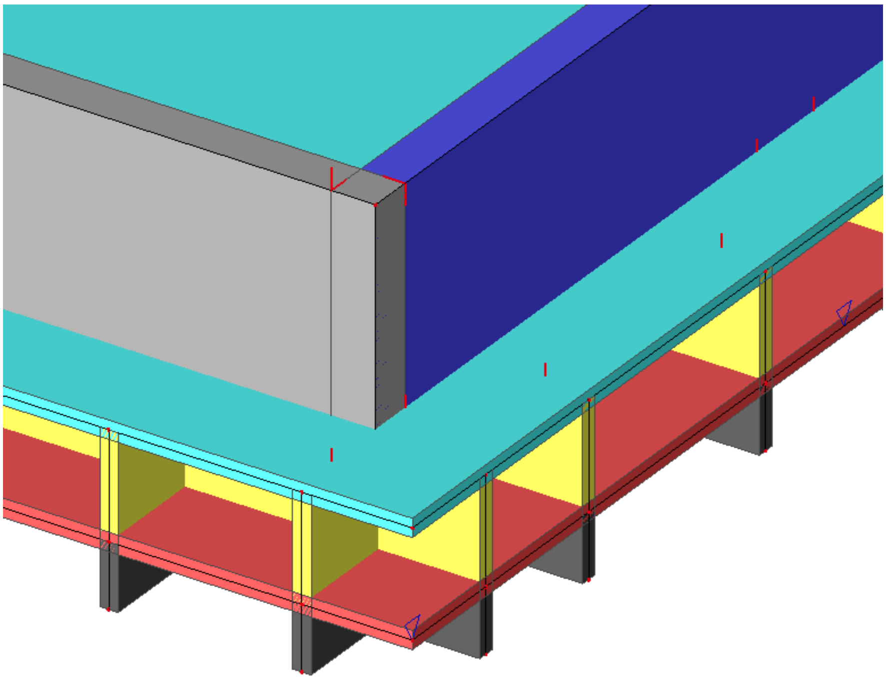
Het grootste probleem van het eerste ontwerp is het ontbreken van voldoende stijfheid aan de randen. Wanneer de wanden worden geplaatst op de randen van het drijflichaam treden er grote vervormingen en spanningen op, zoals te zien is in de onderstaande figuur.



Figuur K1-3: Spanningen in Flexbase vloer, trek in x-richting

SCIA model: Drijflichaam (tweede versie)

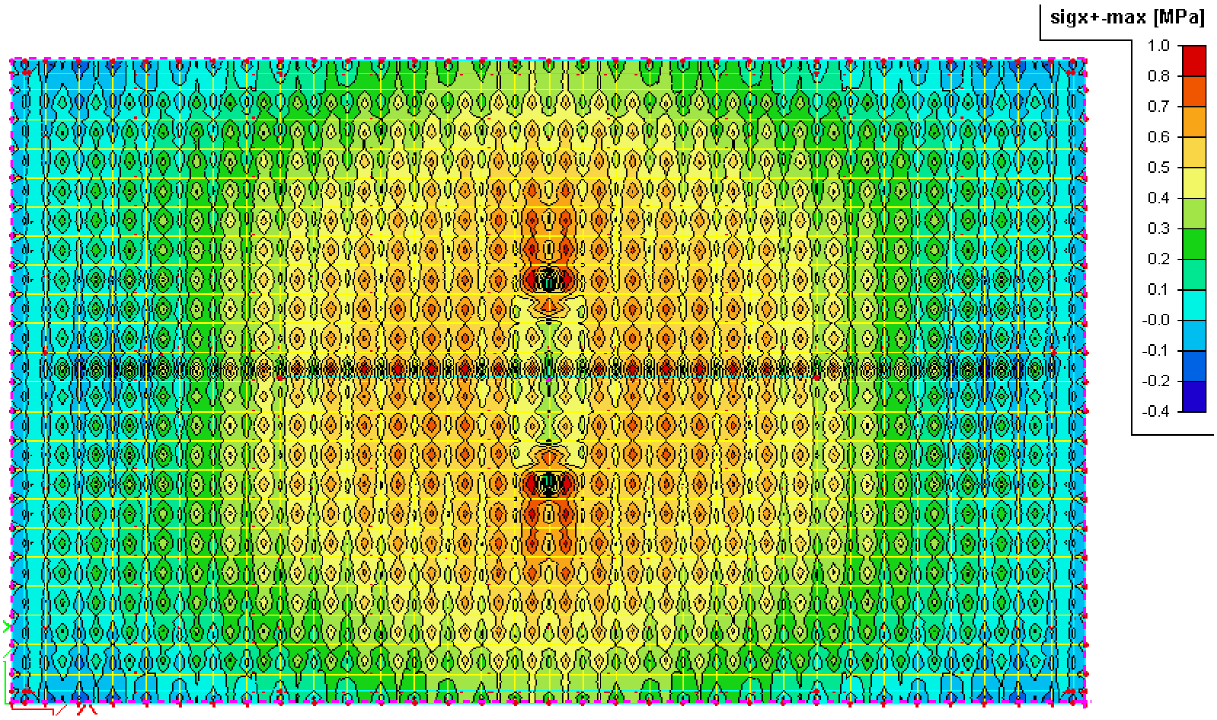
Om dit op te lossen zijn extra wanden in het drijflichaam geplaatst ter hoogte van de wanden van de bovenbouw. Dit is te zien in de onderstaande figuur. Deze wanden maken het drijflichaam stijver aan de randen en zorgen dat deze plaatselijke piekspanningen in minder mate optreden.



Figuur K1-4: Detail van extra wanden Flexbase vloer

In de onderstaande figuur is dit goed te zien. De piekspanningen aan de zijkant van het drijflichaam zijn bijna compleet verdwenen. In plaats van 3,0 N/mm^2 trekspanning is de hoogste trekspanning nu 1,0 N/mm^2. Dit is een aanzienlijke verbetering. Om er zeker van te zijn dat de piekspanningen niet hoger zijn, zijn ook nog sneden ter plaatse van de piekspanning genomen. Hieruit blijkt ook dat de spanningen niet hoger dan 1,0 N/mm^2 zijn. De hoogste spanningen zijn te zien onder de wand in het midden. Deze wand staat niet precies boven een wand in het drijflichaam. Wanneer dit wel wordt gedaan, worden de spanningen hoogstwaarschijnlijk nog verder naar beneden gebracht. Dit zal niet meer in dit model worden aangepast, maar wordt als aanbeveling meegegeven.

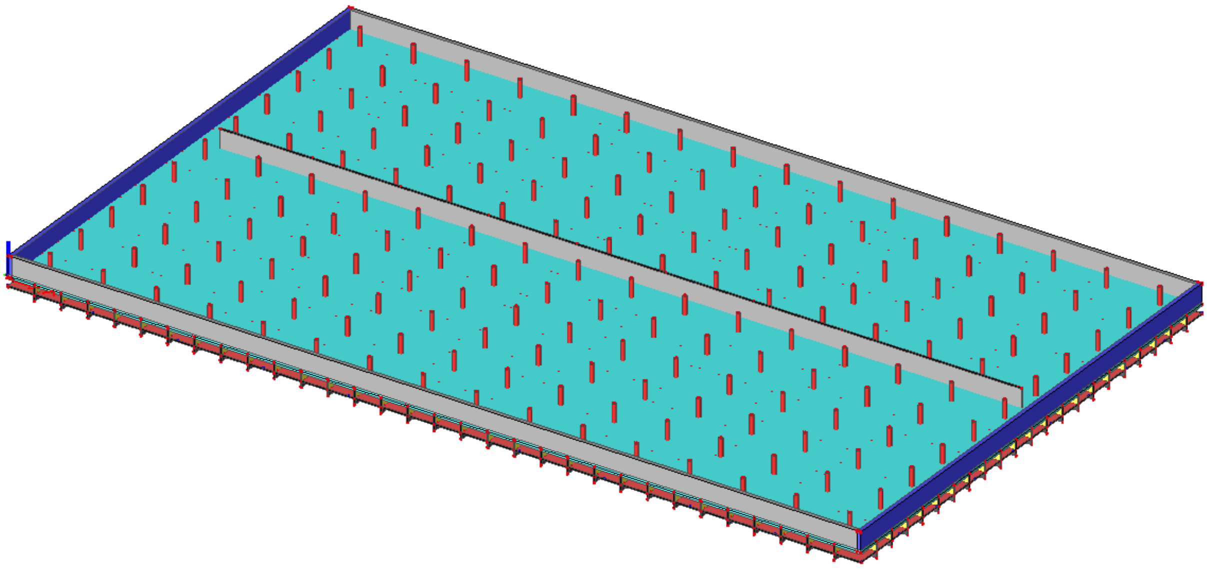
In de figuur zijn ook duidelijke de verstoringen te zien rondom de twee ondersteuningen in het midden. Dit zijn verstoringen die in realiteit niet zullen optreden.



Figuur K1-5: Spanningen in Flexbase vloer, nieuwe model, trek in x-richting

SCIA model: Bovenbouw
De bovenbouw bestaat uit twee lagen, beiden met buitenwanden, één tussenwand en kolommen. De buitenwanden zijn 600 mm dik en 2,60 meter hoog. De tussenwand is 400 mm dik en 2,60 meter hoog. De kolommen hebben een hart op hart afstand van tweemaal de h.o.h. van de wanden van het drijflichaam. De kolommen staan dus allemaal op een kruising van twee balken. Dit is gedaan om pons tegen te gaan. De kolommen zijn rond en hebben een diameter van 600mm.

Om het model te vereenvoudigen zijn de overige binnenwanden, de trappen en de auto-ramps weggelaten. Onderstaande figuur laat het drijflichaam met de onderste laag zien. De tweede laag wordt hetzelfde opgezet.



Figuur K1-6: Model bovenbouw in SCIA Engineer 2010

K2 SCIA Engineer: model control

Het Flexbase drijflichaam is geen traditionele constructie en daarom zal enigszins aangetoond moeten worden dat het model in SCIA de juiste uitkomsten geeft. Hiervoor wordt naar drie punten gekeken, namelijk rotatie, zakking en vervormingen.

Rotatie

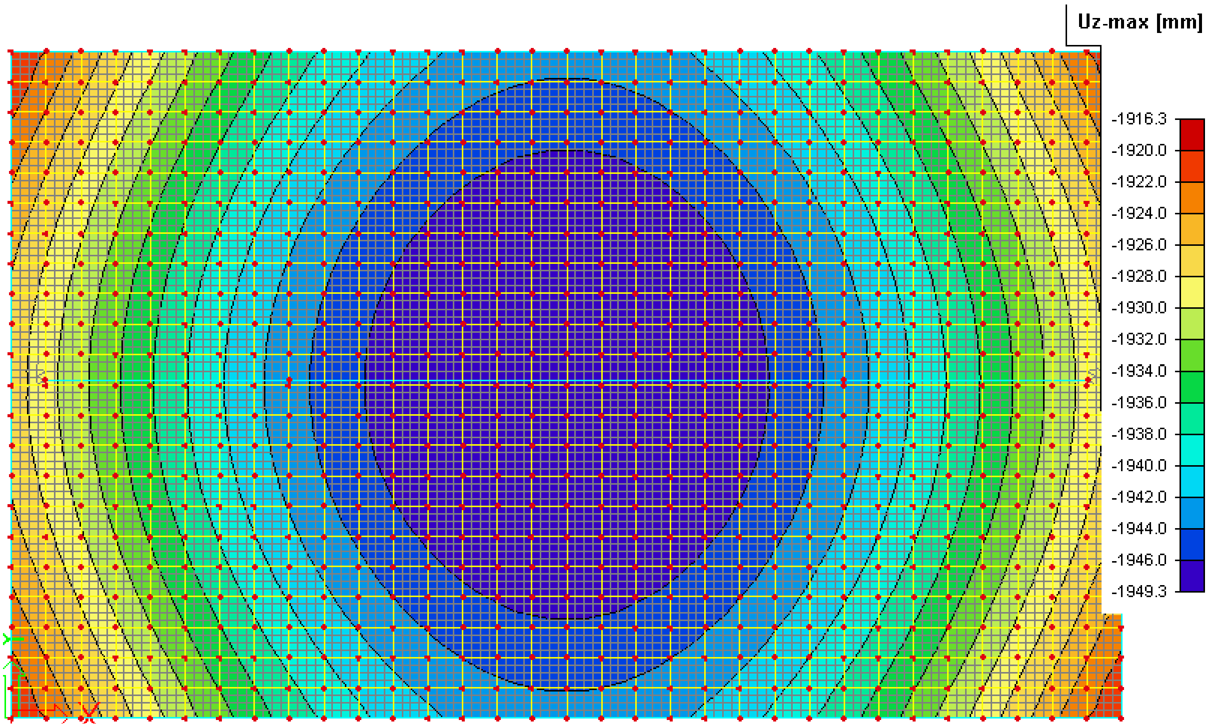
Hiervoor wordt verwezen naar het rapport van Maarten Kuiper waarin werd aangetoond dat SCIA de juiste rotaties geeft voor drijvende vloeren. [M. Kuiper, 2006]

Rotatie → OK

Zakking

In de vorige appendix is het model van de Flexbase vloer beschreven. In de onderstaande figuur zijn de bijbehorende zakkingen te zien. De gemiddelde zakking is 1934 mm, oftewel 1,93 meter. Handmatig was een zakking van 1,97 meter berekend. Dit is iets meer, wat is te verklaren door het ontbrekende EPS in het SCIA model. Voor de zakking en de rotatie geeft SCIA dus correcte uitkomsten.

Zakking → OK



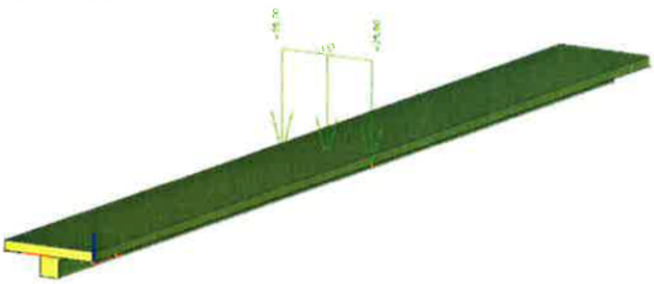
Figuur K2-1: Vervorming drijflichaam [SCIA Engineer 2010]

Vervorming
Hiervoor wordt verwezen naar het rapport van ‘Controle berekeningen Paviljoen’ van Advin BV. Hierin is onder andere de onderstaande test uitgevoerd. De test laat zien dat SCIA ook goede uitkomsten geeft voor de vervorming van een wand en vloer die elastisch zijn ondersteund.

De constructiefase van het Paviljoen, en met name de constructiefase van de Flexbase vloer, is te vergelijken met die van het garage element. Voor het Paviljoen is aangetoond dat SCIA correcte output geeft. Er kan daarom vanuit gegaan worden dat SCIA Engineer ook goede output geeft voor de Flexbase vloer van het garage element.

- Bij de plaat met lijnlast = 25 kN/m^1, sm = 0 kN/m^3 en E = 100 N/mm^2 wordt nu een balk in langsrichting toegevoegd met b x h = 0,2 x 0,2 m met sm = 0 kN/m^3 en E = 100 N/mm^2.

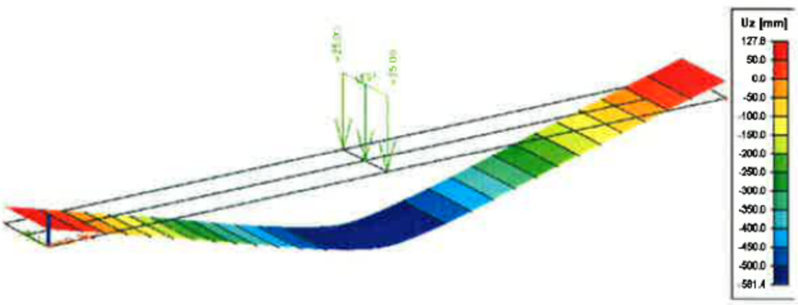
  Scia invoer:



Scia uitvoer:

Som van lasten en reacties.

	[kN]	X	Y	Z
BG 1	last	0.0	0.0	0.0
	knoopreacties	0.0	0.0	0.0
	lijnreactions	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	0.0
BG 2	last	0.0	0.0	-25.0
	knoopreacties	0.0	0.0	0.0
	lijnreactions	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	25.0

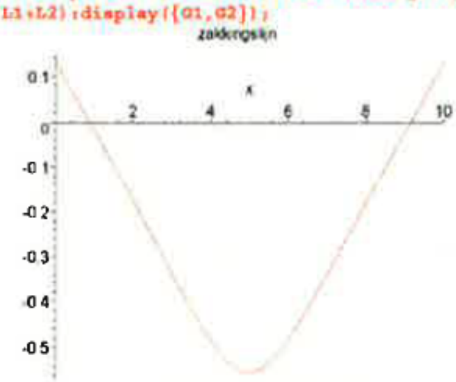


Handmatige controle:
→ gewichtscontrole voldoet
→ De plaat reageert stijver (conform verwachting) → OK
Controle mbv Maple:

```
> restart;
> w1:=C1*exp(beta*x)*sin(beta*x)+C2*exp(beta*x)*cos(beta*x)+C3*exp
  (-beta*x)*sin(beta*x)+C4*exp(-beta*x)*cos(beta*x):
> w2:=D1*exp(beta*x)*sin(beta*x)+D2*exp(beta*x)*cos(beta*x)+D3*exp
  (-beta*x)*sin(beta*x)+D4*exp(-beta*x)*cos(beta*x):
> phi1:=-diff(w1,x):phi2:=-diff(w2,x):
> kappa1:=diff(phi1,x):kappa2:=diff(phi2,x):
> M1:=EI*kappa1:M2:=EI*kappa2:
> V1:=diff(M1,x):V2:=diff(M2,x):
> x:=0:eq1:=M1=0:eq2:=V1=0:
> x:=L1:eq3:=w1=w2:eq4:=phi1=phi2:eq5:=M1=M2:eq6:=V1-V2-F=0:
> x:=L1+L2:eq7:=M2=0:eq8:=V2=0:
> sol:=solve({eq1,eq2,eq3,eq4,eq5,eq6,eq7,eq8},{C1,C2,C3,C4,D1,D2,
  D3,D4}):assign(sol):x:='x':
> EI:=85.95238;L1:=5;L2:=5;F:=25;k:=10;beta:=(k/4/EI)^(1/4):
```

$$EI := 85.95238$$
$$L1 := 5$$
$$L2 := 5$$
$$F := 25$$
$$k := 10$$

```
> with(plots):G1:=plot(-w1,x=0..L1,title="zakkingslijn"):G2:=plot(
  -w2,x=L1..L1+L2):display({G1,G2});
```



$$w\_max := 555.4950747\ mm$$

Doorbuiging$_{\text{Scia Engineer}}$ = 581 mm ≈ Doorbuiging$_{\text{Maple}}$ = 555 mm → OK

Deze test laat zien dat SCIA goede resultaten geeft voor de vervormingen van een constructie die bestaat uit een combinatie van een wand en een vloer die elastisch ondersteund is. Aangezien de Flexbase vloer ook bestaat uit een combinatie van wanden en vloeren, ga ik er van uit dat SCIA hier de juiste output voor geeft. **Vervormingen → OK**

K3 SCIA Engineer: Rotation and distortion

Er zal gekeken worden naar de rotaties en vervormingen in de volgende situaties:

i. Bouw van wanden op Flexbase vloer;
ii. Wind belasting tegen lange zijde van element.

Eisen rotatie

Tijdens de constructiefase zullen de rotaties en vervormingen gecheckt moeten worden. Hiervoor worden de vervormingseisen uit de NEN 6740 gebruikt.

Voor de BGT geldt voor zowel de vervorming als de rotatie maximaal 1:300.

Voor de UGT geldt voor de rotatie maximaal 1:100.

Vervormingen tijdens bouw eerste wanden

Tijdens de bouw van de wanden zijn er twee fases te onderscheiden:

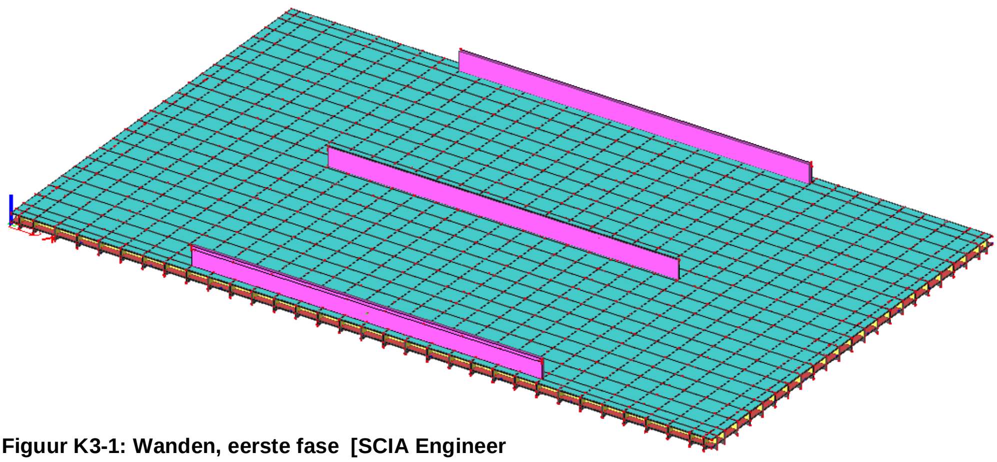
i. Storten van de wanden: Belasting werkt op vloer, maar de wanden werken nog niet constructief mee;
ii. Wanden zijn hard (na 28 dagen): De wanden helpen constructief mee en maken de constructie stijver.

Om dit te simuleren worden eerst de vervormingen bekeken wanneer enkel de belasting van de wanden op de vloer werkt. Daarna worden de wand ingevoerd in SCIA en wordt nogmaals naar de vervormingen gekeken (de verwachting is dat de vervormingen dan kleiner zijn).

De wanden worden niet in één keer gestort, maar in drie fases. De drie fases zullen stuk voor stuk doorlopen worden.

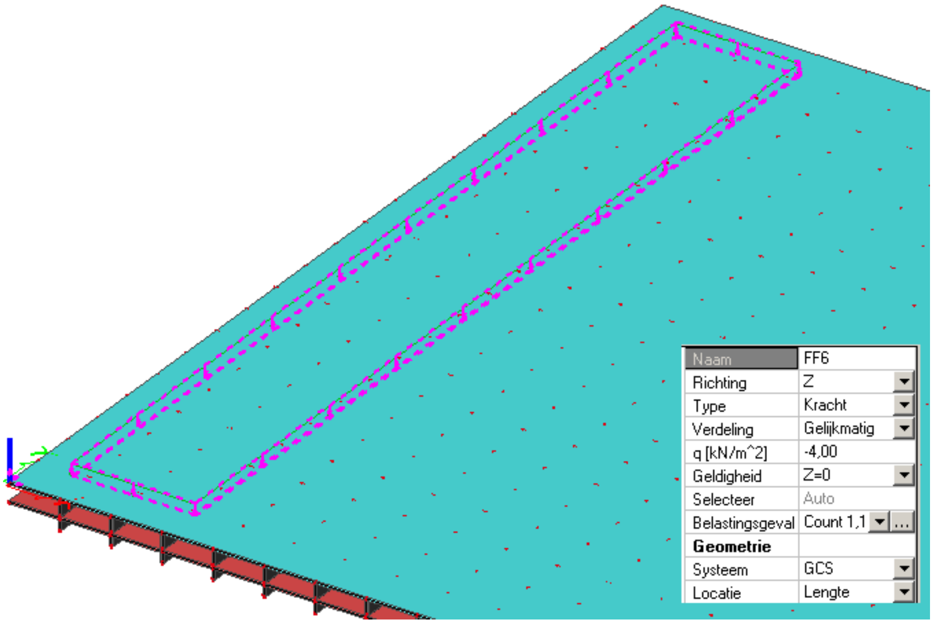
<u>Fase 1</u>

In fase één wordt worden de buitenwanden en de binnenwand gestort over een lengte van 66 meter (de helft van de totale lengte). In de onderstaande figuur zijn de wanden in SCIA ingevoerd. Voor de eerste berekening zijn ze echter als een verdeelde belasting ingevoerd. De wanden zijn 2,60 meter hoog, dit maakt de verdeelde belasting 65 kN/m^2 (2,60 x 25). Hierbij wordt nog een toeslag van 5kN/m^2 bij opgeteld voor de bekisting.



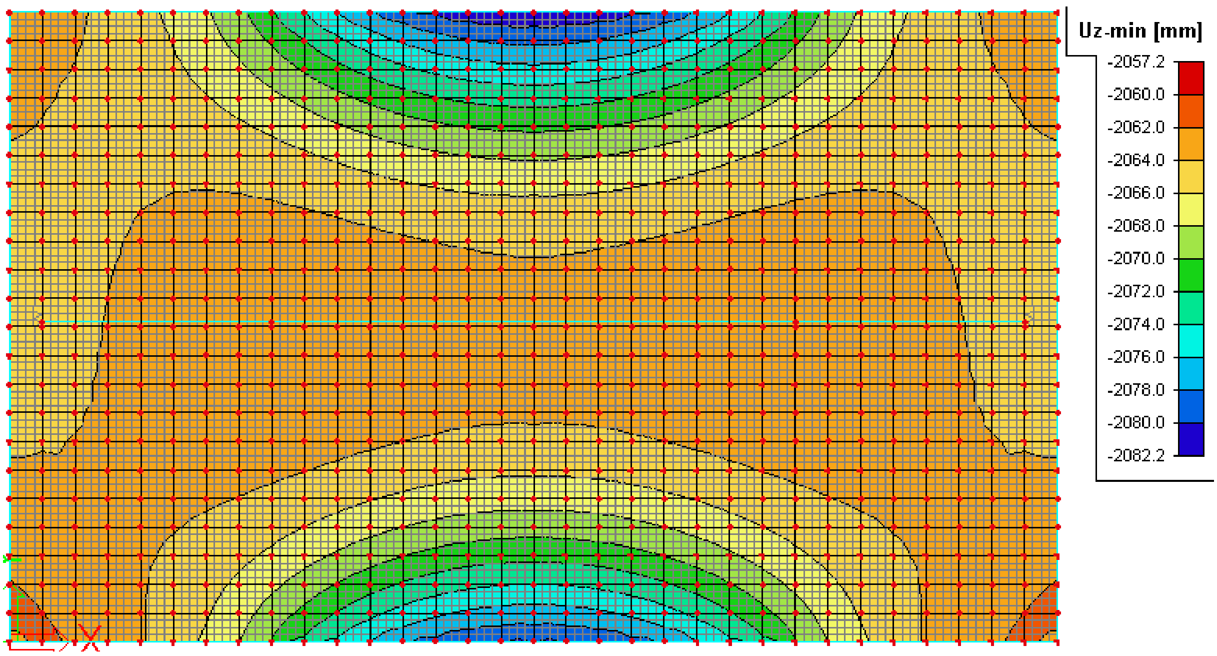
Figuur K3-1: Wanden, eerste fase [SCIA Engineer 2010]

De vervormingen worden te groot wanneer enkel deze wanden op het drijflichaam worden geplaatst. De zakking in het midden wordt meer dan 15 centimeter groter dan aan de zijkanten (is groter dan 1:100) en het drijflichaam gaat hol staan. Om dit tegen te gaan worden contra gewichten geplaatst op beide uiteinden zoals is te zien in de onderstaande figuur. De belasting wordt aangebracht over een oppervlakte van 10 bij 70 meter. De belasting is 4 kN/m^2.



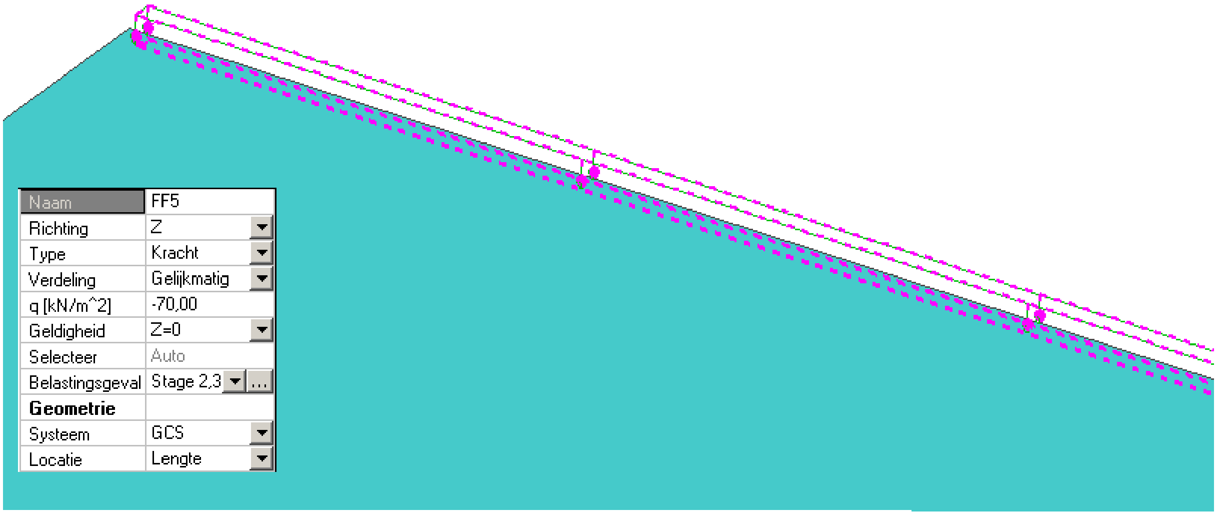
Figuur K3-2: Contra belasting, eerste fase [SCIA Engineer 2010]

Door deze contragewichten te plaatsen wordt het maximale zakkingsverschil 25mm. De maximale rotatie is te zien aan de lange kant in het midden en is 4/3800 = **1:950 → OK.**



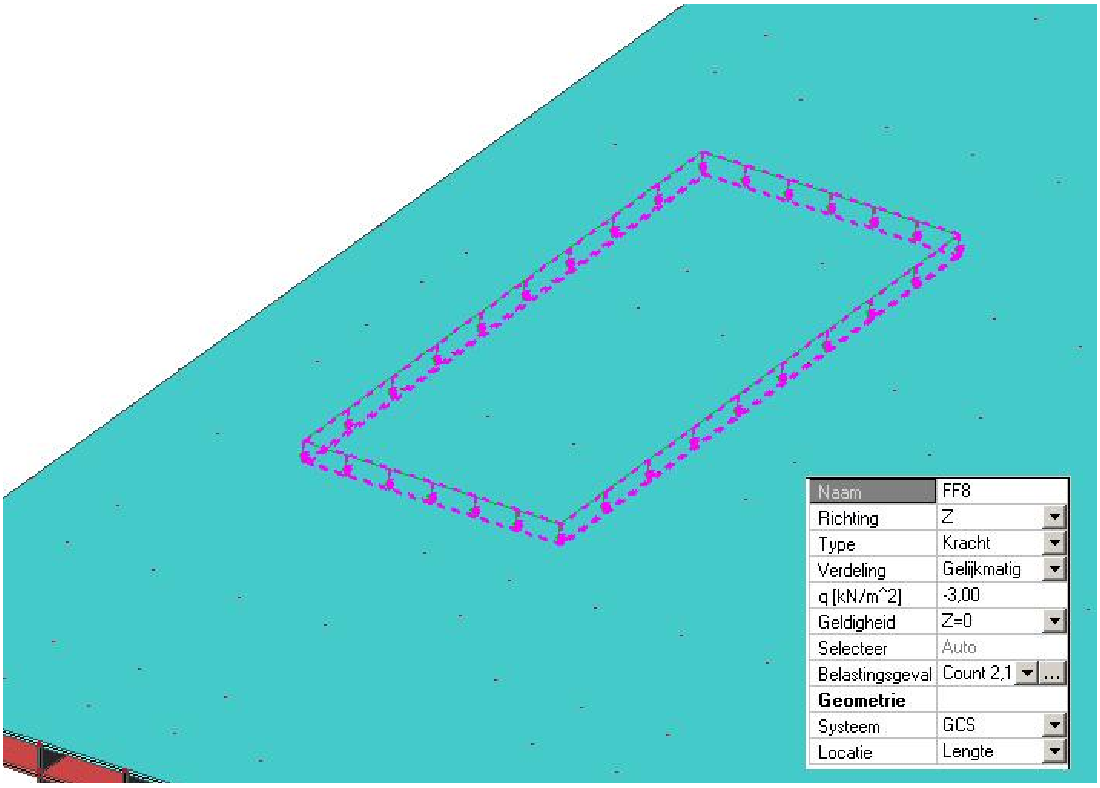
Figuur K3-3: Vervormingen eerste fase [SCIA Engineer 2010]

Fase 2
In de tweede fase wordt de rest van de wanden over de lengte gestort. Er wordt nog vanuit gegaan dat de wanden van fase één niet constructief meedoen. De wanden worden gemodelleerd als een verdeelde belasting. Een deel van de belasting is te zien in figuur K3-4 met de bijbehorende parameters.



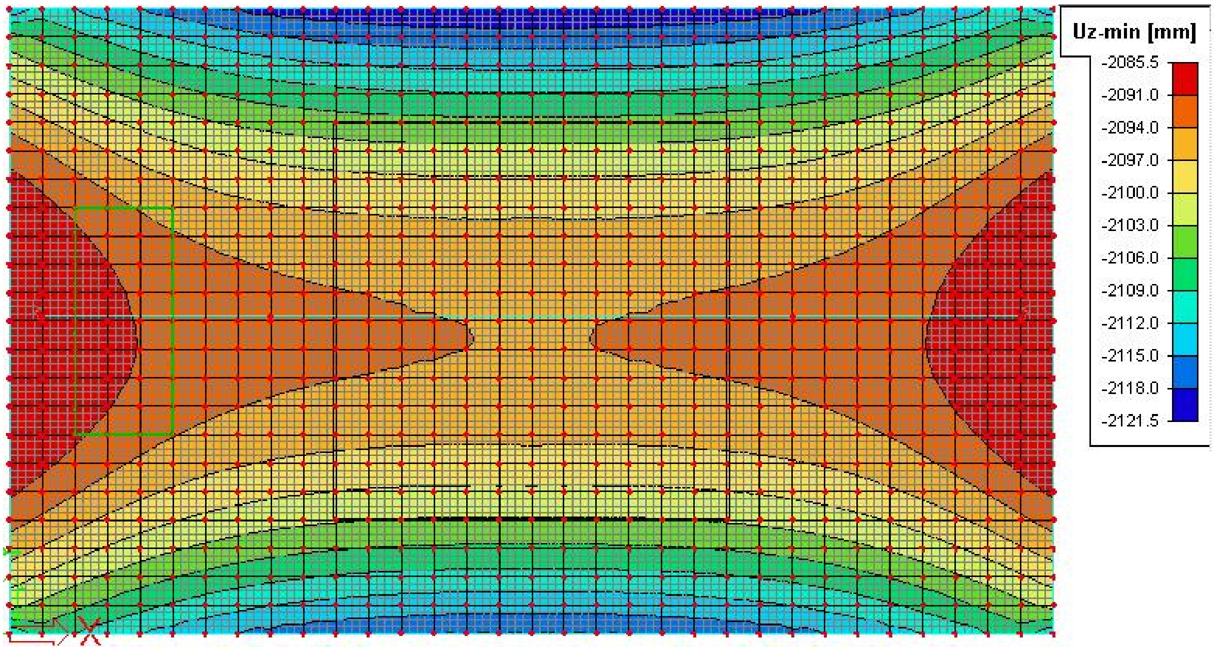
Figuur K3-4: Verdeelde belasting wanden, tweede fase [SCIA Engineer 2010]

Ook in deze fase worden er contragewichten aangebracht om de vervorming te minimaliseren. Deze contragewichten worden aangebracht aan beide zeiden van het drijflichaam. De invoer in SCIA is te zien in de onderstaande figuur. De belasting wordt aangebracht over een oppervlakte van 10 bij 30 meter en bedraagt 3 kN/m^2.



Figuur K3-5: Contra belasting, tweede fase [SCIA Engineer 2010]

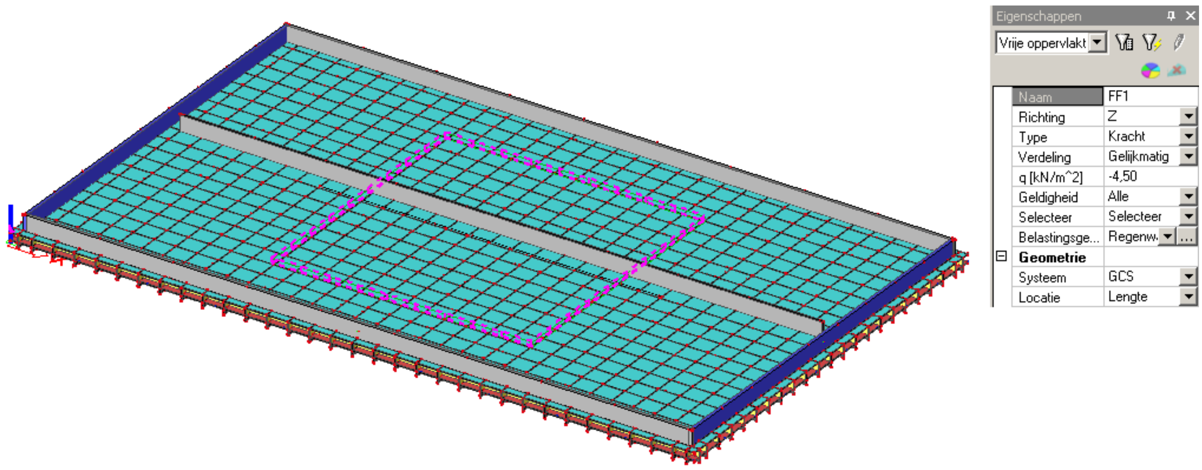
De vervormingen tijdens de tweede fase zijn te zien in de onderstaande figuur. Het maximale zakkingsverschil wordt 25mm. De maximale rotatie is te zien aan de lange kant in het midden en is 10/5500 = **1:550 → OK.**



Figuur K3-6: Vervormingen Flexbase vloer, tweede fase [SCIA Engineer 2010]

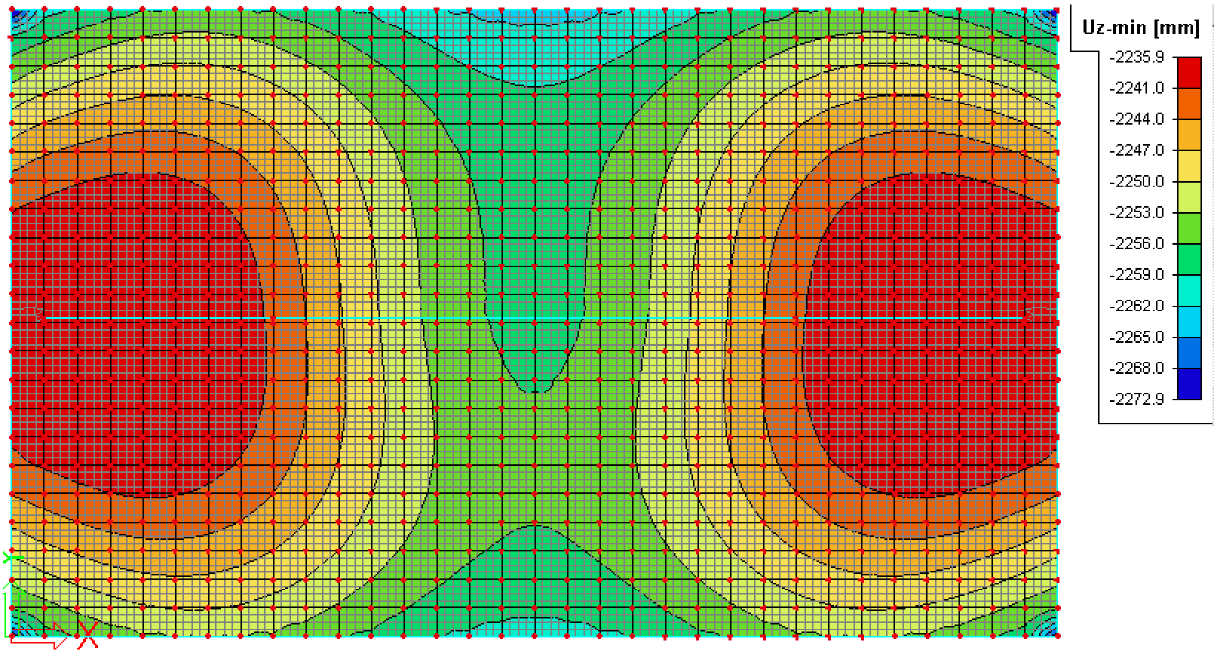
Fase 3

Tijdens de derde en laatste fase worden de wanden op de korte zijde gestort. In eerste instantie is gekeken naar de vervormingen wanneer alle wanden worden ingevoerd als verdeelde belastingen. Dit gaf te grote vervormingen. Daarom zijn ook in deze fase contragewichten geplaatst die te zien zijn in de onderstaande figuur. Het gewicht wordt verdeeld over een oppervlakte van 50 bij 50 meter en bedraagt 4,50 kN/m^2.



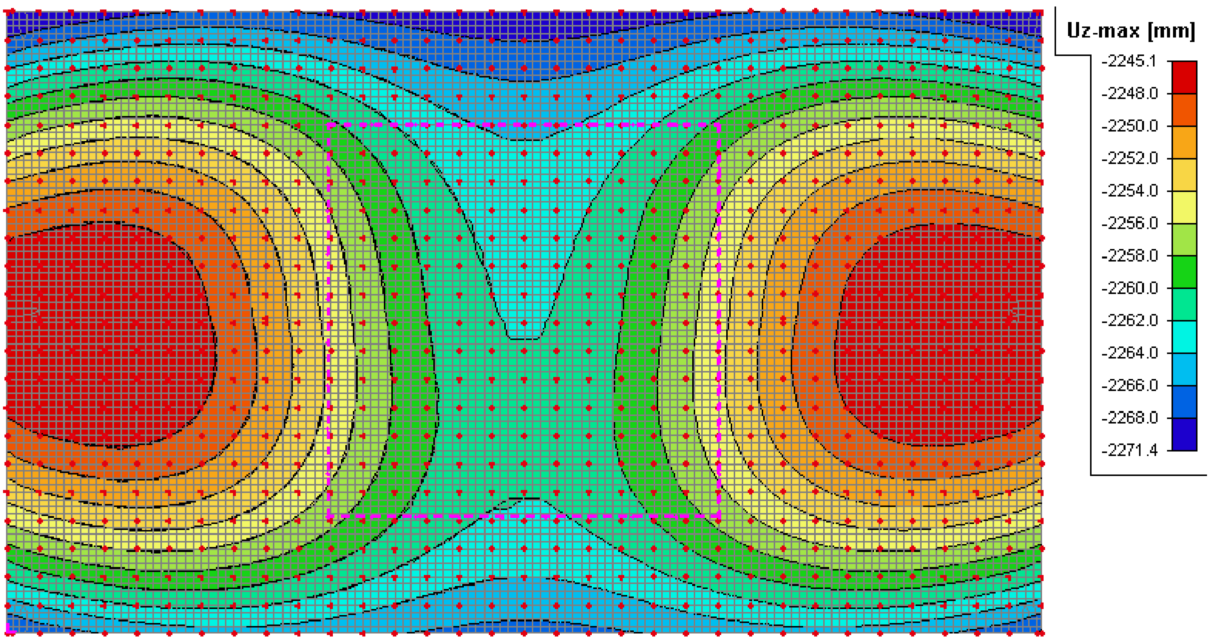
Figuur K3-7: Contra belasting, derde fase [SCIA Engineer 2010]

De bijbehorende vervormingen zijn te zien in figuur K3-8.



Figuur K3-8: Vervormingen derde fase, wanden ingevoerd als verdeelde belastingen [SCIA Engineer 2010]

De vervormingen blijven binnen het maximum van 1:100, maar in de hoeken komt de rotatie dicht in de buurt van het maximum (+/- 1:150). Daarom is er ook gekeken naar de vervormingen als de wanden uit fase één en twee uitgehard zijn en de wanden in fase drie als belastingen worden ingevoerd. Nu werken de wanden uit fase één en twee dus constructief mee en worden de hoeken stijver. De bijbehorende vervormingen zijn te zien in de onderstaande figuur.



Figuur K3-9: Vervormingen derde fase, enkel wanden uit derde fase ingevoerd als verdeelde belastingen [SCIA Engineer 2010]

De figuur laat zien dat de vervormingen in de hoeken en ook de totale vervormingen kleiner worden.
De maximale rotatie is nu 4/5500 = **1:1375 → OK.**

Conclusie:
De rotaties tijdens de bouw zijn, met plaatsing van contragewichten, maximaal 1:550.

Wind belasting op lange zijde element
De windbelastingen zijn in appendix C berekend en de maximale windbelasting is 0,55 kN/ m^2. De onderstaande figuur geeft de factoren aan voor de belastingen op de verschillende delen van het element. De windbelasting werkt over een hoogte van vier meter op de lange wand. Deze belasting bedraagt 0,37 kN/ m^2. Op de eerste vier meter van het dak werkt een zuigende belasting van -0,37 kN/m^2. Op het resterende deel van het dak en op de wand aan de andere zijde werkt een zuigende belasting van -0,18 kN/ m^2.

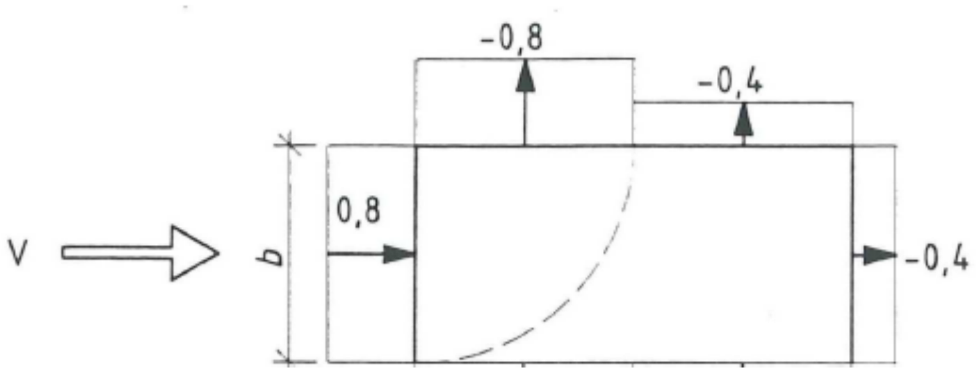
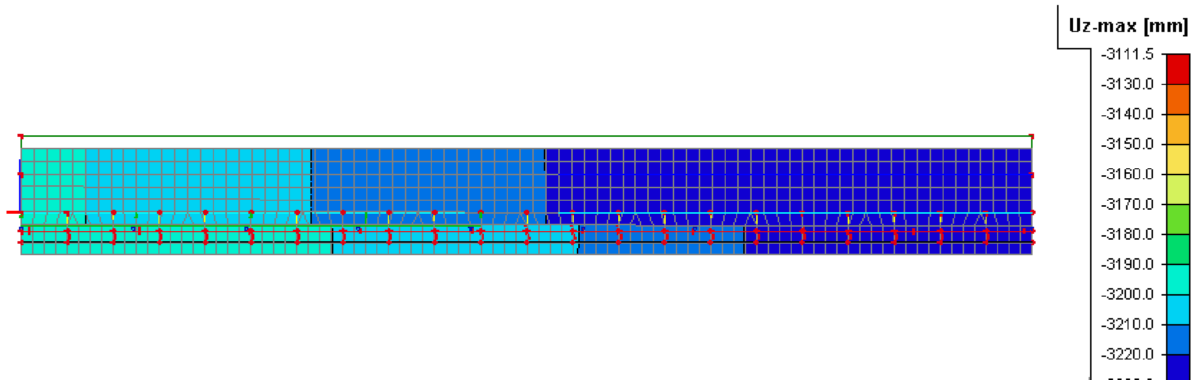


Figure 8-3 wind type factors on vertical walls

Het gehele garage element is gemodelleerd in SCIA en de windbelastingen zijn ingevoerd. De resultaten zijn te zien in de onderstaande figuur. De figuur laat de verplaatsingen zien van de korte zijde. Het maximale verschil in verplaatsing is 4,60 cm over een lengte van 79,2 meter. Dit is een rotatie van **1:1721**. De rotatie door de wind is dus vrijwel nihil.



Figuur K3-10: Rotatie van element door windbelasting, aangezicht: korte zijde [SCIA Engineer 2010]

K4 SCIA Engineer: Pouring scheme walls

Deze appendix beschouwt de bouw van de wanden van de onderste verdieping. Het bouwen van het drijflichaam en de wanden is verdeelt in vier fases, namelijk:

i. Storten van de eerste vloer;
ii. Storten tweede balkenrooster;
iii. Storten tweede vloer;
iv. Storten wanden onderste verdieping.

Tijdens elke fase wordt gekeken naar de maximale spanningen in het beton. Aan de hand van deze spanningen zal de benodigde wapening van de balken en vloeren berekend worden. Het storten van de overige wanden en kolommen wordt niet bekeken, omdat verondersteld wordt dat dit niet de maatgevende situaties zijn wat betreft de spanningen in het beton en EPS. Deze appendix beschouwd fase 4.

Fase 4: Storten wanden onderste verdieping

Wanneer het drijflichaam af is worden de wanden van de eerste verdieping gestort. Dit wordt gedaan in drie fases, zoals is beschreven in appendix K3. In elke fase wordt gekeken naar de spanningen in de balken en de vloeren. Fase 3 bestaat uit twee sub-fases, namelijk 3.1 en 3.2. In fase 3.1 zijn de wanden gestort, maar nog niet uitgehard. In fase 3.2 zijn alle wanden uitgehard en wordt gekeken naar de spanningen in de buitenwanden en tussenwand.

Voor de balken worden de balken in x-richting (lengterichting) en in y-richting (breedterichting) onderscheiden.

Voor de vloeren wordt gekeken naar de maximale spanningen in x- en y-richting aan de bovenkant van de vloer (SigY- en SigX-) en aan de onderkant van de vloer (SigY+ en SigX+). In de vloer zijn namelijk niet de spanningen over de hoogte van de vloer te zien zoals bij de balken. Daarom worden de spanningen aan de onder en bovenkant opgevraagd.

Notitie 1: Tijdens het uitlezen van de waardes die in deze appendix staan was de z-as omgedraaid in SCIA. Normaliter zijn de "+" waardes de bovenkant van de vloer en de "–" waardes de onderkant.

Notitie 2: In deze appendix heeft het drijflichaam nog twee balkenroosters. Het definitieve ontwerp heeft maar één balklaag. Later wordt aangetoond dat de uitkomsten met of zonder het onderste balkenrooster weinig verschillen. (pagina 182)

Counterweight

Om de maximale vervormingen en spanningen te beperken worden er tijdens de constructiefase contragewichten aangebracht. Deze contragewichten worden aangebracht voordat het beton wordt gestort. De spanningen in het beton zullen dus ook bekeken moeten worden in de fases. Dit is ook beschreven in deze appendix.

Conclusie

Vergeleken met de afzinkfase (appendix K7) zijn de spanningen in de Flexbase vloer erg laag. In de twee vloeren is een maximale trekspanning te zien van 1,9N/mm^2. Dit is 475kN/m (1,9 * 250 * 1000). De vloer zal gewapend worden op 1.500kN/m zoals in appendix K7 is beschreven. Dus voor zowel de twee vloeren als het balkenrooster is de afzinkfase maatgevend en niet de bouwfase.

<u>Fase 1 (voor belastingen zie pagina 147)</u>

In deze fase worden de middelste 66 meter van de wanden in de lengte gestort. De bijbehorende spanningen zijn in de figuren op de volgende pagina's te zien en in tabel K4-1.

De figuren staan in de volgende volgorde:

i. Eerste (onderste) balklaag;
ii. Eerste (onderste) vloer;
iii. Tweede balklaag;
iv. Tweede vloer.

	x-trek [N/mm^2]	x-druk [N/mm^2]	y-trek [N/mm^2]	y-druk [N/mm^2]
Eerste balklaag	1,6	2,4	0,4	0,6
Eerste vloer	0,7	1,3	0,7	2,0
Tweede balklaag	0,8	0,8	-	-
Tweede vloer	1,2	1,0	1,5	1,0

Tabel K4-1, spanningen in beton, fase 1

Toelichting tabel

Eerste balklaag

De waardes laten zien wat er verwacht werd. De vloer gaat hol staan in de x-richting en bol in de y-richting. In de x-richting is dus trek te zien aan de onderkant van de balken en in de y-richting aan de bovenkant van de balken. Figuur laat ook de ondersteuning zien die is ingevoerd om onrealistische rotaties in SCIA te voorkomen. Hier treden grote druk- en trekspanningen op, deze worden buiten beschouwing gelaten, omdat deze in werkelijkheid niet zullen optreden.

Eerste vloer

In de onderste vloer gebeurd vrijwel hetzelfde. De trek en druk zijn het grootst in de x-richting in het midden van de vloer. In de y-richting zijn de trek- en drukspanningen vrijwel nihil.

Tweede balklaag

De hoge spanningen aan de zijkanten zijn hier het meeste belangrijk. In dit model staan de wanden nog aan de zijkant en zijn deze niet direct ondersteund door een wand in de Flexbase vloer. In appendix K1 is beschreven hoe dit is verholpen. Hiermee zijn deze piekspanningen aan de uiteinden van de balken verholpen.

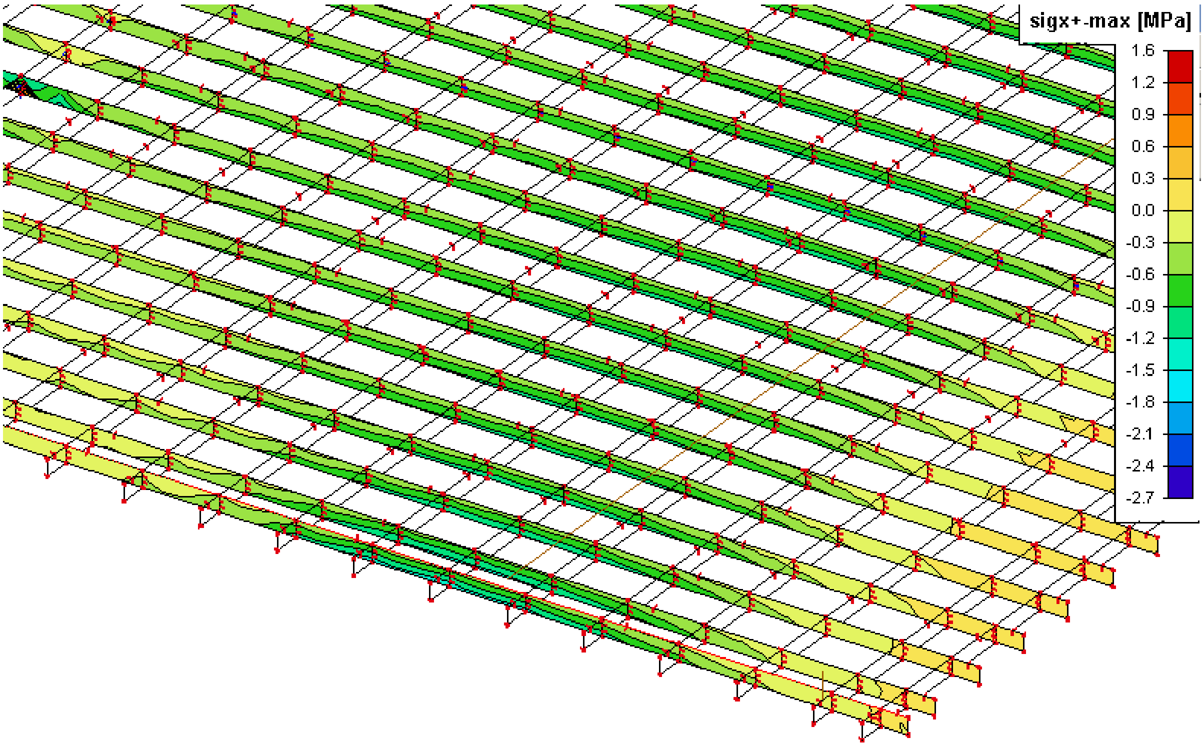
Tweede vloer

De trek- en drukspanningen zijn zowel in de x- als y-richting iets groter dan in de onderste vloer. Dit is te verklaren door het feit dat de belastingen van de wand op deze vloer werken.

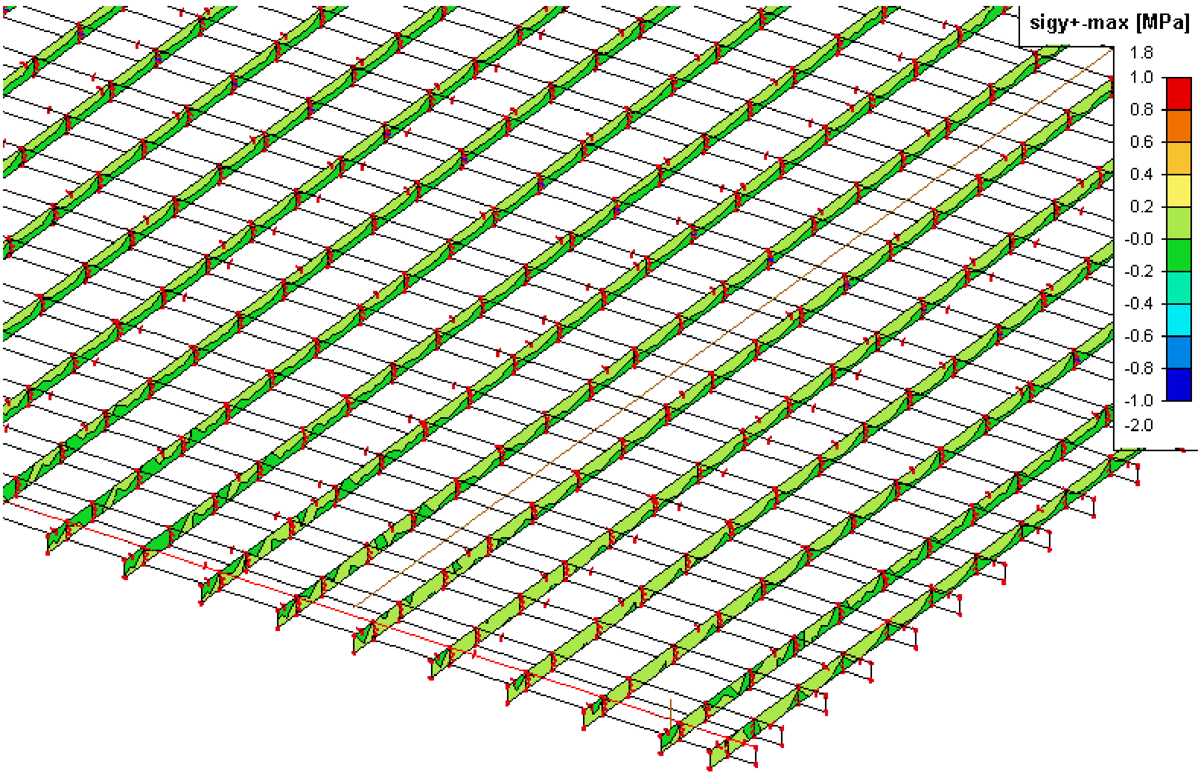
Wat vooral opvalt zijn de lokale trek- en drukspanningen aan de randen van het drijflichaam. De belasting van de vloer werken hier direct op de vloer die lokaal gezien kan worden als een vloer opgelegd op twee steunpunten. In appendix K1 is beschreven hoe dit is verholpen. Hiermee zijn deze piekspanningen aan de zijkanten van de vloer verholpen.

De volgende fases zullen niet meer toegelicht worden. Er zal enkel een conclusie gegeven worden aan het einde van deze appendix.

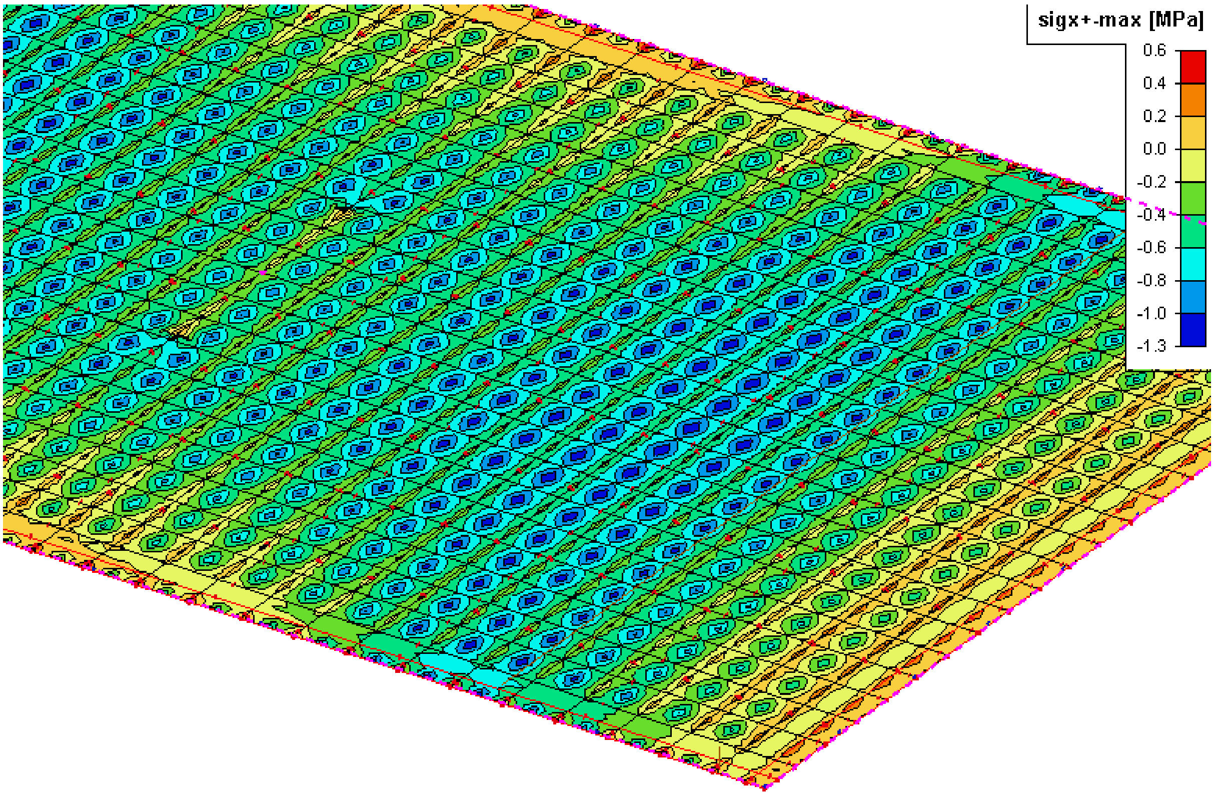
Fase 1, eerste balklaag, spanningen in x-richting



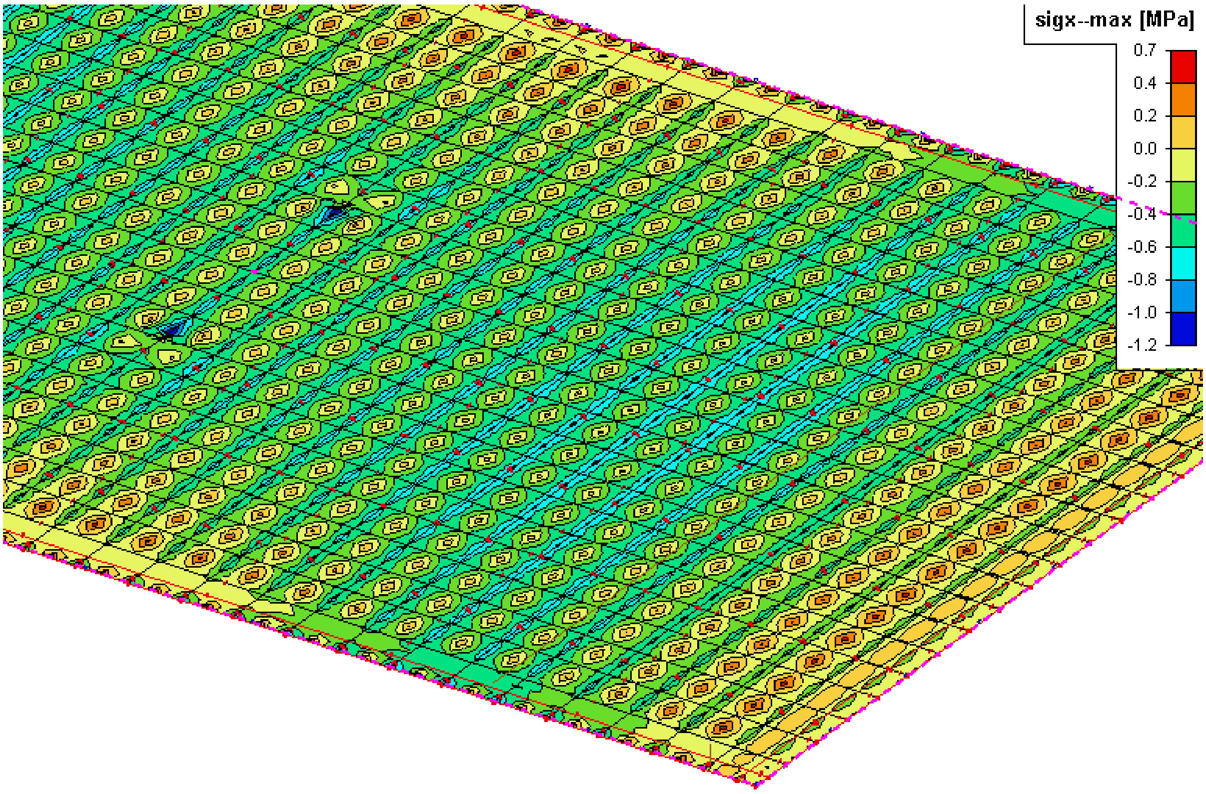
Fase 1, eerste balklaag, spanningen in y-richting



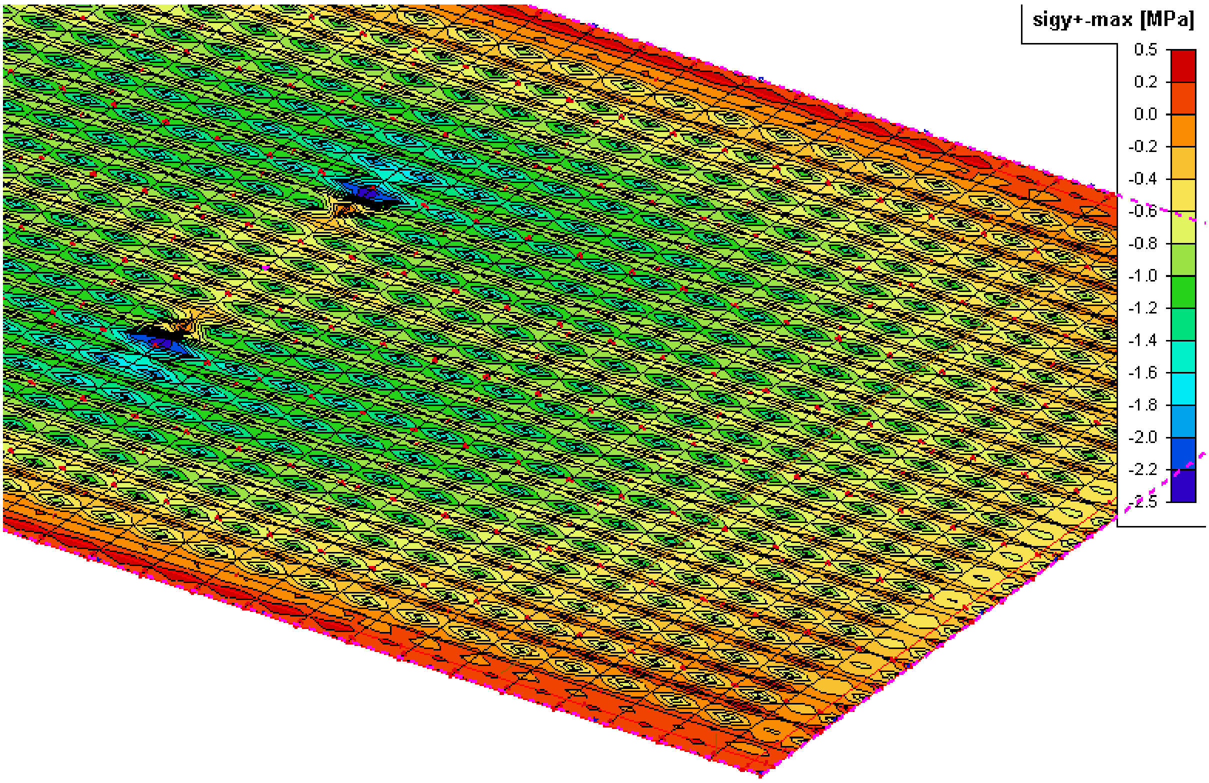
Fase 1, eerste vloer, spanningen in x-richting, onderkant vloer



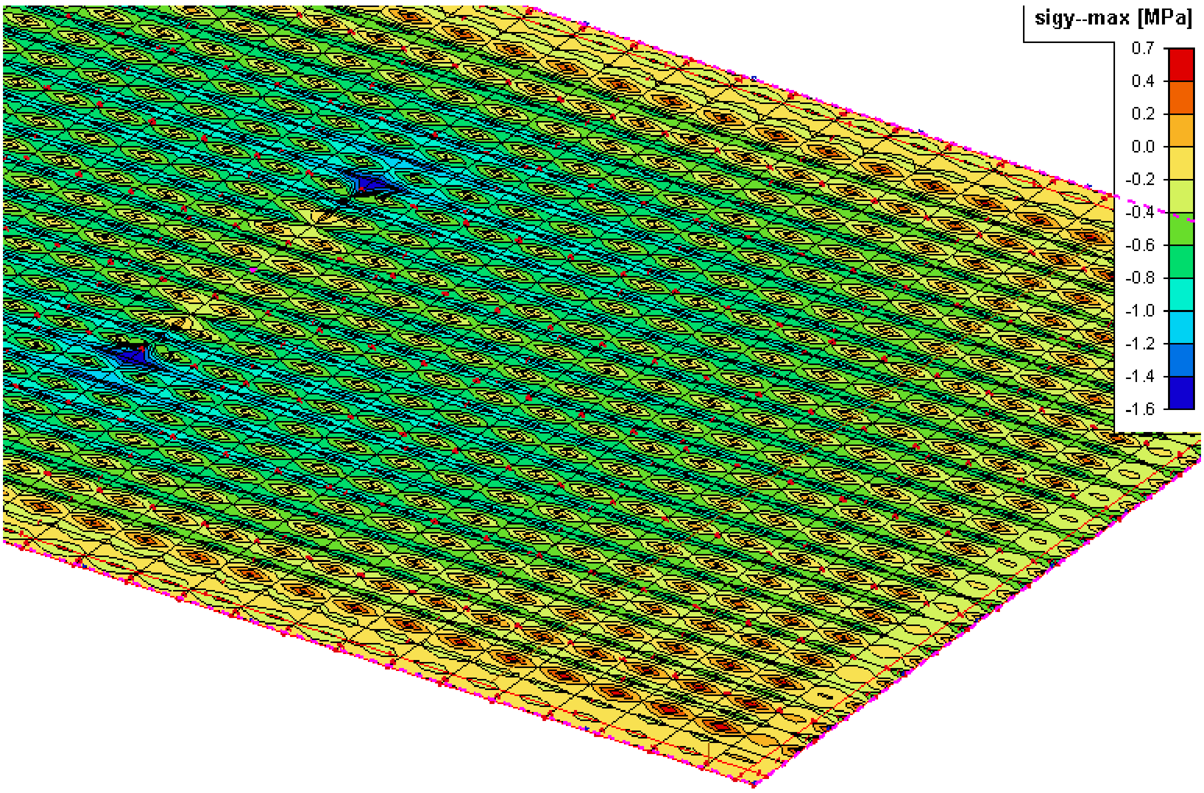
Fase 1, eerste vloer, spanningen in x-richting, bovenkant vloer



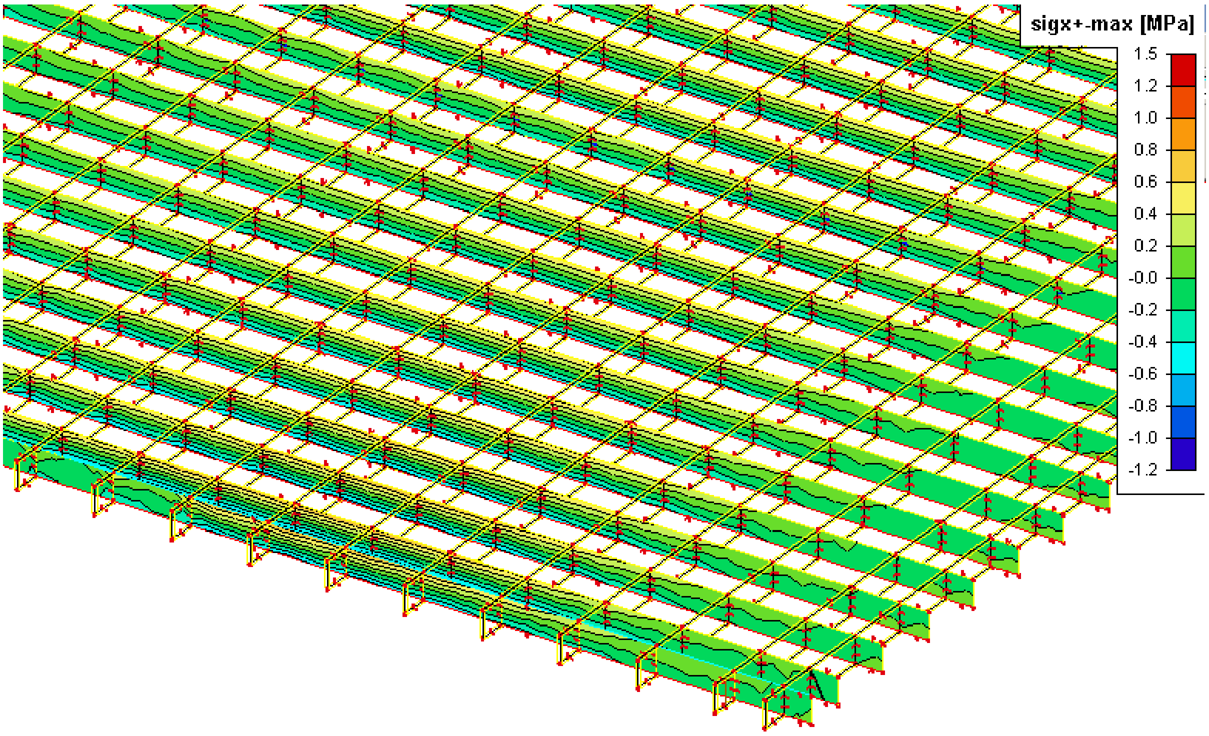
Fase 1, eerste vloer, spanningen in y-richting, onderkant vloer



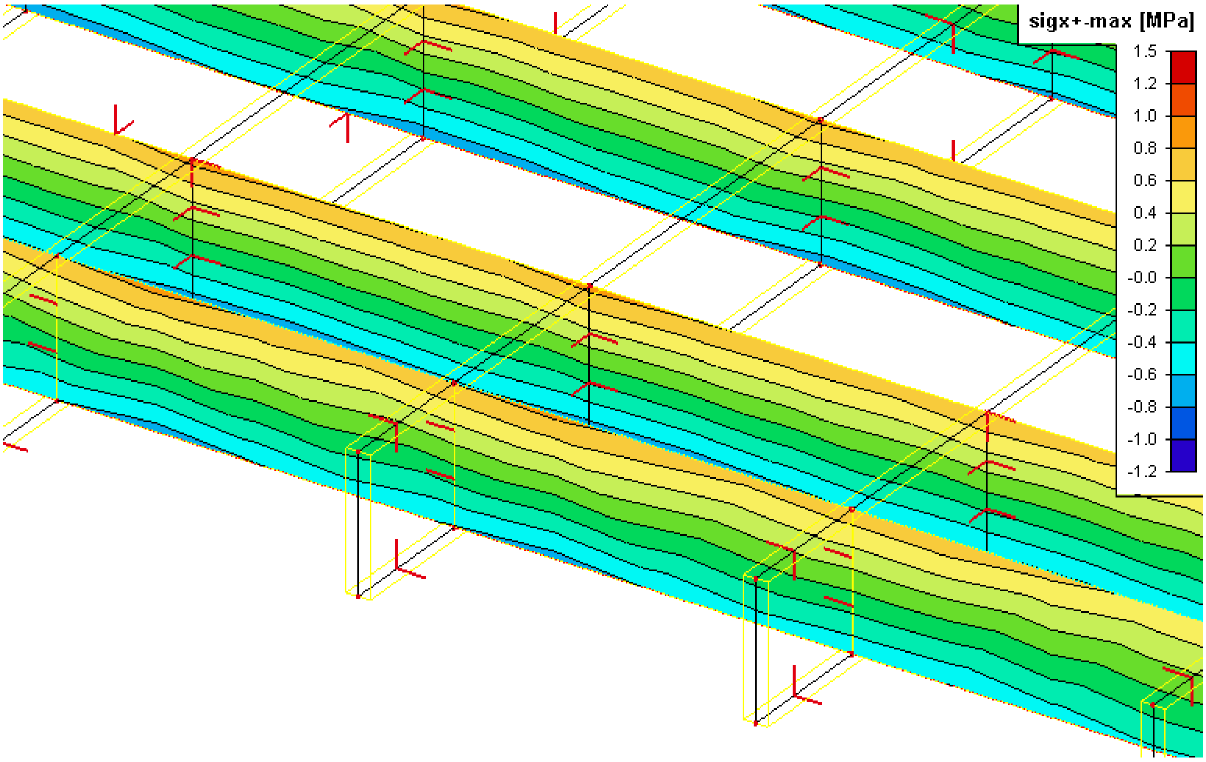
Fase 1, eerste vloer, spanningen in y-richting, bovenkant vloer



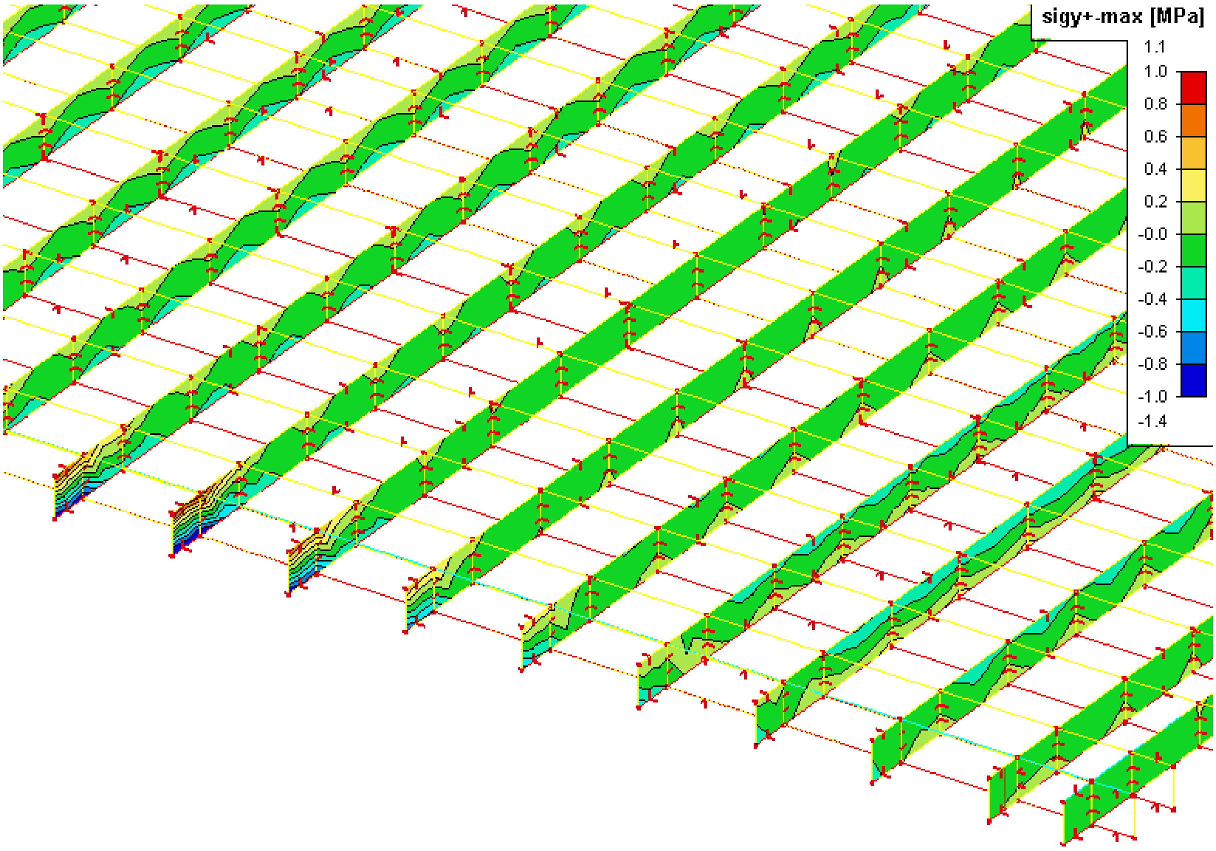
Fase 1, tweede balklaag, spanningen in x-richting



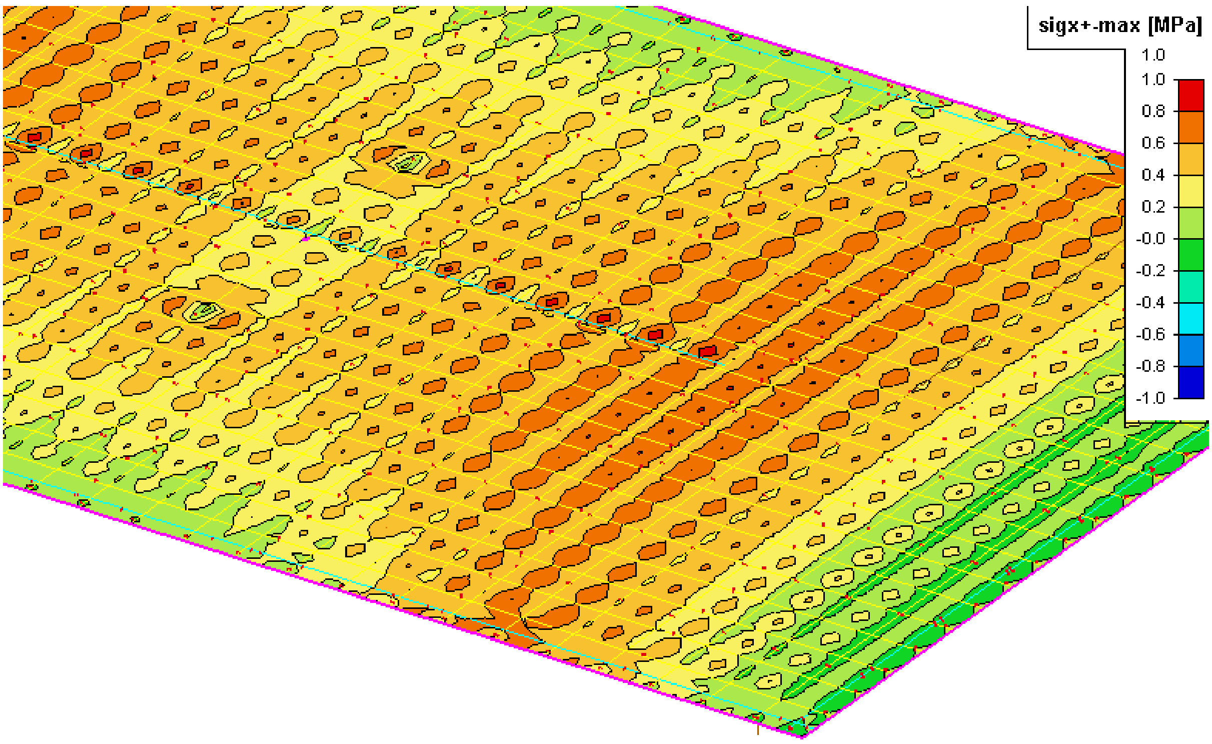
Fase 1, tweede balklaag, spanningen in x-richting, detail



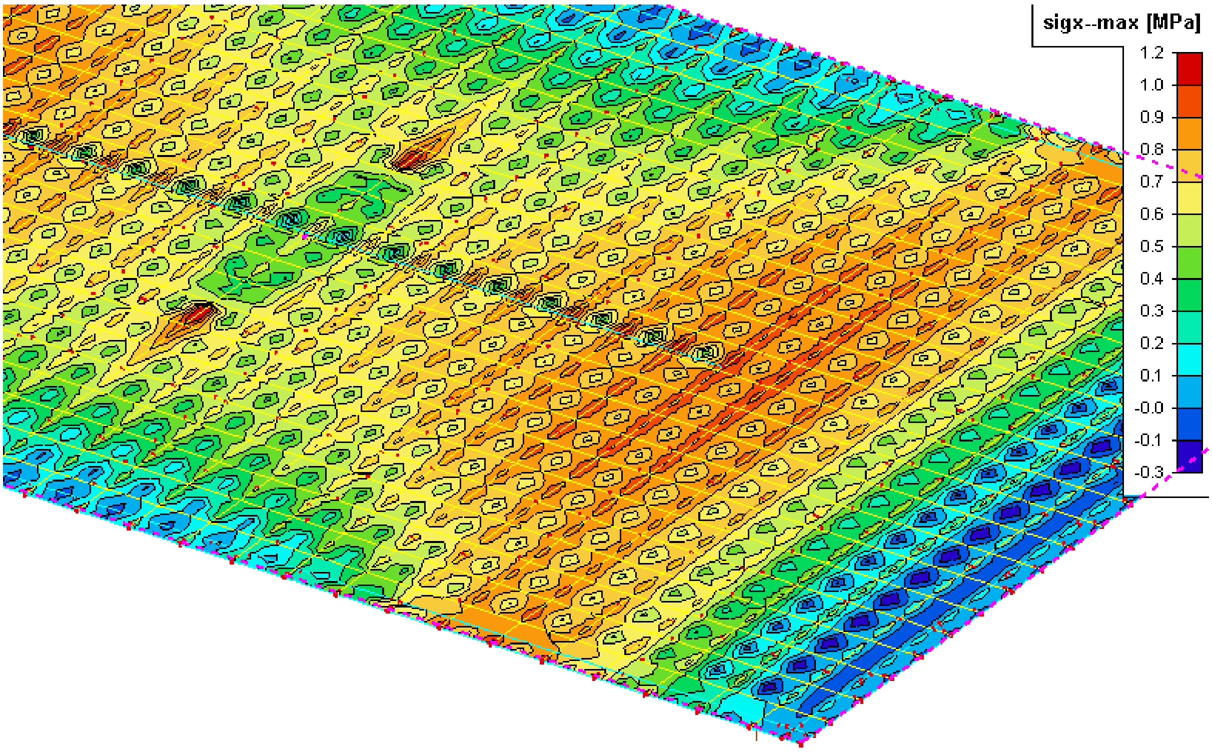
Fase 1, tweede balklaag, spanningenspanningen in y-richting



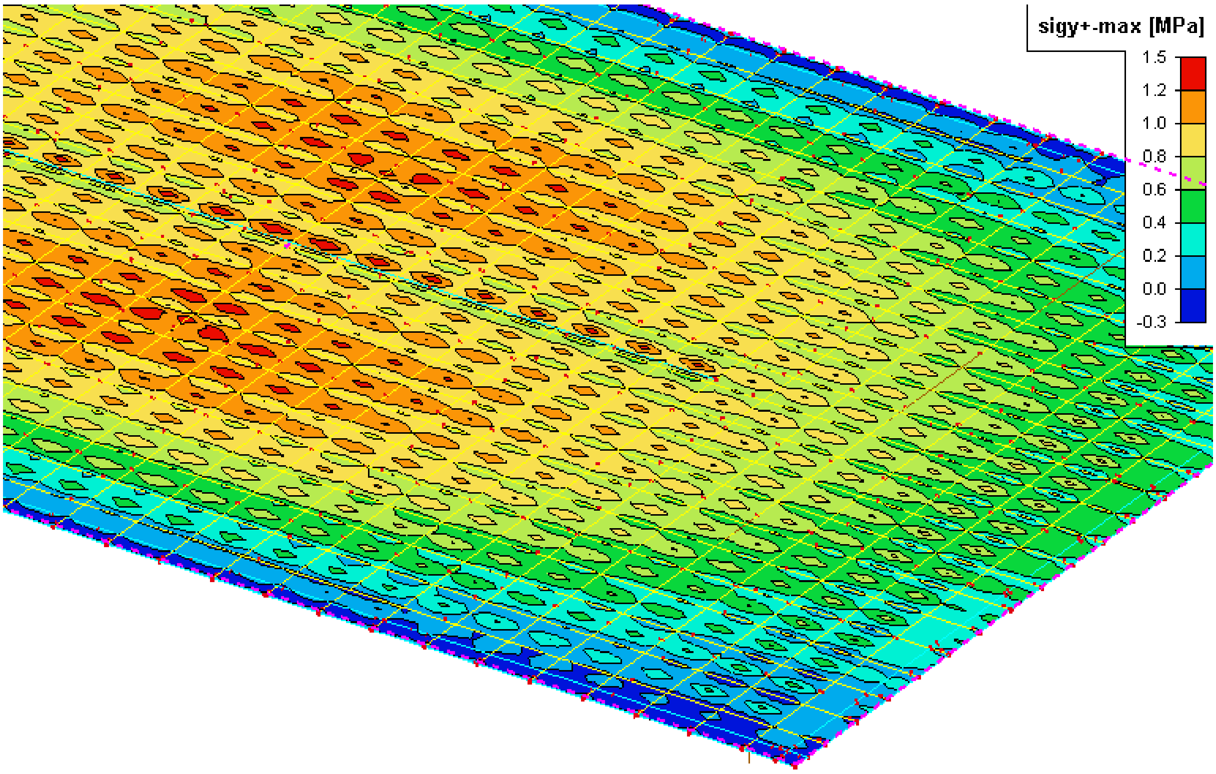
Fase 1, tweede vloer, spanningen in x-richting, onderkant vloer



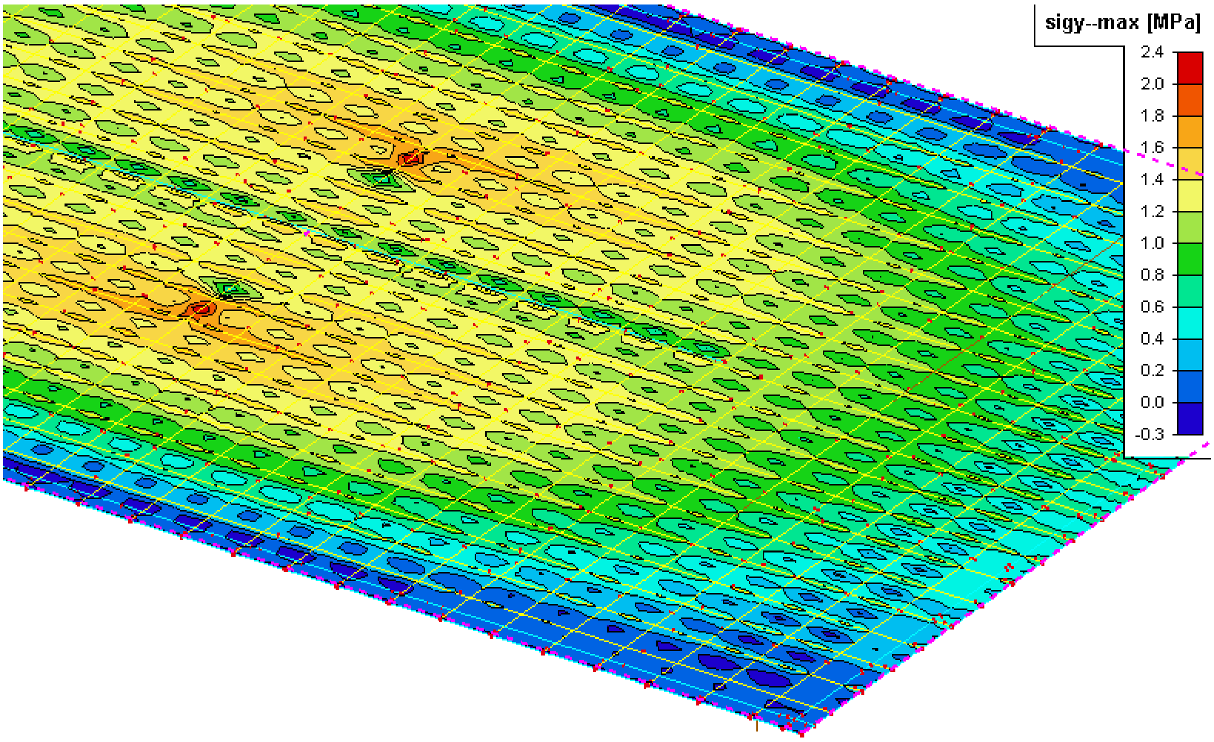
Fase 1, tweede vloer, spanningen in x-richting, bovenkant vloer



Fase 1, tweede vloer, spanningen in y-richting, onderkant vloer



Fase 1, tweede vloer, spanningen in y-richting, bovenkant vloer

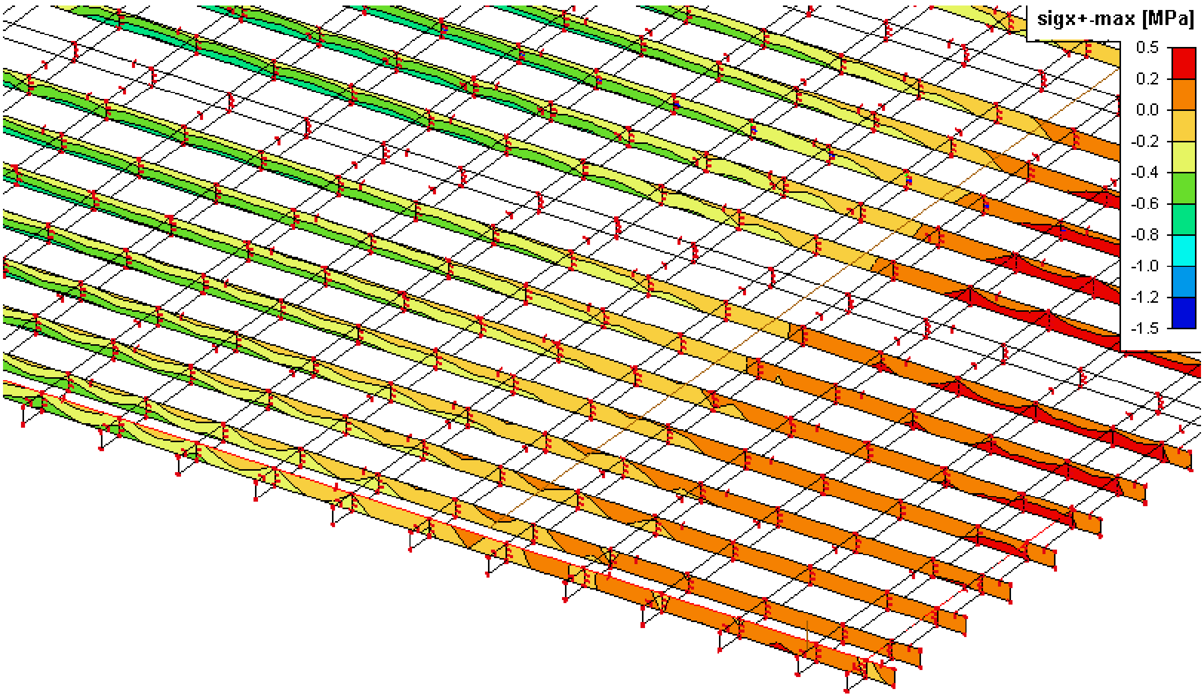


Fase 2 (voor belastingen zie pagina 149)
In deze fase wordt de rest van de wanden in de lengte gestort. De bijbehorende spanningen zijn in de figuren op de volgende pagina's te zien en in tabel K4-2.

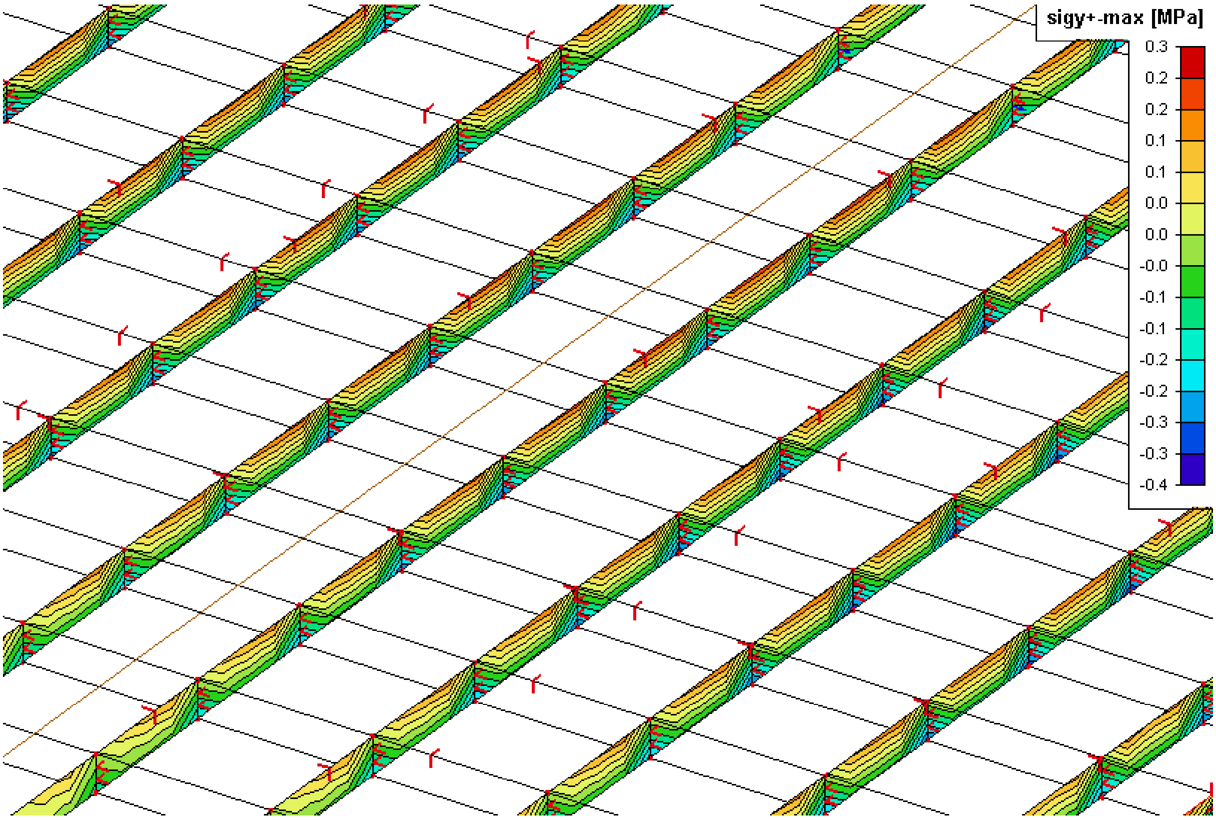
	x-trek [N/mm^2]	x-druk [N/mm^2]	y-trek [N/mm^2]	y-druk [N/mm^2]
Eerste balklaag	0,5	1,5	0,3	0,4
Eerste vloer	0,7	1,4	0,5	2,0
Tweede balklaag	1,0	1,0	0,7	0,6
Tweede vloer	1,2	0,5	1,6	0,6

Tabel K4-2, spanningen in beton, fase 2

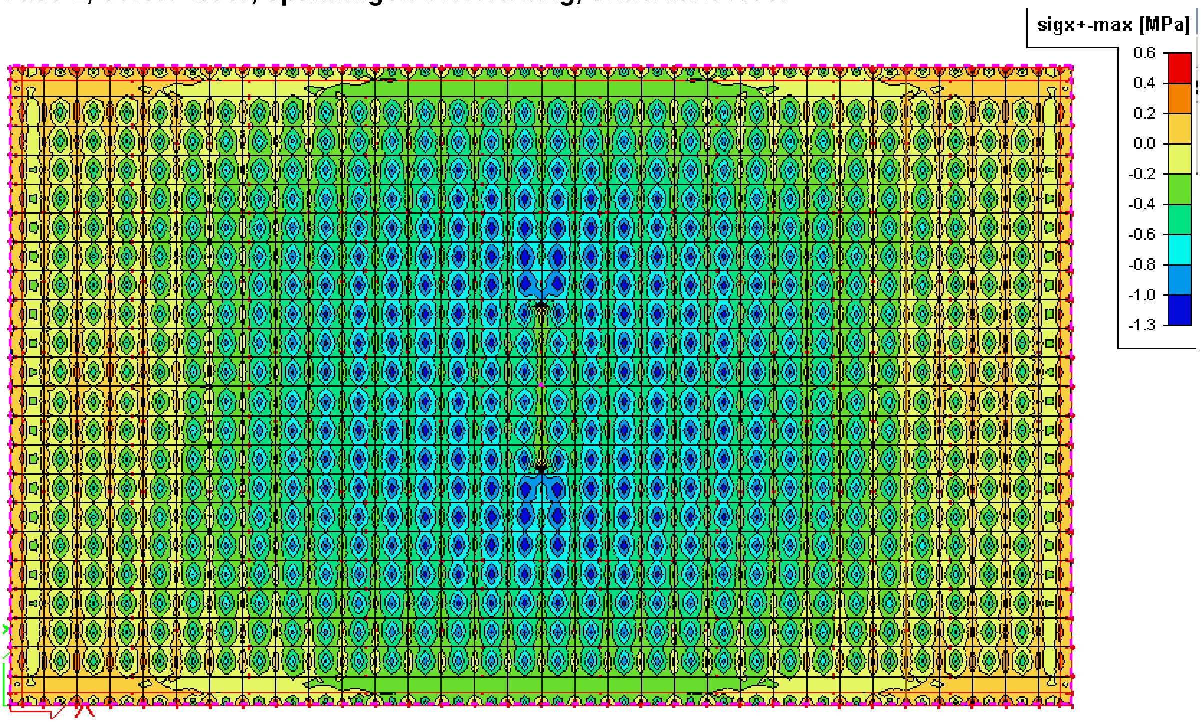
Fase 2, eerste balklaag, spanningen in x-richting



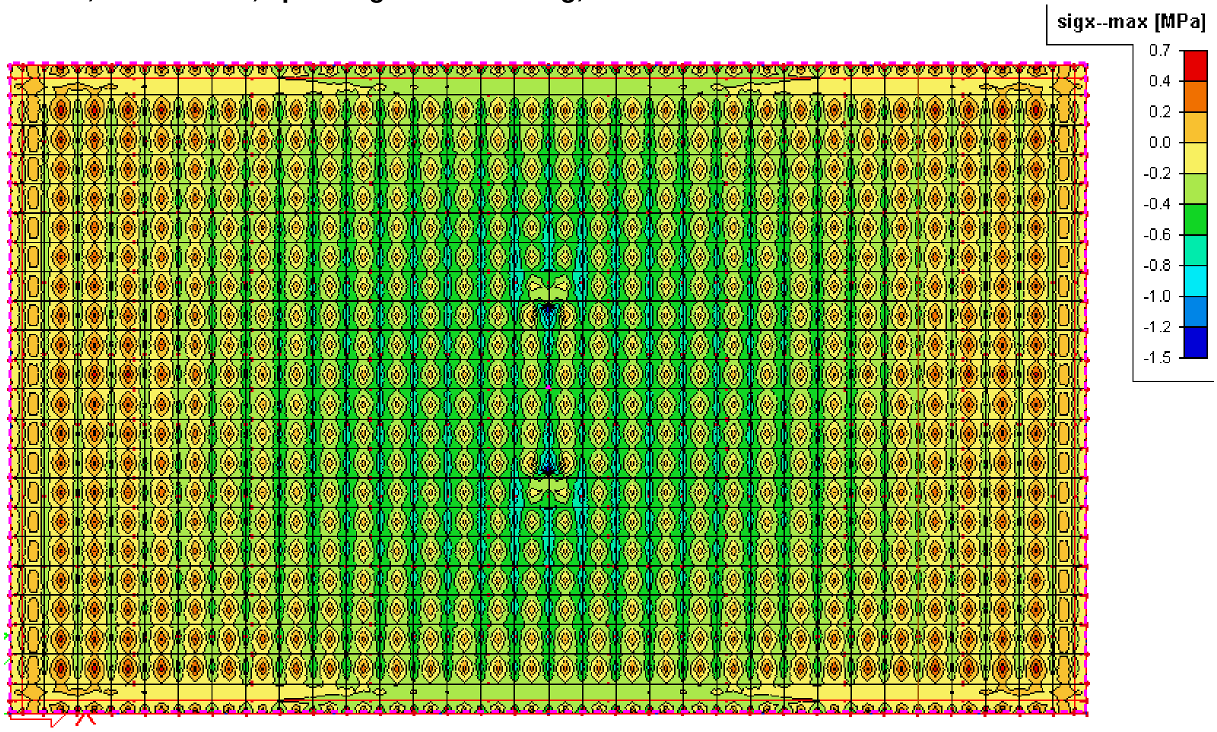
Fase 2, eerste balklaag, spanningen in y-richting



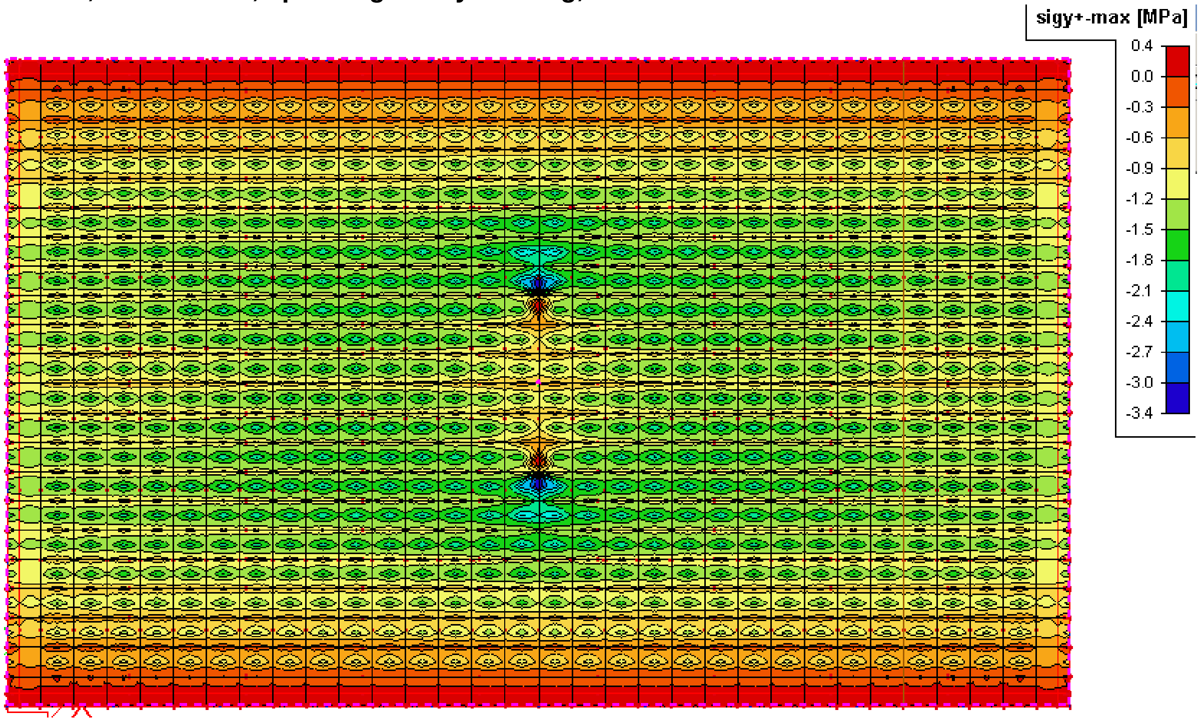
Fase 2, eerste vloer, spanningen in x-richting, onderkant vloer



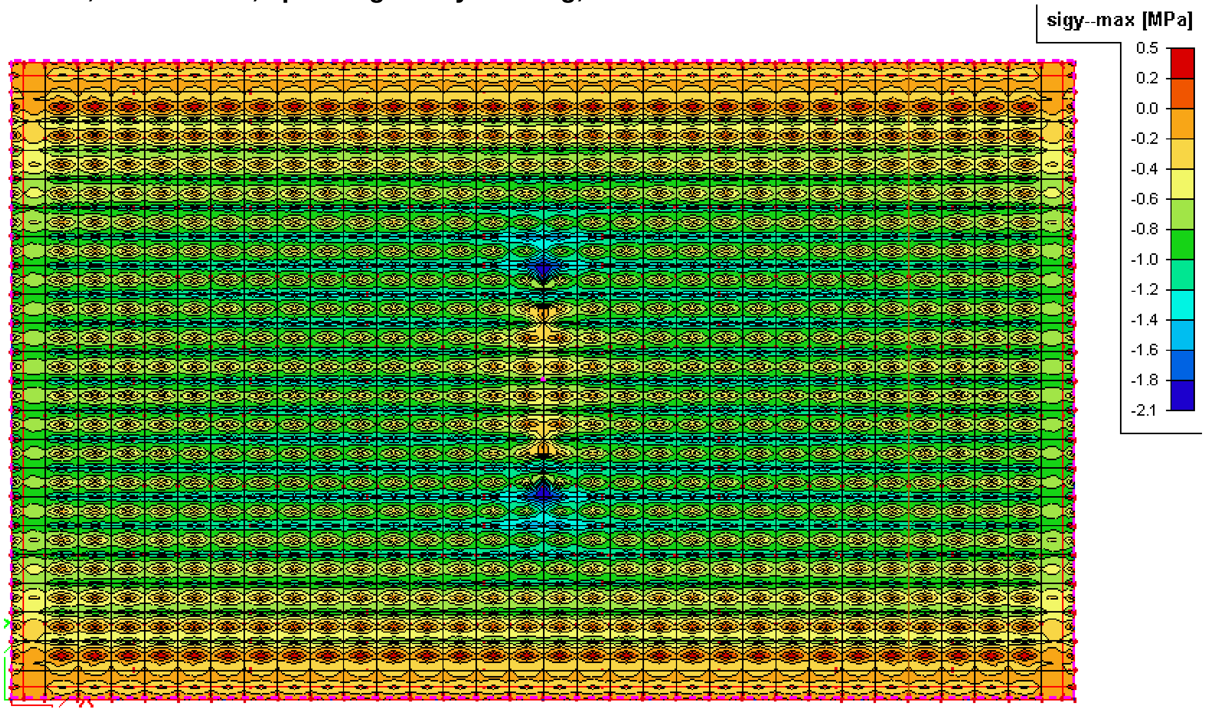
Fase 2, eerste vloer, spanningen in x-richting, bovenkant vloer



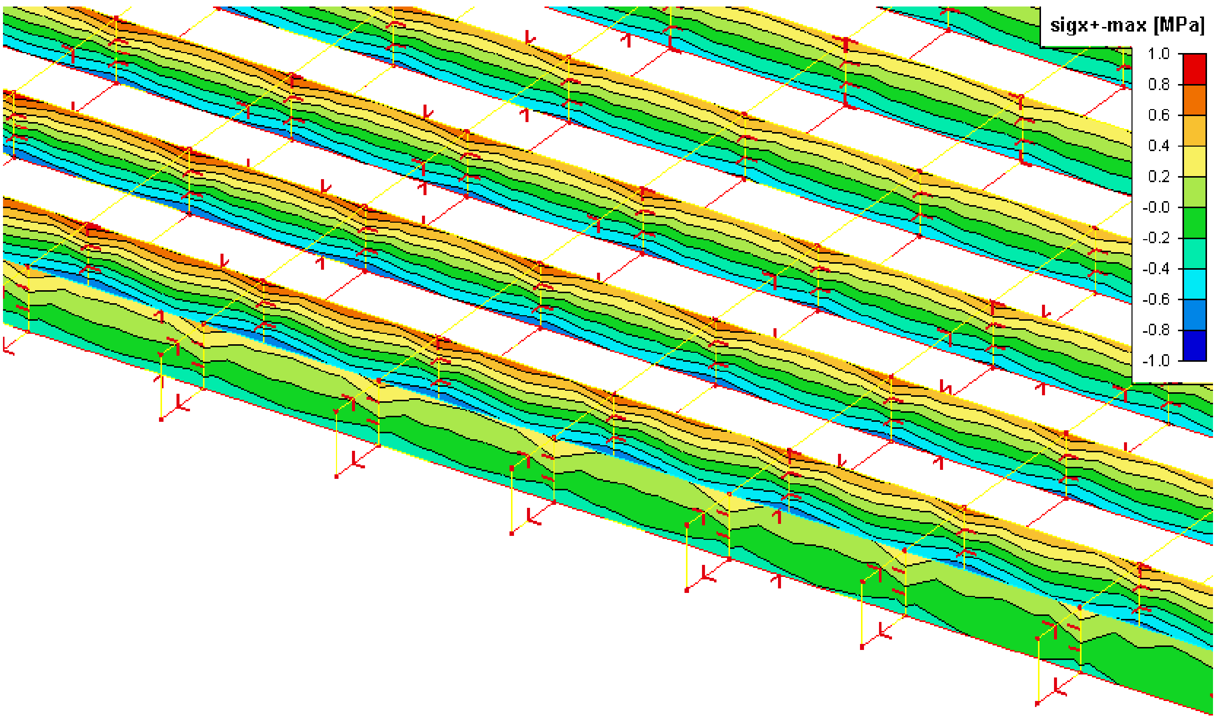
Fase 2, eerste vloer, spanningen in y-richting, onderkant vloer



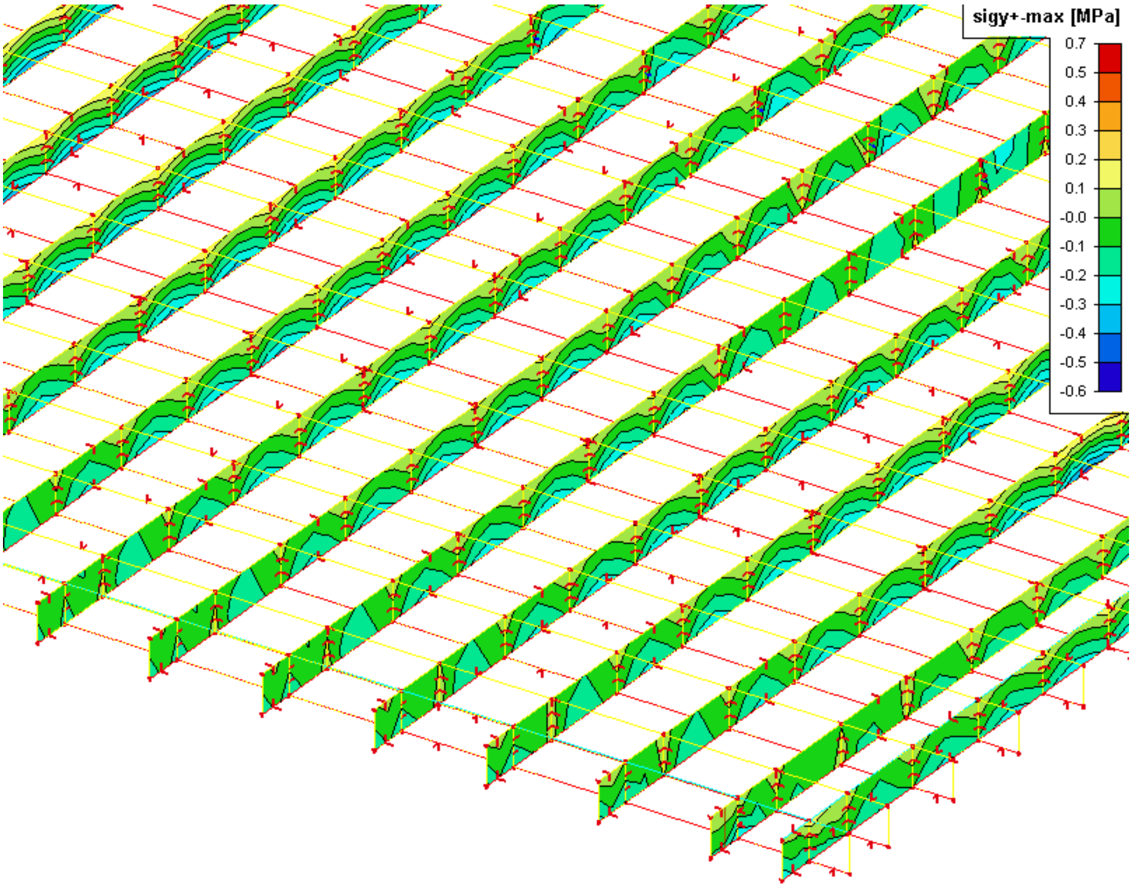
Fase 2, eerste vloer, spanningen in y-richting, bovenkant vloer



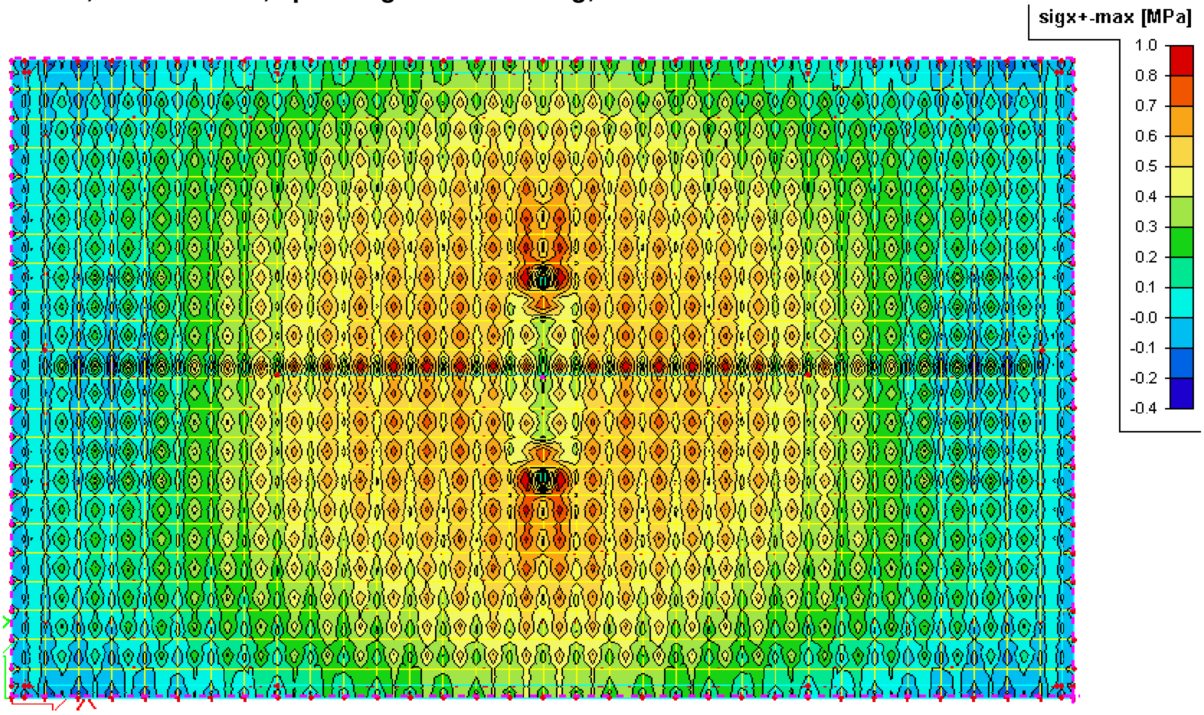
Fase 2, tweede balklaag, spanningen in x-richting



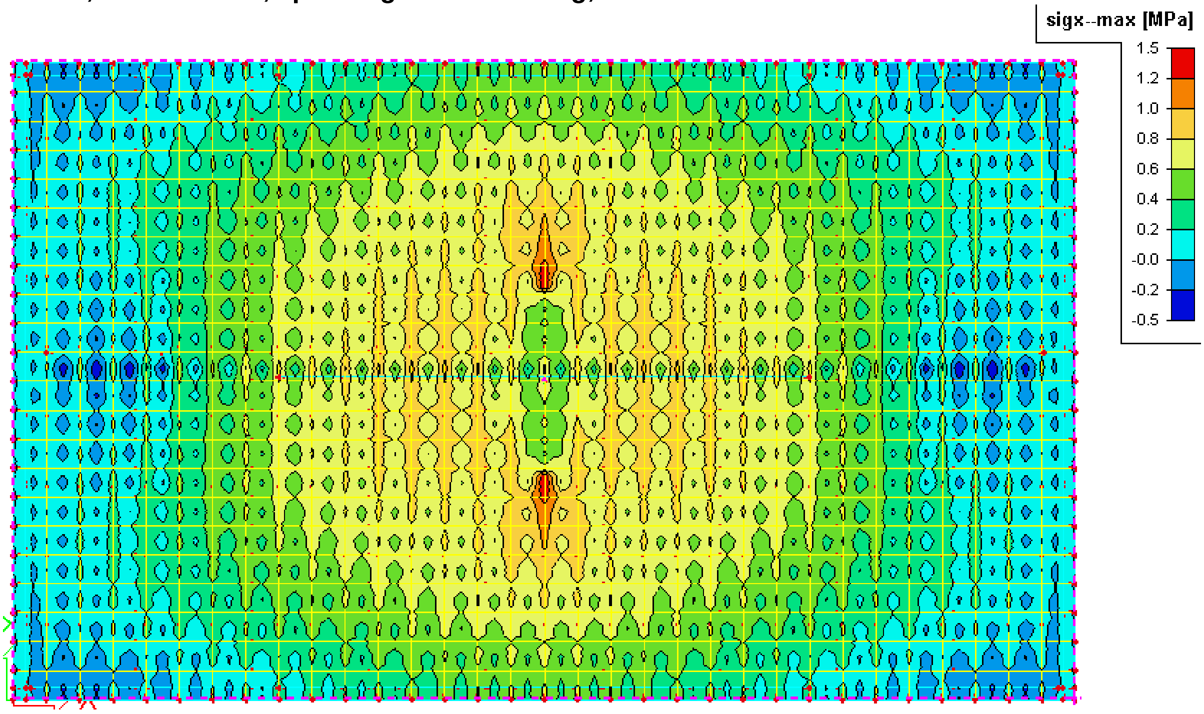
Fase 2, tweede balklaag, spanningen in y-richting



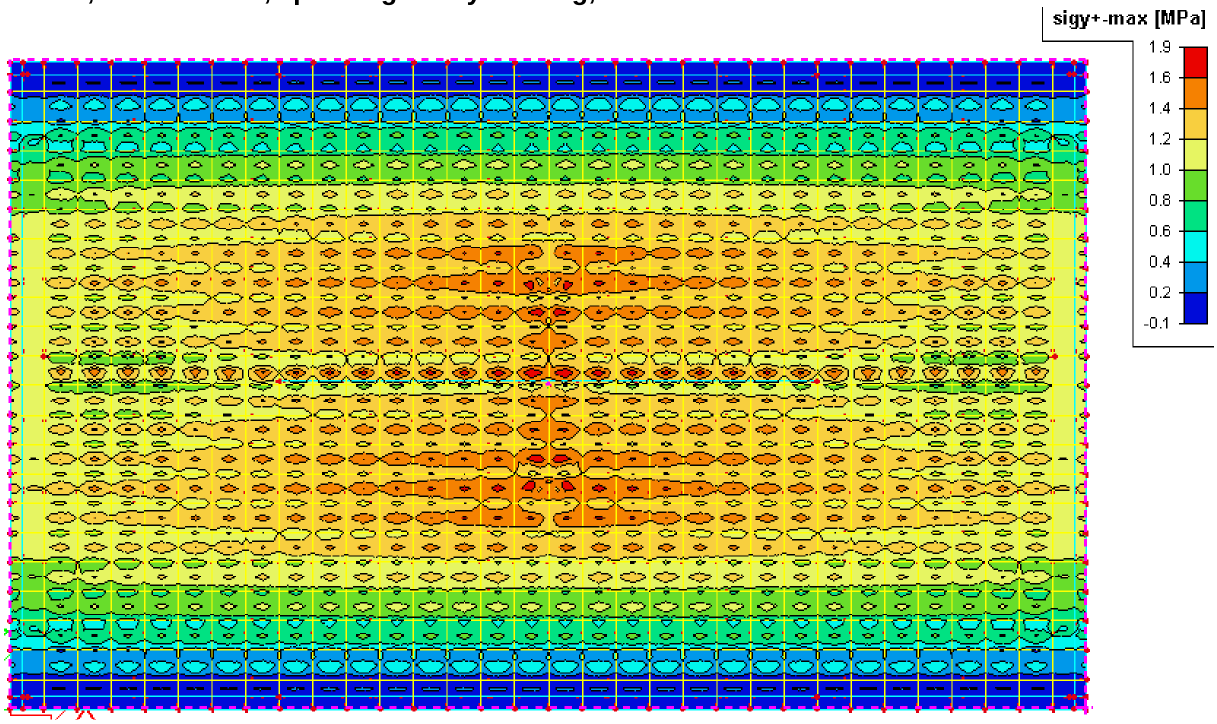
Fase 2, tweede vloer, spanningen in x-richting, onderkant vloer



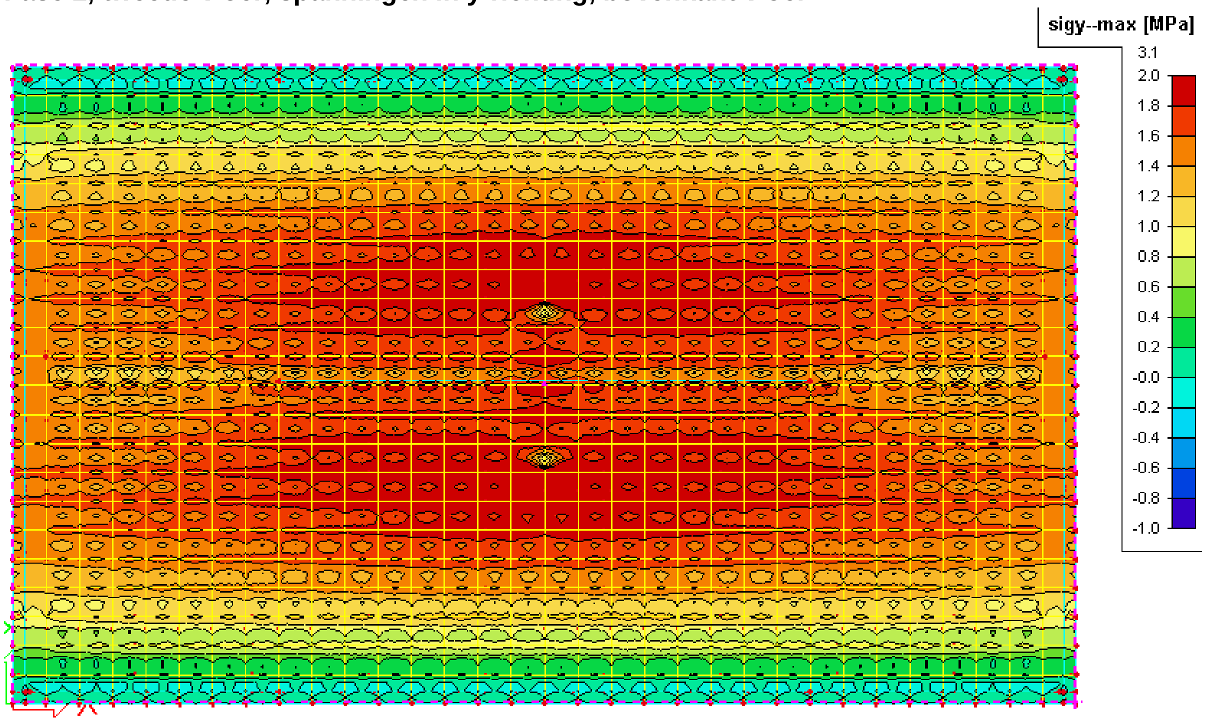
Fase 2, tweede vloer, spanningen in x-richting, bovenkant vloer



Fase 2, tweede vloer, spanningen in y-richting, onderkant vloer



Fase 2, tweede vloer, spanningen in y-richting, bovenkant vloer



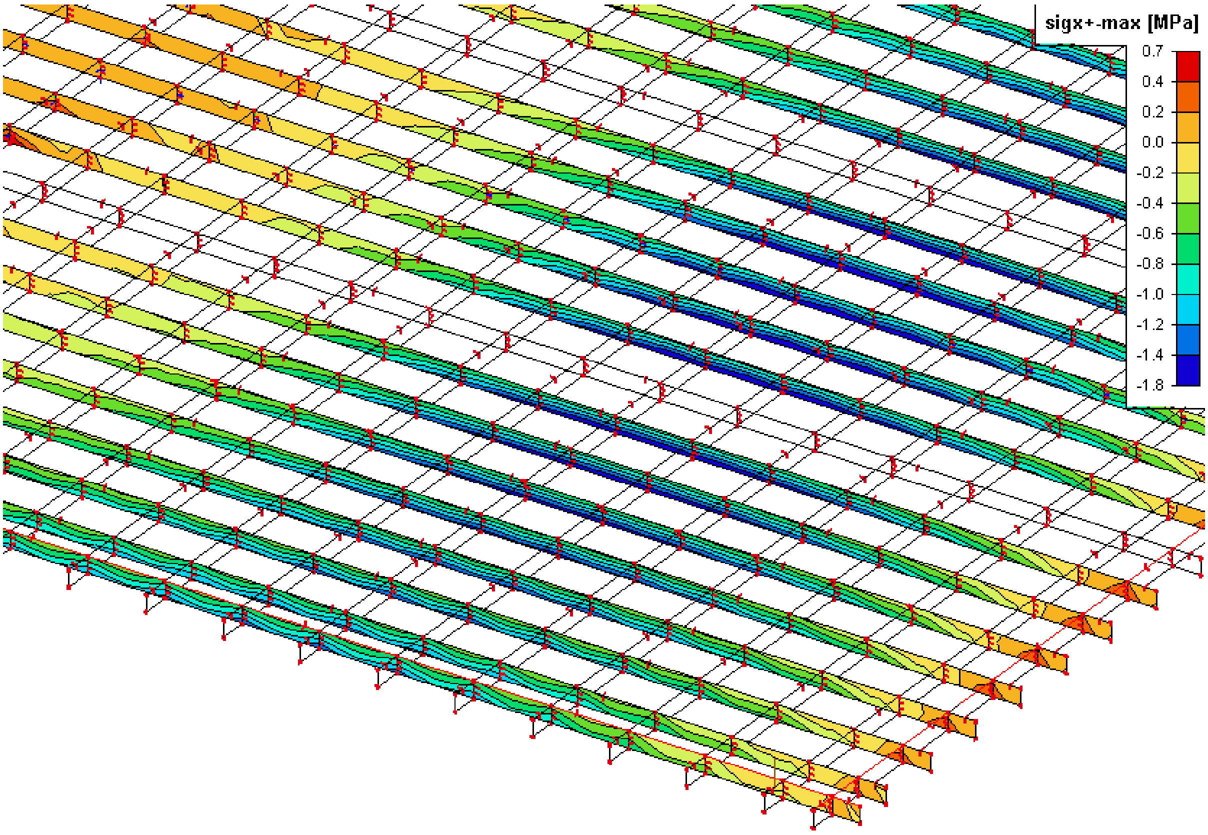
Fase 3.1 (voor belastingen zie pagina 150)

In deze fase worden de buitenwanden over de breedterichting gestort. De bijbehorende spanningen zijn in de figuren op de volgende pagina's te zien en in tabel K4-3. In deze fase is het beton nog niet uitgehard en zijn de wanden ingevoerd als belastingen.

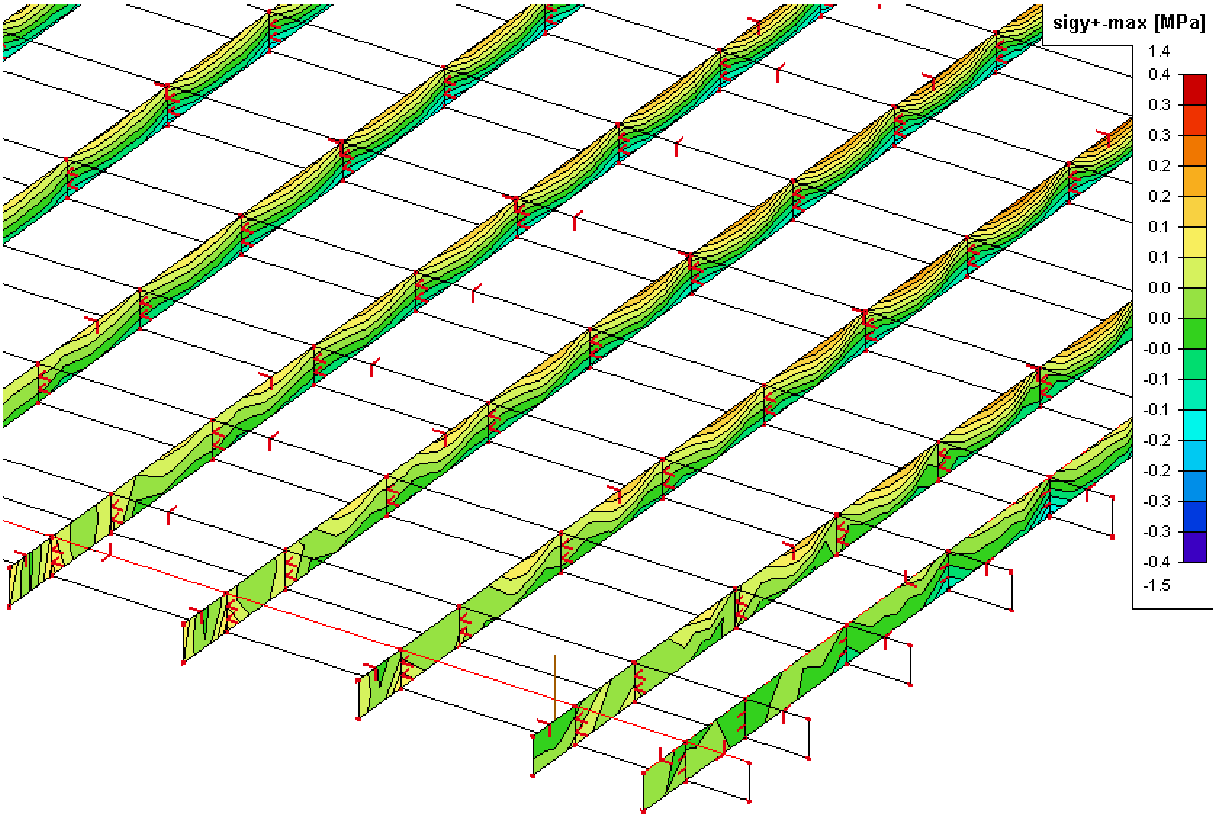
	x-trek [N/mm^2]	**x-druk [N/mm^2]**	**y-trek [N/mm^2]**	**y-druk [N/mm^2]**
Eerste balklaag	0,7	1,8	0,4	0,4
Eerste vloer	-	-	-	-
Tweede balklaag	1,1	1,2	0,4	0,8
Garage wanden in x-richting	**3,6**	0,8	-	-

Tabel K4-3, spanningen in beton, fase 2

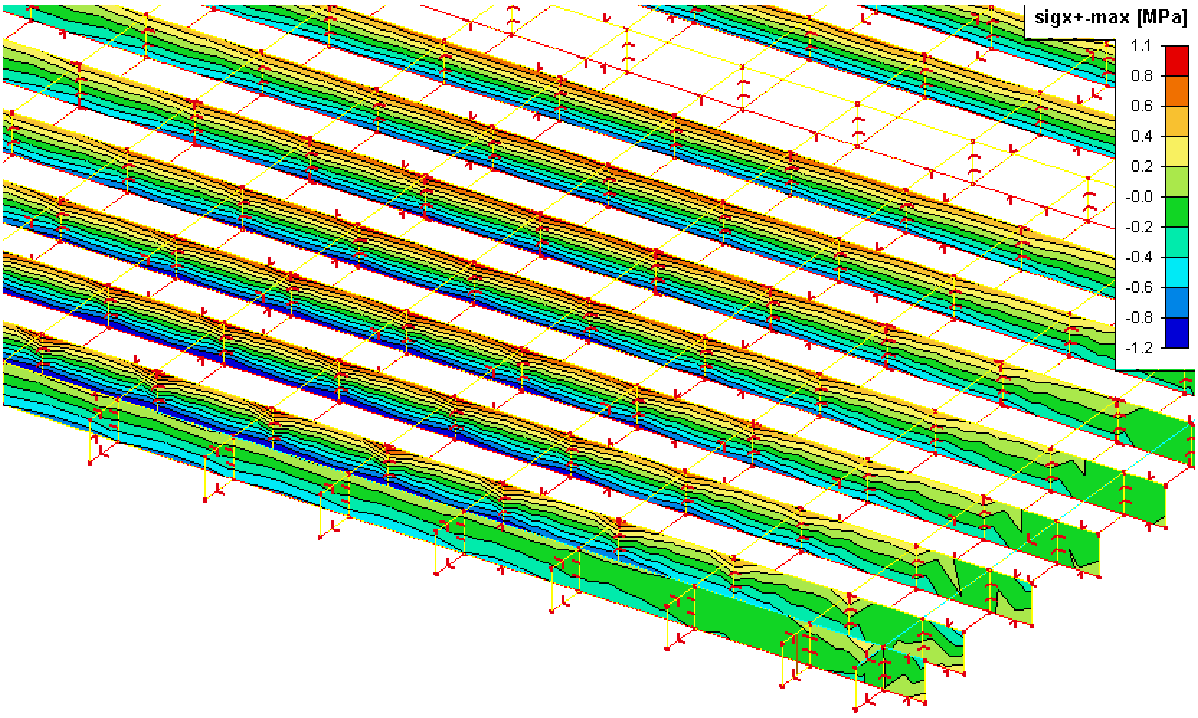
Fase 3.1, eerste balklaag, spanningen in x-richting



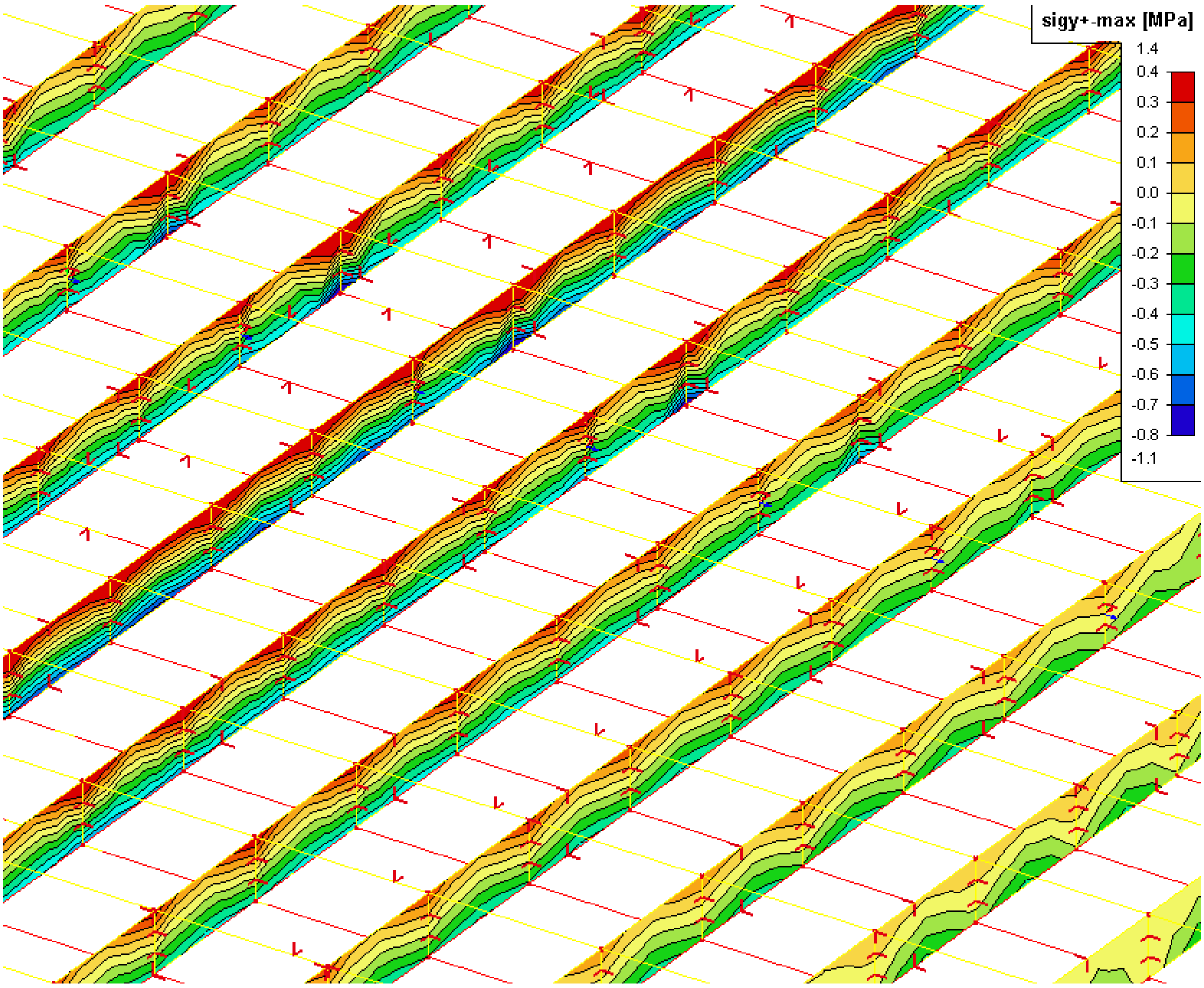
Fase 3.1, eerste balklaag, spanningen in y-richting



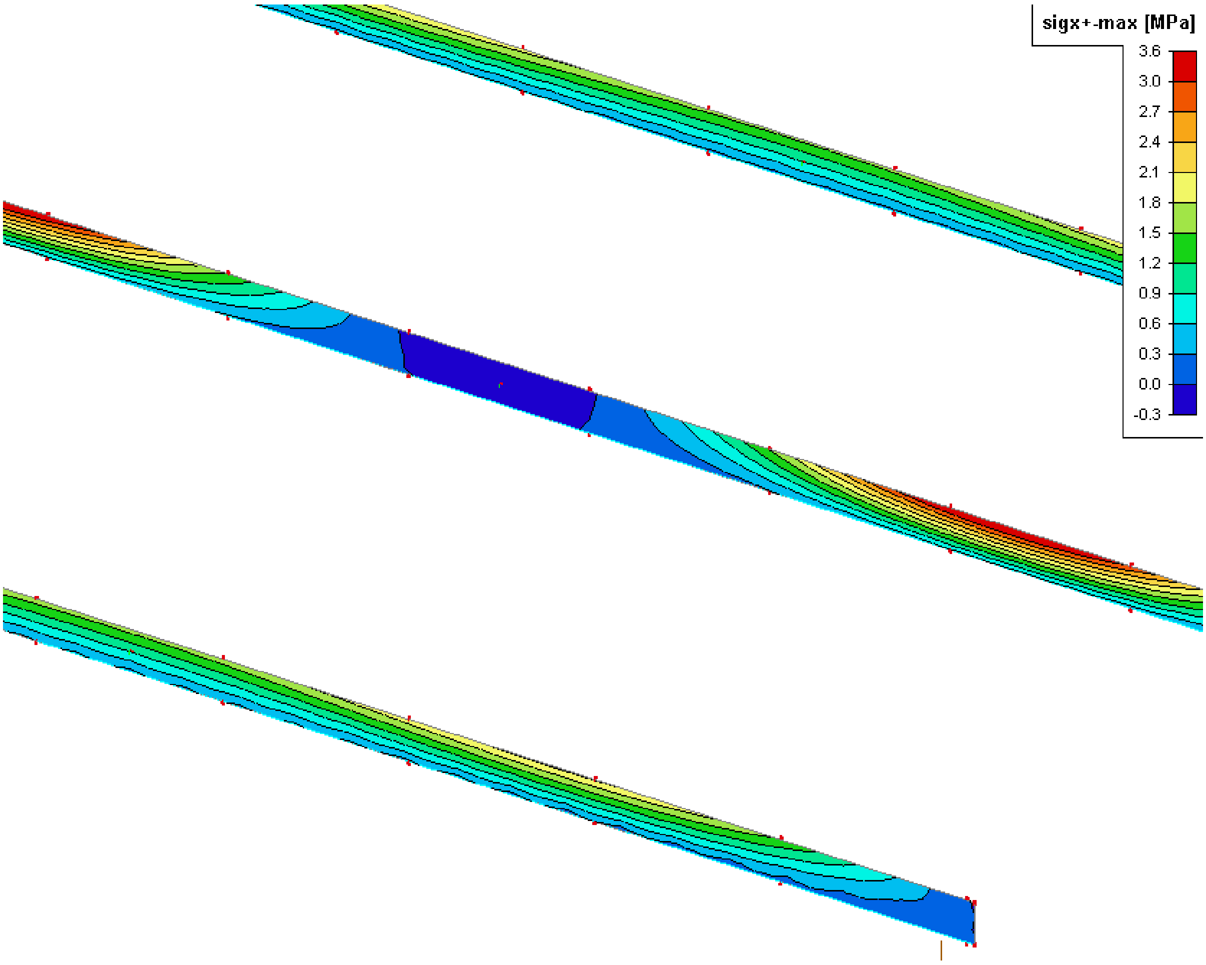
Fase 3.1, tweede balklaag, spanningen in x-richting



Fase 3.1, tweede balklaag, spanningen in y-richting



Fase 3.1, wanden eerste verdieping, spanningen in x-richting



Fase 3.2

In deze fase zijn de wanden uitgehard. Er wordt gekeken naar de maximale spanningen in de buitenwanden en de tussenwand.

	x-trek [N/mm^2]	**x-druk [N/mm^2]**	**y-trek [N/mm^2]**	**y-druk [N/mm^2]**
Wanden x-richting	4,1	0,4	-	-
Wanden y-richting	-	-	3,0	0,1

Tabel K4-4, spanningen in beton, fase 3.2

Toelichting tabel

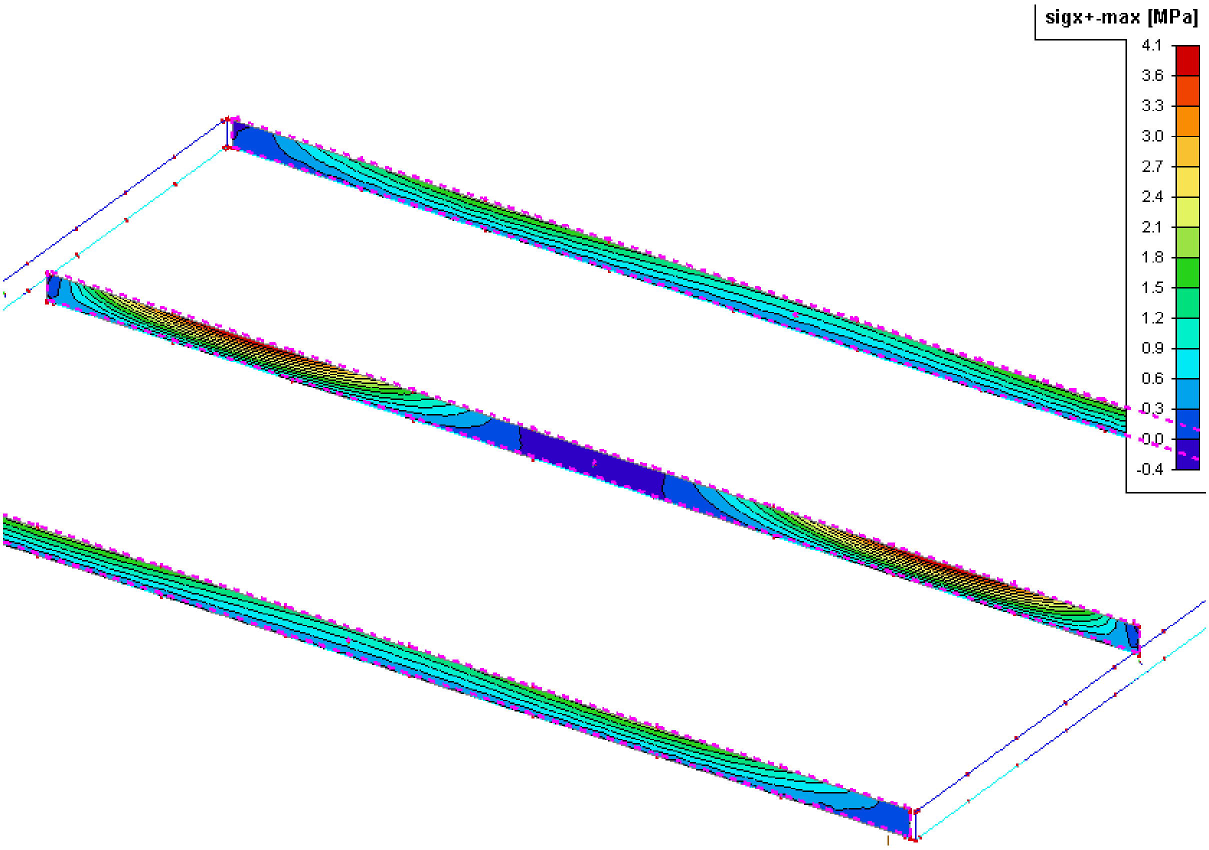
Wanden x-richting

Door het plaatsen van de wanden op de korte kant van vloer komt er een trekspanning in de wanden over de lengte. Deze trekspanning zit boven in de wanden tussen de wanden op de korte zijde en de countergewichten. Deze trekspanningen zijn tijdelijk, maar hier zal wel op gewapend moeten worden. Zodra de vloer van de tweede verdieping wordt gestort zullen deze trekspanningen sterk verminderen en zullen de drukspanningen toenemen.

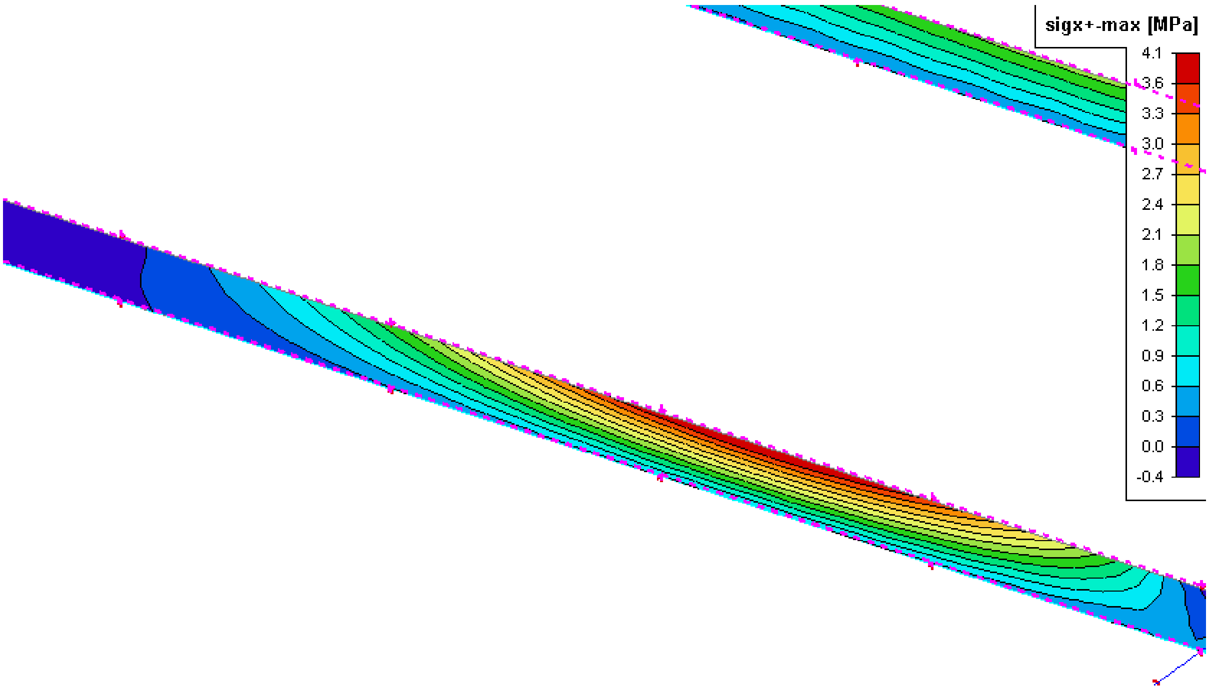
Wanden y-richting

Door de vervormingen in de vloer zullen er ook trek- en drukspanningen in de wanden op de korte zijde komen. Deze waardes zijn echter niet reëel en zullen in realiteit een stuk lager zijn. Wanneer de wanden worden gestort is het beton namelijk nog vloeibaar en de wanden zullen in de vervormde stand uitharden. De trek- en drukspanningen zullen dus nihil zijn wanneer de wanden zijn uitgehard. Pas wanneer de rest van de constructie wordt gestort zullen de wanden vervormen en ‘recht’ worden. De trek- en drukspanningen zullen dan dus juist de omgekeerd worden en er zal een drukspanningen optreden aan de bovenkant van de wanden.

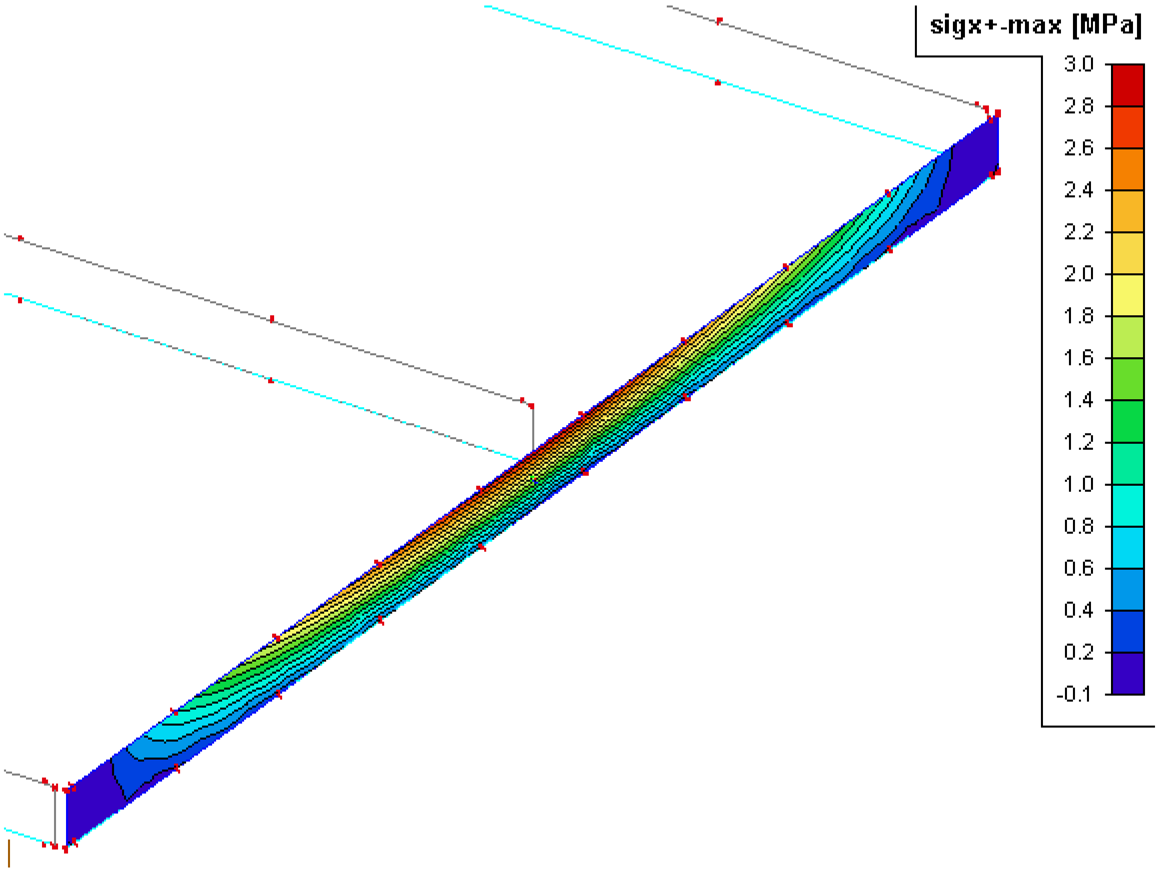
Fase 3.2, wanden eerste verdieping, spanningen in x-richting



Fase 3.2, wanden eerste verdieping, spanningen in x-richting, detail



Fase 3.2, wanden eerste verdieping, spanningen in y-richting



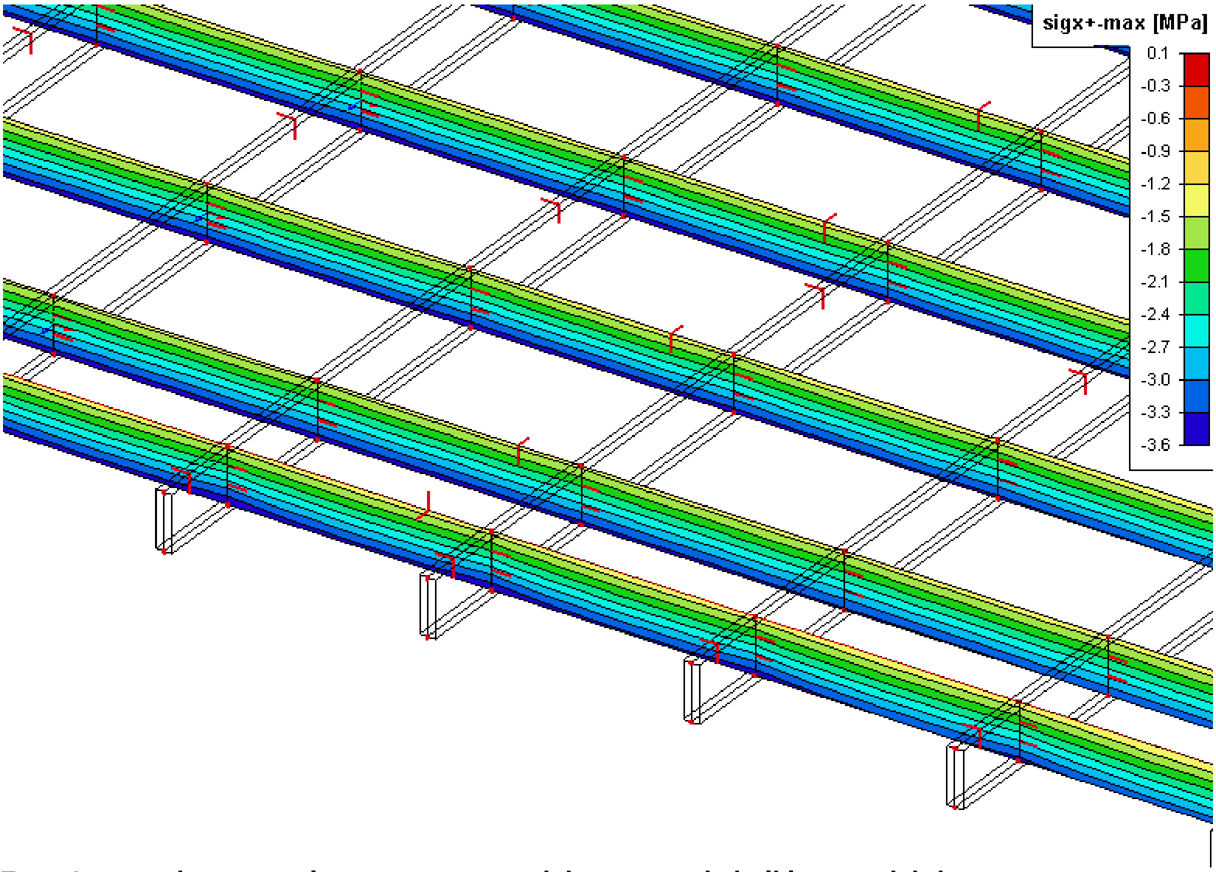
Counterweight
Dit gedeelte bekijkt de maximale spanningen in het beton die optreden door het plaatsen van de contragewichten. De maximale spanningen worden gegeven voor de fases 1, 2 en 3.

	x-trek [N/mm^2]	**x-druk [N/mm^2]**	**y-trek [N/mm^2]**	**y-druk [N/mm^2]**
Eerste balklaag	1,8	3,6	-	-
Eerste vloer	-	-	-	-
Tweede balklaag	2,1	1,6	1,2	1,4
Tweede vloer	1,9	0,1	-	-
Wanden x-richting	0,1	8,4	-	-

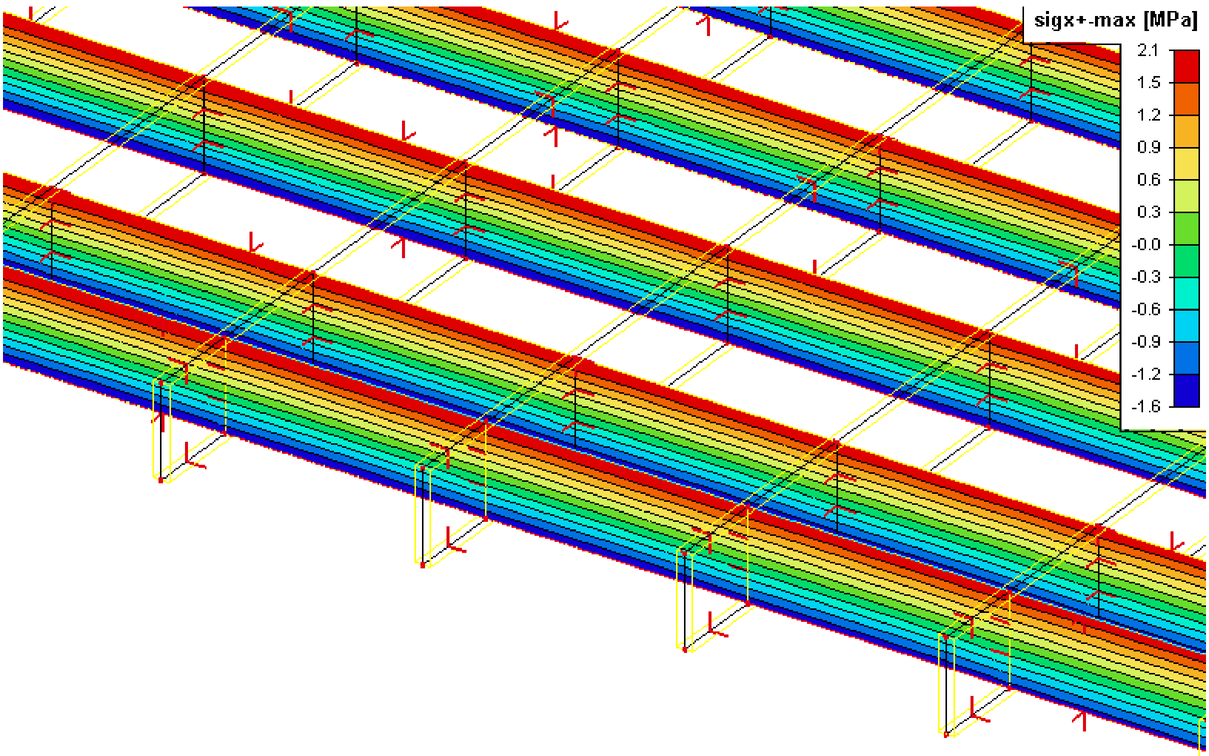
Tabel K4-5, spanningen in beton, contragewichten

Toelichting tabel
Als al verwacht, laten de waardes in elke fase het omgekeerde zien van de waardes na het storten. Dit is logisch, aangezien het countergewichten zijn. Hierdoor worden de spanningen gemiddeld. Er zijn geen extreme waardes te zien.

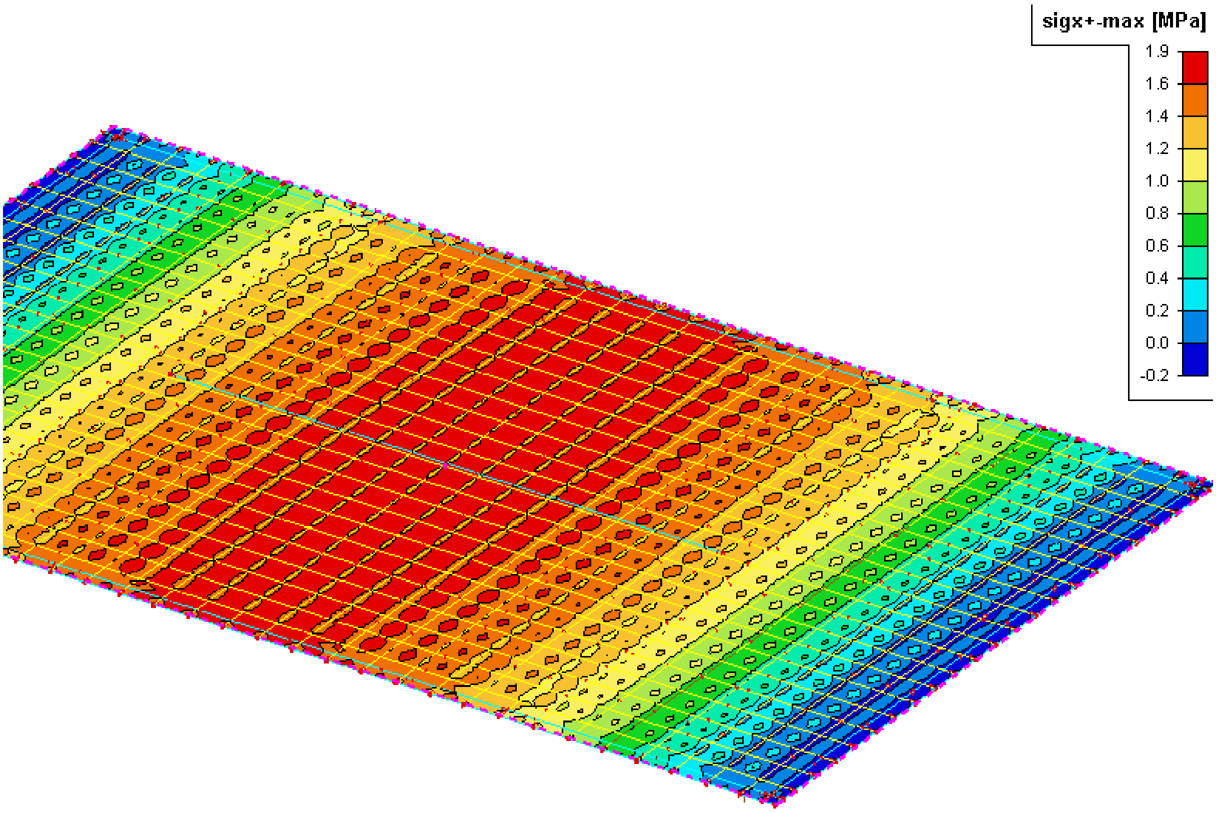
Fase 1, spanningen na plaatsen contragewichten, eerste balklaag, x-richting



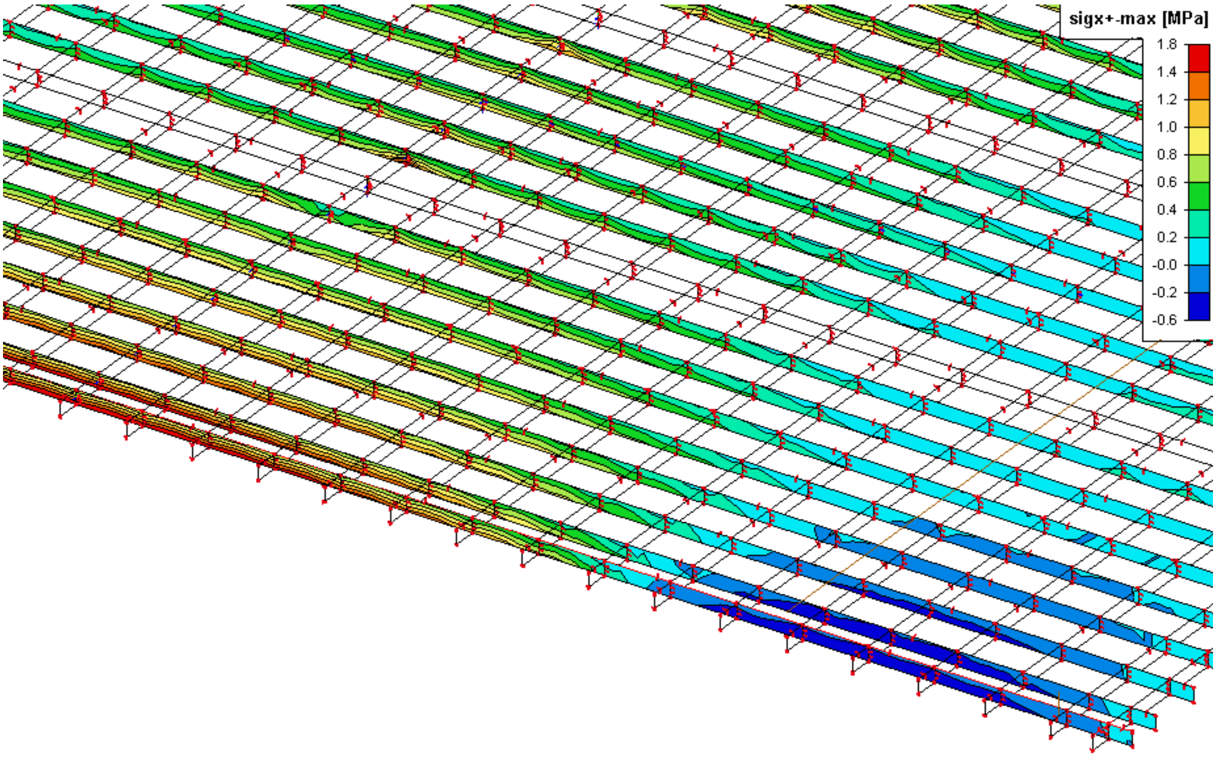
Fase 1, spanningen na plaatsen contragewichten, tweede balklaag, x-richting



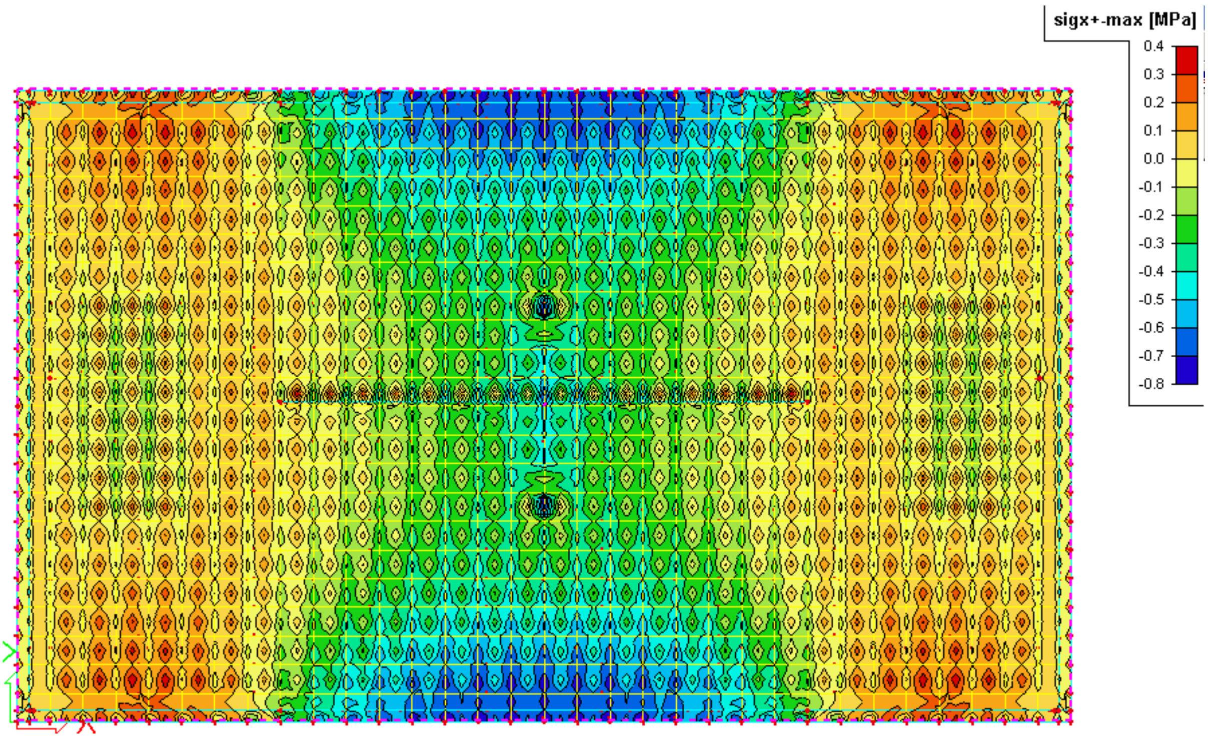
Fase 1, spanningen na plaatsen contragewichten, tweede vloer, spanningen in x-richting



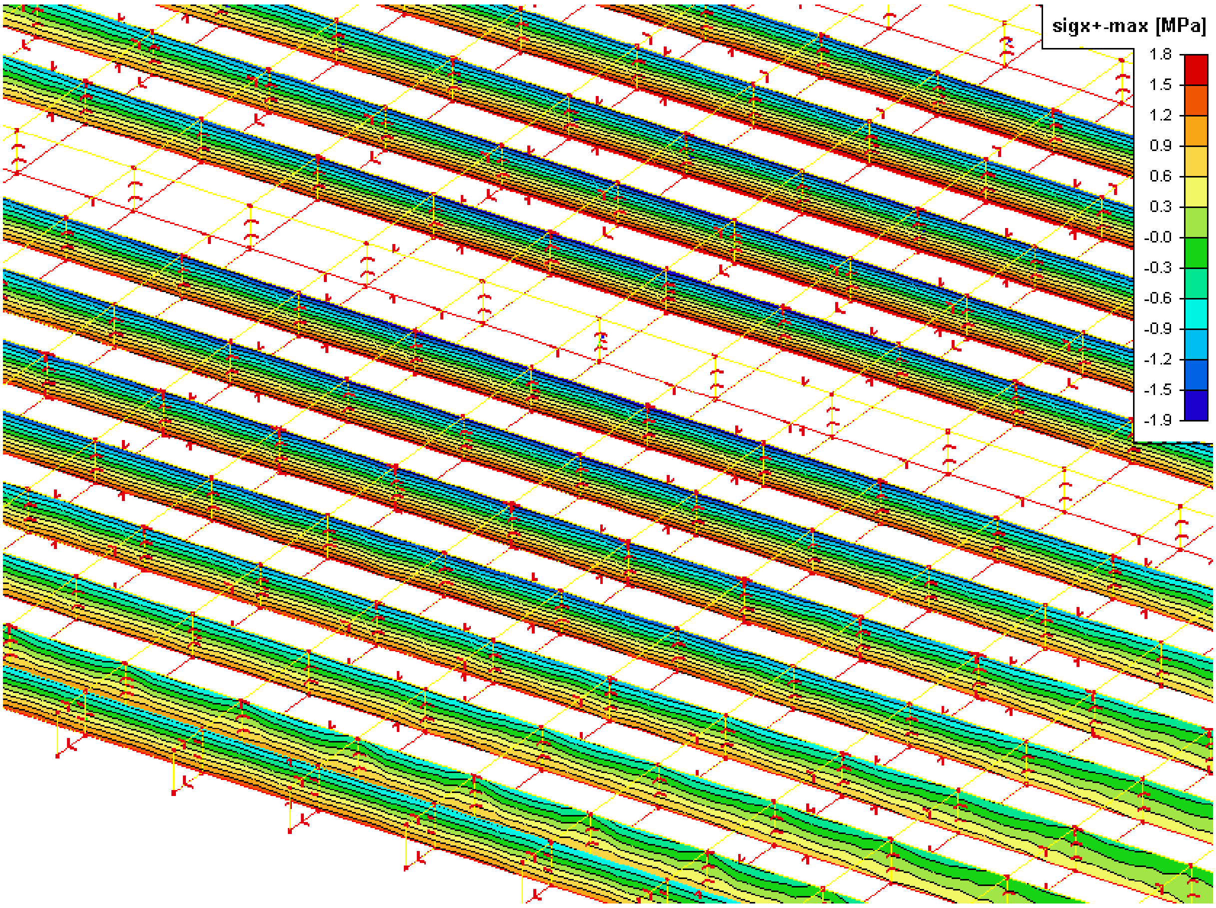
Fase 2, spanningen na plaatsen contragewichten, eerste balklaag, x-richting



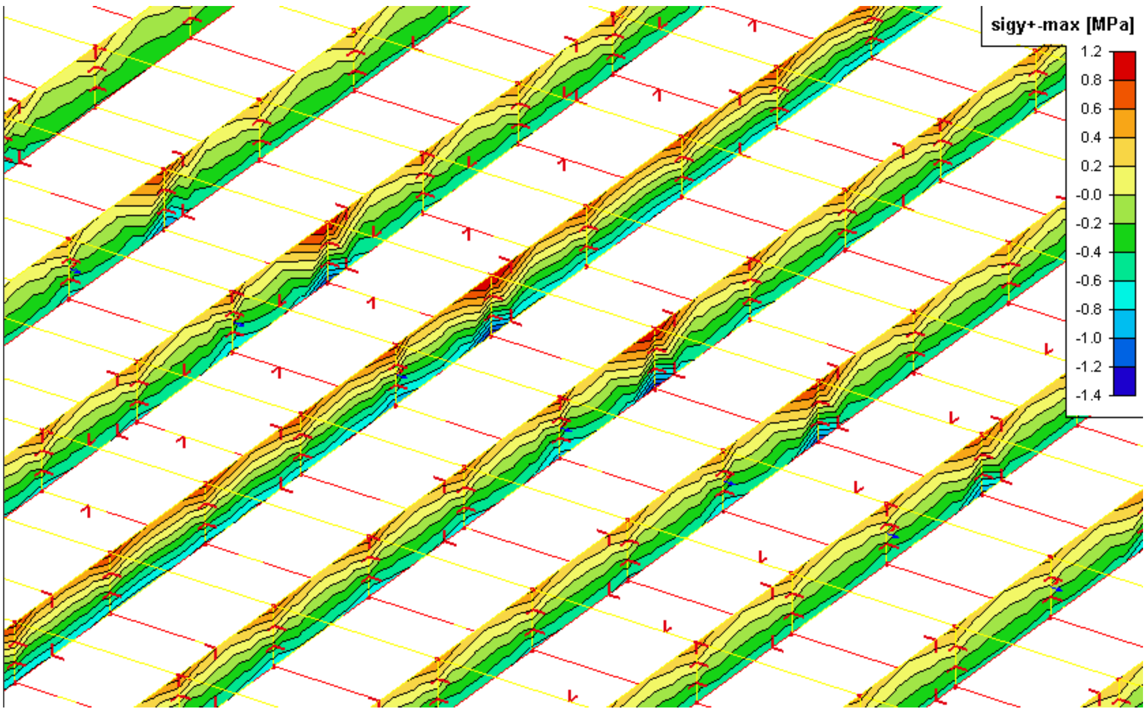
Fase 2, spanningen na plaatsen contragewichten, tweede vloer, spanningen in x-richting



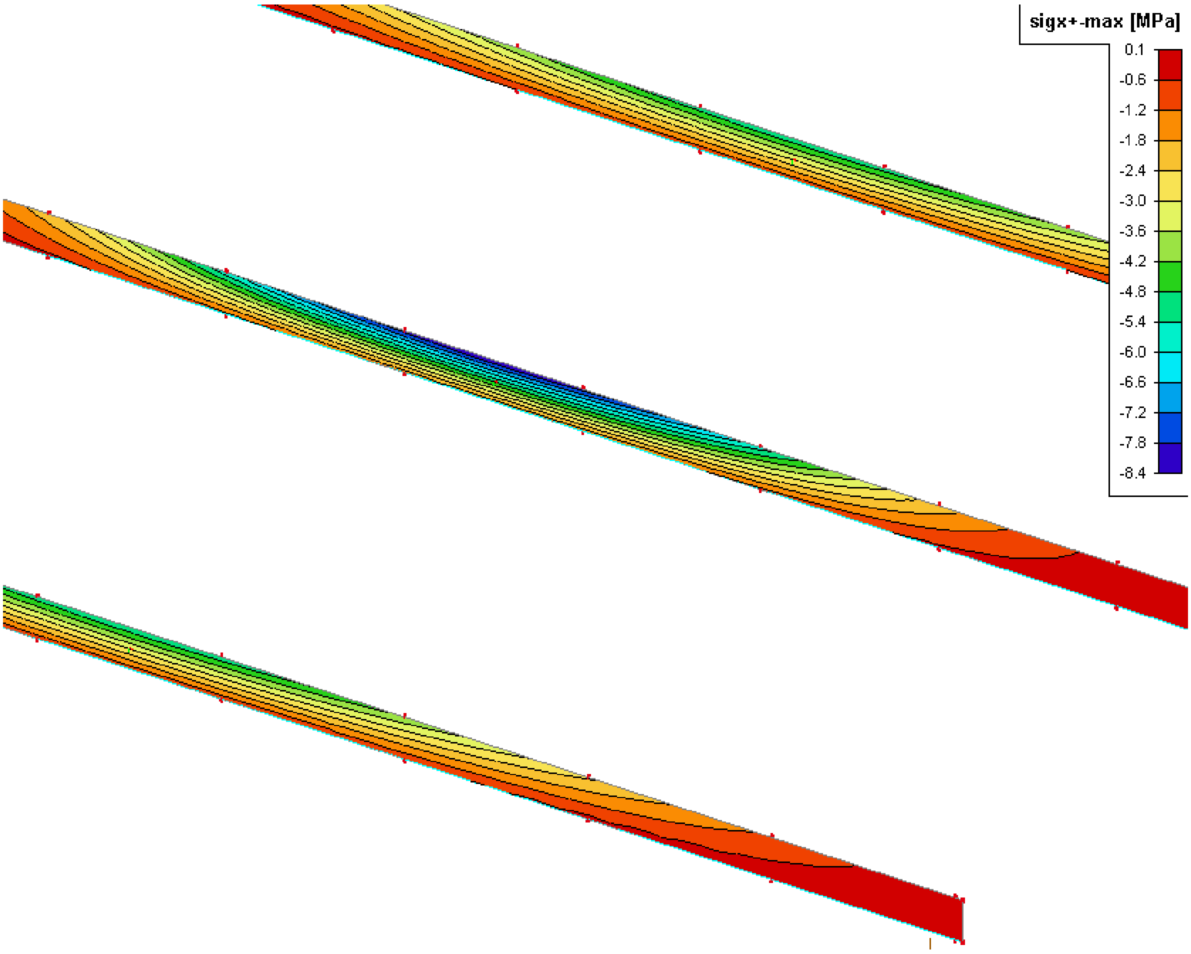
Fase 3, spanningen na plaatsen contragewichten, tweede balklaag, x-richting



Fase 3, spanningen na plaatsen contragewichten, tweede balklaag, y-richting



Fase 3, spanningen na plaatsen contragewichten, wanden eerste verdieping, x-richting



K5 SCIA Engineer: Stresses in EPS

Tijdens het storten van het beton zullen er spanningen en vervormingen optreden in het EPS. In eerste instantie was de vloer zo opgebouwd dat er eerst een balkenlaag gestort zou worden die zou zorgen voor de stijfheid die nodig is voor het storten van de eerste vloer. Dit betekend echter dat de totale Flexbase vloer 80 centimeter dikker wordt. Dit betekend niet alleen meer kosten voor beton, EPS en mankrachten, maar ook meer afgraven om de parkeergarage op de juiste diepte te krijgen.

Flexbase vloer zonder eerste balkenlaag

Bij eerdere ontwerpen van de Flexbase heavy variant is altijd uitgegaan van twee balkenroosters en twee vloeren. Omdat het type EPS 150 werd gebruikt, was het EPS simpelweg niet stijf genoeg om er direct een vloer op te storten. De vervormingen en spanningen in het EPS zouden te groot worden.

Voor de Flexbase vloer van de parkeergarage wordt echter EPS 250 gebruik dat versterkt wordt door lagen glasvezelfolie. Dit type EPS is tot 5 keer stijver. Om uit te zoeken of de Flexbase vloer zonder deze eerste balkenlaag kan zullen er twee test gedaan moeten worden.

<u>Test 1: Spanningen in Flexbase vloer zonder eerste balkenlaag</u>

In de vorige appendices is uitgegaan van een Flexbase vloer met twee balkenroosters en twee vloeren. Nu kan de eerste balkenlaag niet zomaar weggelaten worden. De eerste test houdt in dat er een aantal steekproeven gedaan zullen worden. In de situaties waar de spanningen het hoogst waren in de vloer, zal de eerste balkenlaag weggelaten worden. Daarna wordt nogmaals naar de spanningen gekeken en worden die vergeleken met de eerste resultaten. Als deze waardes dichtbij elkaar liggen kan ervan uitgegaan worden dat de eerdere test niet opnieuw uitgevoerd hoeven worden.

<u>Test 2: Spanningen en vervormingen in EPS tijdens storten vloer</u>

De lagen EPS worden gemodelleerd in SCIA en vervolgens wordt het stortplan gesimuleerd door vlakbelastingen op het EPS aan te brengen. Vervolgens wordt er naar de vervormingen en spanningen in het EPS gekeken. Deze waardes worden getoetst aan de maximale spanningen voor EPS 250 en de maximale vervormingen die mogen optreden tijdens het storten.

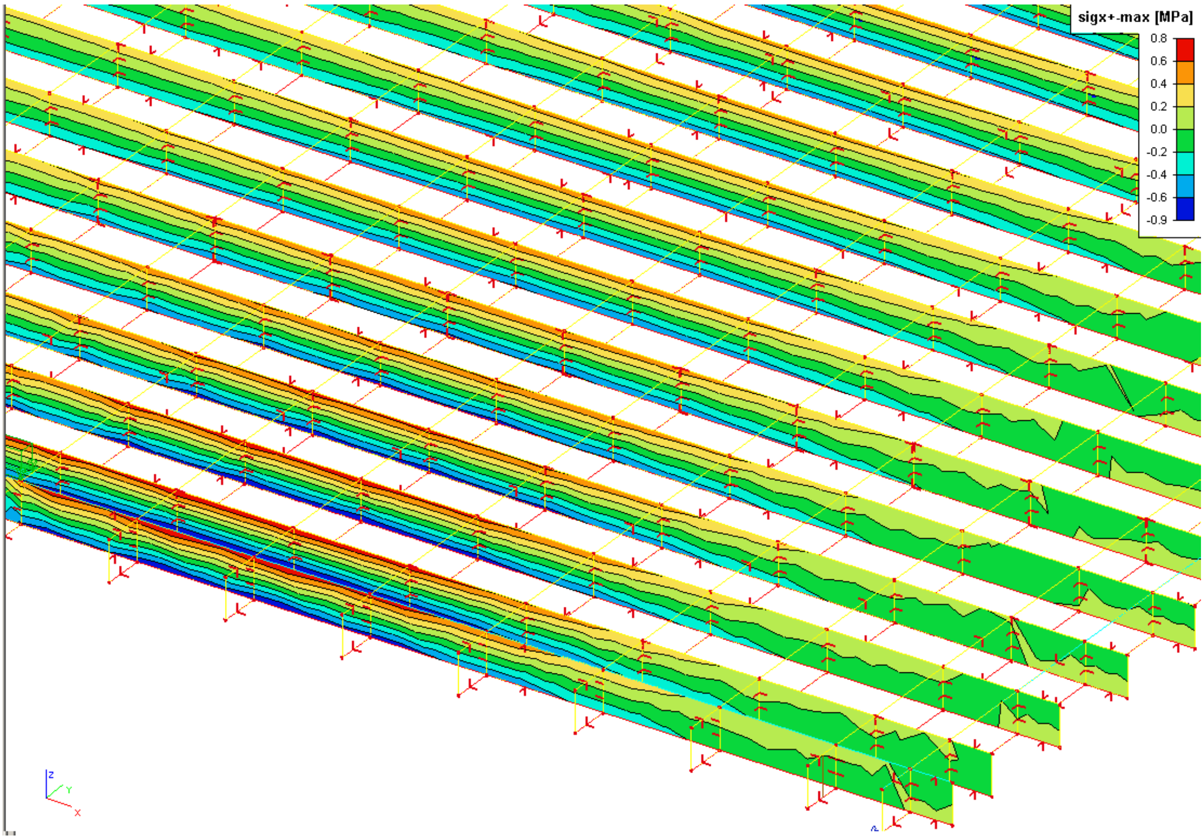
Test 1: Spanningen in Flexbase vloer zonder eerste balklaag

Appendix K4 laat zien dat de trek- en drukspanningen in de tweede balklaag maximaal 0,8N/mm^2 zijn. Deze spanningen zijn erg laag, maar het is interessant om te zien wat de spanningen zijn wanneer de eerste balklaag weg wordt gehaald.

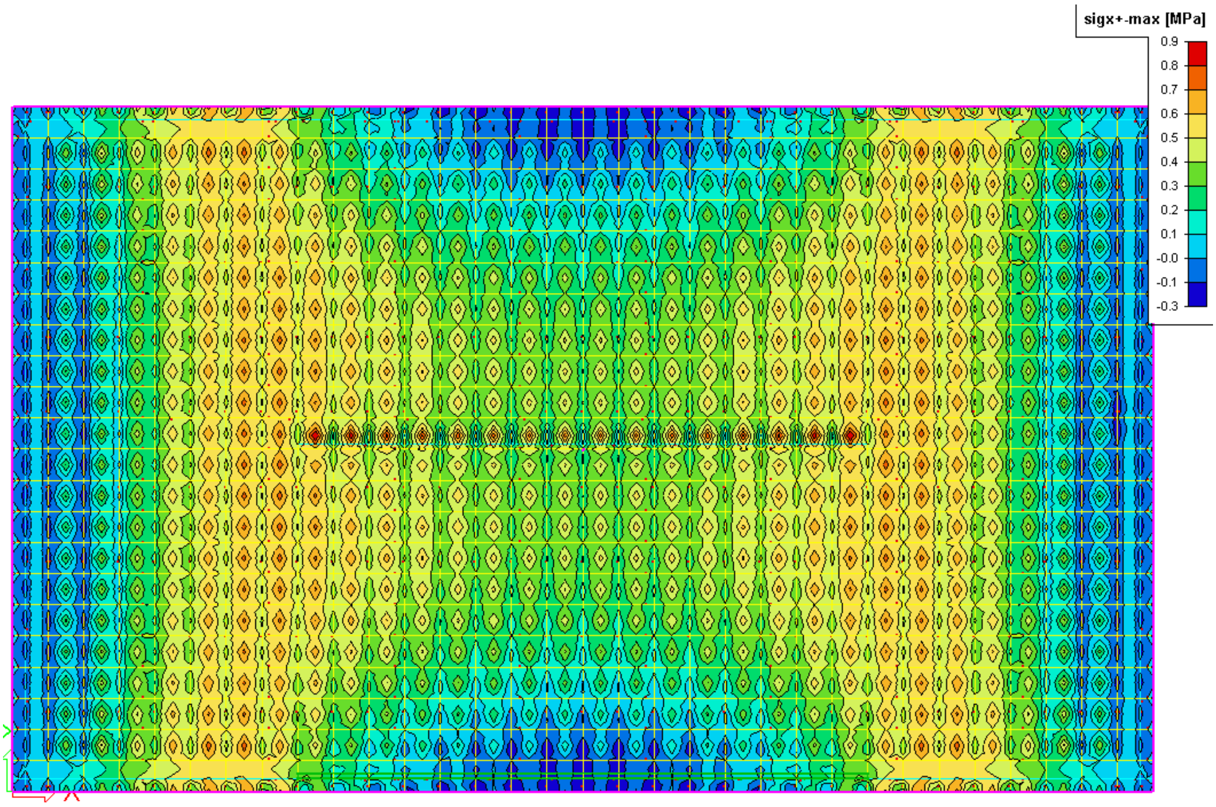
In de figuur op de volgende pagina zijn de spanningen in de tweede balklaag te zien in fase 1.1 wanneer de eerste balklaag is verwijderd. De figuur laat duidelijk zien dat de spanningen tussen de 0,8N/mm^2 trek en de 0,9N/mm^2 druk zitten. Er is dus nauwelijks een verandering te zien.

De tweede figuur laat de tweede vloer zien van de Flexbase vloer. Ook hier is nauwelijks verandering te merken.

Hieruit valt dus te concluderen dat het niet nodig is de modellering opnieuw uit te voeren.



Figuur K5-1: Spanning in x-richting, balkenrooster Flexbase vloer, fase 1.1

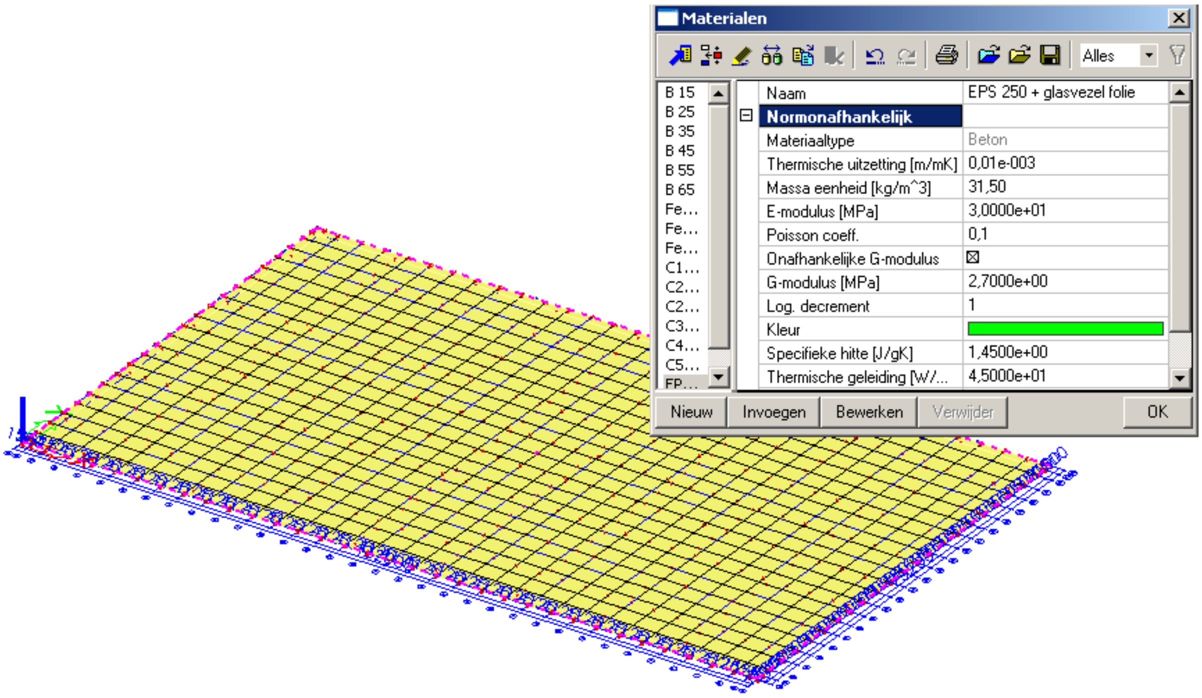


Figuur K5-2: Spanning in bovenste vloer, Flexbase vloer met één balkenrooster, fase 1.1

Test 2: Spanningen en vervormingen in EPS tijdens storten vloer

Als eerste wordt het EPS gemodelleerd in SCIA. De belangrijkste parameter is hier de E-modulus. EPS 250 heeft een E-modulus van 12.000 kPa. De E-modulus van EPS 250 met lagen glasvezelfolie om de 20 centimeter ligt echter hoger. Hier zijn al wel test mee gedaan, maar deze zijn allemaal nog in een begin stadium. Daarom ben ik op bezoek geweest bij Unidek bv en heb ik gesproken met hun engineer op het gebied van EPS. Hij wist mij te vertellen dat ik met een E-modulus van 2 á 3 maal die van EPS 250 aan de veilige kant zou zitten. Dit kan nog niet met documentatie onderbouwd worden en zal dus nog extra onderzoek vergen! Bij het modeleren van het EPS is een E-modulus van 30.000 kPa aangehouden.

Onderstaand figuur laat het model zien. Verder zijn er stramienen ingevoegd op de locaties van de balken in het balkenrooster. Dit maakt het plaatsen van de belastingen makkelijker.



Figuur K5-3: SCIA Model: EPS 250 versterkt met glasvezelfolie

Conclusies

i. De betonnen vloer zal niet in één keer gestort kunnen worden. De vloer van 25 centimeter dik zal dus in twee keer gestort moeten worden, voornamelijk om de vervormingen te beperken. Eerst zal een laag van 10 centimeter aangebracht worden over de gehele vloer. Als tweede stap wordt een laag van 15 centimeter aangebracht, waarmee de totale dikte uitkomt op 25 centimeter. Hierbij kan gekozen worden voor het storten van de tweede laag terwijl de eerste laag nog vloeibaar is of het storten van de tweede laag na uitharden van de eerste laag.

ii. De vervormingen blijven, bij het storten van de vloer in twee keer, onder de 1:20 (maximale hellingshoek is 1:42). Wel wordt de aanbeveling gedaan om beton met een lage consistentie te gebruiken (klasse C2). Deze klasse is ideaal voor het storten op een helling en wordt zelfs gebruik op hellingen met een hoek tot 1:2. Normen op het gebied van hellingshoeken bij het storten van beton zijn er niet. Vanuit de praktijk leert men echter dat er tot een hoek van 1:20 (5%) geen extra aandacht wordt besteed aan het eventueel vloeien van het beton tijdens het storten.

iii. De trekspanningen die ontstaan tijdens het storten van het beton blijven ruim onder de maximaal toelaatbare trekspanningen in EPS250. De treksterkte van het EPS zal in werkelijkheid nog hoger liggen door het toepassen van glasvezelfolie. Daarnaast is hier de lange termijn treksterkte gebruikt (=0,35 * treksterkte EPS250). De waardes blijven dus ver binnen de maximale waardes.

Berekening maximale spanningen

De maximaal toelaatbare spanning in het EPS wordt gehaald uit de documentatie van Kemisol BV. De onderstaande tabel laat voor EPS 250 een waarde zien van 350 kPa, oftewel 0,35N/mm^2 (let op: dit is de lange termijn treksterkte). Hiermee wordt een veilige waarde aangehouden, aangezien het met glasvezelfolie versterkt EPS grotere spanningen aan kan. Hier zijn echter geen waardes voor beschikbaar.

Europees type EPS	Belgisch type EPS	'Level' volgens EN 13163	90/90 ondergrens (kPa)
EPS S		BS50	50
EPS 30		BS50	50
EPS 50		BS75	75
EPS 60	PS 15 & PS 15 SE	BS100	100
EPS 70		BS115	115
EPS 80		BS125	125
EPS 90		BS135	135
EPS 100	PS 20 & PS 20 SE	BS150	150
EPS 120		BS170	170
EPS 150	PS 25 & PS 25 SE	BS200	200
EPS 200	PS 30 & PS 30 SE	BS250	250
EPS 250	PS 35 & PS 35 SE	BS350	350

Tabel K5-1: Maximaal toelaatbare trekspanningen EPS

Uit het stortplan volgt dat de spanningen in de verschillende stortfases erg dicht bij elkaar liggen en het hoogst zijn tijdens de eerste stort. De figuren op de volgende pagina's laten de waardes zien. De trekspanningen liggen in het lineaire deel van het EPS dus het vectorieel combineren van de spanningen is hier toelaatbaar. De maximale spanning in het EPS wordt berekend volgens Huber-Hencky.

Huber-Hencky wordt over het algemeen gebruikt voor doorsnedes met een dominante hoofdrichting zoals liggers. Bij EPS is dit niet het geval, aangezien de doorsnede homogeen is en er geen dominante hoofdrichting is. Het geeft echter wel een goed beeld van de maximale spanningen in het EPS.

$\sigma_{eps250} = 0{,}35$ N/mm^2

$y_m = 1{,}5$ [-]

Resultaten stortplan:

$\sigma_{x+} = 0{,}090$ N/mm^2

$\sigma_{y+} = 0{,}088$ N/mm^2

$\sigma_{xy+} = 0{,}055$ N/mm^2

$\sigma_{x-} = 0{,}086$ N/mm^2

$\sigma_{y-} = 0{,}087$ N/mm^2

$\sigma_{xy-} = 0{,}055$ N/mm^2

Toetsing:

$\Sigma_{EPS250,m} = 0{,}35 / 1{,}5 = 0{,}233$ N/mm^2

Maximale spanning volgens Huber-Hencky

$\sigma_+ = \mathrm{sqrt}(\sigma_{x+}{}^2 + \sigma_{y+}{}^2 + 3 * \sigma_{xy+}{}^2)$

$\sigma_+ = \mathrm{sqrt}(0{,}090 + 0{,}088 + 3 * 0{,}055^2)$

$\boldsymbol{\sigma_+ = 0{,}158}$ **N/mm^2** < 0,233 N/mm^2

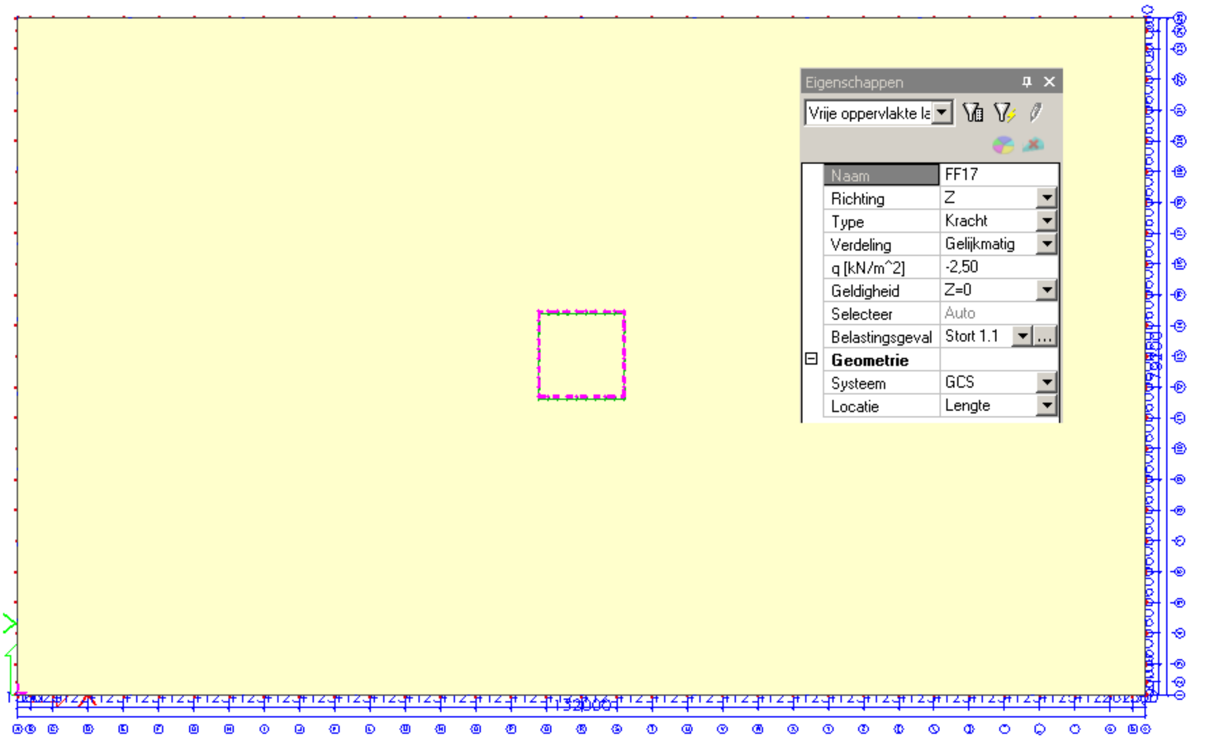
$\sigma_- = \mathrm{sqrt}(\sigma_{x-}{}^2 + \sigma_{y-}{}^2 + 3 * \sigma_{xy-}{}^2)$

$\sigma_- = \mathrm{sqrt}(0{,}086 + 0{,}087 + 3 * 0{,}055^2)$

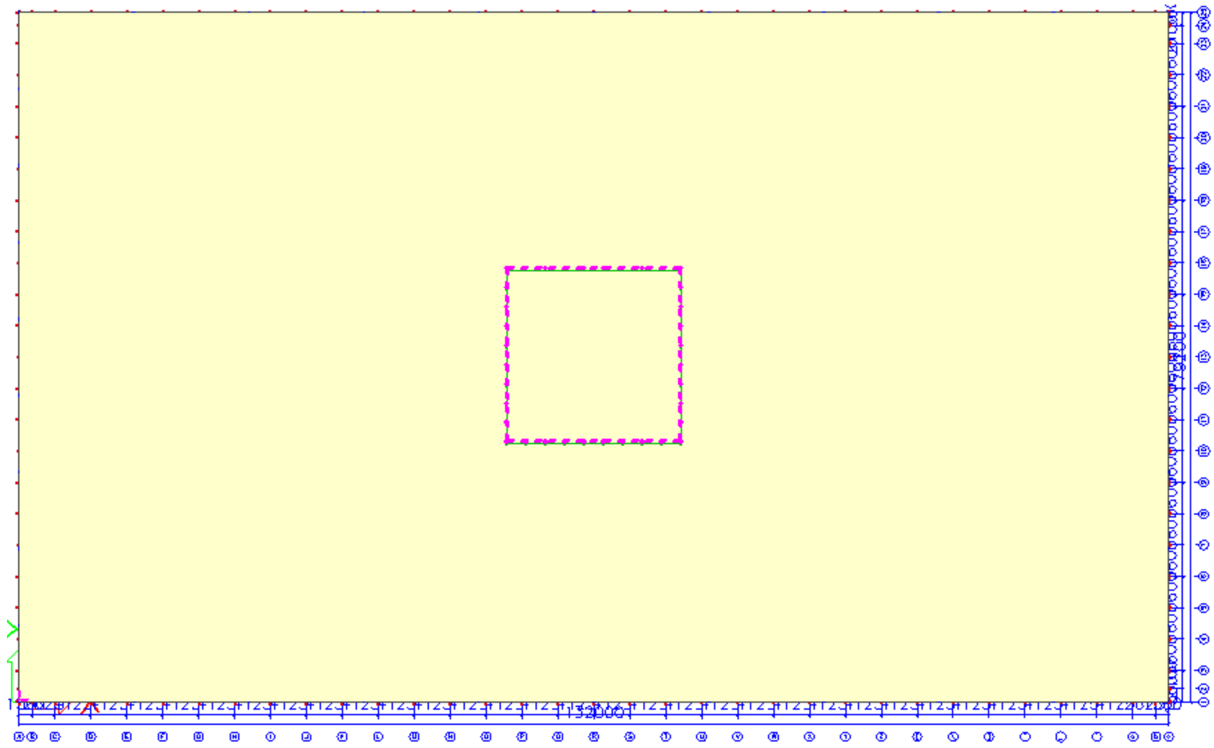
$\boldsymbol{\sigma_- = 0{,}155}$ **N/mm^2** < 0,233 N/mm^2

Unity check: 0,158 / 0,233 = 0,678 < 1,0 → OK

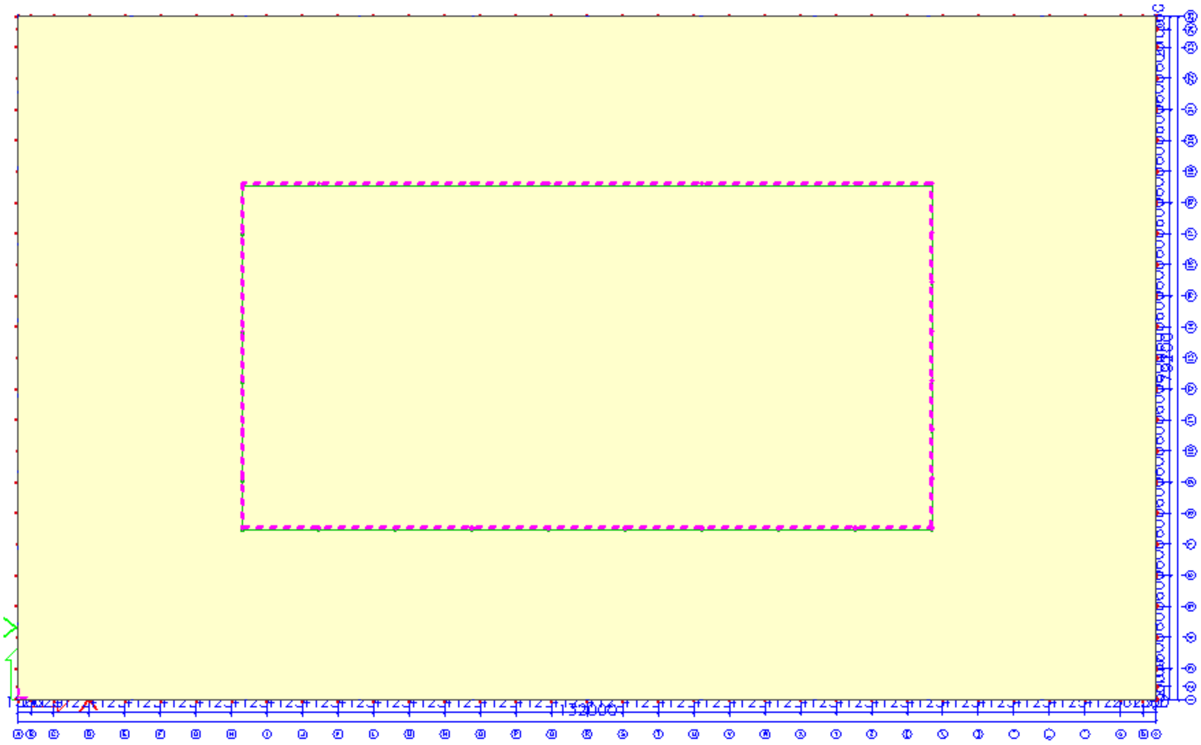
Let op: dit zijn de maximale spanningen en op verschillende locaties in de EPS vloer. Dit is dus een conservatieve berekening! De eigenlijke waardes zullen lager uitvallen.



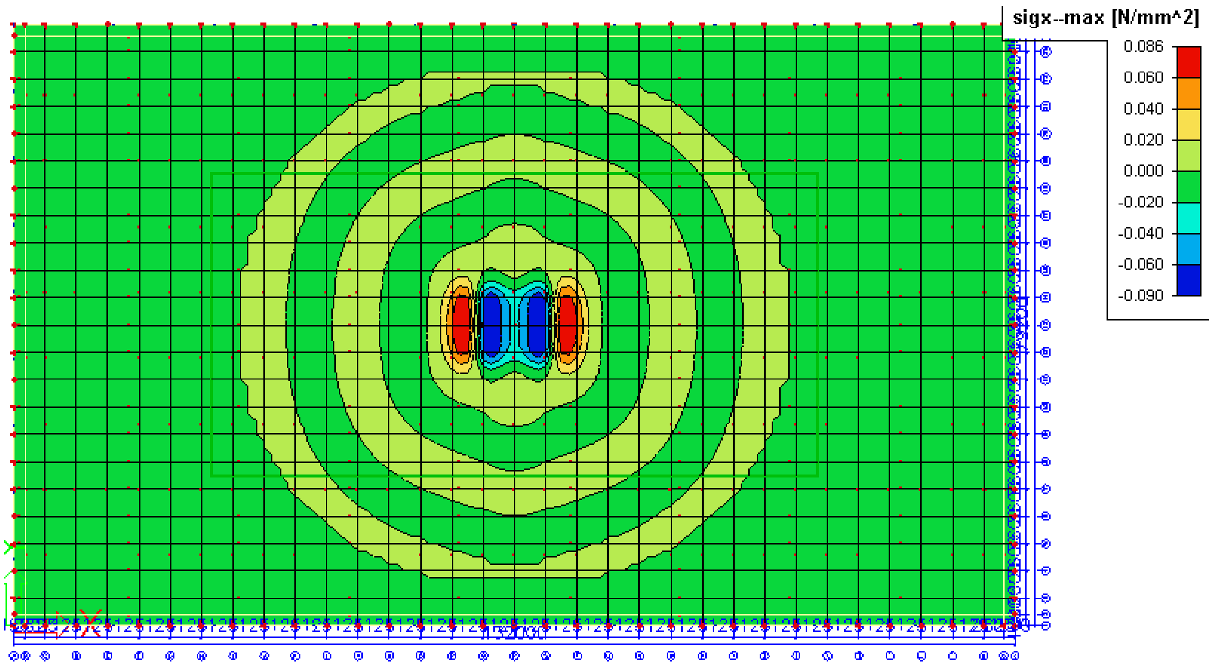
Figuur K5-4: Stortfase 1.1: 10 x 10 meter, 10 cm dikke betonlaag



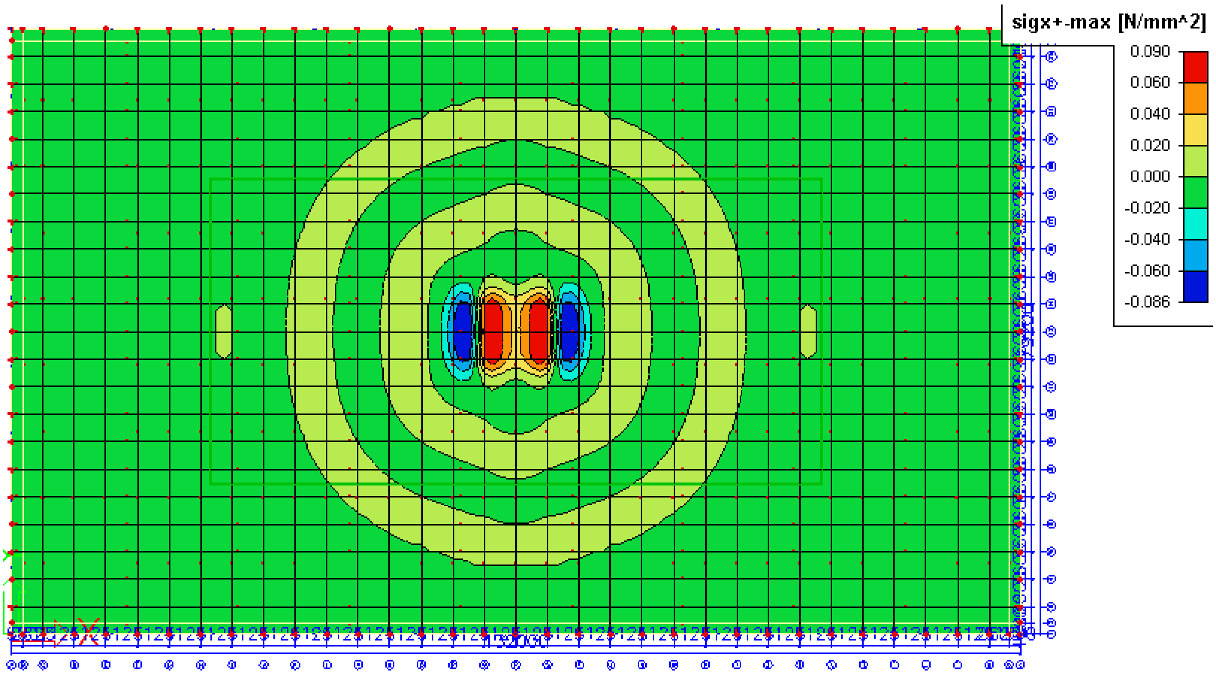
Figuur K5-5: Stortfase 1.2: 25 x 25 meter, 10 cm dikke betonlaag



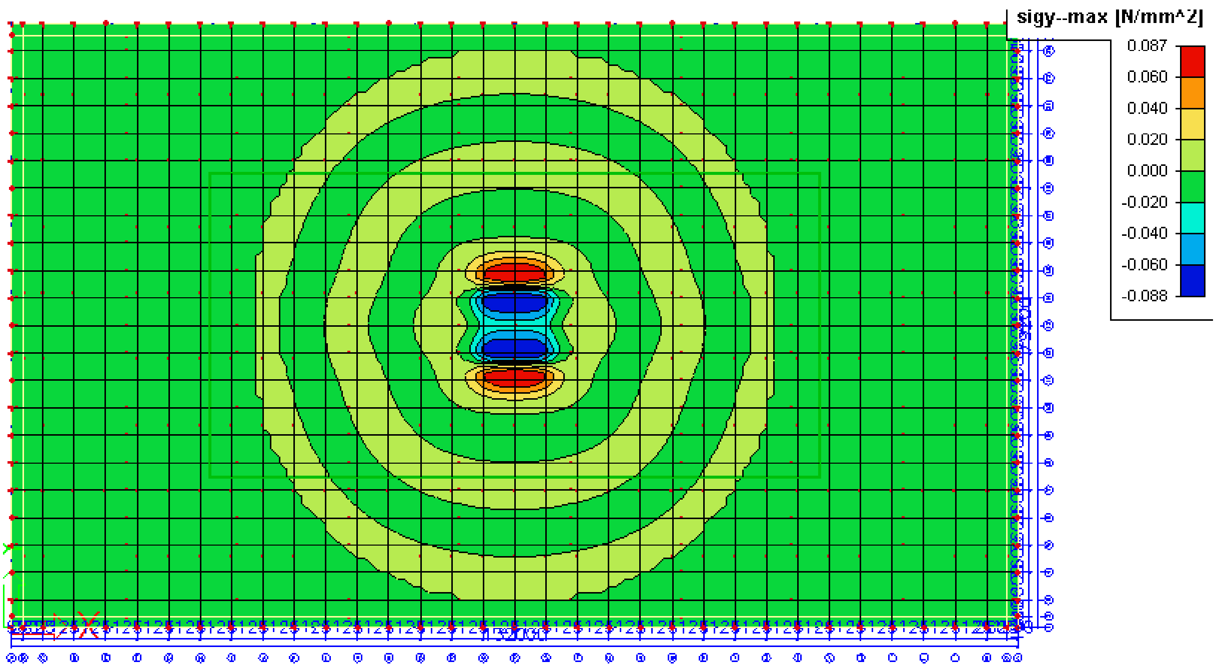
Figuur K5-6: Stortfase 1.3: 25 x 80 meter, 10 cm dikke betonlaag



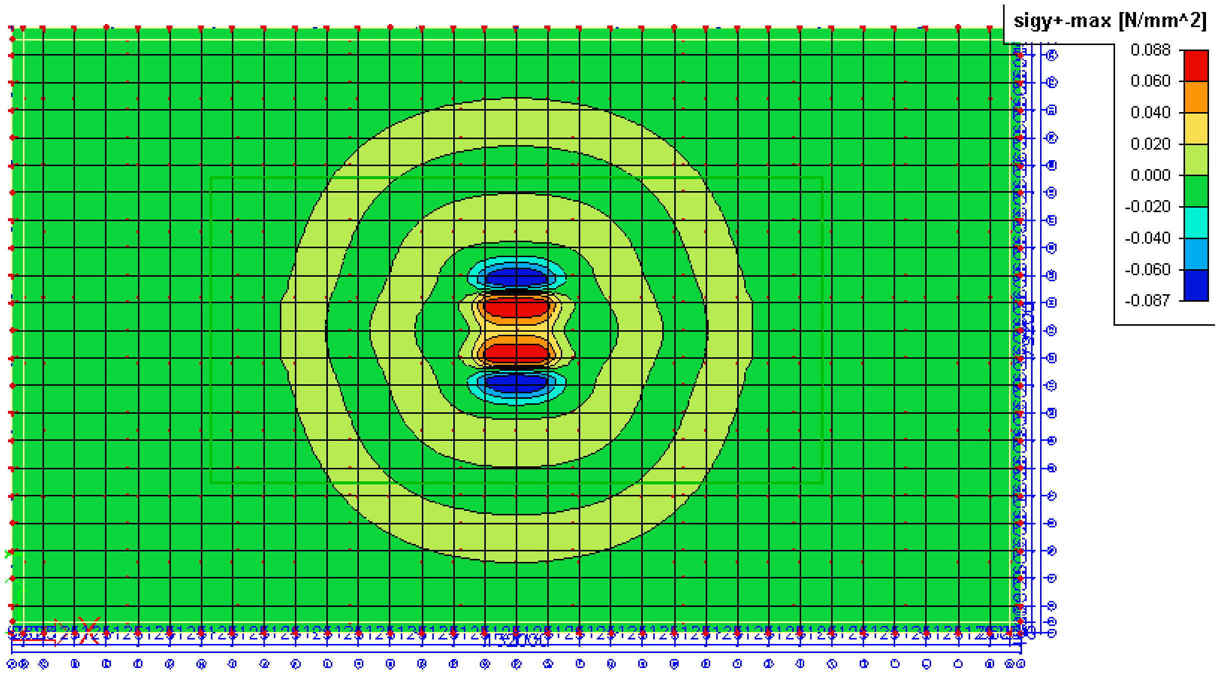
Figuur K5-7: Spanningen in x-richting, onderkant vloer, stortfase 1.1



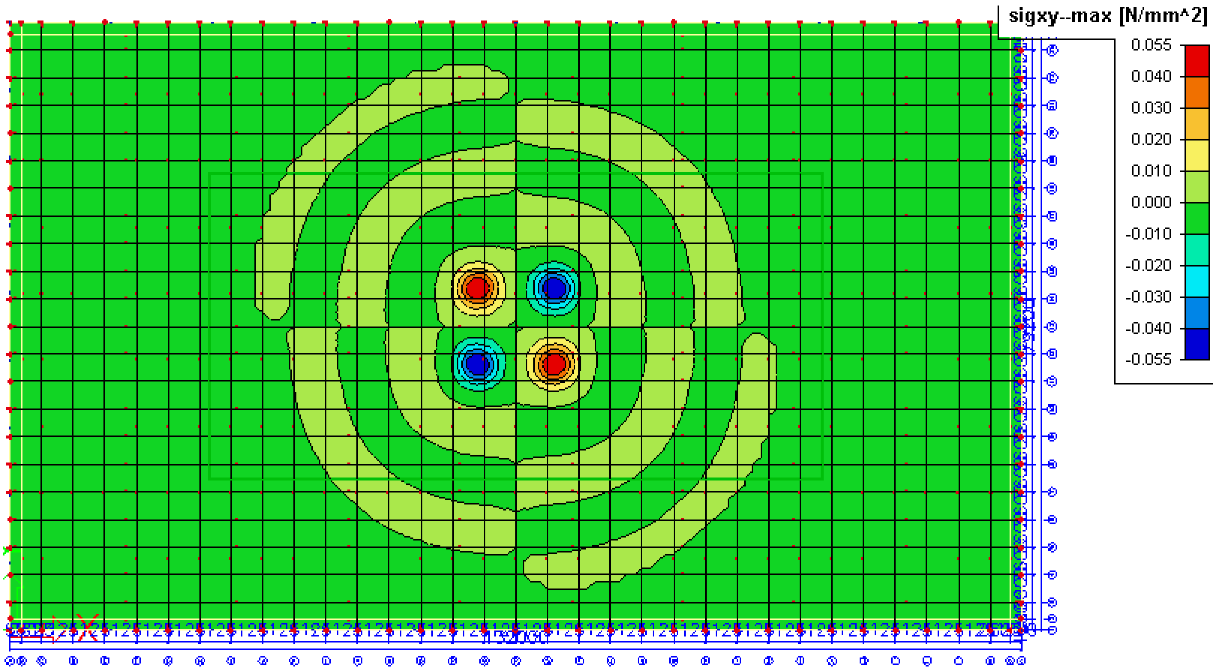
Figuur K5-8: Spanningen in x-richting, bovenkant vloer, stortfase 1.1



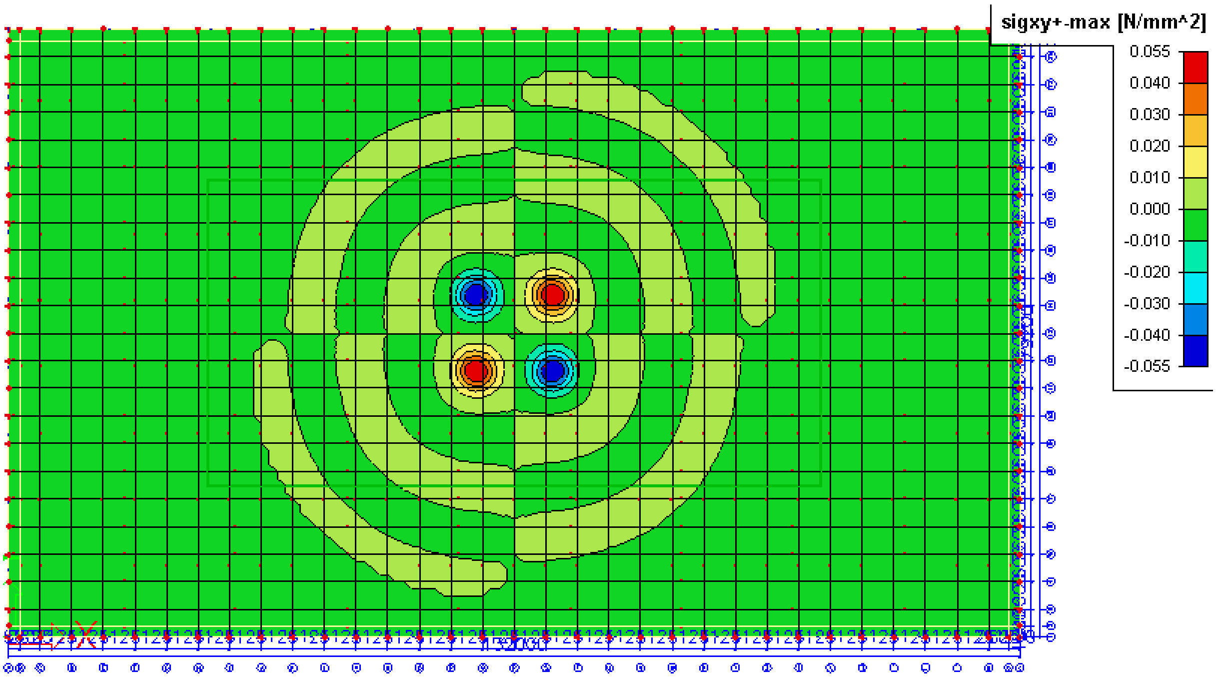
Figuur K5-9: Spanningen in y-richting, onderkant vloer, stortfase 1.1



Figuur K5-10: Spanningen in y-richting, bovenkant vloer, stortfase 1.1



Figuur K5-11: Spanningen in xy-richting, onderkant vloer, stortfase 1.1



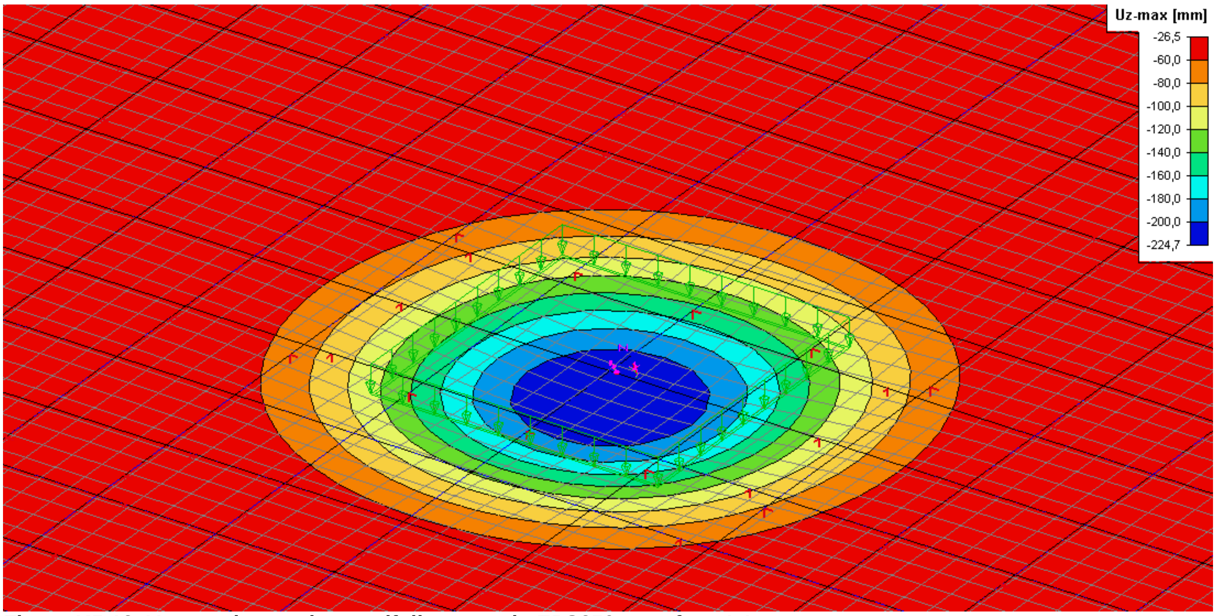
Figuur K5-12: Spanningen in xy-richting, bovenkant vloer, stortfase 1.1

Berekening maximale vervormingen tijdens storten

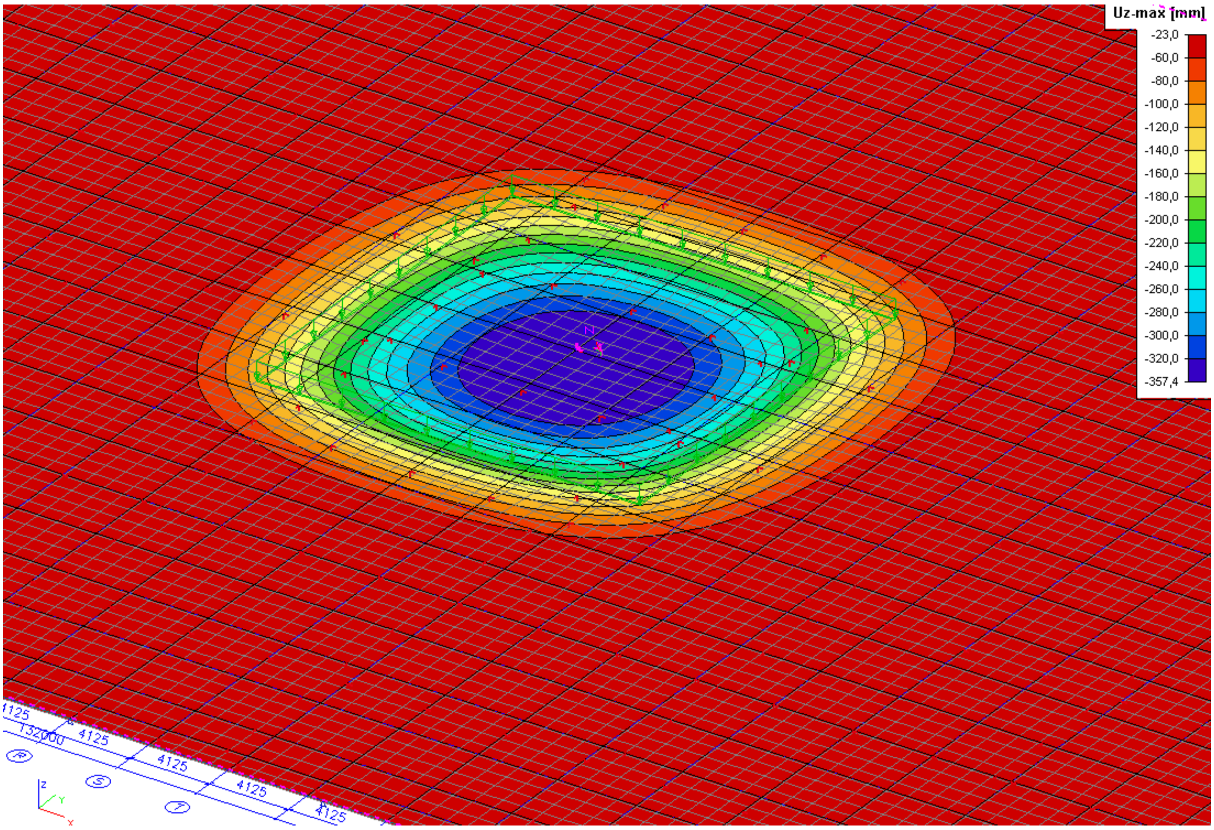
Voor deze berekening worden dezelfde stortfases gebruikt. In elke fase wordt de maximale vervorming berekend. Deze vervorming dient onder de 1:20 te blijven.

De maximale helling wordt aangenomen in fase 1.2. De helling is hier 34:1450, oftewel **1:42.**

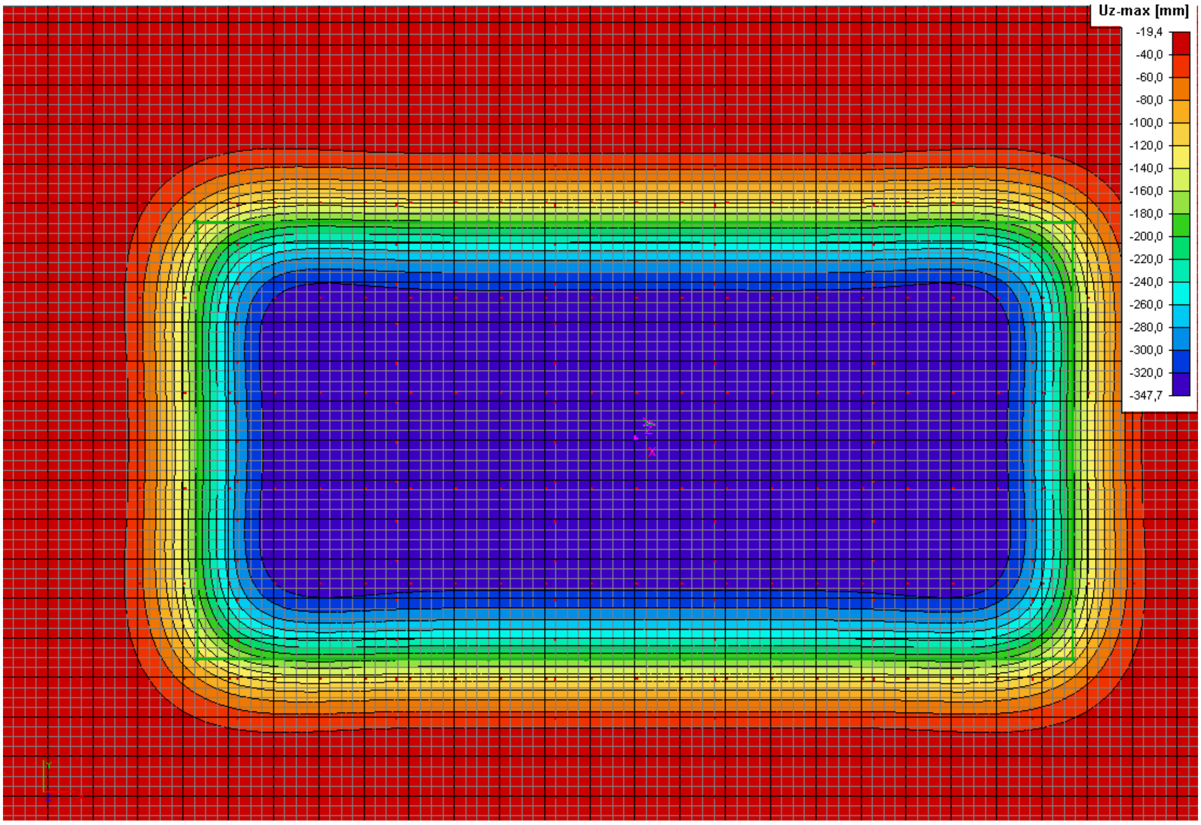
Conclusie: de maximale helling in het EPS tijdens het storten is 1:42. Wanneer een betonmengel wordt gebruikt van consistentieklasse 2 of 3 is dit meer dan voldoende en het zal dan ook geen problemen geven wanneer op een dergelijke helling beton wordt gestort.



Figuur K5-13: Vervormingen glasvezelfolie versterkt EPS250, stortfase 1.1



Figuur K5-14: Vervormingen glasvezelfolie versterkt EPS250, stortfase 1.2



Figuur K5-15: Vervormingen glasvezelfolie versterkt EPS250, stortfase 1.3

K6 SCIA Engineer: Pouring scheme Flexbase floor

In de voorgaande appendices is het stortplan voor de wanden van de parkeergarage en de eerste vloer van de Flexbase vloer besproken. Deze appendix bespreekt het stortplan van het balkenrooster.

Wanneer de eerste vloer op het EPS is gestort is het nog geen stijf geheel. Tijdens het storten van het balkenrooster zullen er dus nog aanzienlijke vervormingen optreden in de vloer. Wanneer de gehele Flexbase vloer als geheel werkt is het beter om naar de spanningen in de vloeren te kijken (aangezien de vloer gezien kan worden als HEA profielen waarin de vloeren de druk en trekspanningen opnemen). In dit geval werkt de vloer alleen en is de som van de druk en trekspanningen nul. Er wordt hier daarom gekeken naar de maximale momenten in de vloer.

Model

De vloer van 250mm dik is ingevoerd in SCIA en ondersteund door een elastische fundering (water). Een stramienrooster is ingevoerd waar het balkenrooster als lijnlasten op wordt gemodelleerd. In 4 fases wordt geleidelijke het balkenrooster gestort.

Het balkenrooster wordt in twee keer gestort. De lijnlast is dan ook 0,4 (breedte) * 0,5 (halve hoogte) * 25 (dichtheid beton) = 5 kN/m^1. De veiligheidsfactor is 1,2 (bouwfase).

De fases en modellen zijn in de figuren op de volgende pagina's te zien.

Berekening

In de figuren op de volgende pagina's is te zien dat het maximale moment tijdens het storten van de balken optreed in fase 2.2. Het moment in de vloer is dan plaatselijk 57kNm.

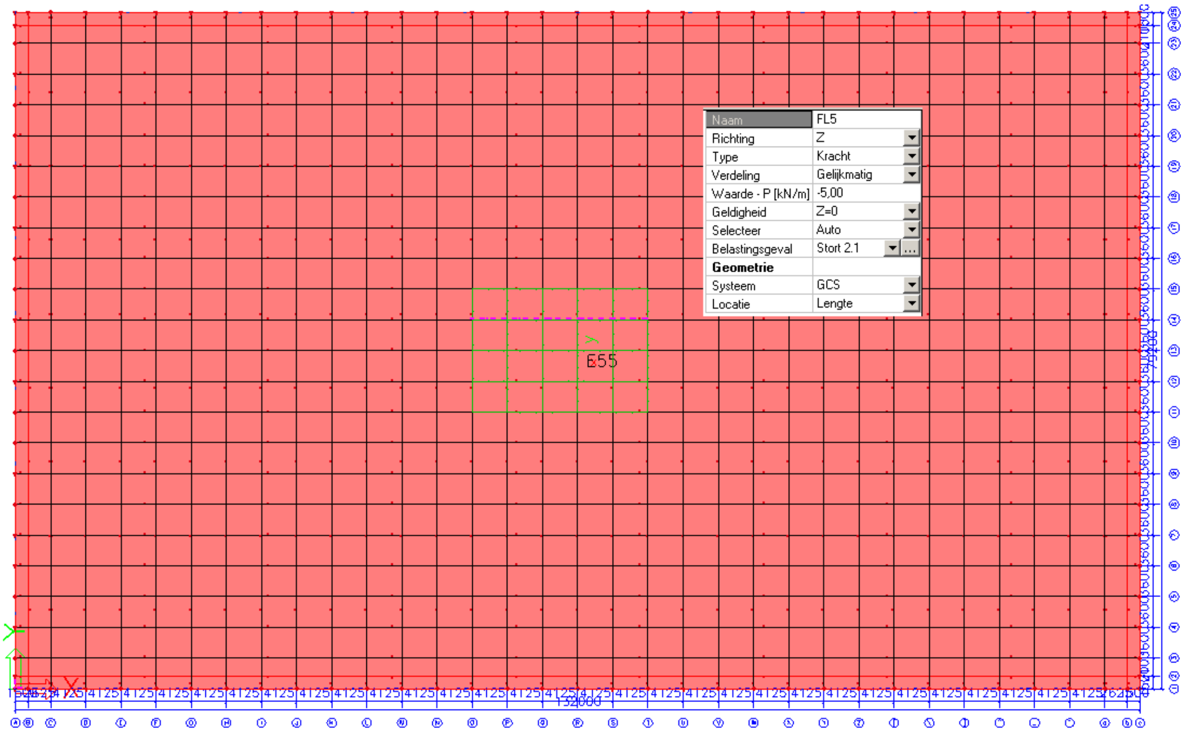
In de onderste vloer zitten twee rijen wapening Ø16-62,5 (appendix K7). Een simpele formule laat zien dat de gewenste wapening is:

$A_s = M /(0,9 * d * f_s) = 57 * 10^6 / (0,9 * 191 * 220) = 1.507 mm^2$

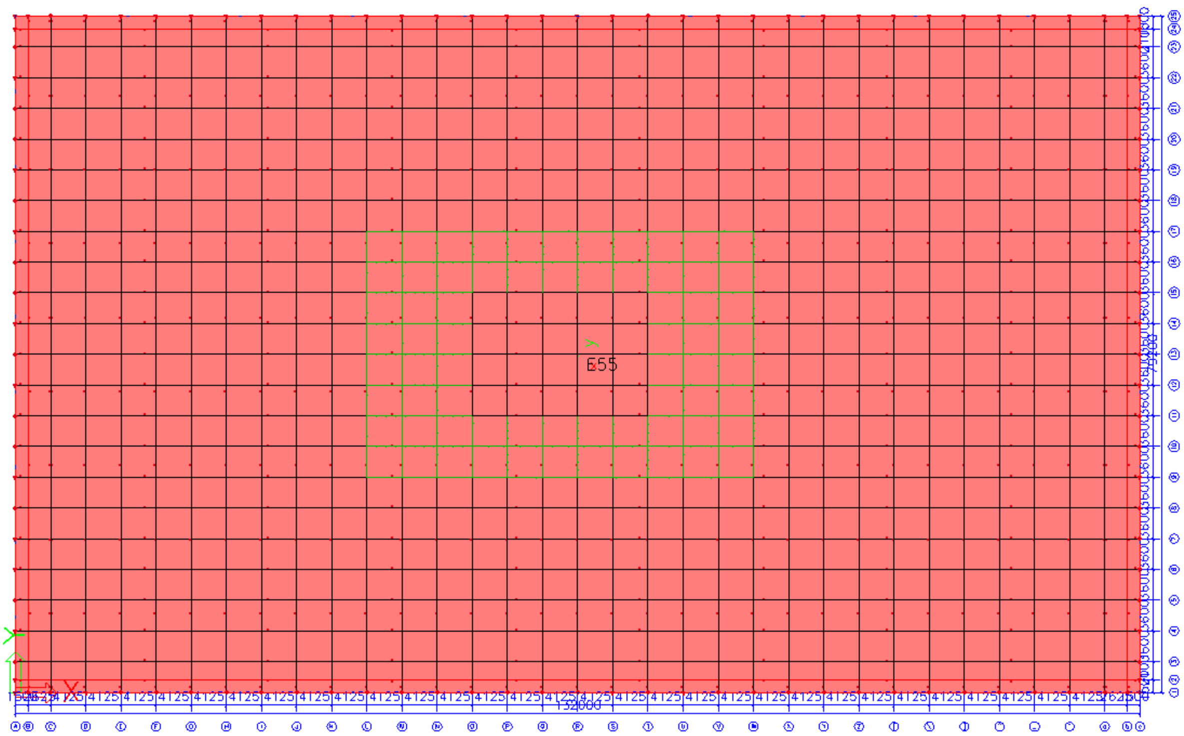
Ø16-62,5 = 6.433mm^2, het op te nemen moment is >200kNm. De wapening voldoet dus ruimschoots.

Conclusie

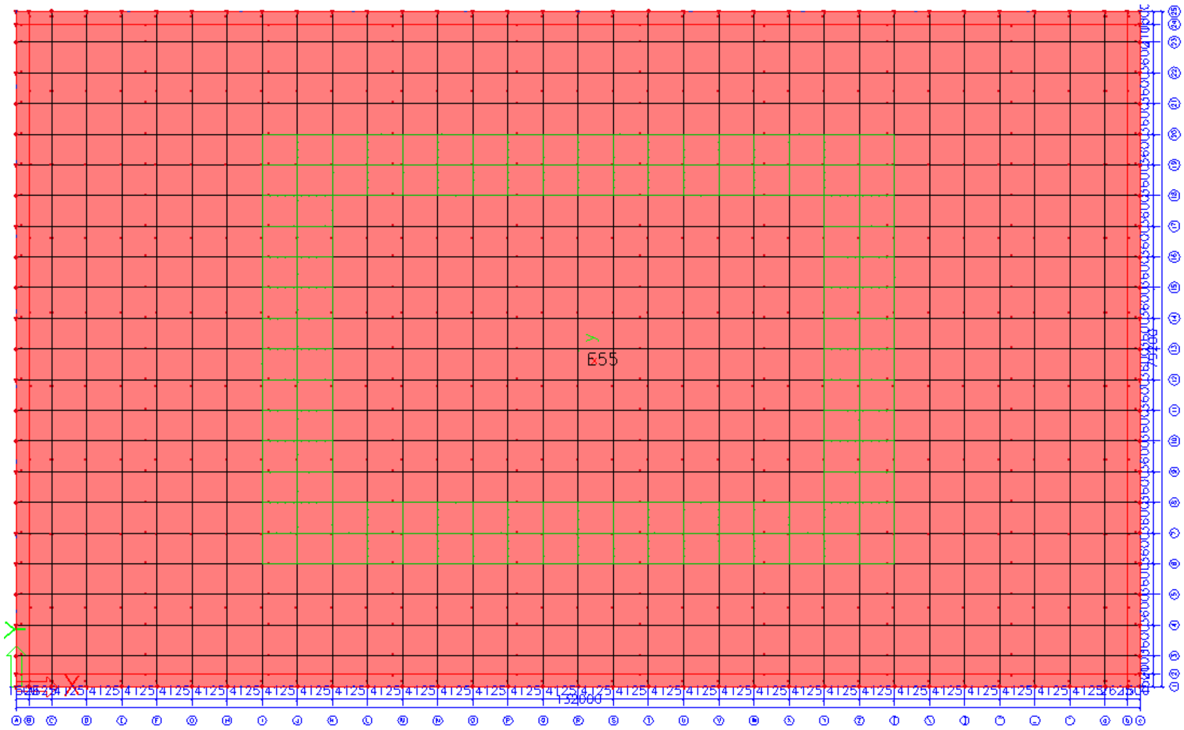
De wapening benodigd om de scheurwijdte te beperken (appendix K7) is ruimschoots voldoende. Nadat de eerste helft van het balkenrooster is gestort en uitgehard wordt het geheel stijf genoeg geacht om de rest van het balkenrooster te storten.



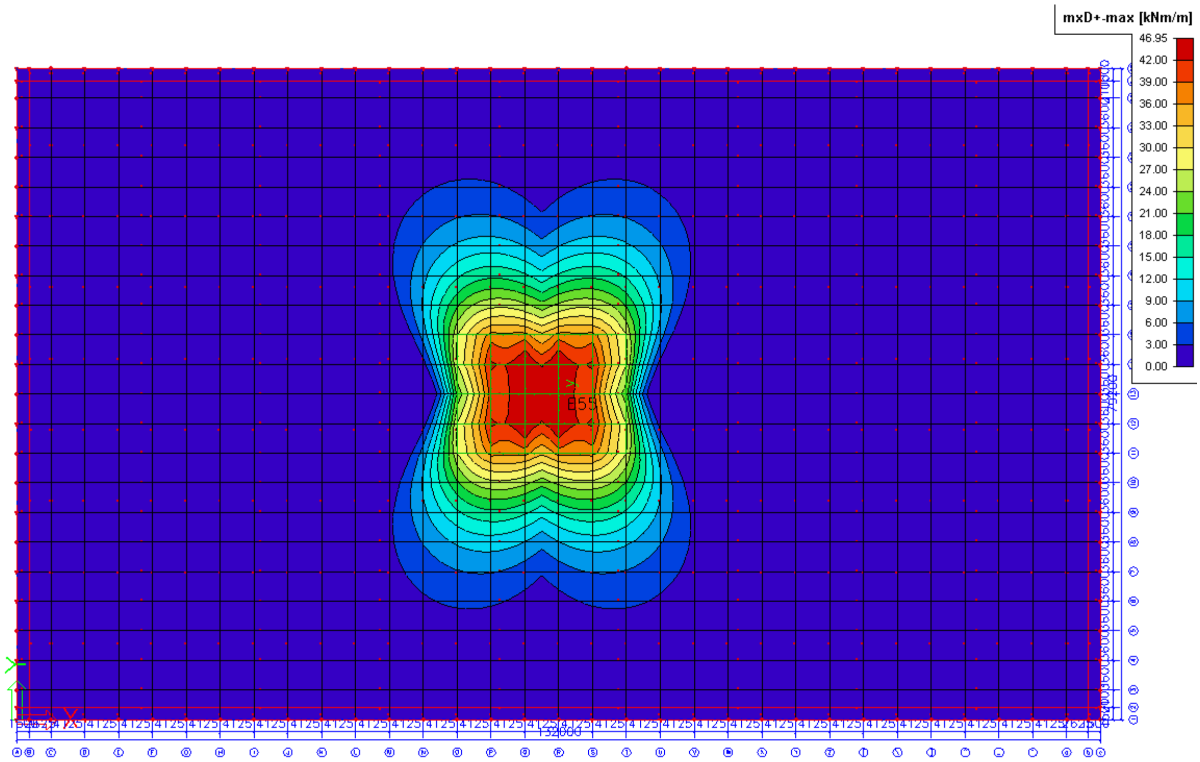
Figuur K6-1: Stortfase 2.1, eerste balken op onderste vloer



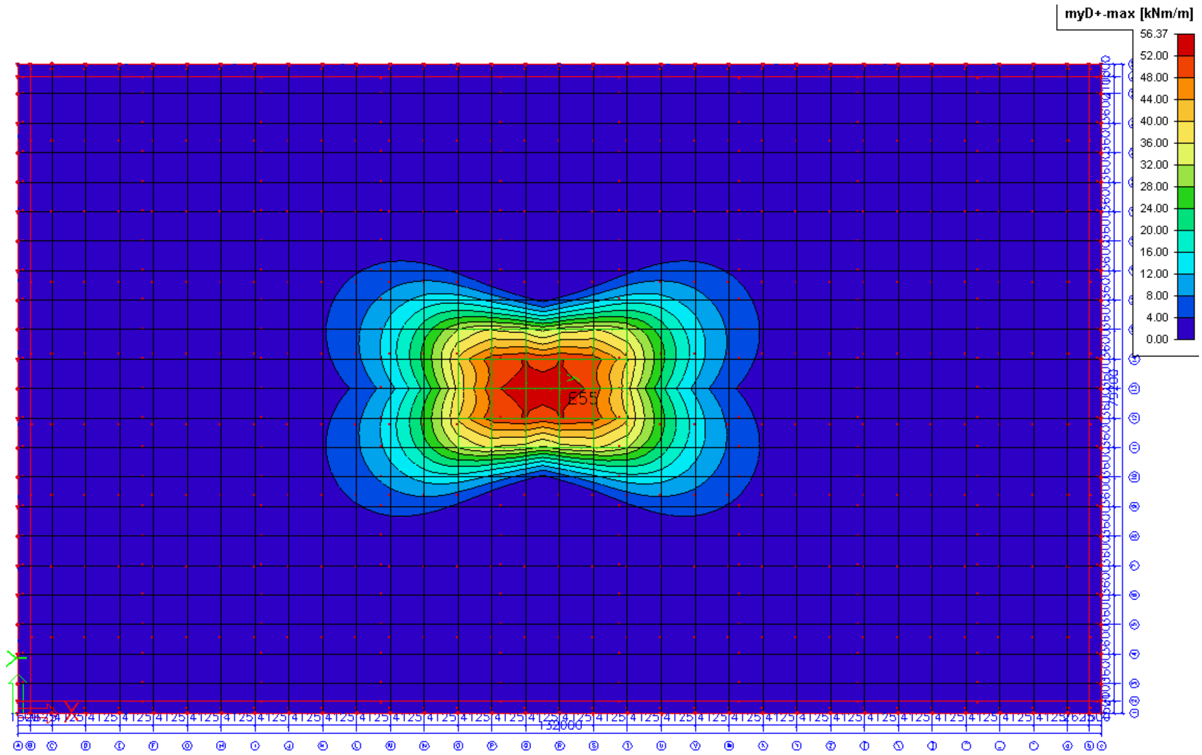
Figuur K6-2: Stortfase 2.2, balken op onderste vloer



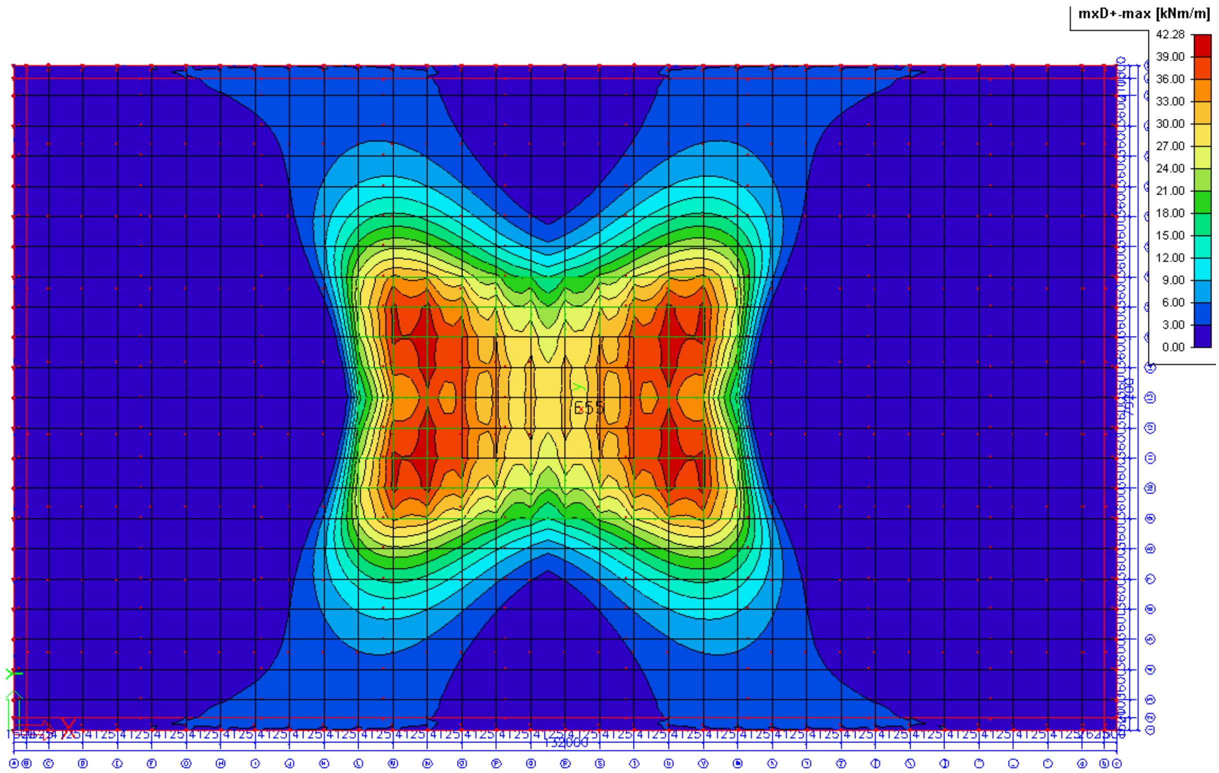
Figuur K6-3: Stortfase 2.3, balken op onderste vloer



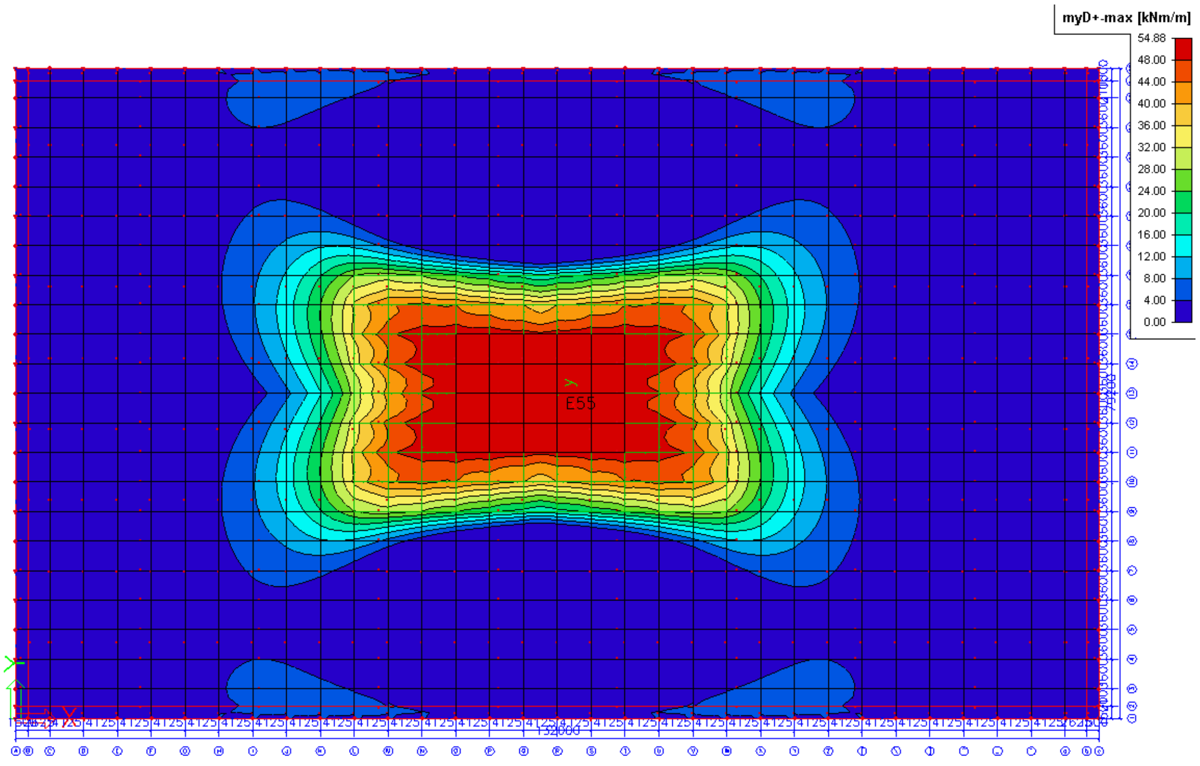
Figuur K6-4: Momenten x-richting, onderste vloer, stortfase 2.1



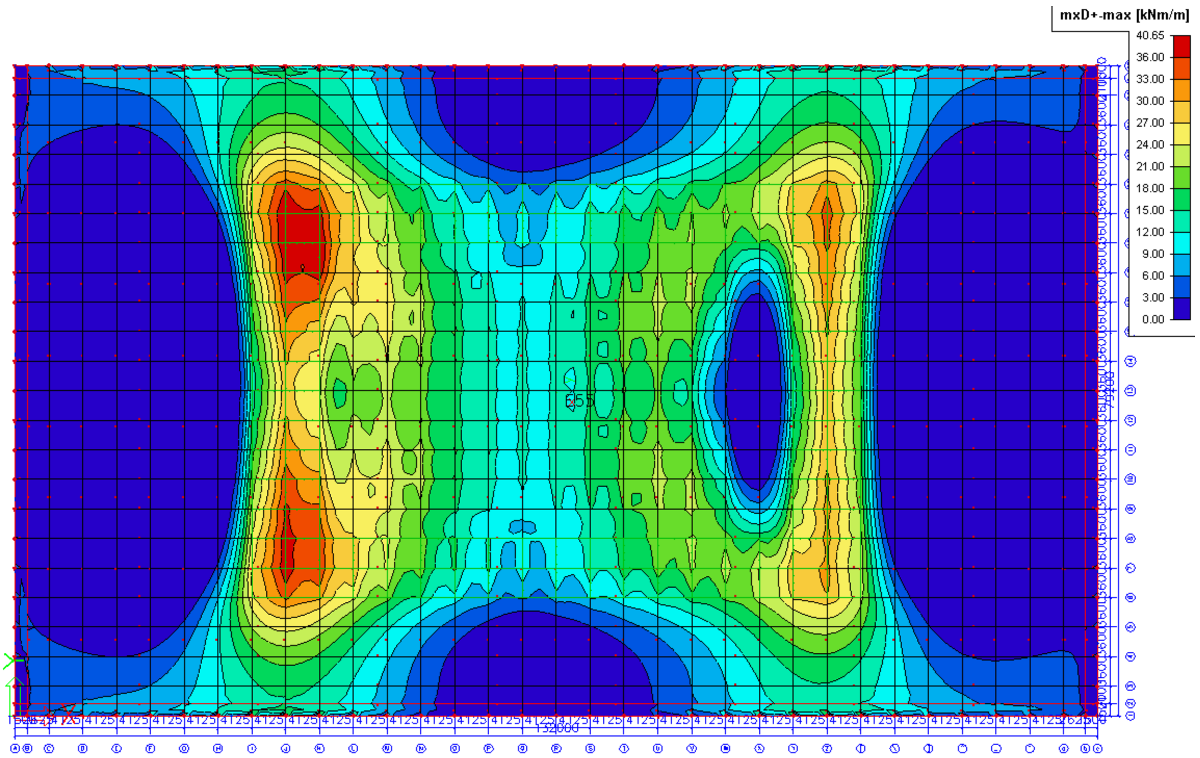
Figuur K6-5: Momenten y-richting, onderste vloer, stortfase 2.1



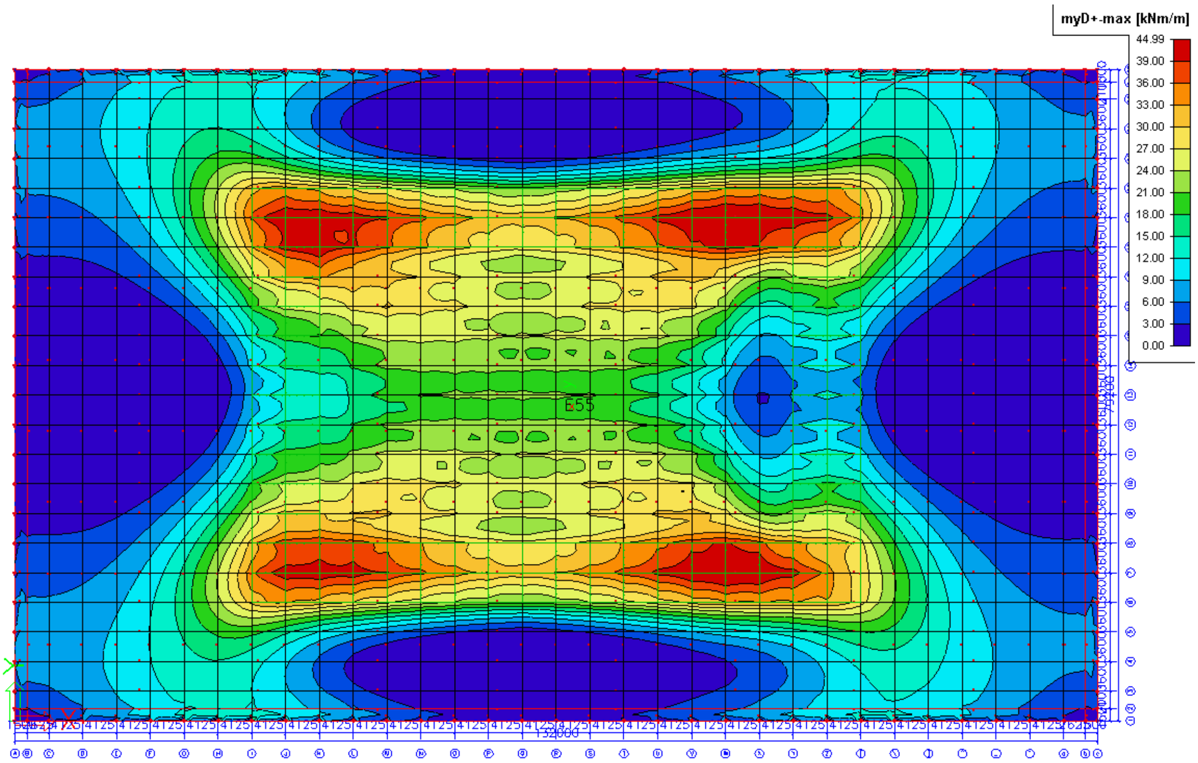
Figuur K6-6: Momenten x-richting, onderste vloer, stortfase 2.2



Figuur K6-7: Momenten y-richting, onderste vloer, stortfase 2.2



Figuur K6-8: Momenten x-richting, onderste vloer, stortfase 2.3



Figuur K6-9: Momenten y-richting, onderste vloer, stortfase 2.3

K7 SCIA Engineer: Operation stage

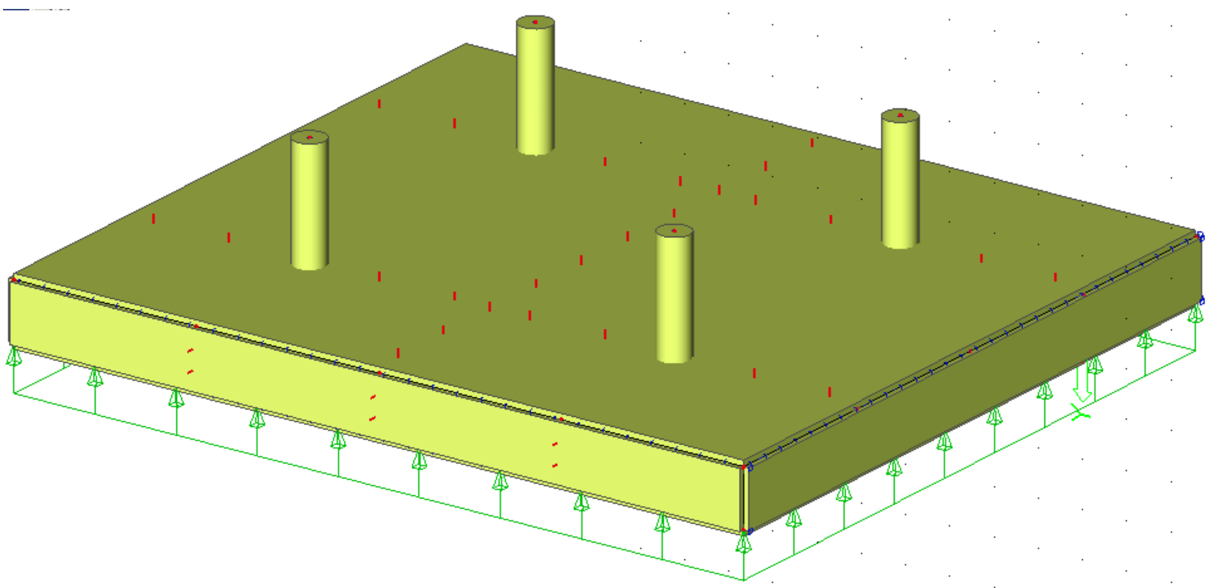
Tijdens de gebruiksfase is de parkeergarage op zijn diepste punt en zijn de krachten die op de wanden, dak en vloer werken het grootst. De constructieve berekeningen voor het dak, de vloer van de tweede verdieping en de wanden zijn te vinden in appendix L. Deze appendix behandeld de Flexbase vloer.

Voor de Flexbase vloer is echter niet de gebruiksfase maatgevend, maar de fase net daarvoor: de afzinkfase. Net voordat de Flexbase vloer de uiteindelijke diepte bereikt werkt al wel de maximale waterdruk op de onderzijde, maar zijn de groutankers nog niet aangebracht. Dit betekend dat de maatgevende overspanning niet degene is tussen de groutankers (4,12 meter) maar de afstanden tussen de kolommen, namelijk 8,25 meter. Dit betekent dat de momenten in de vloer tijdens deze fase maximaal zijn. Deze fase zal dan ook gemodelleerd worden in SCIA. (Zie appendix J voor spanningen in vloer tijdens gebruiksfase)

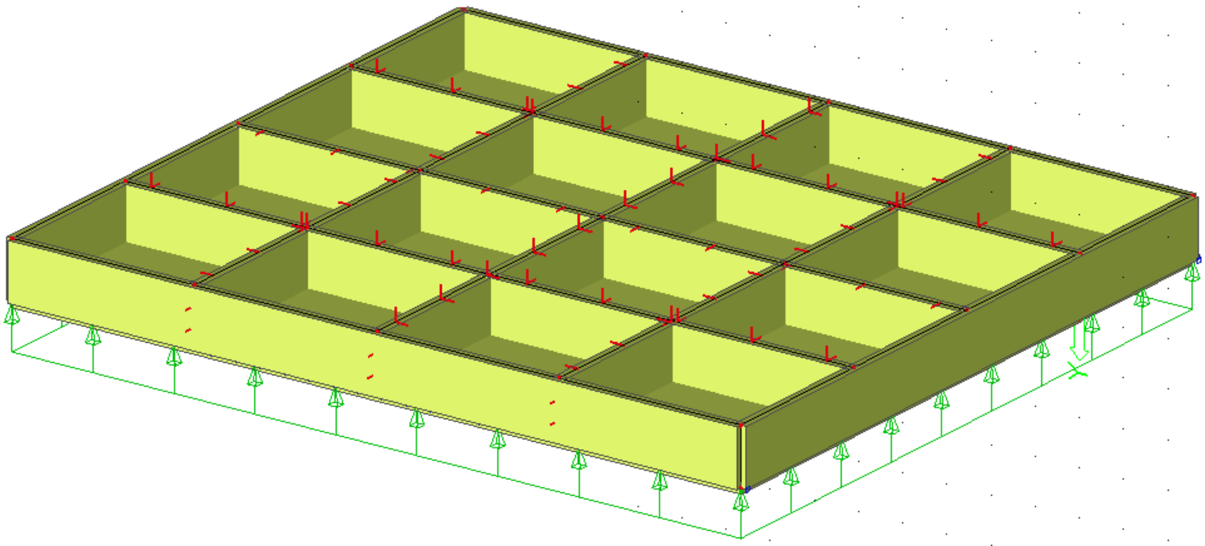
Model

De complete vloer invoeren in SCIA en nauwkeurige gegevens opvragen zou het programma erg langzaam maken, daarom is een maatgevend gedeelte van de vloer ingevoerd. In de twee onderstaande figuren is het model te zien. De h.o.h. afstanden tussen de kolommen zijn 7,20 meter en 8,25 meter. Bovenop de kolommen zitten de opleggingen.

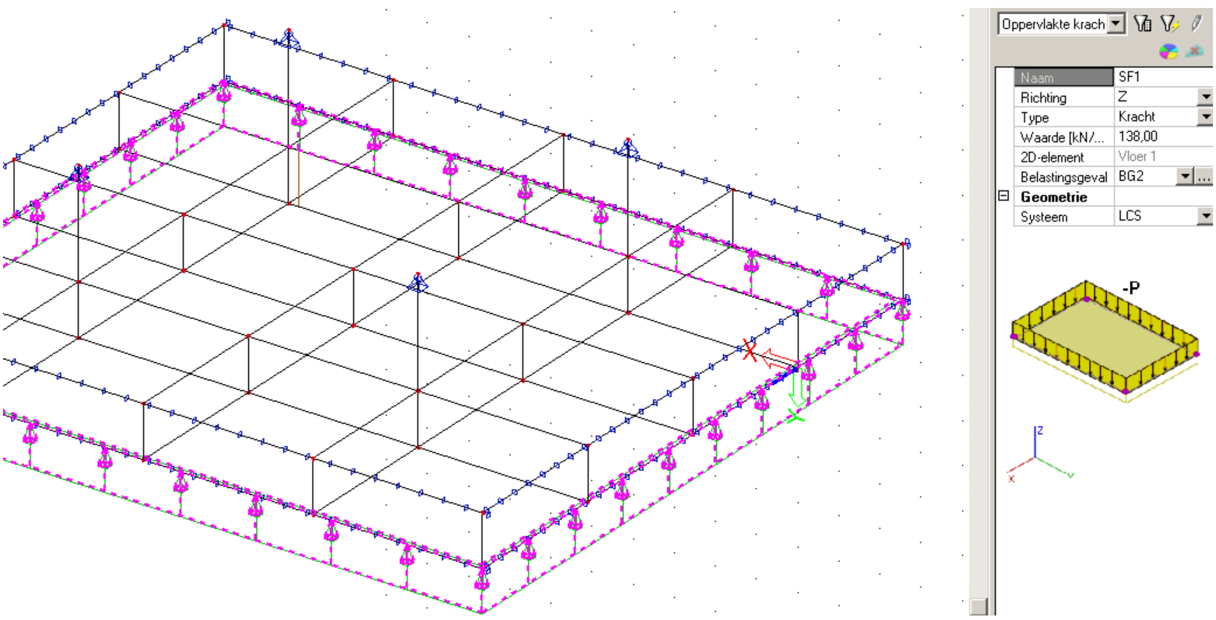
De randen van de vloer zijn momentvast opgelegd om te simuleren dat het een onderdeel is van een vloer. In dit model is enkel de waterdruk tegen de onderzijde ingevoerd. Dit is een conservatieve simulering, aangezien in werkelijkheid de druk van de watertanks als contragewicht aanwezig is en de vervormingen, en daarmee de momenten, verkleind. Tegen de onderzijde is een vlakbelasting aangebracht van 138 kN/m^2. Dit simuleert een waterdruk van 13,8 meter.



Figuur K7-1: SCIA Model, onderdeel Flexbase vloer



Figuur K7-2: SCIA Model, onderdeel Flexbase vloer, bovenste vloer weggelaten



Figuur K7-3: SCIA Model, onderdeel Flexbase vloer, waterdruk als vlaklast tegen onderzijde

Berekeningen

De sandwich-constructie kan gezien worden als een samenwerking van HEA profielen waarin het lijf de dwarskracht opneemt en de flensen (de vloeren) de druk- en trekspanningen opnemen. In deze appendix zullen de volgende berekeningen worden uitgevoerd:

iv. Berekening balken op dwarskracht;
v. Berekening scheurwijdte onderste vloer (wmax 0,3mm);
vi. Berekening scheurwijdte bovenste vloer (wmax 0,1mm).

Het verschil tussen de tweede en derde berekening is dat de onderste vloer niet waterdicht hoeft te zijn, dit zal namelijk niet leiden tot wateroverlast in de garage. Wanneer de bovenste vloer niet waterdicht is zal dit wel leiden tot wateroverlast. De onderste vloer wordt daarom getoetst op de maximale scheurwijdte bij de hoogste milieuklasse. Namelijk 0,3mm volgens EC 1992-1-1.

Berekening balkenrooster

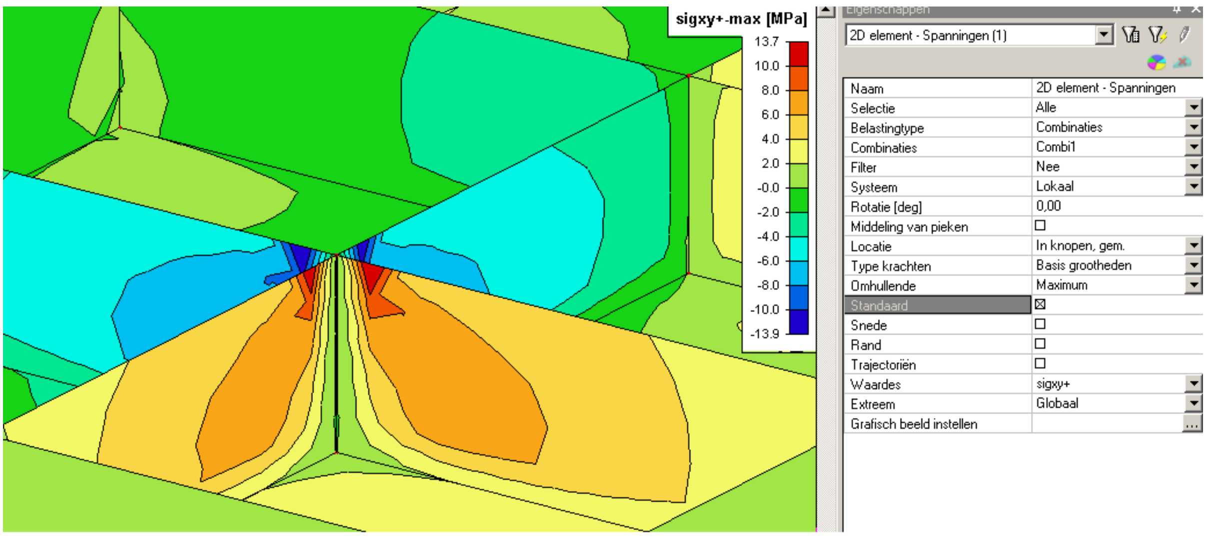
Als eerste wordt de maximale schuifspanning cecontroleerd. Deze waarde mag niet groter zijn dan τ_2 (4,2N/mm^2 voor B28/35). Zoals in de onderstaande figuur te zien is, is de schuifspanning het hoogst waar de balken elkaar kruisen. Hier wordt een snede over de hoogte van de balk genomen en de gemiddelde schuifspanning over deze snede. De gemiddelde schuifspanning is 6,4 N/mm^2 > τ_2.

Hiervoor zijn twee oplossing, wanneer de h.o.h. afstand van de balken gelijk blijft, namelijk:

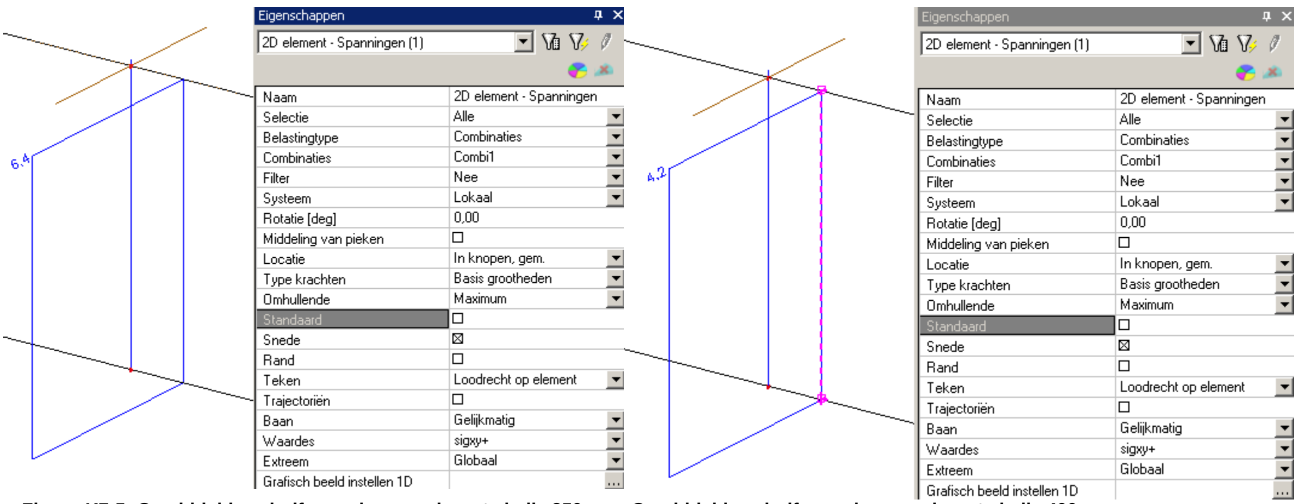
i. Betonklasse verhogen;
ii. Balkafmetingen veranderen.

Ik kies ervoor de balkafmetingen te vergroten. Ik voer de berekening nogmaals uit met een balk van 400mm breed. Zoals te zien is in de onderstaande figuur is de gemiddelde schuifspanning dan 4,2N/mm^2 = τ_2 → OK. Met deze waarde zal de dwarskracht wapening berekend worden.

Let op: dit is de dwarskrachtwapening op de plaats waar de schuifspanning maximaal is. Deze dwarskrachtwapening zal dus niet overal toegepast worden, enkel waar de balken elkaar kruisen.



Figuur K7-4: Schuifspanningen in balkenrooster, 250mm



Figuur K7-5: Gemiddelde schuifspanning over hoogte balk, 250mm   Gemiddelde schuifspanning over hoogte balk, 400mm

Berekening dwarskrachtwapening

Voor het berekenen van de dwarskracht wapening gebruik ik de volgende formule:

As bgsl = (τd * b * d) / (0.9fs)

Td	= schuifspanning in betondoorsnede	[N/mm^2]
b	= breedte balk	[mm]
y	= nuttige hoogte	[mm]
fs	= 435 N/mm^2 ; FeB500	[N/mm^2]

As bgsl = (4,2 * 400 * 1000) / (0.9 * 435)

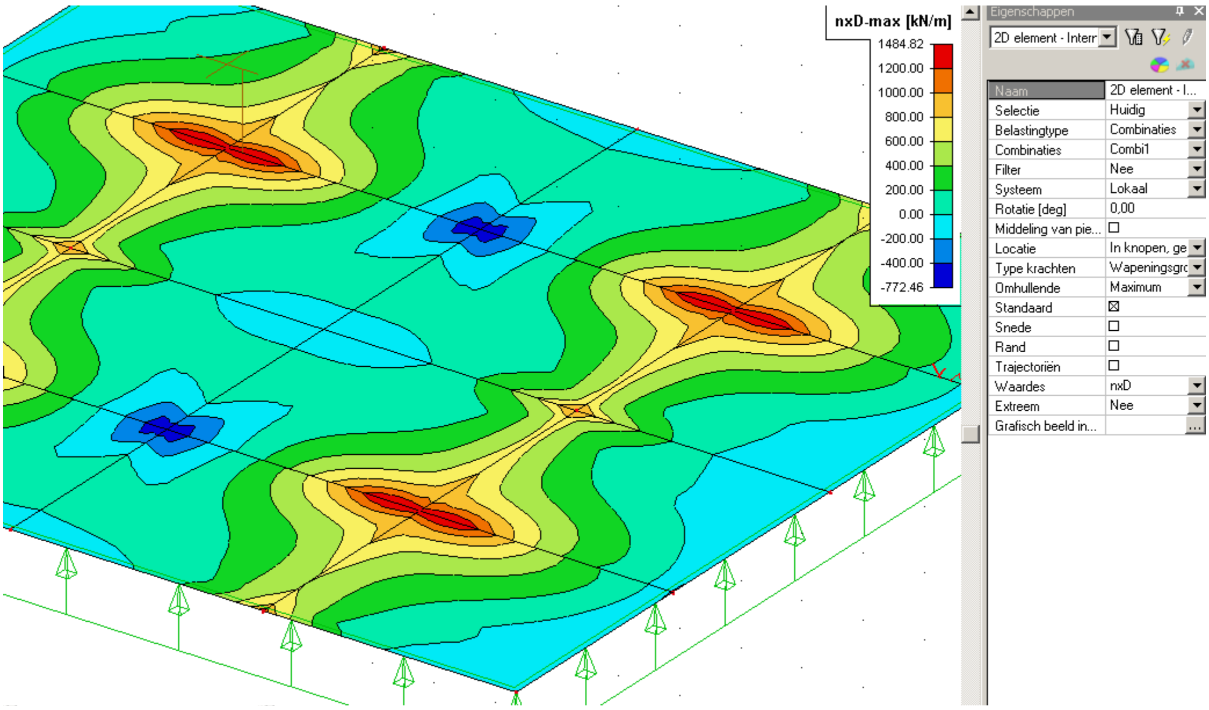
As bgsl = 4.292 mm^2

Bgls Ø 20 – 125 → 5.026mm^2 → OK

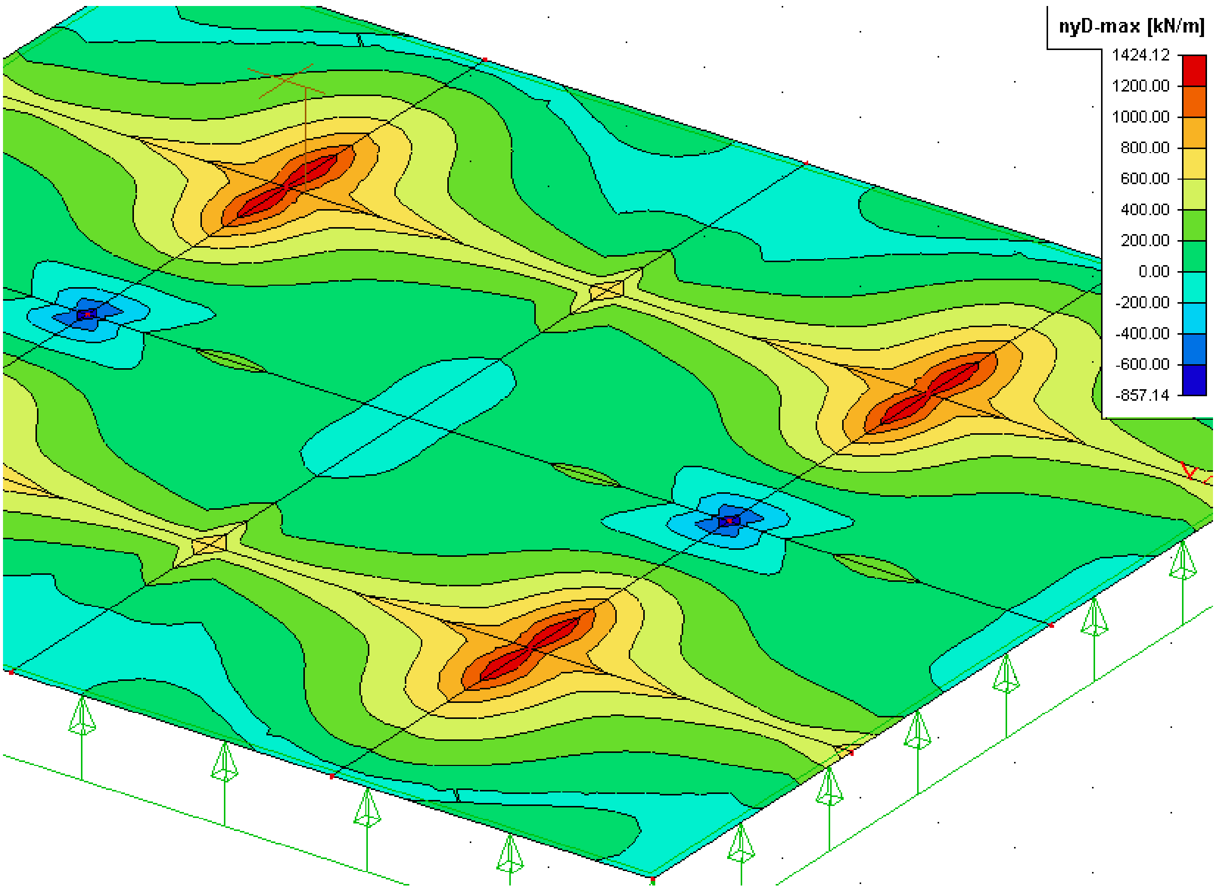
Waar de balken elkaar kruisen worden beugels Ø 20 – 125 toegepast.

<u>Berekening onderste vloer</u>

De vloeren worden gecontroleerd op de scheurwijdte. In de onderstaande figuren zijn de NxD en NxY te zien. Dit is de optredende druk- of trekspanning in de plaat over één strekkende meter. De maximale trekspanning in de onderste vloer is 1.485kN/m. Op deze waarde zal de vloer getoetst worden. Ook hier is te zien, zoals verwacht, dat deze waardes optreden waar de balken elkaar snijden onder de kolommen.



Figuur K7-6: Trek- en drukspanningen in onderste vloer, x-richting



Figuur K7-7: Trek- en drukspanningen in onderste vloer, y-richting

Calculation crack width

Input

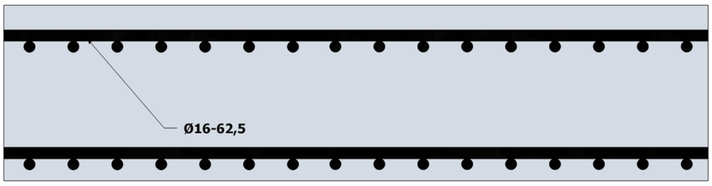
Concrete:	C28/35	
Height:	250	[mm]
f_{cd}:	18,7	[N/mm^2]
f_{ctm}:	2,70	[N/mm^2]
$E_{cm,\ short}$:	32.300	[N/mm^2]
$E_{cm,\ long}$:	17.000	[N/mm^2]
Reinforcement steel:	FeB 500	
Concrete cover:	35	[mm]
f_{yd}:	435	[N/mm^2]
$E_{s:}$	200.000	[N/mm^2]
Ratio E_s/E_{cm}:	11,8	[-]

w_{max} = 0,3mm

Reinforcement

Reinforcement, first guess: Ø 20 – 62,5, two rows

d_x = 250 – 35 – 20 – 8 = 187 mm



Reinforcement ratio ρ_1

$A_s = 10^2 * \pi * 16 * 2 = 10.053 mm^2/m^1$

ρ_1 = 10.053 / (1000 * 187) = 0,052 [-]

Note: there is no compression zone, the entire height of the floor is under tension.

Steel stress in SLS

$$\sigma_s = \frac{NxD}{A_{s,prov}}$$

$$\sigma_s = \frac{1.485 * 10^3}{10.053}$$

$\sigma_s = 147,5 N/mm^2$

Maximum diameter reinforcement for $\sigma_s = 200 N/mm^2 > 147,5 N/mm^2$ → Ø 20 → OK, EC 1992-1-1

Calculation of the height of the tensile member

The height of the tensile member in a beam loaded in bending:

$h_{c,eff} < (h-x)/3$

$h_{c,eff}$ = 2,5 (h-d) = 2,5 (250 – 187) = 157,5mm

Reinforcement ratio of the tensile member

$\rho_{p,eff} = A_s / (h_{eff} * b)$

$\rho_{p,eff}$ = 10.053 / (157,5 * 1000) = 0,064 [-]

The cracking force of the tensile member

$N_{cr} = A_{c,eff} * f_{ctm} (1 + \alpha_e * \rho_{p,eff})$

N_{cr} = 157,5 * 1000 * 2,7 (1 + 11,8 * 0,064)

$N_{cr} = 746 * 10^3$ N

$\sigma_{sr} = N_{cr} / A_s = 74,0 N/mm^2$

$\sigma_{sr} < \sigma_s$ → Established cracking stage

Calculation of the crack width

General expression for maximum crack width in a tensile member:

$$w_{max} = \frac{1}{2} * \frac{f_{ctm}}{\tau_{bm}} * \frac{Ø}{\rho} * \frac{1}{E_s} (\sigma_s - \alpha_e * \sigma_{sr} + \beta * E_s)$$

Long term loading & stabilised cracking stage:

α = 0,3

β = 1,0

τ_{bm} = 2,0 * f_{ctm}

$$w_{max} = \frac{1}{2} * \frac{1}{2} * \frac{20}{0{,}064} * \frac{1}{200.000} (147{,}5 - 0{,}3 * 74{,}0)$$

w_{max} = 0,049mm → OK, when only considering the crack width, the reinforcement could be less. But the stresses in the steel would become too high.

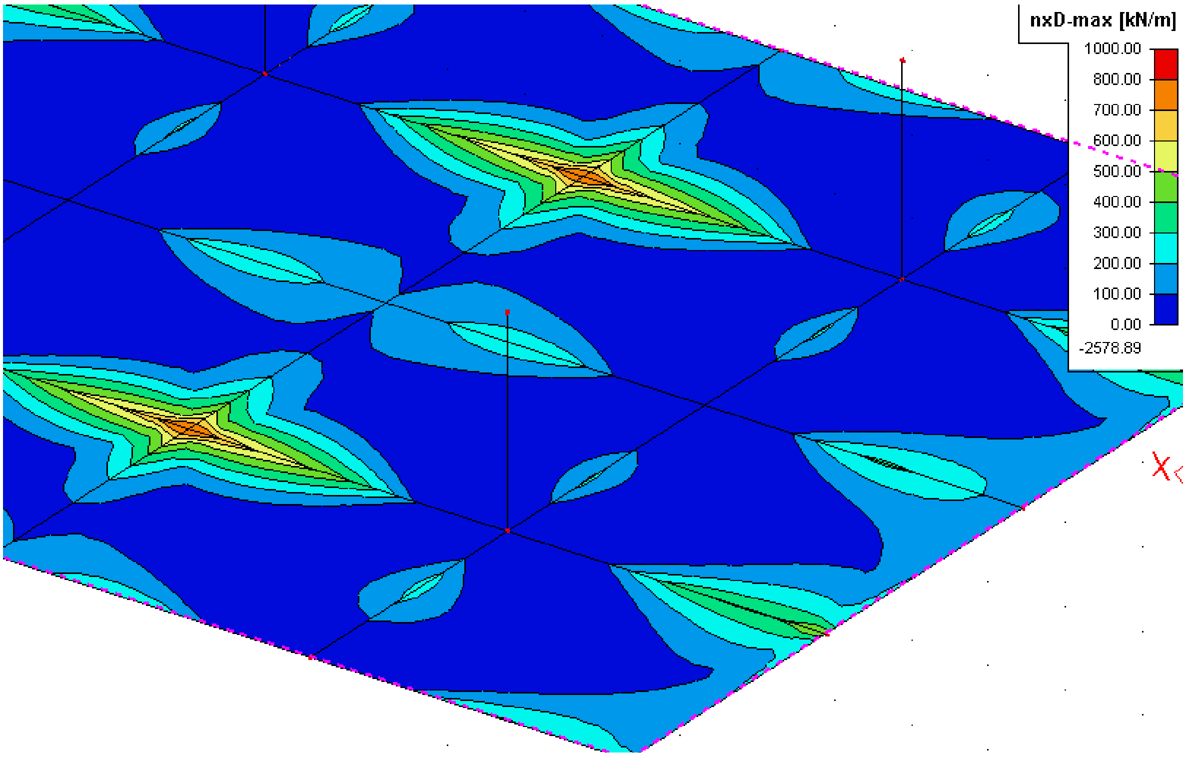
Steel stress [MPa]	Maximum bar size [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Maximum bar size reinforcement in relation with steel stress. [EC1991-1-1]

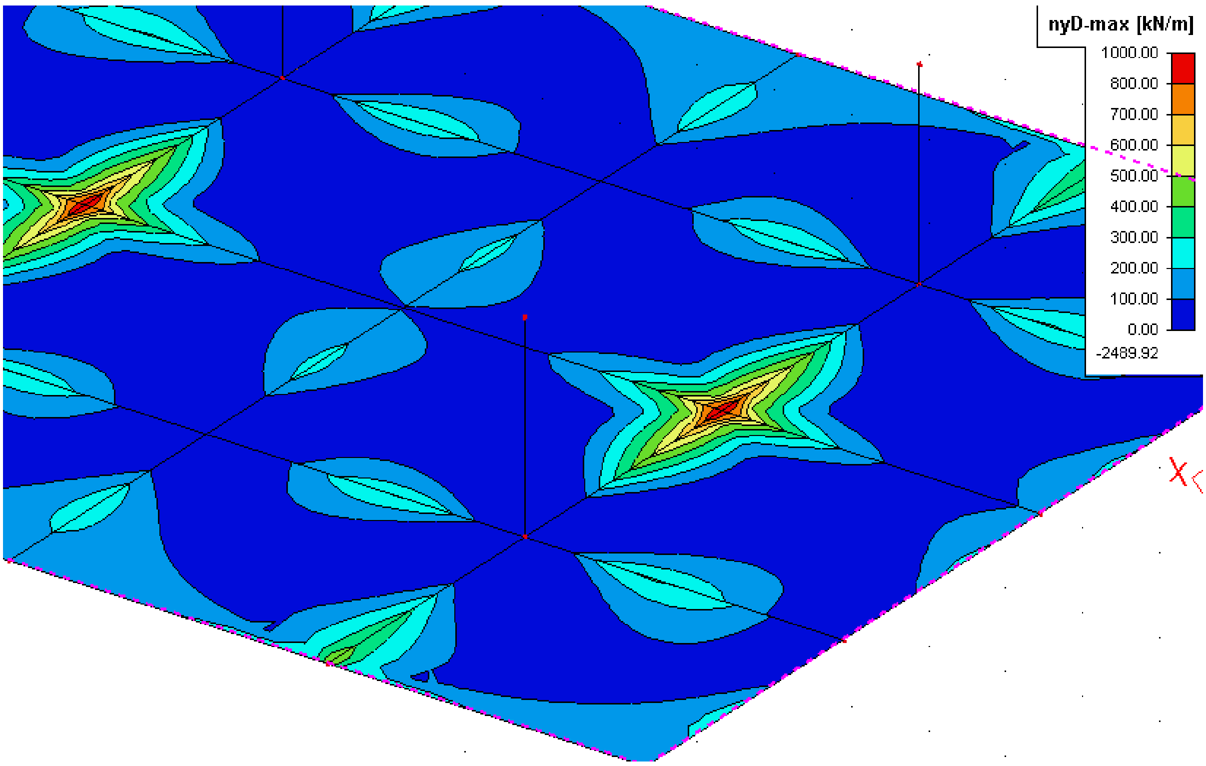
Verification of the bar spacing according to EC1991-1-1 cl. 8.2
When using a concrete mixture having a nominal aggregate particle size of 31,5 mm (C28/35), the clear distance between bars should be at least 31,5 + 5 = 37 mm.

s = (1000 – (16 * 20)) / 16 = 42,5mm > 37 → OK

Berekening bovenste vloer
In de onderstaande figuren zijn de NxD en NyD te zien. Dit is de optredende druk- of trekspanning in de plaat over één strekkende meter. De maximale trekspanning in de bovenste vloer is 1.000kN/m. Op deze waarde zal de vloer getoetst worden. Ook hier is te zien, zoals verwacht, dat deze waardes optreden waar de balken elkaar snijden tussen de kolommen in.



Figuur K7-8: Trek- en drukspanningen in bovenste vloer, x-richting



Figuur K7-9: Trek- en drukspanningen in onderste vloer, y-richting

Calculation crack width

Input

Concrete:	C28/35	
Height:	250	[mm]
f_{cd}:	18,7	[N/mm^2]
f_{ctm}:	2,70	[N/mm^2]
$E_{cm, short}$:	32.300	[N/mm^2]
$E_{cm, long}$:	17.000	[N/mm^2]
Reinforcement steel:	FeB 500	
Concrete cover:	35	[mm]
f_{yd}:	435	[N/mm^2]
$E_{s:}$	200.000	[N/mm^2]
Ratio E_s/E_{cm}:	11,8	[-]

w_{max} = 0,1mm

Reinforcement

Reinforcement, first guess: Ø 16 – 83,3 , two rows

d_x = 250 – 35 – 16 – 8 = 191mm

Reinforcement ratio ρ_1

A_s = 8 * 8 * π * 12 * 2 = 4.825mm^2/m^1

ρ_1 = 4.825 / (1000 * 191) = 0,025 [-]

Note: there is no compression zone, the entire height of the floor is under tension.

Steel stress in SLS

$$\sigma_s = \frac{NxD}{A_{s,prov}}$$

$$\sigma_s = \frac{1.000 * 10^3}{4.825}$$

σ_s = 207,2N/mm^2

Maximum diameter reinforcement for σ_s= 240N/mm^2 > 207,2N/mm^2 → Ø 16 → OK, EC 1992-1-1

Calculation of the height of the “hidden” tensile member

The height of the tensile member in a beam loaded in bending:

$h_{c,eff} < (h-x)/3$

$h_{c,eff} = 2{,}5\ (h-d) = 2{,}5\ (250 - 191) = 147{,}5mm$

$\rho_{p,eff} = A_s / (h_{eff} * b)$

$\rho_{p,eff} = 4.825/ (147{,}5 * 1000) \quad = 0{,}033$ [-]

The cracking force of the "hidden" tensile member

$N_{cr} = A_{c,eff} * f_{ctm}\ (1 + \alpha_e * \rho_{p,eff})$

$N_{cr} = 147{,}5 * 1000 * 2{,}7\ (1 + 11{,}8 * 0{,}033)$

$N_{cr} = 553{,}3 * 10^3$ N

$\sigma_{sr} = N_{cr} / A_s = 114{,}7 N/mm^2$

$\sigma_{sr} < \sigma_s \rightarrow$ Established cracking stage

Calculation of the crack width

General expression for maximum crack width in a tensile member:

$$w_{max} = \frac{1}{2} * \frac{f_{ctm}}{\tau_{bm}} * \frac{Ø}{\rho} * \frac{1}{E_s}\ (\sigma_s - \alpha_e * \sigma_{sr} + \beta * E_s)$$

Long term loading & stabilised cracking stage:

$\alpha = 0{,}3$

$\beta = 1{,}0$

$\tau_{bm} = 2{,}0 * f_{ctm}$

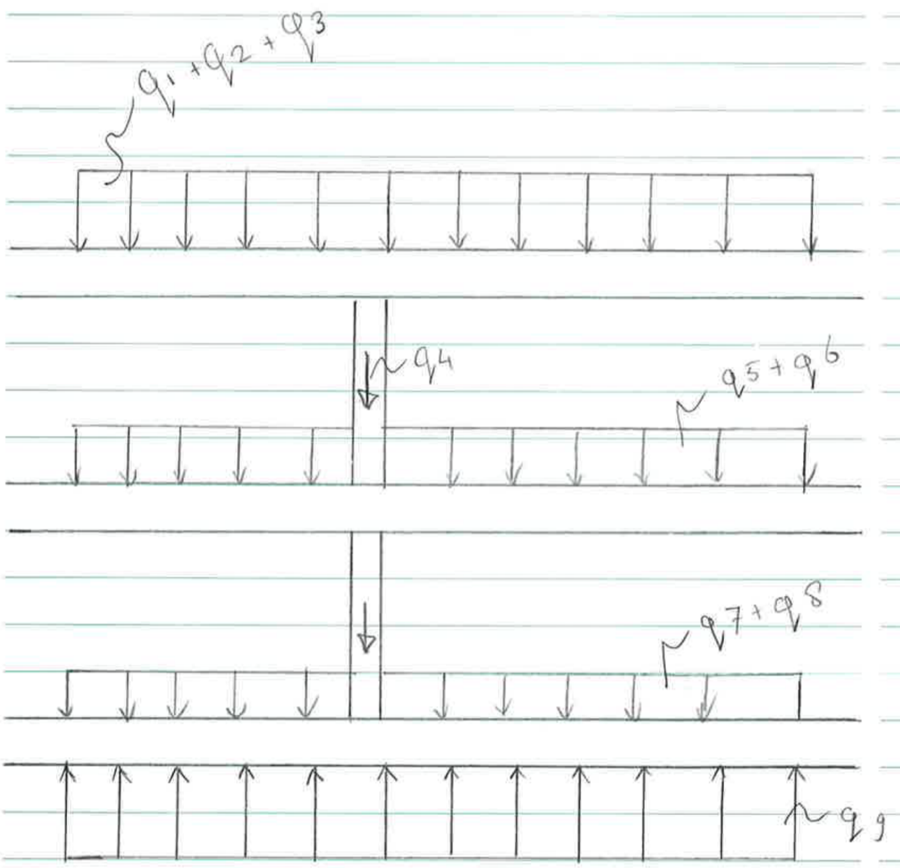
$$w_{max} = \frac{1}{2} * \frac{1}{2} * \frac{16}{0{,}033} * \frac{1}{200.000}\ (207{,}2 - 0{,}3 * 114{,}7)$$

$w_{max} = 0{,}104mm \approx 0{,}1 \rightarrow$ OK

L1 Structural calculations: Columns

The dimensions of the columns are designed for the immersion stage. During immersion the garage element is filled with water ballast tanks and the loads on the floor and columns are larger than in the other stages. The loads acting on the element are shown below.

Calculation: colums, operation stage



q_1 = hydrostatic water pressure [kN/m²]
q_2 = ballast concrete roof [kN/m²]
q_3 = weight roof [kN/m²]
q_4 = weight columns [kN/m']
q_5 = ballast water immersion [kN/m²]
q_6 = weight floor [kN/m²]
q_7 = ballast water immersion [kN/m²]
q_8 = weight flexbase floor [kN/m²]
q_9 = hydrostatic water pressure [kN/m²]

$q_{1+2+3+4+5+6+7+8} = q_9$

The loads and resulting normal force in the columns are shown below. The normal force in the columns at the upper floor is 5.261 kN and at the bottom floor 7.613 kN. This is the normal force at the bottom of the column. The normal force at the top can be found by subtracting the own weight of the column.

Loads

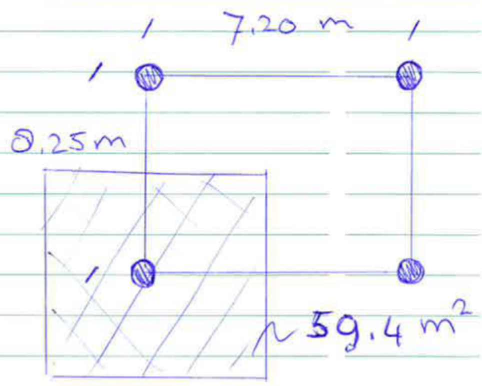
q_1	=	4,10 * 10 * 1,20	= 49,20 kN/m^2
q_2	=	0,6 * 25 * 1,20	= 18,00 kN/m^2
q_3	=	0,7 * 25 * 1,20	= 21,00 kN/m^2
q_4	=	0,3^2 * π * 25 * 1,20	= 8,48 kN/m^1
q_5	=	18 * 1,20	= 21,60 kN/m^2
q_6	=	0,6 * 25 * 1,20	= 18,00 kN/m^2
q_7	=	9 * 1,20	= 10,80 kN/m^2
q_8	=	22,5 * 1,20 (appendix B)	= 49,20 kN/m^2
q_9	=	13,80 * 10 * 1,20	= 165,60 kN/m^2

N'd column 1 =

$(q_1+q_2+q_3) \times 59{,}4$

$+ q_4 \times 2{,}60 =$

$\underline{5.261 \text{ kN}}$



N'd column 2 = $5.261 + (q_5 + q_6) \times 59.4 =$ $\underline{7.613 \text{ kN}}$

The normal force is also checked with SCIA Engineer. A small scale model of is made with columns of 2,60 meters high and a c.t.c. distance of 7,20 meters. The model is shown below.

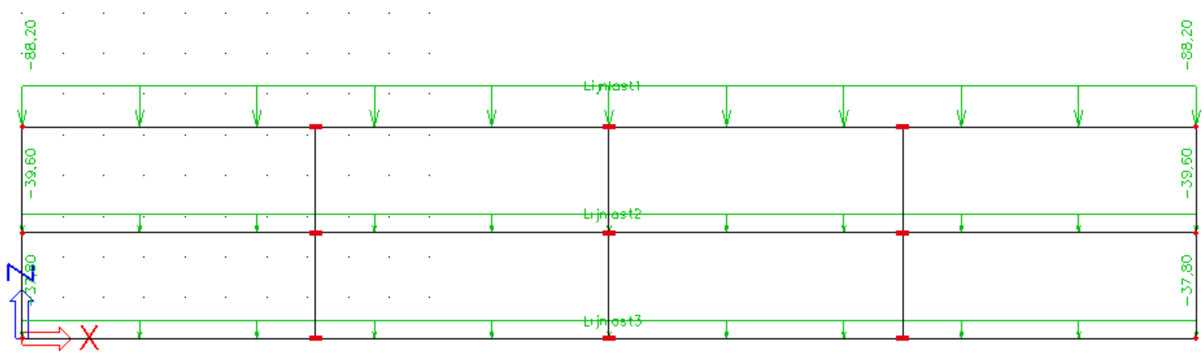


Figure L1-1: Downward forces in model

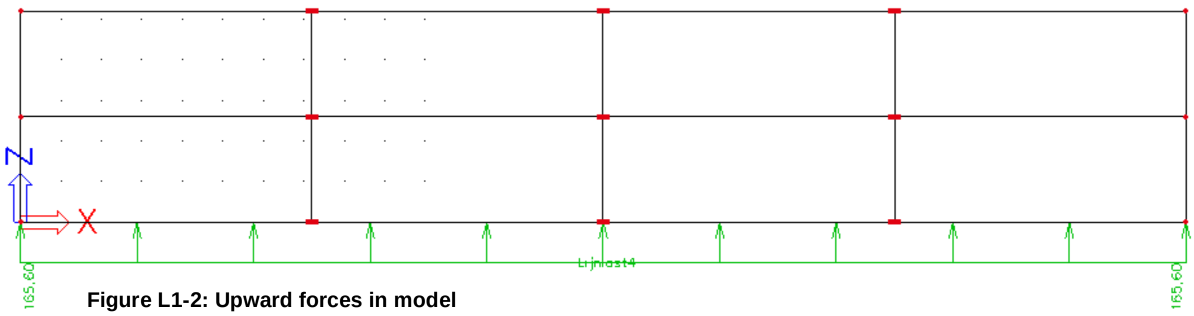


Figure L1-2: Upward forces in model

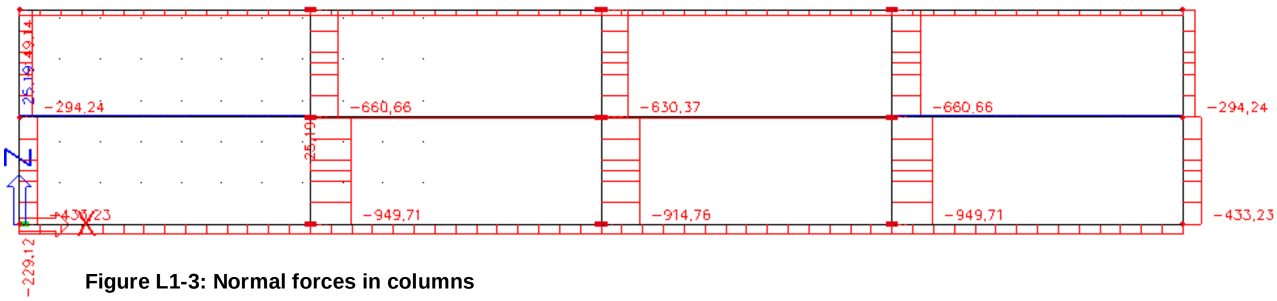


Figure L1-3: Normal forces in columns

The output in SCIA shows the same results. The c.t.c. distance of the columns in the figure is 7,20 meters. In the c.t.c. distance in the perpendicular direction is 8,25 meters. Therefore, the output has to be multiplied by 8,25. This gives for the upper columns:

N'd = 630 * 8,25 = 5.197 kN ≈ 5.261 → OK

And for the columns at the bottom floor:

N'd = 914 * 8,25 = 7.540 kN ≈ 7.613 kN → OK

The moments at the top of the columns are close to zero and in the column in the middle completely zero. Therefore, I can assume that the columns are centrally loaded. The formula for the maximum load on the column is:

$$N_u = A_c * f_{cd} + A_s * f_{yd}$$

In which:

A_c	=	Cross-section concrete	[mm^2]
A_s	=	Cross-section reinforcement steel	[mm^2]
f_{cd}	=	Pressure strength concrete	[N/mm^2]
f_{yd}	=	Pressure strength reinforcement steel	[N/mm^2]
A_c	=	300 * 300 * π = 282.743	[mm^2]
A_s	=	1% * A_c = 2.827	[mm^2]
f_{cd}	=	21 (C28/35)	[N/mm^2]
f_{yd}	=	425 (FeB500)	[N/mm^2]

$$N_u = 282.743 * 21 + 2.827 * 425$$

$$N_u = 7.139 \text{ kN}$$

Upper columns: Unity check = 5.197 / 7.139 = 0,73 < 1,00 → OK

Bottom columns: Unity check = 7.540 / 7.139 = 1,06 > 1,00 → Not OK

Columns at the bottom floor will need a larger cross-section. Next try → D = 700 mm.

A_c	=	350 * 350 * π = 384.845	[mm^2]
A_s	=	1% * A_c = 3.848	[mm^2]
f_{cd}	=	21 (C28/35)	[N/mm^2]
f_{yd}	=	425 (FeB500)	[N/mm^2]

$$N_u = 384.845 * 21 + 3.848 * 425$$

$$N_u = 9.717 \text{ kN}$$

Bottom columns: Unity check = 7.540 / 9.717 = 0,78 < 1,00 → OK

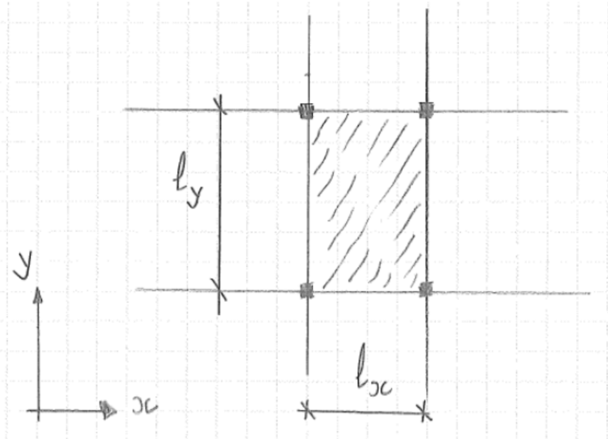
Required dimensions

Bottom columns: **D = 700mm**

Upper columns: **D = 600mm**

L2 Structural calculations: Roof slab

Concrete roof slab:	7,20 x 8,25
Concrete:	C28/35
Height:	700mm
Columns:	Ø 600mm
Reinforcement steel:	FeB 500
Concrete cover:	35mm

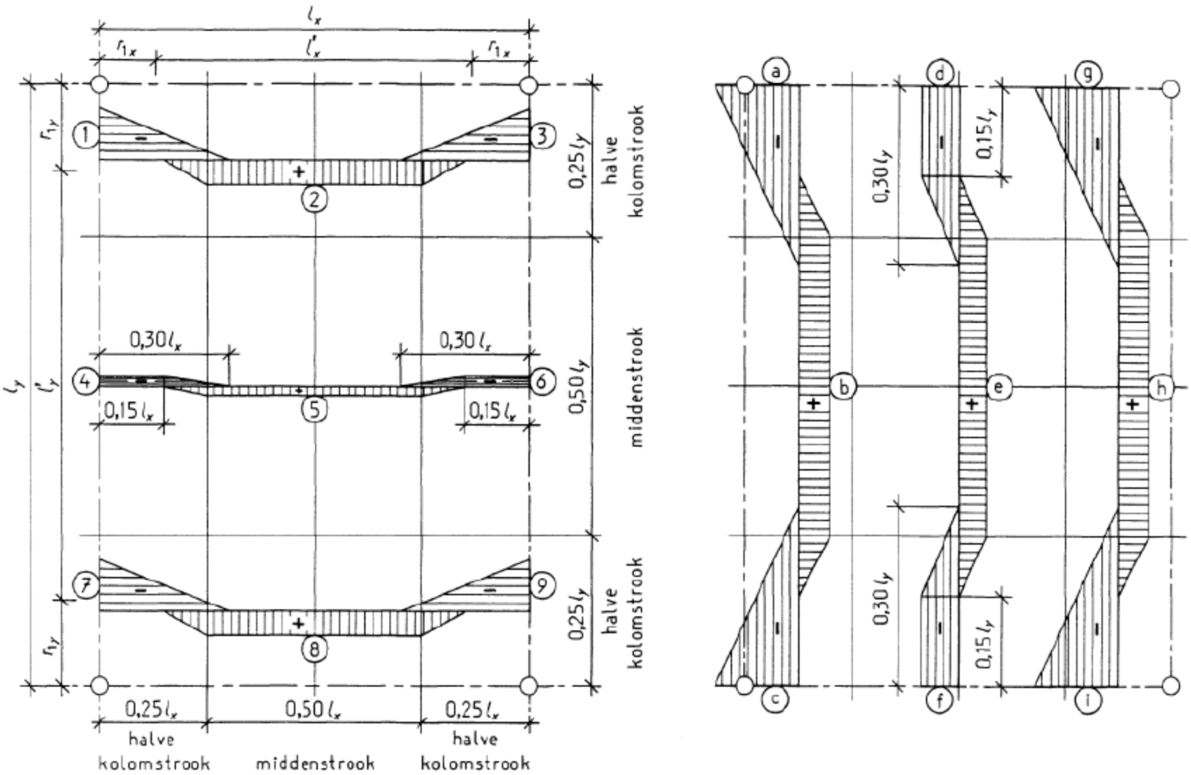


l_x = 8,25 / 7,20 = 1,15

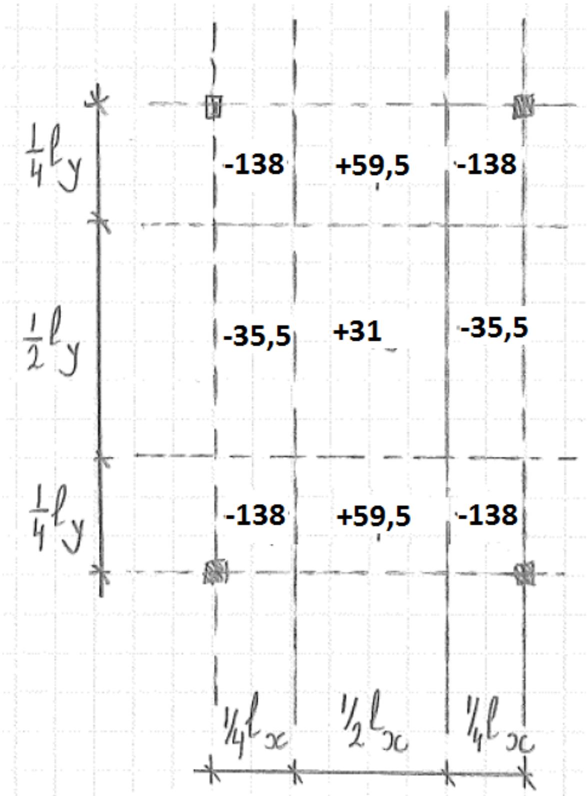
Reinforcement coefficients are interpolated using NEN 6720 table 19.

Momentencoëfficiënten

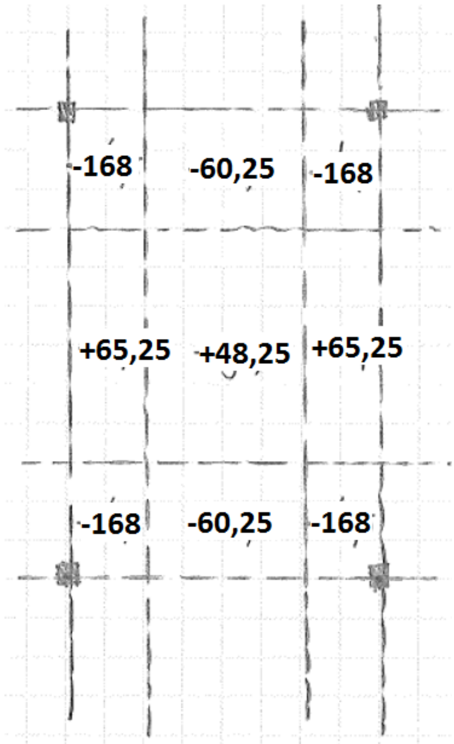
l_y/l_x	$m^*_{xx} = 0{,}001\ p_d\ l_x^2 \times$									$m^*_{yy} = 0{,}001\ p_d\ l_x^2 \times$								
	1	2	3	4	5	6	7	8	9	a	b	c	d	e	f	g	h	i
1,0	−132	+ 54	−132	− 40	+ 34	− 40	−132	+ 54	−132	−132	+ 54	−132	− 40	+ 34	− 40	−132	+ 54	−132
1,2	−140	+ 61	−140	− 34	+ 30	− 34	−140	+ 61	−140	−180	+ 69	−180	− 67	+ 53	− 67	−180	+ 69	−180
1,4	−146	+ 67	−146	− 28	+ 25	− 28	−146	+ 67	−146	−235	+ 89	−235	−102	+ 77	−102	−235	+ 89	−235
1,6	−151	+ 73	−151	− 22	+ 21	− 22	−151	+ 73	−151	−296	+112	−296	−145	+104	−145	−296	+112	−296
1,8	−155	+ 78	−155	− 18	+ 17	− 18	−155	+ 78	−155	−363	+140	−363	−195	+134	−195	−363	+140	−363
2,0	−159	+ 81	−159	− 14	+ 14	− 14	−159	+ 81	−159	−437	+171	−437	−251	+167	−251	−437	+171	−437



Reinforcement coefficients x-direction



Reinforcement coefficients y-direction



Reinforcement, first guess: Ø 25 – 200

$d_y = 700 – 35 – 25/2 \quad = 652{,}5mm$

$d_x = 700 – 35 -25 -25/2 \quad = 627{,}5mm$

$d_{eff} = (d_y + d_x)/2 \quad = 640mm$

Loads

Self-weight: $1{,}2 * 25 * 0{,}7 = 21\ kN/m^2$

Extra safety (sediment built-up etc.): $1{,}2 * 15 = 18{,}0\ kN/m^2$

Hydr. Water pressure: $1{,}2 * 10 * 4{,}1 = 49{,}2\ kN/m^2$

Total load: $88{,}20\ kN/m^2$

Reinforcement moment x-direction (M_{ed}) = coefficient * 0,001 * q_d * l_x^2

Reinforcement moment y-direction (M_{ed}) = coefficient * 0,001 * q_d * l_y^2

x-direction

Coefficient [-]	M_{ed} [kNm/m]	$M_{ed}/(b*d^2)$ kN/m^2	ρ_l [-]	A_s [mm^2]	Reinforcement
-138	-631	1602	0,0037	2403	*
+59,5	272	691	0,0016	1037	Ø16-125
-138	-631	1602	0,0037	2403	*
-35,5	-162	411	0,0015	955	Ø16-125
+31	141,7	361	< ρ_l,min	<$A_{s,min}$	Ø16-125
-35,5	-162	411	0,0015	955	Ø16-125
-138	-631	1602	0,0037	2403	*
+59,5	272	691	0,0016	1037	Ø16-125
-138	-631	1602	0,0037	2403	*

$A_s = ((M_{ed}/b*d^2)/f_{cd})*d_x$

f_{cd} = 435n/mm^2 (f_{yk}/1,15)

Minimum reinforcement required:

$A_{s,min} = 0,26 * f_{ctm} * b_t * d / f_{yk} = 0,26 * 2,1 * 1000 * 652,5 / 500$ = 712mm^2/m

$A_{s,min} > 0,0013 * b_t * d = 0,0013 * 652,5 * 1000$ = 848mm^2/m

Ø16-125 = 1608mm^2 → OK

*** Total reinforcement in column strip:**
- 40% concentrated in strip
- 60% in column strip

Width concentrated strip = s

$s = b_2 + 1,5 * b_1 + 1,5 * h$

$b_1 = b_2$ = diameter column

s = 600 + 1,5 * 600 + 1,5 * 700 = 2550mm

s < 0,7 * width column strip → concentrated strip has to be taken into account

Total reinforcement moment in concentrated strip:

0,4 * (-2603) / 2,55 + 0,6 * (-2603) / 4,125 = -787kNm/m

In other parts of the column strip:

0,6 * (-2603) / 4,125 = -379 kNm/m

Reinforcement needed in concentrated strip:

A_s = 2772mm^2 → **Ø25-125 = 3926mm^2 → OK**

y-direction

Coefficient [-]	M_{ed} [kNm/m]	$M_{ed}/(b*d^2)$ kN/m^2	A_s [mm^2]	Reinforcement
-168	-1008	2560	3693	*
+65,25	391	993	1432	Ø16-125
-168	-1008	2560	3693	*
--60,25	-362	919	1325	Ø16-125
+48,25	290	736	1062	Ø16-125
-60,25	-362	919	1325	Ø16-125
-168	-1008	2560	3693	*
+65,25	391	993	1432	Ø16-125
-168	-1008	2560	3693	*

$A_s = ((M_{ed}/b*d^2)/f_{cd})*d_y$

f_{cd} = 435n/mm^2 (f_{yk}/1,15)

Minimum reinforcement required:

$A_{s,min} = 0{,}26 * f_{ctm} * b_t * d / f_{yk} = 0{,}26 * 2{,}1 * 1000 * 627{,}5 / 500$ = 685mm^2/m

$A_{s,min} > 0{,}0013 * b_t * d = 0{,}0013 * 627{,}5 * 1000$ = 815mm^2/m

Ø16-125 = 1608mm^2 → OK

*** Total reinforcement in column strip:**

- 40% concentrated in strip
- 60% in column strip

Width concentrated strip = s

$s = b_2 + 1{,}5 * b_1 + 1{,}5 * h$

$b_1 = b_2$ = diameter column

s = 600 + 1,5 * 600 + 1,5 * 700 = 2550mm

s > 0,7 * width column strip → concentrated strip doesn't have to be taken into account

Reinforcement needed in column strip:

A_s = 3693mm^2 → **Ø25-125 = 3926mm^2 → OK**

Crack width control

Exactly above the columns the moments are extreme. Therefore, this is the place where I do a crack width control. For the crack width control I use the SLS instead of the ULS. The picture below shows the M-line. The moment above the columns is 840kNm.

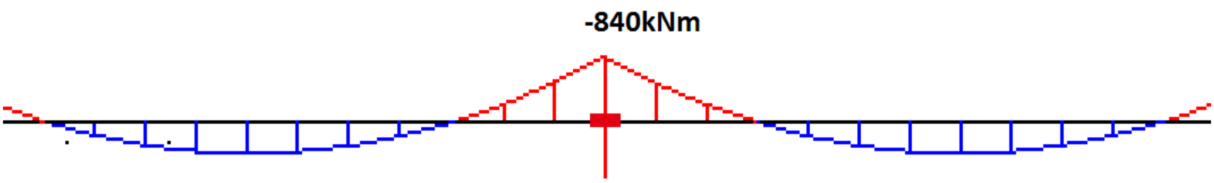
M_{ed} = coefficient * 0,001 * q_d * l_y^2

Loads

Self-weight:	25 * 0,7 =	17,5 kN/m^2
Extra safety (sediment built-up etc.):		15,0 kN/m^2
Hydr. Water pressure:	10 * 4,1 =	41,0 kN/m^2

Total load: 73,5 kN/m^2

M_{ed} = -168 * 0,001 * 73,5 * 8,25^2 = 840kNm



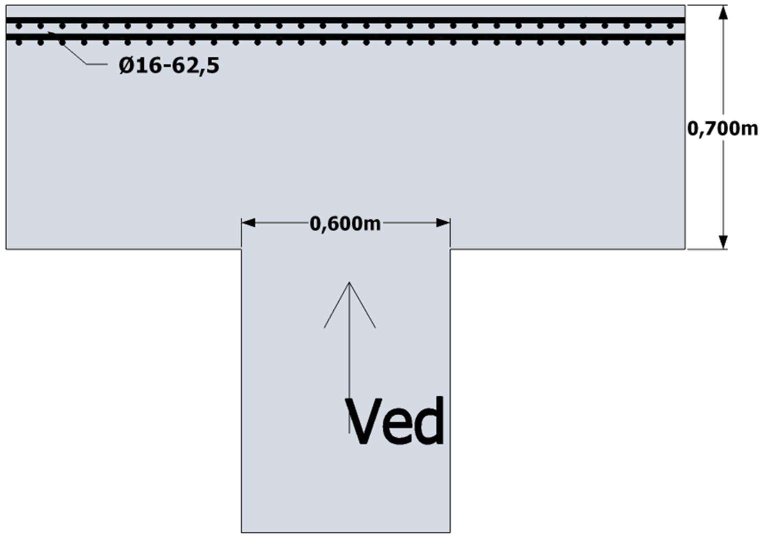
Moment in roof above columns in y-direction

Input

Concrete roof slab:	7,20 x 8,25	[m]
Concrete:	C28/35	
Height:	700	[mm]
Columns:	Ø 600	[mm]
f_{cd}:	18,7	[N/mm^2]
f_{ctm}:	2,70	[N/mm^2]
$E_{cm, short}$:	32.300	[N/mm^2]
$E_{cm, long}$:	17.000	[N/mm^2]
Reinforcement steel:	FeB 500	
Concrete cover:	35	[mm]
f_{yd}:	435	[N/mm^2]
$E_{s:}$	200.000	[N/mm^2]
Ratio E_s/E_{cm}:	11,8	[-]

w_{max} = 0,1mm

Reinforcement



Reinforcement, first guess: Ø 16 – 62,5, two rows

d_y = 700 – 35 – 16 – 16 - 20 = 613mm

Reinforcement ratio ρ_1

A_s = 8 * 8 * π * 16 * 2 = 6434mm²/m¹

ρ_1 = 6434 / (1000 * 631) = 0,0105 [-]

Calculation of the height of the compressive zone in the SLS (assuming linear elastic behaviour of concrete loaded in compression; concrete is assumed to have no tensile strength):

$$\frac{x}{d} = -\alpha_e * \rho 1 + \sqrt{(\alpha_e * \rho 1)^2 + 2 * \alpha_e * \rho 1}$$

$$\frac{x}{613} = -11{,}8 * 0{,}0105 + \sqrt{(11{,}8 * 0{,}0105)^2 + 2 * 11{,}8 * 0{,}0105}$$

x = 238,5 mm

Steel stress in SLS

$$\sigma_s = \frac{M_{eqp}}{A_{s,prov}\,(d - \frac{1}{3X})}$$

$$\sigma_s = \frac{840 * 10^6}{6434 * \ (613 - \frac{1}{3} * 238{,}5)}$$

σ_s = 244,7N/mm² (maximum diameter reinforcement for σ_s= 240N/mm² ≈ 244N/mm² → Ø 16 → OK, EC 1992-1-1)

Calculation of the height of the "hidden" tensile member

The height of the tensile member in a beam loaded in bending:

$h_{c,eff}$ < (h-x)/3

$h_{c,eff}$ = 2,5 (h-d) = 2,5 (700 – 613) = 217,5mm

(h-x)/3 = (700 – 238,5)/3 = 153,8mm → Use 153,8mm

Reinforcement ratio of the "hidden" tensile member

$\rho_{p,eff} = A_s / (h_{eff} * b)$

$\rho_{p,eff}$ = 6435/ (153,8 * 1000) = 0,0418 [-]

The cracking force of the "hidden" tensile member

$N_{cr} = A_{c,eff} * f_{ctm} (1 + \alpha_e * \rho_{p,eff})$

$N_{cr} = 153,8 * 1000 * 2,7 (1 + 11,8 * 0,0418)$
$N_{cr} = 620 * 10^3$ N

$\sigma_{sr} = N_{cr} / A_s = 96,4 N/mm^2$

$\sigma_{sr} < \sigma_s$ → Established cracking stage

Calculation of the crack width

General expression for maximum crack width in a tensile member:

$$w_{max} = \frac{1}{2} * \frac{f_{ctm}}{\tau_{bm}} * \frac{Ø}{\rho} * \frac{1}{E_s} (\sigma_s - \alpha_e * \sigma_{sr} + \beta * E_s)$$

Long term loading & stabilised cracking stage:

$\alpha = 0,3$

$\beta = 1,0$

$\tau_{bm} = 2,0 * f_{ctm}$

$$w_{max} = \frac{1}{2} * \frac{1}{2} * \frac{16}{0,0418} * \frac{1}{200.000} (244,7 - 0,3 * 96,4)$$

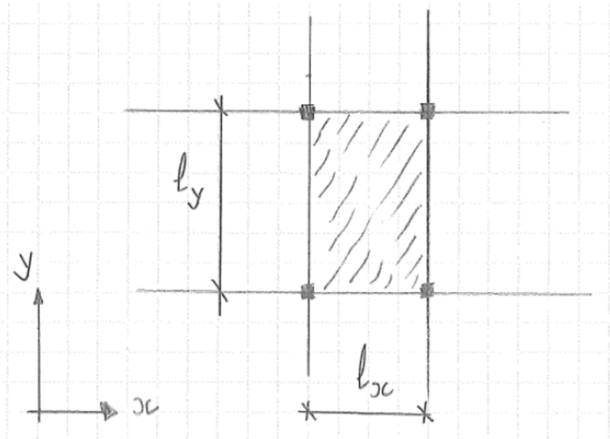
w_{max} = 0,103mm → OK

Verification of the bar spacing according to EC1991-1-1 cl. 8.2
When using a concrete mixture having a nominal aggregate particle size of 31,5 mm (C28/35), the clear distance between bars should be at least 31,5 + 5 = 37 mm.

s = (1000 – (16 * 16)) / 16 = 46,5mm > 37 → OK

L3 Structural calculations: Intermediate floor slab

Concrete floor slab:	7,20 x 8,25
Concrete:	C28/35
Height:	600mm
Columns:	Ø 700mm
Reinforcement steel:	FeB 500
Concrete cover:	25mm

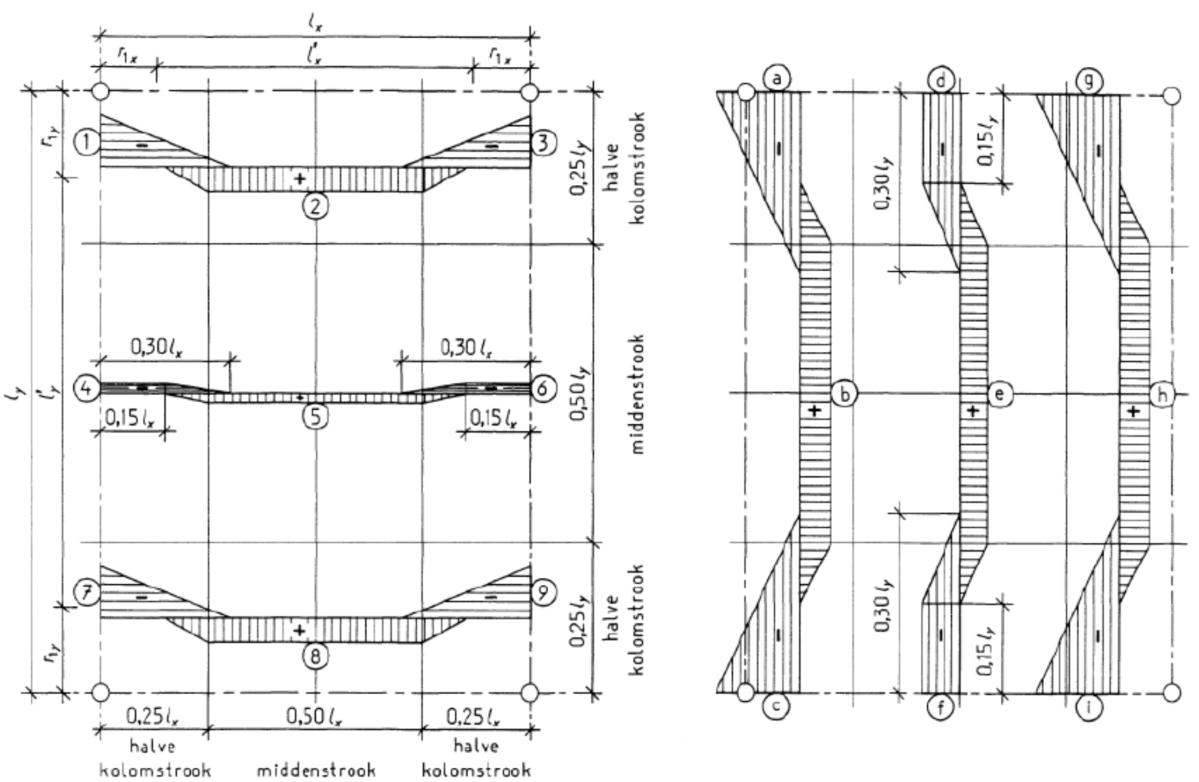


l_x = 8,25 / 7,20 = 1,15

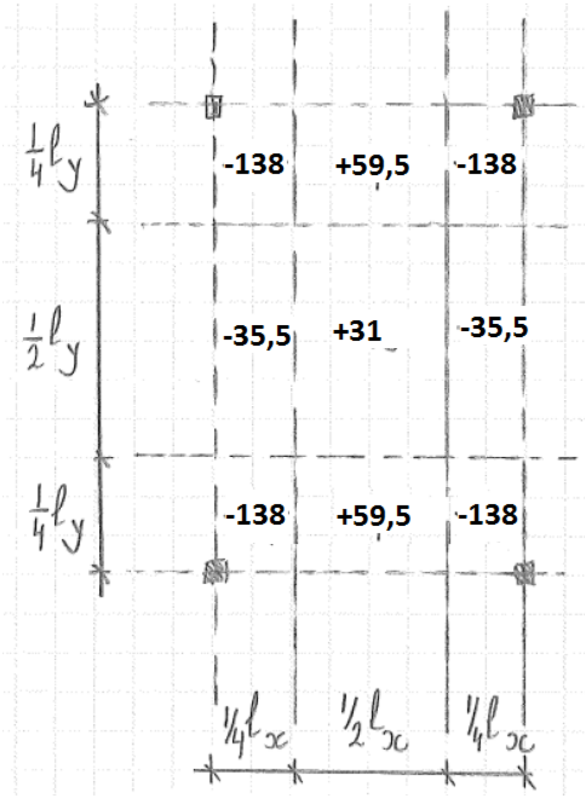
Reinforcement coefficients are interpolated using NEN 6720 table 19.

Momentencoëfficiënten

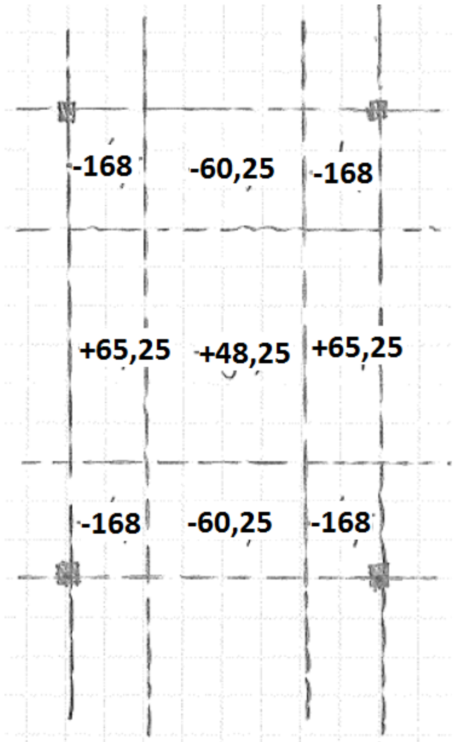
l_y/l_x	$m^*_{xx} = 0,001\ p_d\ l_x^2\ \times$ 1	2	3	4	5	6	7	8	9	$m^*_{yy} = 0,001\ p_d\ l_x^2\ \times$ a	b	c	d	e	f	g	h	i
1,0	−132	+ 54	−132	− 40	+ 34	− 40	−132	+ 54	−132	−132	+ 54	−132	− 40	+ 34	− 40	−132	+ 54	−132
1,2	−140	+ 61	−140	− 34	+ 30	− 34	−140	+ 61	−140	−180	+ 69	−180	− 67	+ 53	− 67	−180	+ 69	−180
1,4	−146	+ 67	−146	− 28	+ 25	− 28	−146	+ 67	−146	−235	+ 89	−235	−102	+ 77	−102	−235	+ 89	−235
1,6	−151	+ 73	−151	− 22	+ 21	− 22	−151	+ 73	−151	−296	+112	−296	−145	+104	−145	−296	+112	−296
1,8	−155	+ 78	−155	− 18	+ 17	− 18	−155	+ 78	−155	−363	+140	−363	−195	+134	−195	−363	+140	−363
2,0	−159	+ 81	−159	− 14	+ 14	− 14	−159	+ 81	−159	−437	+171	−437	−251	+167	−251	−437	+171	−437



Reinforcement coefficients x-direction



Reinforcement coefficients y-direction



Reinforcement, first guess: Ø 20 – 200

d_y = 600 – 25 – 20/2 = 565mm

d_x = 600 – 25 -20 - 20/2 = 545mm

d_{eff} = $(d_y + d_x)/2$ = 555mm

Loads

Self-weight:	1,2 * 25 * 0,6	=	18 kN/m^2
Ballast water*:	1,2 * 18	=	21,6 kN/m^2

Total load: 39,60 kN/m^2

* The immersion stage is the most unfavourable situation. The ballast tanks that are placed in this stage induce a load of 18kN/m^2, which is much higher than the load that the cars induce during the operation stage (1,5 – 2,5 kN/m^2).

Reinforcement moment x-direction (M_{ed}) = coefficient * 0,001 * q_d * l_x^2

Reinforcement moment y-direction (M_{ed}) = coefficient * 0,001 * q_d * l_y^2

x-direction

Coefficient [-]	M_{ed} [kNm/m]	$M_{ed}/(b*d^2)$ kN/m^2	A_s [mm^2]	Reinforcement
-138	-283	936	1194	*
+59,5	122	396	505	Ø16-250
-138	-283	936	1194	*
-35,5	-73	237	302	Ø16-250
+31	63,6	206	262	Ø16-250
-35,5	-73	237	302	Ø16-250
-138	-283	936	1194	*
+59,5	122	396	505	Ø16-250
-138	-283	936	1194	*

$A_s = ((M_{ed}/b*d^2)/f_{cd})*d_x$

f_{cd} = 435n/mm^2 (f_{yk}/1,15)

Minimum reinforcement required:

$A_{s,min} = 0{,}26 * f_{ctm} * b_t * d / f_{yk} = 0{,}26 * 2{,}1 * 1000 * 545 / 500$ = 606mm^2/m

$A_{s,min} > 0{,}0013 * b_t * d = 0{,}0013 * 545 * 1000$ = 721mm^2/m

Ø16-250 = 804mm^2 → OK

*** Total reinforcement in column strip:**

- 40% concentrated in strip
- 60% in column strip

Width concentrated strip = s

$s = b_2 + 1{,}5 * b_1 + 1{,}5 * h$

$b_1 = b_2$ = diameter column

s = 700 + 1,5 * 700 + 1,5 * 700 = 2800mm

s < 0,7 * width column strip → concentrated strip has to be taken into account

Total reinforcement moment in concentrated strip:

0,4 * (-1167) / 2,80 + 0,6 * (-1167) / 4,125 = -336kNm/m

In other parts of the column strip:

0,6 * (-1167) / 4,125 = -169,75 kNm/m

Reinforcement needed in concentrated strip:

A_s = 1391mm^2 → **Ø20-200 = 1570mm^2 → OK**

y-direction

Coefficient [-]	M_{ed} [kNm/m]	$M_{ed}/(b*d^2)$ kN/m^2	A_s [mm^2]	Reinforcement
-168	-452	1415	1838	*
+65,25	175	548	711	Ø16-125
-168	-452	1415	1838	*
-60,25	-162	516	670	Ø16-125
+48,25	129	404	524	Ø16-125
-60,25	-162	516	670	Ø16-125
-168	-452	1415	1838	*
+65,25	175	548	711	Ø16-125
-168	-452	1415	1838	*

$A_s = ((M_{ed}/b*d^2)/f_{cd})*d_y$

f_{cd} = 435n/mm^2 (f_{yk}/1,15)

Minimum reinforcement required:

$A_{s,min} = 0{,}26 * f_{ctm} * b_t * d / f_{yk} = 0{,}26 * 2{,}1 * 1000 * 565 / 500$ = 616mm^2/m

$A_{s,min} > 0{,}0013 * b_t * d = 0{,}0013 * 565 * 1000$ = 735mm^2/m

Ø16-125 = 1608mm^2 → OK

*** Total reinforcement in column strip:**

- 40% concentrated in strip
- 60% in column strip

Width concentrated strip = s

$s = b_2 + 1{,}5 * b_1 + 1{,}5 * h$

$b_1 = b_2$ = diameter column

s = 700 + 1,5 * 700 + 1,5 * 700 = 2800mm

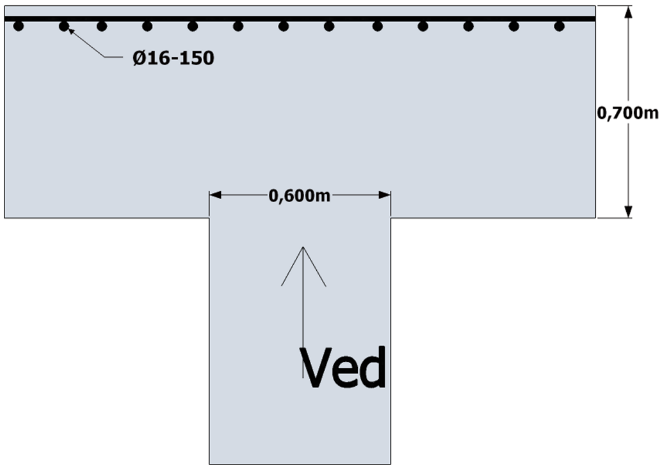
s > 0,7 * width column strip → concentrated strip doesn't have to be taken into account

Reinforcement needed in column strip:

A_s = 1838mm^2 → **Ø20-125 = 2513mm^2 → OK**

L4 Structural calculations: Punching shear reinforcement

Punching shear roof



Concrete: C28/35

Reinforcement: Ø16-150

Cover: 35mm

V_{ed}: 5.261kN

Slab area: 59,4m^2

$d_x = 700 – 35 – 16/2 = 657mm$

$d_y = 700 – 35 – 16 – 16/2 = 641mm$

$d_{eff} = (657 + 641)/2 = 649mm$

Reinforcement ratio ρ_{lx}

Ø16-150 = 1340mm^2/m

Total reinforcement = $(c + 6 * d_x)$ 1340 = (0,6 + 6 * 0,657) * 1340 = 6086mm^2

$\rho_{lx} = 6086 / ((c + 6 * d_x)d_x) = 6086 / ((600 + 6 * 657) * 657) = 0,0020$ [-]

Reinforcement ratio ρ_{ly}

Ø16-150 = 1340mm^2/m

Total reinforcement = $(c + 6 * d_y)$ 1340 = (0,6 + 6 * 0,641) * 1340 = 5957mm^2

$\rho_{lx} = 5957 / ((c + 6 * d_y)d_y) = 5957 / ((600 + 6 * 641) * 641) = 0,0021$ [-]

Reinforcement ratio ρ_l

$\rho_l = \mathrm{sqrt}(\rho_{lx} * \rho_{ly}) = 0,00205$ [-]

First control perimeter u_1

$u_1 = \pi\ (c + 4 * d_{eff}) = \pi\ (600 + 4 * 649) = 10.040mm$

Design value of punching shear stress

$V_{ed} = (\beta * V_{ed})/(u_1 * d_{eff})$

$\beta = 1,15$ (centre column)

$V_{ed} = 1,15 * 5.261.000 / 10.040 * 649 = 0,93N/mm^2$

Concrete capacity

$V_{rd,c} = 0,12 * k\ (100 * \rho_l * f_{ck})$^(1/3)

$k = 1 + \mathrm{sqrt}(200 / d_{eff}) = 1 + \mathrm{sqrt}(200/649) = 1,55$ [-]

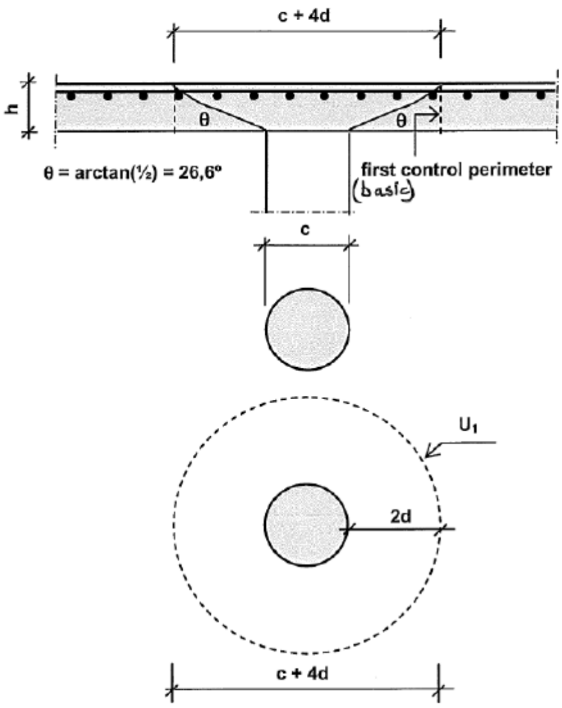
$V_{rd,c}$ = 0,12 * 1,55 (100 * 0,00205* 35)^(1/3) = 0,36N/mm^2
V_{min} = 0,0035 * k^1,5 * f_{ck}^0,5

V_{min} = 0,0035 * 1,55^1,5 * 35^0,5 = 0,399N/mm^2

$V_{min} < V_{ed}$ → punching shear reinforcement required (or other measure).

Two measures are possible:
i. Adding a column head;
ii. Applying punching shear reinforcement.

Only applying a column head is not enough and punching shear reinforcement is still necessary. Therefore, the choice is made to only use punching shear reinforcement.



Applying punching shear reinforcement

Reinforcement:
x-direction: Ø25-125
y-direction: Ø25-125

d_y = 700-35-25-25/2 = 627,5mm
d_x = 700-35-25/2 = 652,5mm

d_{eff} = (627,5 + 652,5)/2 = 640mm

Reinforcement ratio ρ_{lx}

Ø25-125 = 3927mm^2/m

Total reinforcement = (c + 4 * d_x) 3927 = (0,6 + 4 * 0,652) * 3927 = 12.597mm^2

ρ_{lx} = 12597 / ((c + 4 * d_x)d_x) = 12957 / ((600 + 4 * 652) * 652) = 0,0062 [-]

Reinforcement ratio ρ_{ly}

Ø25-125 = 3927mm^2/m

Total reinforcement = (c + 4 * d_y) 3927 = (0,6 + 4 * 0,627) * 3927 = 12.205mm^2

ρ_{lx} = 12205 / ((c + 4 * d_y)d_y) = 12205 / ((600 + 4 * 627) * 627) = 0,0063 [-]

Reinforcement ratio ρ_l

ρ_l = sqrt(ρ_{lx} * ρ_{ly}) = 0,00625 [-]

First control perimeter u_0

u_0 = π * 600 = 1.884mm

Design value of punching shear stress

V_{ed} = (β * V_{ed})/(u_0 * d_{eff})

β = 1,15 (centre column)

V_{ed} = 1,15 * 5.261.000 / 1,884 * 640 = 5,01N/mm^2

$V_{rd,max}$ = 0,5 * v * f_{cd}

v = 0,6 * (1 – f_{ck}/250)

v = 0,6 * (1 – 35/250)

v = 0,516 [-]

$V_{rd,max}$ = 0,5 * 0,516 * 35/1,5

$V_{rd,max}$ = 6,02N/mm^2

$V_{ed} < V_{rd,max}$ → No concrete failure next to column.

Concrete capacity

$V_{rd,c}$ = 0,12 * k (100 * ρ_l * f_{ck})^(1/3)

k = 1 + sqrt(200 / d_{eff}) = 1 + sqrt(200/640) = 1,56 [-]

$V_{rd,c}$ = 0,12 * 1,56 (100 * 0,00625 * 35)^(1/3) = 0,52N/mm^2

V_{min} = 0,0035 * k^1,5 * f_{ck}^0,5

V_{min} = 0,0035 * 1,56^1,5 * 35^0,5 = 0,40N/mm^2

$V_{min} < V_{ed}$ → punching shear reinforcement required (or other measure).

Second control perimeter u_1

u_1 = π (c + 4 * d_{eff}) = π (600 + 4 * 640) = 9.927mm

Design value of punching shear stress

V_{ed} = (β * V_{ed})/(u_1 * d_{eff})

β = 1,15 (centre column)

V_{ed} = 1,15 * 5.261.000 / 9.927 * 640 = 0,95N/mm^2

$V_{ed} > V_{rd,c}$ → punching shear reinforcement required

Input

V_{ed} = 0,95 N/mm^2

$V_{rd,c}$ = 0,52 N/mm^2

$f_{ywd,ef}$ = 250 + 0,25 * d

$f_{ywd,ef}$ = 250 + 0,25 * 640 = 410 N/mm^2

Maximum radial spacing

$S_{r,max} = 0{,}75 * d_{eff}$

$S_{r,max} = 0{,}75 * 640 = 480mm$

$S_r = 450mm$

Required reinforcement

$A_{sw} = (V_{ed} – 0{,}75 * V_{rd,c}) * ((S_r * u_1)/(1{,}5 * f_{ywd,ef}))$

$A_{sw} = (0{,}95 – 0{,}75 * 0{,}52) * ((450 * 9.927)/(1{,}5 * 410))$

$A_{sw} = 4.067mm^2$ (in each reinforcement perimeter)

U_{out}

For which perimeter is $V_{ed} = V_{rd,c}$

$U_{out} = β * V_{ed} / V_{rd,c} * d_{eff}$

$U_{out} = β * 5.261 * 10^3 / 0{,}52 * 640$

$U_{out} = 18.179mm$

Distance column-U_{out}

$U_{out} = π(600 + x)$

$X = 8 * d_{eff}$

Distance column-$U_{out} = 4 * d_{eff}$

Outer punching shear reinforcement at $4 * d_{eff} – 1{,}5 * d_{eff} = 2{,}5 * d_{eff}$

Punching shear perimeters

$0{,}3 * d_{eff} – 0{,}5 * d_{eff} – 1{,}25 * d_{eff} – 2{,}0 * d_{eff} – 2{,}75 * d_{eff}$

190 mm – 320 mm – 800 mm – 1250 mm – 1750 mm

Reinforcement in perimeters

Stirrups Ø16

$A_s = 2 * 8^2 * π = 402mm^2$

10 stirrups required per perimeter

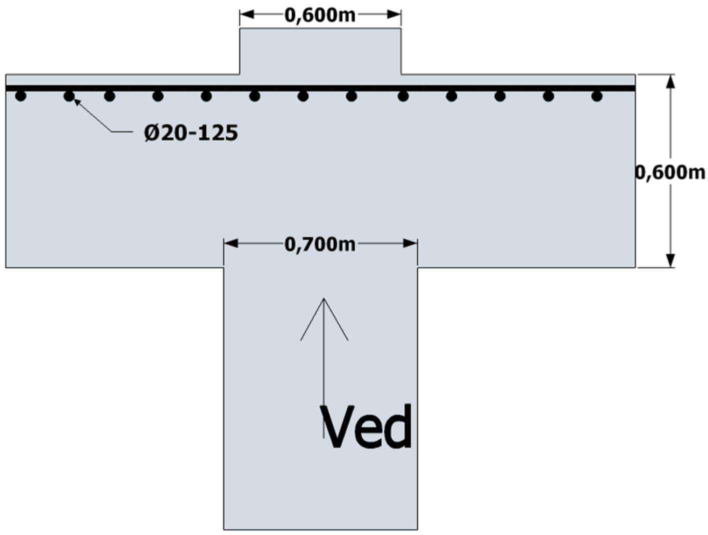
Check minimum cross-section needed per link leg

$A_{sw,min} > (0{,}08 * s_r * s_t * sqrt(f_{ck}))/(1{,}5 * f_{yk})$

$A_{sw,min} > (0{,}08 * 450 * 1{,}5 * 640 * sqrt(35))/(1{,}5 * 500)$

$A_{sw,min} > 272mm^2 < 402 \rightarrow$ OK

Punching shear intermediate floor



Concrete: C28/35

Reinforcement: Ø20-125

Cover: 25mm

V_{ed}: 2.343kN

Slab area: 59,4m^2

d_x = 600 – 25 – 20 – 20/2 = 545mm

d_y = 600 – 25 – 20/2 = 565mm

d_{eff} = (545 + 565)/2 = 555mm

Reinforcement ratio ρ_{lx}

Ø20-125 = 2.513mm^2/m

Total reinforcement = (c + 4 * d_x) 2.513 = (0,7 + 4 * 0,545) * 2.513 = 7.237mm^2

ρ_{lx} = 7.237 / ((c + 4 * d_x)d_x) = 7.237 / ((700 + 4 * 545) * 545) = 0,0046

Reinforcement ratio ρ_{ly}

Ø20-125 = 2.513mm^2/m

Total reinforcement = (c + 4 * d_y) 2.513 = (0,7 + 4 * 0,565) * 2.513 = 7.438mm^2

ρ_{lx} = 7.438 / ((c + 4 * d_y)d_y) = 7.438 / ((700 + 4 * 565) * 565) = 0,0044

Reinforcement ratio ρ_l

ρ_l = sqrt(ρ_{lx} * ρ_{ly}) = 0,0045

First control perimeter u_0

u_0 = π * 700 = 2.199mm

Design value of punching shear stress

V_{ed} = (β * V_{ed})/(u_0 * d_{eff})

β = 1,15 (centre column)

V_{ed} = 1,15 * 2.343.000 / 2.199 * 555 = 2,21N/mm^2

$V_{rd,max}$ = 0,5 * v * f_{cd}

v = 0,6 * (1 – f_{ck}/250)

v = 0,6 * (1 – 35/250)

v = 0,516 [-]

$V_{rd,max}$ = 0,5 * 0,516 * 35/1,5

$V_{rd,max}$ = 6,02N/mm^2

$V_{ed} < V_{rd,max}$ → No concrete failure next to column.

Concrete capacity

$V_{rd,c}$ = 0,12 * k (100 * ρ_l * f_{ck})^(1/3)

k = 1 + sqrt(200 / d_{eff}) = 1 + sqrt(200/555) = 1,60 [-]

$V_{rd,c}$ = 0,12 * 1,60 (100 * 0,0045 * 35)^(1/3) = 0,48N/mm^2

V_{min} = 0,0035 * k^1,5 * f_{ck}^0,5

V_{min} = 0,0035 * 1,60^1,5 * 35^0,5 = 0,42N/mm^2

$V_{min} < V_{ed}$ → punching shear reinforcement required (or other measure).

Second control perimeter u_1

u_1 = π (c + 4 * d_{eff}) = π (700 + 4 * 555) = 9.173mm

Design value of punching shear stress

V_{ed} = (β * V_{ed})/(u_1 * d_{eff})

β = 1,15 (centre column)

V_{ed} = 1,15 * 2.343.000 / 9.173 * 555 = 0,53N/mm^2

$V_{ed} > V_{rd,c}$ → punching shear reinforcement required

Input

V_{ed} = 0,53 N/mm^2

$V_{rd,c}$ = 0,48 N/mm^2

$f_{ywd,ef}$ = 250 + 0,25 * d

$f_{ywd,ef}$ = 250 + 0,25 * 555 = 388,75 N/mm^2

Maximum radial spacing

$S_{r,max}$ = 0,75 * d_{eff}

$S_{r,max}$ = 0,75 * 555 = 416mm

S_r = 410mm

Required reinforcement

A_{sw} = (V_{ed} – 0,75 * $V_{rd,c}$) * ((S_r * u_1)/(1,5 * $f_{ywd,ef}$))

A_{sw} = (0,53 – 0,75 * 0,48) * ((410 * 9.173)/(1,5 * 388))

A_{sw} = 1.099m^2 (in each reinforcement perimeter)

U_{out}

For which perimeter is V_{ed} = $V_{rd,c}$

U_{out} = β * V_{ed} / $V_{rd,c}$ * d_{eff}

U_{out} = β * 2.343 * 10^3 / 0,48 * 555

U_{out} = 10.144mm

Distance column-U_{out}

U_{out} = π(700 + x)

X = 5 * d_{eff}

Distance column-U_{out} = 2,5 * d_{eff}

Outer punching shear reinforcement at 1,5 * d_{eff}

Punching shear perimeters

0,3 * d_{eff} – 0,5 * d_{eff} – 1,25 * d_{eff}

165 mm – 275 mm – 690 mm

Reinforcement in perimeters (first guess)

Stirrups Ø8

A_s = 2 * 4^2 * π = 100,4mm^2

12 stirrups required per perimeter

Check minimum cross-section needed per link leg

$A_{sw,min}$ > (0,08 * s_r * s_t * sqrt(f_{ck}))/(1,5 * f_{yk})

$A_{sw,min}$ > (0,08 * 410 * 1,5 * 555 * sqrt(35))/(1,5 * 500)

$A_{sw,min}$ > 215mm^2 < 402 → Not OK

Reinforcement in perimeters

Use 4 * Stirrups Ø20

A_s = 2 * 10^2 * π = 314mm^2 → OK

4 stirrups required per perimeter

L5 Structural calculations: Walls

Compared to the rest of the structure, the moments in the walls are very low. Because the crack width control was the leading factor in determining the needed reinforcement for the roof and floors, only the crack width control will be done for the walls.

Crack width control

The maximum moment in the walls is 112,7kNm. This follows from SCIA calculations with the water pressure and soil pressure acting on the wall. The maximum moment is found where the wall and Flexbase floor connect.

For the loads on the wall see page 62.

Input

Concrete:	C28/35	
Thickness wall:	600	[mm]
f_{cd}:	18,7	[N/mm^2]
f_{ctm}:	2,70	[N/mm^2]
$E_{cm,\ short}$:	32.300	[N/mm^2]
$E_{cm,\ long}$:	17.000	[N/mm^2]
Reinforcement steel:	FeB 500	
Concrete cover:	35	[mm]
f_{yd}:	435	[N/mm^2]
E_s:	200.000	[N/mm^2]
Ratio E_s/E_{cm}:	11,8	[-]

w_{max} = 0,1mm

Reinforcement

Reinforcement, first guess: Ø 20 – 150

d_y = 600 – 35 – 10 = 555mm

Reinforcement ratio ρ_1

A_s = 10 * 10 * π * (1000/150) = 2094mm^2/m^1

ρ_1 = 2094 / (1000 * 555) = 0,0038 [-]

Calculation of the height of the compressive zone in the SLS (assuming linear elastic behaviour of concrete loaded in compression; concrete is assumed to have no tensile strength):

$$\frac{x}{d} = -\alpha_e * \rho 1 + \sqrt{(\alpha_e * \rho 1)^2 + 2 * \alpha_e * \rho 1}$$

$$\frac{x}{613} = -11{,}8 * 0{,}0038 + \sqrt{(11{,}8 * 0{,}0038)^2 + 2 * 11{,}8 * 0{,}0038}$$

x = 143,17 mm

Steel stress in SLS

$$\sigma_s = \frac{M_{eqp}}{A_{s,prov}\,(d - \frac{1}{3X})}$$

$$\sigma_s = \frac{112{,}7 * 10^6}{2094 * \ (555 - \frac{1}{3} * 143{,}17)}$$

σ_s = 106N/mm^2

Calculation of the height of the “hidden” tensile member

The height of the tensile member in a beam loaded in bending:

$h_{c,eff}$ < (h-x)/3

$h_{c,eff}$ = 2,5 (h-d) = 2,5 (600 – 555) = 112,5mm

(h-x)/3 = (600 – 143,17)/3 = 152,1mm → Use 112,5mm

Reinforcement ratio of the "hidden" tensile member

$\rho_{p,eff}$ = A_s / (h_{eff} * b)

$\rho_{p,eff}$ = 2094/ (112,5 * 1000) = 0,0186 [-]

The cracking force of the "hidden" tensile member

N_{cr} = $A_{c,eff}$ * f_{ctm} (1 + α_e * $\rho_{p,eff}$)

N_{cr} = 112,5 * 1000 * 2,7 (1 + 11,8 * 0,0186)
N_{cr} = 370,4 *10^3 N

σ_{sr} = N_{cr} / A_s = 176,8N/mm^2

σ_{sr} > σ_s → Not in established cracking stage!

M1 Construction costs: Comparison

De kosten voor de parkeergarage, wanneer deze in-situ gebouwd zou worden, worden gebaseerd op het rapport 'Kostprijsbepaling parkeergarage Geldersekade' van Stadsdeel Centrum Amsterdam [Oosterdok, 2009]. In dit rapport worden de kosten bepaald voor het bouwen van een parkeergarage in het Oosterdok. Het ontwerp van deze parkeergarage is, op de vloer na, hetzelfde als het ontwerp dat gebruikt wordt in deze thesis. Het enige verschil is de afmeting. De kostenraming in het rapport van Stadsdeel Centrum Amsterdam is gebaseerd op een garage van 78 bij 78 meter. De parkeergarage in deze thesis is 129 bij 76 meter, de Flexbase vloer is 132 bij 79 meter. De kostenraming zal daarom ook aangepast worden zodat beide parkeergarages dezelfde afmetingen hebben. Het ontwerp is te zien op pagina 31 en 32 van het hoofdrapport.

De hoofddimensies zijn:

Lengte:	129	[m]
Breedte:	76	[m]
Wanddikte:	600	[mm]
Vloerdikte:	800	[mm]
Dak:	700	[mm]
Kolommen, diameter:	600	[mm]
Aantal parkeerlagen:	2	[-]

De uitgangspunten bij het opstellen van de begroting zijn:

i. Het ontwerp van de parkeergarage is, op de vloer na, voor beide uitvoeringsmethodes gelijk. Het betreft het ontwerp op de pagina's 31 en 32;
ii. Het hoofddoel is niet de exacte bouwkosten bepalen, maar het verschil tussen de twee bouwmethodes. Daarom worden de eenheidsprijzen uit de kostenraming van Stadsdeel Centrum Amsterdam gebruikt voor beide begrotingen;
iii. Niet inbegrepen in de kosten zijn: grondverwerving en schade, bodem- of andere saneringen, BTW, risico's, winst, rentekosten;
iv. Niet inbegrepen in de kosten zijn: de toegang voor auto's, de drie toegangen voor voetgangers en aanpassingen aan de omliggende infrastructuur. Deze kosten zullen nagenoeg hetzelfde zijn voor beide varianten en voegen dan ook geen waarde toe aan de vergelijking.

Vergelijking van bouwmethodes
Het bouwen van de wanden, kolommen, dak en toegangen zal voor beide methodes vrijwel hetzelfde zijn. Daar ligt dan ook niet het gebied van interesse. De belangrijke punten zijn juist de verschillen tussen de twee methodes en de bijbehorende kosten. Aan de ene kant van de schaal hebben we de punten die wegvallen bij het drijvend bouwen, zoals het bouwen van een bouwkuip. Aan de andere kant zijn de extra punten die nodig zijn voor het drijvend bouwen, zoals de Flexbase vloer, en dus extra geld kosten. Als het verschil tussen deze twee positief uitvalt voor het drijvend bouwen kunnen we spreken van een geslaagd alternatief.

Deze afweging is gemaakt in tabel M1-1. De tabel laat zien dat het verschil positief is.

Traditionele bouwmethode vs. drijvend bouwen

Niet benodigd voor drijvend bouwen	
Tijdelijke voorzieningen	
Toepassen tijdelijke damwand	€ 1.126.400,00
Aanbrengen gordingen	€ 160.000,00
Aanbrengen groutankers onder water	€ 688.000,00
Funderingswerken	
Aanbrengen groutinjectiepalen	€ 1.912.500,00
Betonwerken	
Aanbrengen onderwaterbeton	€ 2.065.560,00
Aanbrengen uitvullaag OWB	€ 172.500,00
Aanbrengen beton vloer	€ 2.745.050,00
Totaal	**€ 8.870.010,00**
Extra kosten drijvend bouwen	
Tijdelijke voorzieningen	
Toepassen tijdelijke damwand, incl. gording en ankers	€ 205.000,00
Toepassen golfbreker	€ 30.000,00
Extra voorzieningen positionering Flexbase vloer	€ 25.000,00
Grondwerk	
Extra kosten grond in den natte ontgraven	€ 68.950,00
Extra kosten vervoeren grond	€ 258.562,50
Extra kosten aanvulzand	€ 137.500,00
Extra kosten verwijderen, vervoeren, acceptatie slib	€ 69.000,00
Funderingswerken	
Aanbrengen GEWI-ankers	€ 872.600,40
Aanbrengen betontegels	€ 200.000,00
Flexbase vloer	
Flexbase vloer, 132 x 79 meter	€ 4.666.935,22
Betonwerken	
Extra kosten betonwerken	€ 53.160,00
Afzinken garage element	
Waterballast tanks, huur, vullen, legen	€ 60.000,00
Afzink materieel, pontons, kranen	€ 25.000,00
Totaal	**€ 6.671.708,12**
Verschil	**€ 2.198.308,88**

Tabel M1-1

M2 Construction costs: Traditional method

Oosterdok, in-situ, tweelaags, 132 x 79 meter, exclusief toeritten				
750 parkeerplaatsen				
Voorbereidende werkzaamheden				
Inrichten werkterrein	EUR	1	€ 10.000,00	€ 10.000,00
Tijdelijke voorzieningen				
Toepassen tijdelijke damwand, AZ26-700 135m x 85m, 32m lang	m2	14.080		
Aanbrengen gordingen	ton	160		
Aanbrengen groutankers onder water, h.o.h. 2,5m	st	172		
Grondwerken				
Grond in den natte ontgraven, 85 x 135, 9 meter diep	m3	103.275		
Vervoeren grond, enkele reis tot 10 km	m3	103.275		
Aanbrengen aanvulzand tussen constructie en damwand	m3	5.500		
Verwijderen en afvoeren slib, exclusief acceptatiekosten	m3	5.737		
Acceptatiekosten slib	ton	8.606		
Funderingswerken				
Aanbrengen groutinjectiepalen, lang ca. 11 m, h.o.h. 3m	st	1.275		
Betonwerken				
Aanbrengen onderwaterbeton, dik 1500 mm	m3	17.213		
Aanbrengen uitvullaag OWB, dik 100 mm	m3	1.150		
Aanbrengen beton vloer, dik 800 mm	m3	7.843		
Aanbrengen tussenvloer, dik 500 mm	m3	4.902		
Aanbrengen beton wanden, dik 600 mm	m3	1.772		
Aanbr. beton dekconstructie, dik 700 mm	m3	6.863		
Aanbrengen hellingsbanen, dik 500 mm	m3	670		
Aanbrengen kolommen	st	300		
Aanbrengen afwerklaag	m2	20.856		
Overige				
Trappenhuizen voetgangers	st.	3		
Totaal directe kosten				**€ 17.778.977,50**

Oosterdok, in-situ, tweelaags, 79 x 106 meter, exclusief toeritten

<u>550 parkeerplaatsen</u>

Voorbereidende werkzaamheden				
Inrichten werkterrein	EUR	1	€ 10.000,00	€ 10.000,00
Tijdelijke voorzieningen				
Toepassen tijdelijke damwand, AZ26-700 110m x 85m, 32m lang	m2	12.480	€ 80,00	€ 998.400,00
Aanbrengen gordingen	ton	140		
Aanbrengen groutankers onder water, h.o.h. 2,5m	st	156		
Grondwerken				
Grond in den natte ontgraven, 85 x 110, 9 meter diep	m3	84.150		
Vervoeren grond, enkele reis tot 10 km	m3	84.150		
Aanbrengen aanvulzand tussen constructie en damwand	m3	4.875		
Verwijderen en afvoeren slib, exclusief acceptatiekosten	m3	4.675		
Acceptatiekosten slib	ton	7.013		
Funderingswerken				
Aanbrengen groutinjectiepalen, lang ca. 11 m, h.o.h. 3m	st	1.039		
Betonwerken				
Aanbrengen onderwaterbeton, dik 1500 mm	m3	14.025		
Aanbrengen uitvullaag OWB, dik 100 mm	m3	935		
Aanbrengen beton vloer, dik 800 mm	m3	6.700		
Aanbrengen tussenvloer, dik 500 mm	m3	4.187		
Aanbrengen beton wanden, dik 600 mm	m3	1.554		
Aanbr. beton dekconstructie, dik 700 mm	m3	5.862		
Aanbrengen hellingsbanen, dik 500 mm	m3	670		
Aanbrengen kolommen	st	300		
Aanbrengen afwerklaag	m2	16.748		
Overige				
Trappenhuizen voetgangers	st.	3		
Totaal directe kosten				**€ 15.113.377,50**

Oosterdok, in-situ, tweelaags, 79 x 79 meter, exclusief toeritten

350 parkeerplaatsen

Voorbereidende werkzaamheden				
Inrichten werkterrein	EUR	1	€ 10.000,00	€ 10.000,00
Tijdelijke voorzieningen				
Toepassen tijdelijke damwand, AZ26-700 85m x 85m, 32m lang	m2	10.880		
Aanbrengen gordingen	ton	112		
Aanbrengen groutankers onder water, h.o.h. 2,5m	st	136		
Grondwerken				
Grond in den natte ontgraven, 85 x 85, 9 meter diep	m3	65.025		
Vervoeren grond, enkele reis tot 10 km	m3	65.025		
Aanbrengen aanvulzand tussen constructie en damwand	m3	4.250		
Verwijderen en afvoeren slib, exclusief acceptatiekosten	m3	4.250		
Acceptatiekosten slib	ton	6.375		
Funderingswerken				
Aanbrengen groutinjectiepalen, lang ca. 11 m, h.o.h. 3m	st	784		
Betonwerken				
Aanbrengen onderwaterbeton, dik 1500 mm	m3	10.838		
Aanbrengen uitvullaag OWB, dik 100 mm	m3	723		
Aanbrengen beton vloer, dik 800 mm	m3	4.992		
Aanbrengen tussenvloer, dik 500 mm	m3	3.120		
Aanbrengen beton wanden, dik 600 mm	m3	1.327		
Aanbr. beton dekconstructie, dik 700 mm	m3	4.369		
Aanbrengen hellingsbanen, dik 500 mm	m3	670		
Aanbrengen kolommen	st	180		
Aanbrengen afwerklaag	m2	12.482		
Overige				
Trappenhuizen voetgangers	st.	3		
Totaal directe kosten				**€ 11.774.202,50**

M3 Construction costs: Floating construction method

Oosterdok, floating construction, tweelaags, 132 x 79 meter, exclusief toeritten

750 parkeerplaatsen

Voorbereidende werkzaamheden				
Inrichten werkterrein	EUR	1	€ 10.000,00	€ 10.000,00
Tijdelijke voorzieningen				
Toepassen tijdelijke damwand kade, AZ18 2 x 60m, 20m lang	m2	2.400		
Aanbrengen gordingen	ton	25		
Aanbrengen groutankers, h.o.h. 2,5m	st	48		
Toepassen golfbreker	st	1		
Extra voorzieningen positionering Flexbase vloer	st	1		
Grondwerken				
Grond in den natte ontgraven, 95 x 145, 10 meter diep	m3	137.750		
Vervoeren grond, enkele reis tot 10 km	m3	137.750		
Aanbrengen aanvulzand	m3	16.500		
Verwijderen en afvoeren slib, exclusief acceptatiekosten	m3	6.887		
Acceptatiekosten slib	ton	10.331		
Funderingswerken				
Aanbrengen GEWI-ankers, 15m1	st	651		
Aanbregen betontegels, 2000 x 2000 mm	st	100		
Flexbase vloer				
EPS250, vier lagen van 200 mm, glasfolie versterkt	m2	10.428		
Lijm, 4 lagen	m2	41.712		
Wokkels, 4 st/m2	st	42.000		
Installatie EPS, 4 lagen	m2	41.712		
Eerste vloer, dik 250 mm, C28/35, incl. bekisting	m3	2.607		
EPS60, dik 1000 mm	m3	7.689		
EPS200, randen tweede laag	m3	754		
Installatie EPS, 2 lagen	m2	16.888		
Balkenrooster, hoog 1000 mm, breed 400 mm	m3	2.088		
Omega profielen, bekisting balken	st	85		
Tweede vloer, dik 250 mm, C28/35, incl. bekisting	m3	2.607		
Ballast tegen vervormen vloer tijdens stort	st	2		
Betonwerken				
Aanbrengen tussenvloer, dik 500 mm	m3	4.902		
Aanbrengen beton wanden, dik 600 mm	m3	1.772		
Aanbr. beton dekconstructie, dik 700 mm	m3	6.863		
Aanbrengen hellingsbanen, dik 500 mm	m3	670		
Aanbrengen kolommen	st	300		

Aanbrengen afwerklaag	m2	20.856		
Afzinken garage element				
Waterballast tanks, huur, vullen, legen	st.	400		
Afzink materieel, pontons, kranen	dag	1		
Overige				
Trappenhuizen voetgangers	st.	3	€ 20.000,00	€ 60.000,00
Totaal directe kosten				**€ 15.580.675,62**

Oosterdok, floating construction, tweelaags, 79 x 106 meter, exclusief toeritten

550 parkeerplaatsen

Voorbereidende werkzaamheden				
Inrichten werkterrein	EUR	1	€ 10.000,00	€ 10.000,00
Tijdelijke voorzieningen				
Toepassen tijdelijke damwand kade, AZ18 2 x 30m, 20m lang	m2	1.200		
Aanbrengen gordingen	ton	12,5		
Aanbrengen groutankers, h.o.h. 2,5m	st	24		
Toepassen golfbreker	st	1		
Extra voorzieningen positionering Flexbase vloer	st	1		
Grondwerken				
Grond in den natte ontgraven, 95 x 120, 10 meter diep	m3	114.000		
Vervoeren grond, enkele reis tot 10 km	m3	114.000		
Aanbrengen aanvulzand	m3	14.780		
Verwijderen en afvoeren slib, exclusief acceptatiekosten	m3	5.700		
Acceptatiekosten slib	ton	8.550		
Funderingswerken				
Aanbrengen GEWI-ankers, 15m1	st	522		
Aanbregen betontegels, 2000 x 2000 mm	st	80		
Flexbase vloer				
EPS250, vier lagen van 200 mm, glasfolie versterkt	m2	8.374		
Lijm, 4 lagen	m2	33.496		
Wokkels, 4 st/m2	st	33.496		
Installatie EPS, 4 lagen	m2	33.496		
Eerste vloer, dik 250 mm, C28/35, incl. bekisting	m3	2.094		
EPS60, dik 1000 mm	m3	6.175		
EPS200, randen tweede laag	m3	660		
Installatie EPS, 2 lagen	m2	13.562		
Balkenrooster, hoog 1000 mm, breed 400 mm	m3	1.670		
Omega profielen, bekisting balken	st	70		
Tweede vloer, dik 250 mm, C28/35, incl. bekisting	m3	2.094		
Ballast tegen vervormen vloer tijdens stort	st	2		

Betonwerken				
Aanbrengen tussenvloer, dik 500 mm	m3	4.187		
Aanbrengen beton wanden, dik 600 mm	m3	1.554		
Aanbr. beton dekconstructie, dik 700 mm	m3	5.862		
Aanbrengen hellingsbanen, dik 500 mm	m3	670		
Aanbrengen kolommen	st	240		
Aanbrengen afwerklaag	m2	16.748		
Afzinken garage element				
Waterballast tanks, huur, vullen, legen	st.	325		
Afzink materieel, pontons, kranen	dag	1		
Overige				
Trappenhuizen voetgangers	st.	3	€ 20.000,00	€ 60.000,00
Totaal directe kosten				**€ 12.948.523,90**

Oosterdok, floating construction, tweelaags, 79 x 79 meter, exclusief toeritten

350 parkeerplaatsen

Voorbereidende werkzaamheden				
Inrichten werkterrein	EUR	1	€ 10.000,00	€ 10.000,00
Tijdelijke voorzieningen				
Toepassen golfbreker	st	1	€ 30.000,00	€ 30.000,00
Extra voorzieningen positionering Flexbase vloer	st	1	€ 25.000,00	€ 25.000,00
Grondwerken				
Grond in den natte ontgraven, 95 x 95, 10 meter diep	m3	90.250		
Vervoeren grond, enkele reis tot 10 km	m3	90.250		
Aanbrengen aanvulzand	m3	13.060		
Verwijderen en afvoeren slib, exclusief acceptatiekosten	m3	4.500		
Acceptatiekosten slib	ton	6.750		
Funderingswerken				
Aanbrengen GEWI-ankers, 15m1	st	390		
Aanbregen betontegels, 2000 x 2000 mm	st	60		
Flexbase vloer				
EPS250, vier lagen van 200 mm, glasfolie versterkt	m2	6.241		
Lijm, 4 lagen	m2	24.964		
Wokkels, 4 st/m2	st	24.964		
Installatie EPS, 4 lagen	m2	24.965		
Eerste vloer, dik 250 mm, C28/35, incl. bekisting	m3	1.560		
EPS60, dik 1000 mm	m3	4.602		
EPS200, randen tweede laag	m3	565		
Installatie EPS, 2 lagen	m2	12.482		
Balkenrooster, hoog 1000 mm, breed 400 mm	m3	1.250		
Omega profielen, bekisting balken	st	55		
Tweede vloer, dik 250 mm, C28/35, incl. bekisting	m3	1.560		
Ballast tegen vervormen vloer tijdens stort	st	2		

Betonwerken				
Aanbrengen tussenvloer, dik 500 mm	m3	3.121		
Aanbrengen beton wanden, dik 600 mm	m3	1.327		
Aanbr. beton dekconstructie, dik 700 mm	m3	4.369		
Aanbrengen hellingsbanen, dik 500 mm	m3	670		
Aanbrengen kolommen	st	180		
Aanbrengen afwerklaag	m2	12.482		
Afzinken garage element				
Waterballast tanks, huur, vullen, legen	st.	250	€ 150,00	€ 37.500,00
Afzink materieel, pontons, kranen	dag	1	€ 25.000,00	€ 25.000,00
Overige				
Trappenhuizen voetgangers	st.	1	€ 20.000,00	€ 20.000,00
Totaal directe kosten				**€ 9.860.903,15**

M4 Construction costs: Walls of parking garage

N Financial analysis

In deze financiële analyse worden drie punten beschouwd, namelijk:

i. Kosten;
ii. Baten;
iii. Netto Contante Waarde (NCW).

Kosten

De kosten bestaan uit twee delen, namelijk:

i. Investeringskosten (bouwkosten);
ii. Exploitatiekosten.

De bouwkosten zijn beschouwt in appendix M. Dit zijn echter niet te totale investeringskosten. De bouwkosten zijn namelijk exclusief BTW, engineering, winst en risico etc. Als eerste zal er dus een beeld gevormd moeten worden van de totale investeringskosten.

<u>Investeringskosten</u>

De investeringskosten bestaan uit alle kosten die tijdens en voor de bouw gemaakt moeten worden. Deze kosten bestaan voornamelijk uit percentages van de bouwkosten. De extra kosten zijn:

i. Nader te detailleren (10%);
ii. Eenmalige kosten / uitvoeringskosten (10%);
iii. Algemene kosten / winst en risico (10%);
iv. Object onvoorzien (10%);
v. Engineeringskosten (20%);
vi. Bouwkosten toegangen voetgangers en auto's.

De bouwkosten voor de toegang van de auto's wordt geraamd op € 1.200.000,-. Hierin is de volgende verdeling aangehouden:

i. Bouwkuip 30 x 15 meter, € 200.000,-
ii. Betonwerk, € 800.000,-
iii. Sloopwerk, grondwerk, funderingswerk, € 200.000,-

De bouwkosten van de drie toegangen voor voetgangers worden geraamd op € 300.000,-.

Een overzicht van de investeringskosten is te zien in de onderstaande tabel.

Omschrijving	Totaal
Voorbereidende werkzaamheden	€ 10.000,00
Tijdelijke voorzieningen	€ 260.000,00
Grondwerken	€ 1.928.110,00
Funderingswerken	€ 1.072.600,00
Flexbase vloer	€ 4.666.935,22
Betonwerken	€ 7.498.030,00
Afzinken garage element	€ 85.000,00
Bouwkosten toegangen	€ 1.500.000,00
Overige	€ 60.000,00
Totale bouwkosten	€ 17.080.675,22
Nader te detailleren (10%)	€ 1.708.067,52
Totaal directe kosten	**€ 18.788.742,74**
Eenmalige kosten / uitvoeringskosten (10%)	€ 1.878.874,27
Algemene kosten / winst en risico (10%)	€ 1.878.874,27
Totaal indirecte kosten	€ 3.757.748,55
Object onvoorzien (10%)	€ 2.254.649,13
Engineering, overhead (20%)	€ 4.509.298,26
Totale investeringskosten	**€ 29.310.438,68**

Exploitatiekosten

De exploitatiekosten zijn alle kosten die nodig zijn op de parkeergarage draaiende te houden tijdens de operatiefase. De onderverdeling en kosten per jaar zijn de zien in de onderstaande tabel en zijn gebaseerd op de kosten uit [Oosterdok, 2009].

Exploitatiekosten per jaar	
Omschrijving	**Totaal**
Management en administratie	€ 100.000,00
Bewaking (twee pers. Per dienst, drie diensten)	€ 288.000,00
Nuts	€ 90.000,00
Dagelijks onderhoud (twee pers.)	€ 96.000,00
Verzekeringen en belastingen	€ 70.000,00
Reserveringen	€ 80.000,00
Overige/onvoorzien	€ 40.000,00
Totale exploitatiekosten per jaar	**€ 764.000,00**

Baten

De baten zijn de opbrengsten tijdens de exploitatiefase. Hier komen de baten uit twee bronnen, namelijk:

i. Opbrengsten uit parkeerabonnementen;
ii. Opbrengsten uit parkeergeld (bezoekers).

De parkeer abonnementen in Amsterdam kosten voor inwoners €30,- per maand en voor bedrijven €48,- per maand. Het parkeertarief in dit gebied is €5,- per uur.

De Gemeente Amsterdam houdt voor de Oosterdok Parkeergarage de volgende verdeling aan:

i. 70% abonnementen, waarvan 67% bewoners en 33% bedrijven;
ii. 30% bezoekers.

De baten uit abonnementen zijn eenvoudig te berekenen door het aantal abonnementen te vermenigvuldigen met de kosten per maand. Voor de baten uit bezoekers is dat echter lastiger. De Gemeente Amsterdam heeft op basis van andere parkeergarages in dit gebied berekend dat een parkeerplaats gemiddeld 5,3 uur per dag bezet is. Dit betekent 1.935 uur per jaar en een opbrengst van €9.675,- per jaar per parkeerplaats.

Dubbelgebruik
Voor het berekenen van de opbrengsten gaan we uit van het zogeheten dubbelgebruik. Bewoners zullen hun auto voornamelijk buiten kantooruren parkeren, terwijl bezoekers en abonnementhouders van bedrijven dat vooral binnen kantooruren doen. Dit betekend dat de capaciteit van de parkeergarage meer dan 100% is. Daarom wordt het aantal bezoekersplaatsen niet op 225 gehouden (30% * 750), maar op 400.

Wanneer in acht wordt genomen dat de parkeergarage 750 parkeerplaatsen heeft, betekent dit een opbrengst per jaar van:

Abonnementen bewoners (352 plaatsen):	€ 126.720,-
Abonnementen bedrijven (173 plaatsen):	€ 99.648,-
Bezoekers (400 plaatsen):	€ 3.869.000,-

Wat direct opvalt is de hoge inkomsten uit bezoekers vergeleken met de abonnementen. Het is daarom, in het kader van de haalbaarheid, nuttig om te berekenen wat de opbrengsten zijn wanneer alle baten uit bezoekers komen. Hierdoor is ook goed te zien bij welke parkeertarieven de parkeergarage rendabel wordt. De parkeertarieven in Amsterdam zijn namelijk extreem hoog en de haalbaarheid van dit concept in andere plaatsen is minder snel bereikt. Het volgende hoofdstuk zal inzicht geven in de kosten versus de baten en daarmee de haalbaarheid van dit concept.

Netto contante waarde

70%/30%, dubbelgebruik bezoekers
Financiering door lening
Parkeertarief is €5,-/uur

Year	Exploitation costs	Payments (lending rate 10%)	Benefits	Cashflow	NCW (5,0%)
Engineering fase	€ 0,00	-2720000	€ 0,00	-€ 2.720.000,00	-€ 2.720.000,00
Construction year 1	€ 0,00	-2720000	€ 0,00	-€ 2.720.000,00	-€ 2.590.476,19
Construction year 2	€ 0,00	-2720000	€ 0,00	-€ 2.720.000,00	-€ 2.467.120,18
Exploitation year 1	-€ 764.000,00	-2720000	€ 3.276.294,00	-€ 207.706,00	-€ 179.424,25
Exploitation year 2	-€ 783.100,00	-2720000	€ 3.685.831,00	€ 182.731,00	€ 150.333,25
Exploitation year 3	-€ 802.677,50	-2720000	€ 4.095.368,00	€ 572.690,50	€ 448.717,99
Exploitation year 4	-€ 822.744,44	-2720000	€ 4.197.752,20	€ 655.007,76	€ 488.776,88
Exploitation year 5	-€ 843.313,05	-2720000	€ 4.302.696,01	€ 739.382,96	€ 525.465,66
Exploitation year 6	-€ 864.395,87	-2720000	€ 4.410.263,41	€ 825.867,53	€ 558.979,65
Exploitation year 7	-€ 886.005,77	-2720000	€ 4.520.519,99	€ 914.514,22	€ 589.504,02
Exploitation year 8	-€ 908.155,92	-2720000	€ 4.633.532,99	€ 1.005.377,07	€ 617.214,31
Exploitation year 9	-€ 930.859,81	-2720000	€ 4.749.371,31	€ 1.098.511,50	€ 642.276,92
Exploitation year 10	-€ 954.131,31	-2720000	€ 4.868.105,60	€ 1.193.974,29	€ 664.849,56
Exploitation year 11	-€ 977.984,59	-2720000	€ 4.989.808,24	€ 1.291.823,65	€ 685.081,66
Exploitation year 12	-€ 1.002.434,21	-2720000	€ 5.114.553,44	€ 1.392.119,24	€ 703.114,81
Exploitation year 13	-€ 1.027.495,06	-2720000	€ 5.242.417,28	€ 1.494.922,22	€ 719.083,15
Exploitation year 14	-€ 1.053.182,44	-2720000	€ 5.373.477,71	€ 1.600.295,27	€ 733.113,70
Exploitation year 15	-€ 1.079.512,00	-2720000	€ 5.507.814,65	€ 1.708.302,66	€ 745.326,79
Exploitation year 16	-€ 1.106.499,80	-2720000	€ 5.645.510,02	€ 1.819.010,22	€ 755.836,32
Exploitation year 17	-€ 1.134.162,29	-2720000	€ 5.786.647,77	€ 1.932.485,48	€ 764.750,12
Exploitation year 18	-€ 1.162.516,35	-2720000	€ 5.931.313,97	€ 2.048.797,61	€ 772.170,27
Exploitation year 19	-€ 1.191.579,26	-2720000	€ 6.079.596,81	€ 2.168.017,55	€ 778.193,35
Exploitation year 20	-€ 1.221.368,74	-2720000	€ 6.231.586,74	€ 2.290.217,99	€ 782.910,73
Exploitation year 21	-€ 1.251.902,96	-2720000	€ 6.387.376,40	€ 2.415.473,44	€ 786.408,84
Exploitation year 22	-€ 1.283.200,53	-2720000	€ 6.547.060,81	€ 2.543.860,28	€ 788.769,44
Exploitation year 23	-€ 1.315.280,55	-2720000	€ 6.710.737,33	€ 2.675.456,79	€ 790.069,80
Exploitation year 24	-€ 1.348.162,56	-2720000	€ 6.878.505,77	€ 2.810.343,21	€ 790.382,99
Exploitation year 25	-€ 1.381.866,63	-2720000	€ 7.050.468,41	€ 2.948.601,79	€ 789.778,03
Exploitation year 26	-€ 1.416.413,29	-2720000	€ 7.226.730,12	€ 3.090.316,83	€ 788.320,16
Exploitation year 27	-€ 1.451.823,62	-2720000	€ 7.407.398,37	€ 3.235.574,75	€ 786.070,98
Exploitation year 28	-€ 1.488.119,21	-2720000	€ 7.592.583,33	€ 3.384.464,12	€ 783.088,67
Exploitation year 29	-€ 1.525.322,19	-2720000	€ 7.782.397,92	€ 3.537.075,72	€ 779.428,15
Exploitation year 30	-€ 1.563.455,25	-2720000	€ 7.976.957,87	€ 3.693.502,62	€ 775.141,24
Exploitation year 31	-€ 1.602.541,63	-2720000	€ 8.176.381,81	€ 3.853.840,18	€ 770.276,82
Exploitation year 32	-€ 1.642.605,17	-2720000	€ 8.380.791,36	€ 4.018.186,19	€ 764.881,03
Exploitation year 33	-€ 1.683.670,30	-2720000	€ 8.590.311,14	€ 4.186.640,84	€ 758.997,31
Exploitation year 34	-€ 1.725.762,06	-2720000	€ 8.805.068,92	€ 4.359.306,86	€ 752.666,65

Exploitation year 35	-€ 1.768.906,11	-2720000	€ 9.025.195,64	€ 4.536.289,53	€ 745.927,64
Exploitation year 36	-€ 1.813.128,76	-2720000	€ 9.250.825,53	€ 4.717.696,77	€ 738.816,62
Exploitation year 37	-€ 1.858.456,98	-2720000	€ 9.482.096,17	€ 4.903.639,19	€ 731.367,81
Exploitation year 38	-€ 1.904.918,41		€ 9.719.148,58	€ 7.814.230,17	€ 1.109.977,66
Exploitation year 39	-€ 1.952.541,37		€ 9.962.127,29	€ 8.009.585,92	€ 1.083.549,62
Exploitation year 40	-€ 2.001.354,90		€ 10.211.180,47	€ 8.209.825,57	€ 1.057.750,82
				Total NPV	**€ 20.540.348,82**

Netto contante waarde

100% bezoekers
Financiering tijdens bouwfase
Parkeertarief is €1,46,-/uur

Year	Exploitation costs	Benefits	Cash flow	NPV (5,0%)
Engineering phase	-€ 5.000.000,00	€ 0,00	-€ 5.000.000,00	-€ 5.000.000,00
Construction year 1	-€ 10.000.000,00	€ 0,00	-€ 10.000.000,00	-€ 9.523.809,52
Construction year 2	-€ 14.310.000,00	€ 0,00	-€ 14.310.000,00	-€ 12.979.591,84
Exploitation year 1	-€ 764.000,00	€ 1.694.622,00	€ 930.622,00	€ 803.906,27
Exploitation year 2	-€ 783.100,00	€ 1.906.449,75	€ 1.123.349,75	€ 924.182,62
Exploitation year 3	-€ 802.677,50	€ 2.118.277,50	€ 1.315.600,00	€ 1.030.807,02
Exploitation year 4	-€ 822.744,44	€ 2.171.234,44	€ 1.348.490,00	€ 1.006.264,00
Exploitation year 5	-€ 843.313,05	€ 2.225.515,30	€ 1.382.202,25	€ 982.305,33
Exploitation year 6	-€ 864.395,87	€ 2.281.153,18	€ 1.416.757,31	€ 958.917,11
Exploitation year 7	-€ 886.005,77	€ 2.338.182,01	€ 1.452.176,24	€ 936.085,75
Exploitation year 8	-€ 908.155,92	€ 2.396.636,56	€ 1.488.480,64	€ 913.798,00
Exploitation year 9	-€ 930.859,81	€ 2.456.552,47	€ 1.525.692,66	€ 892.040,90
Exploitation year 10	-€ 954.131,31	€ 2.517.966,29	€ 1.563.834,98	€ 870.801,83
Exploitation year 11	-€ 977.984,59	€ 2.580.915,44	€ 1.602.930,85	€ 850.068,45
Exploitation year 12	-€ 1.002.434,21	€ 2.645.438,33	€ 1.643.004,12	€ 829.828,73
Exploitation year 13	-€ 1.027.495,06	€ 2.711.574,29	€ 1.684.079,23	€ 810.070,90
Exploitation year 14	-€ 1.053.182,44	€ 2.779.363,65	€ 1.726.181,21	€ 790.783,50
Exploitation year 15	-€ 1.079.512,00	€ 2.848.847,74	€ 1.769.335,74	€ 771.955,32
Exploitation year 16	-€ 1.106.499,80	€ 2.920.068,93	€ 1.813.569,13	€ 753.575,43
Exploitation year 17	-€ 1.134.162,29	€ 2.993.070,65	€ 1.858.908,36	€ 735.633,16
Exploitation year 18	-€ 1.162.516,35	€ 3.067.897,42	€ 1.905.381,07	€ 718.118,09
Exploitation year 19	-€ 1.191.579,26	€ 3.144.594,85	€ 1.953.015,59	€ 701.020,04
Exploitation year 20	-€ 1.221.368,74	€ 3.223.209,73	€ 2.001.840,98	€ 684.329,08
Exploitation year 21	-€ 1.251.902,96	€ 3.303.789,97	€ 2.051.887,01	€ 668.035,53
Exploitation year 22	-€ 1.283.200,53	€ 3.386.384,72	€ 2.103.184,18	€ 652.129,92
Exploitation year 23	-€ 1.315.280,55	€ 3.471.044,34	€ 2.155.763,79	€ 636.603,02
Exploitation year 24	-€ 1.348.162,56	€ 3.557.820,45	€ 2.209.657,88	€ 621.445,81
Exploitation year 25	-€ 1.381.866,63	€ 3.646.765,96	€ 2.264.899,33	€ 606.649,48
Exploitation year 26	-€ 1.416.413,29	€ 3.737.935,11	€ 2.321.521,81	€ 592.205,44

Exploitation year 27	-€ 1.451.823,62	€ 3.831.383,48	€ 2.379.559,86	€ 578.105,31
Exploitation year 28	-€ 1.488.119,21	€ 3.927.168,07	€ 2.439.048,86	€ 564.340,90
Exploitation year 29	-€ 1.525.322,19	€ 4.025.347,27	€ 2.500.025,08	€ 550.904,21
Exploitation year 30	-€ 1.563.455,25	€ 4.125.980,95	€ 2.562.525,70	€ 537.787,45
Exploitation year 31	-€ 1.602.541,63	€ 4.229.130,48	€ 2.626.588,85	€ 524.982,98
Exploitation year 32	-€ 1.642.605,17	€ 4.334.858,74	€ 2.692.253,57	€ 512.483,39
Exploitation year 33	-€ 1.683.670,30	€ 4.443.230,21	€ 2.759.559,91	€ 500.281,40
Exploitation year 34	-€ 1.725.762,06	€ 4.554.310,96	€ 2.828.548,90	€ 488.369,94
Exploitation year 35	-€ 1.768.906,11	€ 4.668.168,74	€ 2.899.262,63	€ 476.742,09
Exploitation year 36	-€ 1.813.128,76	€ 4.784.872,96	€ 2.971.744,19	€ 465.391,08
Exploitation year 37	-€ 1.858.456,98	€ 4.904.494,78	€ 3.046.037,80	€ 454.310,34
Exploitation year 38	-€ 1.904.918,41	€ 5.027.107,15	€ 3.122.188,74	€ 443.493,43
Exploitation year 39	-€ 1.952.541,37	€ 5.152.784,83	€ 3.200.243,46	€ 432.934,06
Exploitation year 40	-€ 2.001.354,90	€ 5.281.604,45	€ 3.280.249,55	€ 422.626,11
			Total NPV	**€ 190.912,10**

Netto contante waarde

100% bezoekers
Financiering door lening
Parkeertarief is €2,16,-/uur

Year	Exploitation costs	Payments (lending rate 10%)	Benefits	Cashflow	NVP (5,0%)
Engineering phase	€ 0,00	-2720000	€ 0,00	-€ 2.720.000,00	-€ 2.720.000,00
Construction year 1	€ 0,00	-2720000	€ 0,00	-€ 2.720.000,00	-€ 2.590.476,19
Construction year 2	€ 0,00	-2720000	€ 0,00	-€ 2.720.000,00	-€ 2.467.120,18
Exploitation year 1	-€ 764.000,00	-2720000	€ 2.507.112,00	-€ 976.888,00	-€ 843.872,58
Exploitation year 2	-€ 783.100,00	-2720000	€ 2.820.501,00	-€ 682.599,00	-€ 561.575,89
Exploitation year 3	-€ 802.677,50	-2720000	€ 3.133.890,00	-€ 388.787,50	-€ 304.625,18
Exploitation year 4	-€ 822.744,44	-2720000	€ 3.212.237,25	-€ 330.507,19	-€ 246.629,55
Exploitation year 5	-€ 843.313,05	-2720000	€ 3.292.543,18	-€ 270.769,87	-€ 192.431,09
Exploitation year 6	-€ 864.395,87	-2720000	€ 3.374.856,76	-€ 209.539,11	-€ 141.824,32
Exploitation year 7	-€ 886.005,77	-2720000	€ 3.459.228,18	-€ 146.777,59	-€ 94.614,14
Exploitation year 8	-€ 908.155,92	-2720000	€ 3.545.708,88	-€ 82.447,03	-€ 50.615,33
Exploitation year 9	-€ 930.859,81	-2720000	€ 3.634.351,61	-€ 16.508,21	-€ 9.652,01
Exploitation year 10	-€ 954.131,31	-2720000	€ 3.725.210,40	€ 51.079,09	€ 28.442,75
Exploitation year 11	-€ 977.984,59	-2720000	€ 3.818.340,66	€ 120.356,06	€ 63.827,39
Exploitation year 12	-€ 1.002.434,21	-2720000	€ 3.913.799,17	€ 191.364,97	€ 96.652,31
Exploitation year 13	-€ 1.027.495,06	-2720000	€ 4.011.644,15	€ 264.149,09	€ 127.060,23
Exploitation year 14	-€ 1.053.182,44	-2720000	€ 4.111.935,26	€ 338.752,82	€ 155.186,57
Exploitation year 15	-€ 1.079.512,00	-2720000	€ 4.214.733,64	€ 415.221,64	€ 181.159,83
Exploitation year 16	-€ 1.106.499,80	-2720000	€ 4.320.101,98	€ 493.602,18	€ 205.101,90
Exploitation year 17	-€ 1.134.162,29	-2720000	€ 4.428.104,53	€ 573.942,23	€ 227.128,43
Exploitation year 18	-€ 1.162.516,35	-2720000	€ 4.538.807,14	€ 656.290,79	€ 247.349,10

Exploitation year 19	-€ 1.191.579,26	-2720000	€ 4.652.277,32	€ 740.698,06	€ 265.867,91
Exploitation year 20	-€ 1.221.368,74	-2720000	€ 4.768.584,25	€ 827.215,51	€ 282.783,52
Exploitation year 21	-€ 1.251.902,96	-2720000	€ 4.887.798,86	€ 915.895,90	€ 298.189,42
Exploitation year 22	-€ 1.283.200,53	-2720000	€ 5.009.993,83	€ 1.006.793,30	€ 312.174,29
Exploitation year 23	-€ 1.315.280,55	-2720000	€ 5.135.243,68	€ 1.099.963,13	€ 324.822,16
Exploitation year 24	-€ 1.348.162,56	-2720000	€ 5.263.624,77	€ 1.195.462,21	€ 336.212,67
Exploitation year 25	-€ 1.381.866,63	-2720000	€ 5.395.215,39	€ 1.293.348,76	€ 346.421,29
Exploitation year 26	-€ 1.416.413,29	-2720000	€ 5.530.095,77	€ 1.393.682,48	€ 355.519,53
Exploitation year 27	-€ 1.451.823,62	-2720000	€ 5.668.348,17	€ 1.496.524,54	€ 363.575,13
Exploitation year 28	-€ 1.488.119,21	-2720000	€ 5.810.056,87	€ 1.601.937,66	€ 370.652,25
Exploitation year 29	-€ 1.525.322,19	-2720000	€ 5.955.308,29	€ 1.709.986,10	€ 376.811,64
Exploitation year 30	-€ 1.563.455,25	-2720000	€ 6.104.191,00	€ 1.820.735,75	€ 382.110,83
Exploitation year 31	-€ 1.602.541,63	-2720000	€ 6.256.795,77	€ 1.934.254,14	€ 386.604,29
Exploitation year 32	-€ 1.642.605,17	-2720000	€ 6.413.215,67	€ 2.050.610,50	€ 390.343,55
Exploitation year 33	-€ 1.683.670,30	-2720000	€ 6.573.546,06	€ 2.169.875,76	€ 393.377,40
Exploitation year 34	-€ 1.725.762,06	-2720000	€ 6.737.884,71	€ 2.292.122,65	€ 395.751,97
Exploitation year 35	-€ 1.768.906,11	-2720000	€ 6.906.331,83	€ 2.417.425,72	€ 397.510,93
Exploitation year 36	-€ 1.813.128,76	-2720000	€ 7.078.990,13	€ 2.545.861,36	€ 398.695,55
Exploitation year 37	-€ 1.858.456,98	-2720000	€ 7.255.964,88	€ 2.677.507,90	€ 399.344,86
Exploitation year 38	-€ 1.904.918,41		€ 7.437.364,00	€ 5.532.445,59	€ 785.860,01
Exploitation year 39	-€ 1.952.541,37		€ 7.623.298,10	€ 5.670.756,73	€ 767.149,06
Exploitation year 40	-€ 2.001.354,90		€ 7.813.880,55	€ 5.812.525,65	€ 748.883,60
				Total NPV	**€ 187.133,90**

O Costs floating parking garage

Oosterdok, floating construction,vierlaags, 132 x 79 meter, exclusief toeritten, drijvend				
Voorbereidende werkzaamheden				
Inrichten werkterrein	EUR	1	€ 10.000,00	€ 10.000,00
Tijdelijke voorzieningen				
Toepassen golfbreker	st	1	€ 30.000,00	€ 30.000,00
Extra voorzieningen positionering Flexbase vloer	st	1	€ 25.000,00	€ 25.000,00
Flexbase vloer				
EPS250, vier lagen van 200 mm, glasfolie versterkt	m2	10.428		
Lijm, 4 lagen	m2	41.712		
Wokkels, 4 st/m2	st	42.000		
Installatie EPS, 4 lagen	m2	41.712		
Eerste vloer, dik 250 mm, C28/35, incl. bekisting	m3	2.607		
EPS60, dik 1000 mm	m3	7.689		
EPS200, randen tweede laag	m3	754		
Installatie EPS, 2 lagen	m2	16.888		
Balkenrooster, hoog 1000 mm, breed 400 mm	m3	2.088		
Omega profielen, bekisting balken	st	85		
Tweede vloer, dik 250 mm, C28/35, incl. bekisting	m3	2.607		
Ballast tegen vervormen vloer tijdens stort	st	2		
Betonwerken				
Aanbrengen tussenvloer, dik 500 mm	m3	4.902		
Aanbrengen beton wanden, dik 600 mm	m3	1.772		
Aanbr. beton dekconstructie, dik 500 mm	m3	4.902		
Aanbrengen hellingsbanen, dik 500 mm	m3	670		
Aanbrengen kolommen	st	450		
Aanbrengen afwerklaag	m2	20.856		
Aanbr. beton vloer, dik 500 mm	m3	4.902		
Totaal directe kosten				**€ 13.643.415,22**