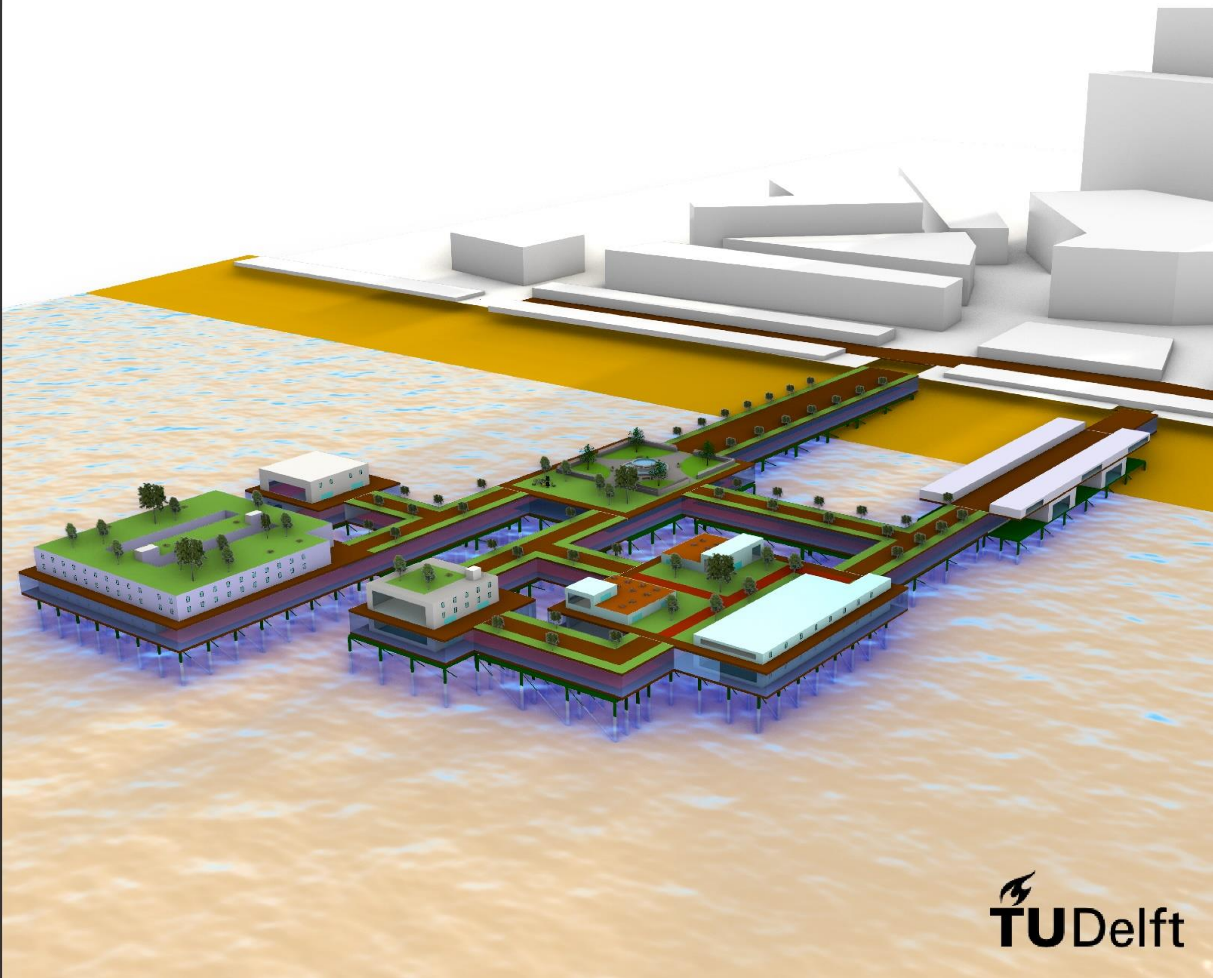
A New Design for the

SCHEVENINGEN PIER

R.E. VALERIANO



TU Delft

A New Design for the Scheveningen Pier

MSc. Thesis

By

R.E. Valeriano

in partial fulfilment of the requirements for the degree of

Master of Science

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Preface

This thesis, named “A New Design for the Scheveningen Pier”, is written in fulfilment of the requirements of the degree Master of Science in Civil Engineering. This is within the department of Hydraulic engineering, with specialisation of Hydraulic Structures and Flood Risk. This thesis has been undertaken at the Delft University of Technology.

This subject was proposed by Dr.Ing. M.Z. Voorendt via the network of the TU Delft. My interest is in hydraulic structures and I have always been fascinated by floating cities on water. Even though the Scheveningen Pier is not an entire city, I saw this thesis as an opportunity to research the possibilities for cities to expand onto the water. In 2020 I approached Dr.Ing. Voorendt to discuss the possibilities to start a master thesis with this subject. In September 2020, I started writing a thesis proposal, where the objective and the methodology were set. In October my kick-off meeting took place to officially start my master thesis.

Unfortunately, my master thesis took place during the COVID-19 pandemic, causing me to work from home. However, I had great support from Dr.Ing. Voorendt, who made time to provide me with helpful feedback almost weekly, and the other members of my graduation committee: Dr.Ir. B.Hofland and Dr. J.R.T. Van der Velde. For that I would like to thank the members of my graduation committee. On top of that, I would also like to thank my friends and family for their support during my thesis.

Raoul Eriberto Valeriano

Delfgauw, June 11th 2021

Summary

The current owners of the Scheveningen Pier are starting to head into the second phase of their exploitation plan, which is to replace the current pier. This new pier is larger than the current pier and can exploit at a higher level.

This thesis creates a design for a new Scheveningen Pier. A design method is used, which combines the engineering design method and the spatial design method. This combined design method consists of two design loops. One design loop for the complete design and the functional / spatial design and one design loop for the technical aspects of the design. These technical aspects are the morphology, hydrodynamics and the structural mechanics.

The design has to take the requirements, criteria and boundary conditions into account. A stakeholder analysis has been performed. From this a set of requirements and criteria is created. The relevant boundary conditions which are researched in this thesis are the bathymetry of the seabed, the soil, the water level of the sea and the height of incoming waves.

The first design that is made is the functional / spatial design. The new pier will consist of multiple platforms, each containing different kinds of facilities. This creates a good separation of functions. All the platforms can be reached on foot or by bike. The plan contains two floors of which one outdoor and one indoor. To get to this design, multiple concepts are made. Most of these concepts are verified with the requirements. After that, the remaining alternatives are compared with each other in a multi-criteria analysis. This analysis supports the final decision on the functional / spatial design.

After this decision, the technical design loop starts. This begins with the morphological design loop. Multiple alternatives were made for the structure of the pier. For all these alternatives, the impact on the surrounding beach is being researched with Delft3D, a process-based coastal model. The result of this analysis is that an open construction has the least negative impact on the surrounding beach of Scheveningen.

This conclusion is used to start with the structural design loop. A pile construction will be used to support the pier deck, since this is an open construction and therefore has the least negative impact on the beach. The structure consists of a deck and piles, which go from the pier deck to around 10 meters into the seabed. The deck consists of beams and plates which transfer the loads from above to the piles. The piles and the elements of the deck are made out of concrete and contain shear and tensile reinforcement. Semi-probabilistic structural calculations are performed on the structure of the pier to verify the dimensioning of the elements.

The pier will be constructed using a jack-up barge with spud piles. This is a platform with piles attached to it, so it can anchor itself into the seabed. Because of this, the platform will not drift away during construction. From this platform it is possible to place the piles, which will be Fundex piles.

In the end of this thesis, a total integrated design has been made. This design accomplishes the goal of this thesis and suffices the requirements and scores good on the criteria. However, to get to a fully functional and realisable design, more detail needs to be added to the design and the calculations. Some design steps should also be revised because new information is gathered over the course of this thesis. The design method used in this thesis is functional, but a more effective design method is given in the end. These changes are based on the experiences gathered in this thesis.

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1 Introduction

This chapter explains the motivation of this study. It explores the current situation with a landscape analysis and it sets up an objective for this thesis.

1.1 Motivation of the study

The Scheveningen Pier has been around for more than 65 years. It has always been a landmark for Scheveningen and the city of The Hague. But in 2013, the Pier went bankrupt and was closed for visitors. A year later in 2014, De Pier BV., consisting out of Kondor Wessels Vastgoed and Danzep BV bought the Pier and brought it back to life. They created a two phase plan for the pier. The first phase was planned for the first five years of the pier after its reopening, where the current pier has been refurbished and has been put into use. The pier is a landmark, accessible for everyone which contains facilities for a variety of audiences. Examples of these facilities are bars, restaurant, shops, a hotel, an indoor playground and a Ferris wheel.

According to the municipality of The Hague, the pier is very important to the economy of city (Revis, 2018). It attracts tourists from the Netherlands and from abroad. After the big reopening in 2014, it attracted around two million visitors in the first year (Revis, 2018). This improvement of the tourism sector creates many employment opportunities for the people of The Hague as well.

In 2017, De Pier BV. sent a letter to the municipality to notify them that they want to start with the development of the second phase of the plan for the pier. While phase one was more oriented on the short term, which is five years, the second phase of the plan focusses on the long term. They want to use their experience from the first phase to come up with a new plan for the pier, which is economically beneficial for The Hague and is supported by the community.

This thesis makes a new design for the pier for this second phase, so for the long term. It should comply with the vision of the municipality of The Hague and De Pier BV.

1.2 Problem exploration with a landscape analysis

For the problem exploration a landscape analysis has been performed. The following is the diagnosis of this landscape analysis

The pier is located in Scheveningen. This is a part of city of The Hague which is situated on the Dutch West coast. The location of Scheveningen in the Netherlands is displayed in Figure 1.

Figure 1: Location of Scheveningen in the Netherlands (Google, 2020)

Before the Scheveningen was built, the location consisted of a dune landscape. When the Scheveningen was built, it started as a recreational area. Since then it started expanding becoming a part of the big city The Hague. Over the years Scheveningen has been expanding towards the beach as well.

The pier is located in a recreational area which is surrounded by nature and accommodation. Therefore people from the surrounding area and visitors like to visit the pier to have a good time. A new pier should keep this function or improve on it. The Pier itself offers a large variability of different functions. This should be remained in a new design. The pier does contain many walkways. People stay in transition a lot. People are able to stand still and enjoy the view, but there are not many public spaces on the pier for this function.

The current pier and the boulevard are not well aligned in terms of style. The pier has a rather different style and atmosphere in comparison to the boulevard. The boulevard is wide and modern, while the pier is relatively small and has relatively old aesthetics.

The full landscape analysis can be found in Appendix A. Photos, which were taken at the location are found in Appendix B.

The pier is situated in the North sea, which brings challenges with it. A new pier needs to withstand the loads coming from this environment. In extreme scenarios, the water level could increase with more than 5 meters above the average sea level, and wavs in extreme scenarios can reach over 7 meters.

1.3 Thesis objective

The objective of this thesis is to make a new design for the pier of Scheveningen which fits the vision of the city, has a positive impact on the surrounding area in the long term and can be exploited at a high level. This design is visualised with models and drawings and is backed up with calculations and analyses. Although this is an integrated project and many factors are taken into account. The main focus of this thesis is on the technical and hydraulic aspects of the new pier, since this is more in line with the curriculum of the Master Hydraulic Engineering. The main aspects which are researched and designed in this thesis are.

- Functional/spatial design
- Morphological design
- Hydrodynamical design
- Structural design

The idea of this thesis is to make a new Scheveningen Pier to replace the old pier. However, it is not excluded that the new design could reuse the old pier.

1.4 Reading guide

Chapter 2 explains the methodology of this thesis. It starts with explaining the engineering design method, the spatial design method and the combined design method which will be used in this thesis to design a new Scheveningen Pier. The order of the chapters in this thesis is not exactly the same as the order of the design steps in the methodology. This is to create a better overview in the report.

After the methodology, Chapter 3 creates the set of requirements and criteria and determines the boundary conditions of this project. These are both used in later chapters, where the design is made.

Chapter 4 creates the functional / spatial design of the pier. Multiple alternatives are made which are verified and then evaluated. At the end of Chapter 4 a final decision is made on the functional / spatial plan of the new Scheveningen Pier.

The technical design loop is performed in Chapter 5 and Chapter 6. The morphology is treated in Chapter 5. This chapter explains how the morphological analysis is performed and gives the result of this analysis. The structure and the construction method is treated in Chapter 6. The dimensioning of the different elements of the structure is explained in this chapter. The total design of the new Scheveningen Pier is given in Chapter 7.

The final chapter, Chapter 8, reviews the process of this thesis. It gives recommendations on how this design can be improved to make it more feasible. On top of that, Chapter 8 gives recommendations on how the design method from Chapter 2 can be improved for possible other projects.

The appendices of this thesis mainly contain calculations, elaborations on the analysis and visualisations.

2 Methodology

This project contains both technical and functional design aspects. These have different design methods which are both explained briefly in this chapter. After that, the design method followed in this thesis is explained.

2.1 Engineering design method

The new Scheveningen Pier has similar aspects to a hydraulic structure, because it is located at sea. It is not a hydraulic structure, because the goal is not to influence the water or facilitate shipping. Usually, hydraulic structures have one main goal. They focus on solving one main problem, for example retaining or crossing water. A design for this type of structure is often acquired by the design cycle of Roozenburg and Eekels (1995). This cycle starts with an analysis of the problem which results in criteria which are used for the synthesis of provisional designs or concept designs. These designs are then being simulated to know what to expect from it and to see how they perform. After that, they are evaluated with criteria gained from the analysis. Usually a multi-criteria analysis (MCA) or a cost-benefit analysis is used for this, and the result of this analysis can help make a final decision on the chosen design. It could happen that new information is gained during the process. If this is the case, previous design steps should be revised, because his new information could alter the results of previous steps. But overall, this method is a relatively linear process.

The abduction for the engineering method is better defined in comparison to the abduction of the spatial design method. Abduction is the step of giving shapes and materials to ideas. For the engineering method this is more simple because there are physical laws and formulas on how given shapes and materials will behave. The result can be calculated or modelled.

2.2 Spatial design method

Besides a technical design, this project also needs a spatial design. This however, requires a different approach than the standard design cycle used by Roozenburg and Eekels (1995). Spatial design often has ill-defined problems, which could complicate the project. One of the methods mentioned by Voorendt (2017) is the method of Lawson and Dorst (2009). Their method does not follow a certain path like the engineering method does. They distinguish five different activities, which have to be repeated multiple times, to come to a design. These steps are formulating, representing, moving, evaluating and managing.

The abduction of the spatial design method is relatively less defined than in the engineering method. This is explained well by Dorst (2011).

In his paper, Dorst (2011) describes the challenges that come with design. First, he mentions a common equation used in different kinds of design.

What + How = Value

In this equation, “*What”* refers to the physical end product. “*How”* is the working principle behind the design and “*Value”* is the function which is fulfilled by the end product. Dorst (2011) uses this to explain the different kinds of abduction. Abduction-1 is the most conventional and is commonly used in technical designs. In this type of abduction, “*How”* and “*Value”* are known. So therefore, only the “*What”* has to be specified by designers.

The second type of abduction is abduction-2. This one is more complex than the first type, because in this abduction, only the value is known. Therefore, a working principle needs to be discovered and a design has to be made.

As an answer to this rather complex system, Dorst (2011) uses frames. He explains this as the connection between the working principle and the Value. " IF we look at the problem situation from this viewpoint, and adopt the working principle associated with that position, THEN we will create the value we are striving for"(Dorst, 2011).

This project has both a technical and a spatial aspect. The technical aspect of the design needs an abduction-1. The value or goal this part wants to achieve is structural safety and protection from the sea. The working principles are clearly stated in design codes and the laws of nature. However, the spatial and functional aspects are not as well defined as the technical aspects. A value for this part can be stated, but the working principles are not set in stone. Therefore this is an abduction-2, which is common for functional design methods. So for this part a frame is needed.

2.3 Design method used for the new pier of Scheveningen

Nowadays it is common to build structures on water which serve other purposes than only crossing or retaining water. Examples of this are the Hong Kong International Airport or the Palm islands in Dubai. These are man-made islands which provide space for different functions. A pier has the same goal. It is supposed to be well protected against the loads from the sea, but it should also make space for accommodation, recreation and businesses. Voorendt (2017) describes a design method which can be used for designing multifunctional flood defences. It combines the engineering approach and the spatial approach, using elements from both in the integrated design method. This design method, with multiple alterations, has been used to create a new Scheveningen Pier in this thesis. Although this thesis has an integrated design, it is still a thesis from the department of hydraulic structures from the faculty of civil engineering. That is why the technical specifications and calculations are elaborated in more detail than the spatial plan.

Figure 2 gives a visualisation for the proposed design method. There are 9 main steps in this process.

1. Problem exploration
2. Development of concepts
3. Analysis
4. Functional specification
5. Verification of concepts
6. Evaluation of concepts
7. Technical design cycle (Figure 3)
8. Integrations of subsystems
9. Validation

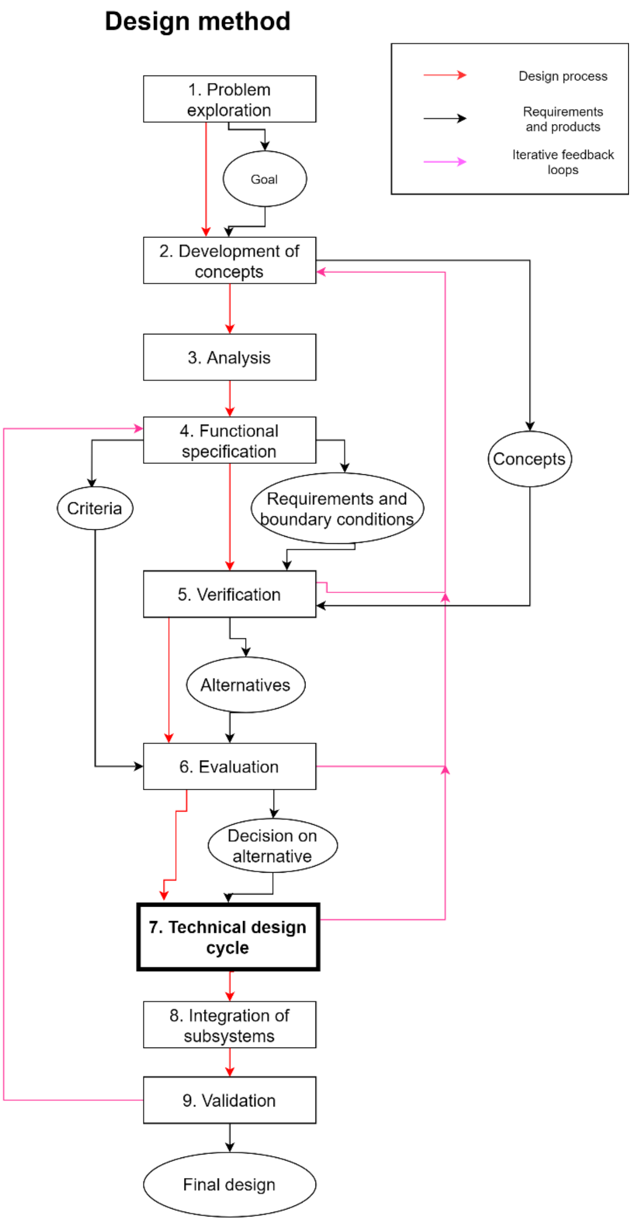


Figure 2: Flowchart of the combined design method.

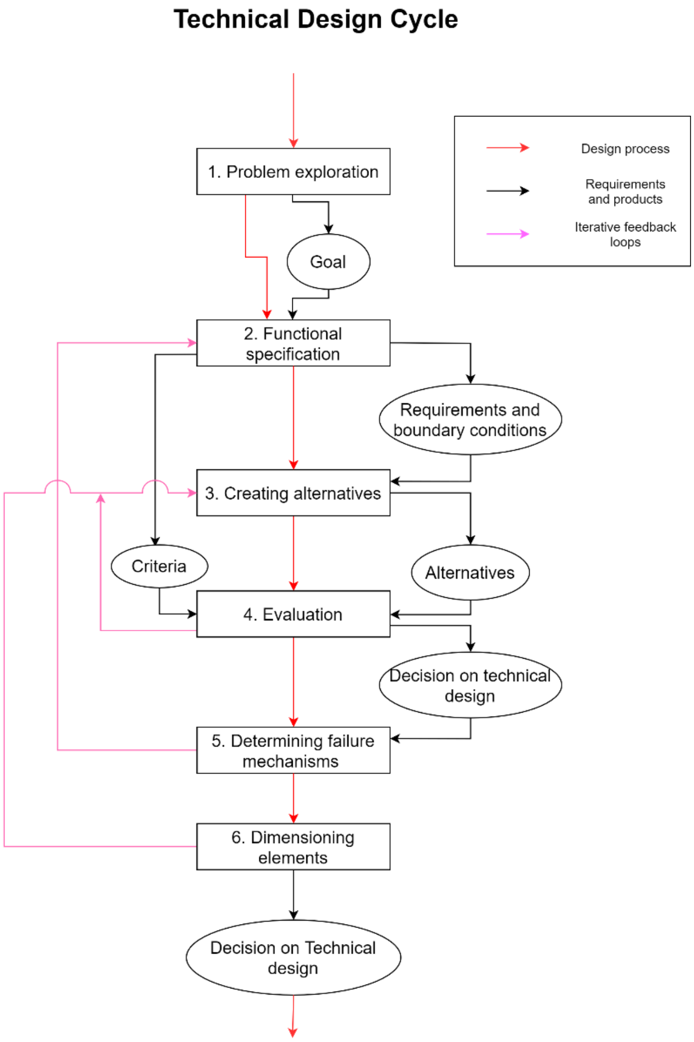


Figure 3: Flowchart of the technical design cycle.

The first step is the problem exploration which results in a goal for which a new Pier has been designed. This is followed by step 2, the development of concepts. Concept designs are made, where the functional design and the hydraulic design are already integrated with each other. After the concepts are developed, more analysis is required to be able to set up a set of requirements, criteria and boundary conditions. These are then used to verify and evaluate the different concepts. After the evaluation, one concept is chosen to proceed with. This concept is further designed in the technical design cycle. This technical cycle needs to be repeated for the morphological, hydrodynamical and structural design. The structure and the construction method of the new pier are designed in the technical design cycle as well. This step has been more elaborated in comparison to the other steps, because in this thesis the focus is more on the technical aspects than on the spatial aspects. Therefore this part of the design method has its own design cycle. After this step, the design gets more detail when the subsystems are integrated. The final step is the validation of the chosen design. A more accurate explanation of these design steps are given in Appendix C.

Two different design loops can be distinguished from each other. The first design loop is step 1 to 9, which puts more emphasis on the functional and spatial design. This loop is displayed in Figure 2. The second design loop is the technical design loop, which is step 7 in the main design loop. This design cycle will be repeated for the morphology, hydrodynamics and the structural design. The engineering method, mentioned in 2.1, will be used for the technical design cycle. The technical design cycle is displayed in Figure 3.

This process seems linear. However, it is necessary to review previous steps. Therefore iterative feedback loops have been added to the design method.

As mentioned before, the method which is used in this thesis is similar to the method of Voorendt (2017). However, there are several differences between the two methods. The main difference can be found in the separation of the two different design loops. Voorendt (2017) has one design loop, which is repeated multiple times for the different aspects of the design. However, in the method which is be used in this thesis, there is a clear separation between the functional aspects and the technical aspects. This can be seen in the fact that in the method used for this thesis, there is a separate design cycle in step 7, the technical design cycle.

In Section 2.2 it was mentioned what a frame is. To be able to set up this frame, the desired values should be clarified. First the target group needs to be specified. The target group are the people that make use of the new pier. These are tourists and new residents of the pier. They have their different values. Tourists value a good time and new residents value a good home environment.

A frame for the tourists is to have many recreational possibilities for different groups of tourists. The assumption that is made here is that tourists aim to have diverse activities when they visit the pier and that they can visit it with a diverse group of people with different interests. For example a family with elderly, who would like to enjoy the view, and young children who would prefer more interactive activities.

IF we look at the problem situation from the viewpoint of the tourists, and adopt having many diverse recreational possibilities, THEN we will create the value we are striving for.

A frame for new residents is to have a safe and tranquil home environment. The assumption that is made here is that residents want a place for themselves, where they can rest and unwind.

IF we look at the problem situation from the viewpoint of new residents, and adopt creating a safe and tranquil home environment, THEN we will create the value we are striving for.

These frames help to describe how the desired values for the different target audiences can be accomplished.

3 Functional specification

This chapter describes step 3 and step 4 of the design method which has been described in Chapter 2 and Appendix C. These design steps are the analysis and the functional specification. The results of these design steps are the boundary conditions and the set of requirements and criteria.

3.1 Stakeholder analysis and set of requirements and criteria

This section makes the set of requirements and criteria. The stakeholders of this project, as well certain criteria from the Eurocodes are analysed to create the set of requirements and criteria.

3.1.1 Stakeholder analysis

Different parties are involved in this new project. Appendix E explains these various stakeholders in more depth. Table 1 gives a summary of all the stakeholders, what their interests are and how they are treated in this project.

Table 1: A summary of the stakeholders, their interests and how they are treated in this project

Stakeholder	**Interest**	**Treatment**	**Requirement or criterion**
Client	• Make revenue • Have low cost • Easy to maintain • Exploitation at high level • Usable for the long term • The new pier must be an improvement in comparison to the old pier	Manage closely	Requirement
Municipality	• Increase the touristic value of the area • Should be an icon • Should have public support • Accessible to everyone	Manage closely	Requirement
Local residents	• No nuisance during construction and usage period	Keep informed	Criterion
Local businesses	• Become more attractive • Be accessible for people	Keep informed	Criterion
New residents of the pier	• Safe and calm environment • Accessible • No nuisance during the usage period	Keep informed	Criterion
New businesses on the pier	• Become attractive • Become accessible	Keep informed	Criterion
Tourists	• Have a variety of recreational activities • Have a good view on the beach and the sea	Monitor	Criterion
New Hotel on the pier	• Become attractive • Become accessible • Being separated from tourists	Keep informed	Criterion
New conference venues	• Space for a conference venue • Being separated from the tourists • Become accessible	Keep informed	Criterion
Environmental organisations	• Nature should remain intact	Keep informed	Criterion

3.1.2 Design lifetime and reliability

The design life time of the new Scheveningen Pier and the required reliability of the structure supporting the pier are determined in this subsection.

Lifetime

The new Scheveningen Pier is a monument to the city of The Hague. According to the Eurocode (Nederlands Normalisatie-instituut, 1990), monumental buildings belong to design lifetime class 5, which is the highest class. Because the pier is also a monument, the pier is assigned to lifetime class 5 as well. This results in a design life time of 100 years.

Reliability

The Scheveningen Pier contains buildings which include public buildings. It is expected that a large number of people will visit the pier. A structural collapse of the Scheveningen Pier will lead to a large loss of life and a large loss of economic value as well. According to the Eurocode (Nederlands Normalisatie-instituut, 1990), a structure which has large consequences with respect to loss of life and to economic values should be assigned to consequence class 3 (CC3), which is the highest class possible. Examples for buildings which are in this consequence class are concert halls and tribunes. The number of visitors for the new Scheveningen Pier is expected to be in the same magnitude as these examples or even more. This is another argument for why the new Scheveningen Pier belongs in CC3.

According to the Eurocode (Nederlands Normalisatie-instituut, 1990), the consequence classes and the reliability classes are directly related to each other, which means that a structure in CC3 also belongs to reliability class 3 (RC3). This means that the Scheveningen Pier belong to RC3. The reliability index for 1 year for the Scheveningen Pier is 5.2.

$$\beta_{1\ year} = 5.2$$

This gives the following yearly failure probability:

$$P_{f,1\ year} = \Phi(-\beta) = \Phi(-5.2) = 10^{-7}$$

3.1.3 Set of requirements and criteria

The new design has to suffice the requirements. These requirements often come from the stakeholders, with relatively much power in the project, or from the design codes like the Eurocode. If any of the requirements is not met, it is not feasible to realise the design. The criteria are used to evaluate the different alternatives. These are often given by the stakeholders. The landscape analysis from Appendix A has also been used to create the set of requirements and criteria.

Because there is a difference between the functional and the technical design, requirements and the criteria are also separated in functional and technical parts.

Functional specification

Requirements for the functional/spatial design

- The new pier must increase the touristic value of the Scheveningen beach resort.
- The new pier must include accommodation.
- The new pier must have a hotel.

- The new pier must have recreational functions.
- The new pier must have space for conference venues.
- The new pier must be accessible to everyone.
- The new pier must be accessible throughout the year.
- The new pier must be exploitable for the long term.
- The new pier must be an improvement in comparison to the current pier.

Criteria for the functional/spatial design

- The new pier should be as accessible as possible.
- The new pier should cause low nuisance to the environment and to the new facilities on the new pier.
- The new pier should fit in with the environment and the boulevard.
- There should be a good separation between the transitional routes and the functional areas.
- The new pier should have a good view of the sea and the beach, and it should not deteriorate the view from other perspectives.
- The new Pier must be visually pleasing to its users and its surroundings.
- The new pier should interrupt the beach as little as possible.
- The natural environment should be damaged as little as possible.
- There should be much space for recreational possibilities.
- A large part of the facilities should be usable throughout the year.

Requirements for the technical design

- The yearly failure probability of the pier is 10^{-7}.
- The design life time of the pier is 100 years.
- The pier must have sufficient strength, stability and stiffness.
- The height of the pier should be sufficient enough so that is does not get flooded.
- The coastal defence cannot be deteriorated due to the construction of the pier.
- The beach and the sea must be safe for visitors to visit.

Criteria for the technical design

- The construction of the new pier should be as cheap as possible.
- The construction of the new pier should be time-efficient.
- New beach area which can be used for recreation is desired.
- The beach width should not be too large so that the walking distance to the sea would be too large.
- As little erosion as possible should occur due to construction of the pier.
- The flow velocities in the sea near the coast should not decrease the swimmer safety.

3.2 Determining the boundary conditions

The relevant boundary conditions for the calculations of the pier are given in Table 2. This section explains how these conditions were determined.

Table 2: Relevant boundary conditions

Bed level at the end of the pier	NAP – 4 m
Soil type	Sand
Extreme high water level	NAP + 5.66 m

Design wave height	7.728 m

3.2.1 Bathymetry of the seabed

The new Scheveningen Pier is almost 500 meters long, of which 300 meters into the sea. The largest bottom depth at the end of the Scheveningen Pier is around NAP – 4m according to Figure 4 and Figure 5.

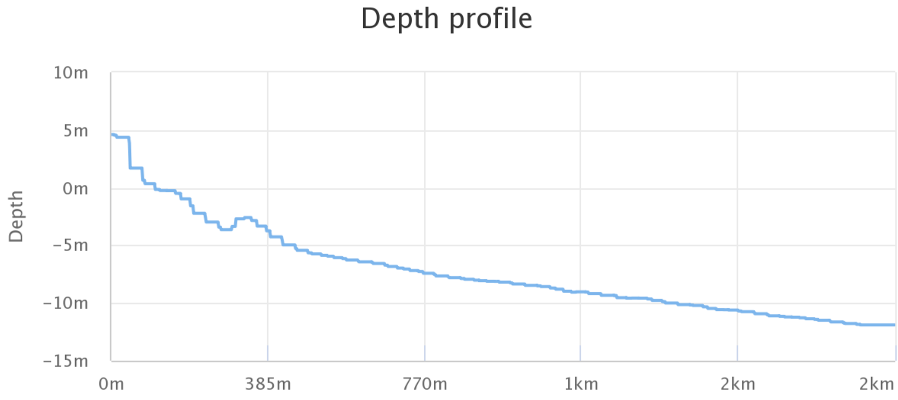


Figure 4: Bathymetry length profile of the coast near Scheveningen. Positive x-direction is in the offshore direction. 0m on the X-axis is on the on-shore side of the green line of Figure 5 (EMODnet, 2020).

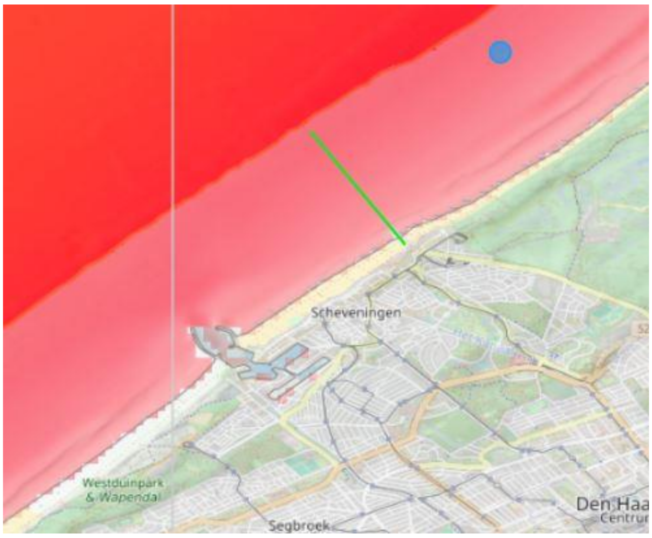


Figure 5: Location of the length profile from Figure 4, Indicated with the green line. x = 0 is located on the beach (EMODnet, 2020).

3.2.2 Soil analysis

A cone penetration test (CPT) has been used to determine the soil properties of the project area. The soil mainly consists of sand. At a depth of NAP – 7m, a clay layer can be observed and at NAP – 18m a peat layer is observed. However, these are relatively thin layers. For further calculations, the soil is assumed to consist of sand only. More explanation about the soil analysis is given in Appendix T.

3.2.3 Determining the design water level

From the scenarios for which the water level is used in the calculations, the highest value of the reliability factor is $\beta = 5.7$ and for a dominant load $\alpha_s = -0.70$. Key figures (Rijkswaterstaat, 2013) for the water level at Scheveningen were used to determine a design water level. This results in a water level of NAP + 4.48m.

To determine the design water level, the distribution of the water level needs to be known. It is assumed that the water level at Scheveningen has a lognormal distribution. The key figures (Rijkswaterstaat, 2013) for the water level show a similar distribution. These key figures show the water levels in Scheveningen and their respective return period. A logarithmic fit through these points is given in Figure 6.

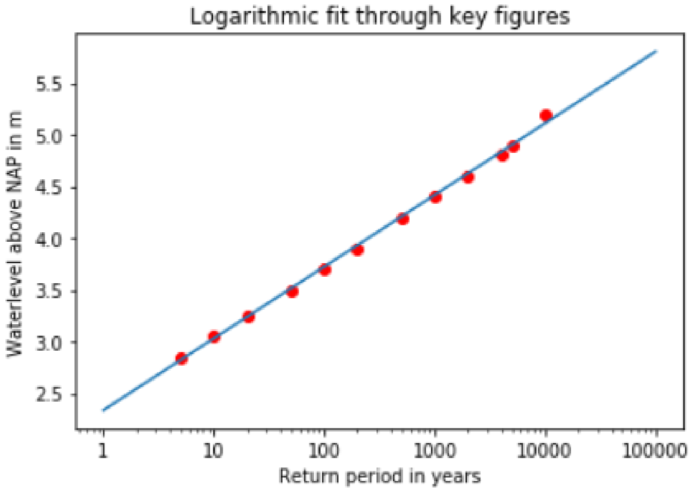


Figure 6: Logarithmic fit through data for the water level. Data from the key figures (Rijkswaterstaat, 2013)

The key figures (Rijkswaterstaat, 2013) are used to determine the distribution. Exceedance probability and the water levels are given. The exceedance frequency is equal to the exceedance probability, and the value of the cumulative distribution function can then be determined.

With python, a lognormal distribution has been fitted through the data points. This results in a lognormal distribution with the following parameters.

$$\mu = NAP + 2.586 \text{ m}$$

$$\sigma = 0.357 \text{ m}$$

$$V = \frac{\sigma}{\mu} = 0.138$$

The water level is a dominant load in governing load situations. This mean, $\alpha = -0.70$. The fitted distribution is illustrated in Figure 7.

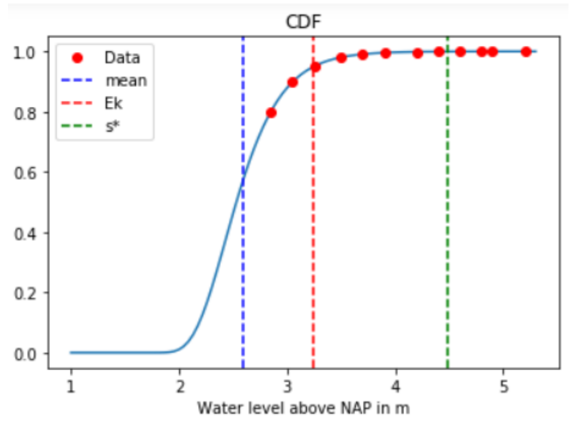


Figure 7a : Fitted cumulative density function for the water level at Scheveningen.α_s = -0.70, β= is 5.7

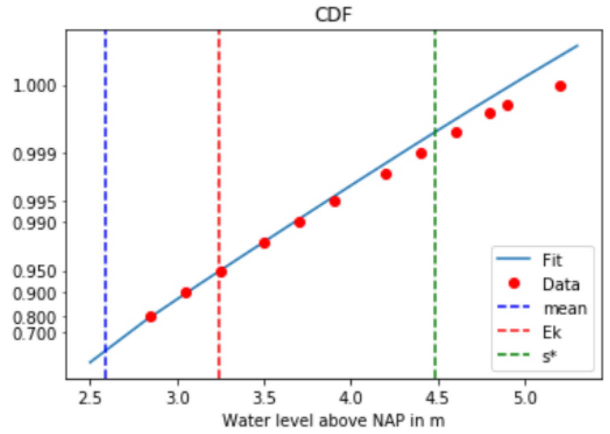


Figure 7b: Fitted cumulative density function for the water level at Scheveningen.α_s = -0.70, β= is 5.7, zoomed in at the end on logarithmic scale.

With the distribution, the characteristic value and the design value can be determined. The design value depends on the reliability index β and the factor α.
Figure 8 illustrates how the design water level depends on the reliability factor β.

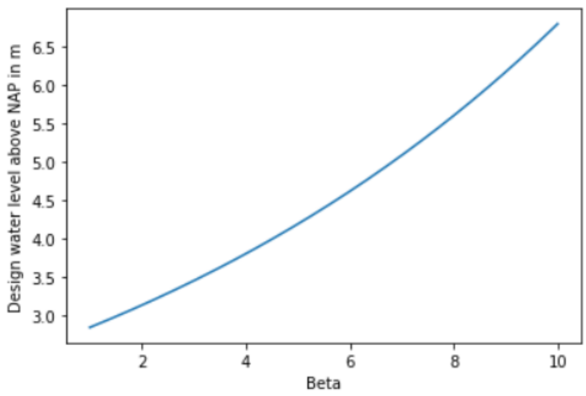


Figure 8: Design water level dependent of β for α_s = 0.70

From the scenarios for which the water level is used in the calculations, the highest $\beta = 5.7$ and for a dominant load $\alpha_s = -0.70$. This results in a water level of NAP + 4.48m. An additional 1.178 m is added to this water level to account for climate change. This is based on the climate scenarios from the KNMI. The design should assume a scenario with warm temperatures and a high change of the air flow pattern. A further elaboration of the sea level rise due to climate change is given in Appendix Q. The design water level and the addition of climate change results in a water level of NAP + 5.66 m.

Only for the failure mechanisms where the deck is hit by the water, which are overtopping and waves hitting the deck from below, has a higher reliability factor than 5.7. But no calculations are performed for this failure mechanism, this failure mechanism only determines the height of the pier deck.

The exact calculations for the design water level can be found in Appendix R.

3.2.4 Determining the design wave height

The design wave height is calculated at 8.74 m. But the waves break before they reach this height, therefore the design have height is dependant of the water depth instead. This results in a design wave height of 7.728 m for the design water level of NAP + 5.66 m.

To determine the design wave height, a time series has been analysed with python. This data was gathered via Rijkswaterstaat (Rijkswaterstaat, 2020).The data has been taken at the Europlatform.

The timespan is from 01-01-2000 to 31-12-2018. To reduce the computing time, the daily maxima have been used for this analysis instead of every measurement. Only the storm waves are taken into account. For this, a threshold has been set-up at a wave height 250 cm. This means that waves below this threshold are be taken into account. The significant wave height is average of the highest third of the storm waves. This results in:

$$H_s = 3.97 \text{ m}$$

The design wave height is the wave height with a 10% exceedance probability (Rijkswaterstaat, 2018). For this, a Rayleigh distribution is assumed for the significant wave height. In Dutch conditions the following can be assumed.

$$H_d = 2.2 \cdot H_S = 8.74 \text{ m}$$

Figure 9 shows how the significant wave height and the design wave height relate to the data.

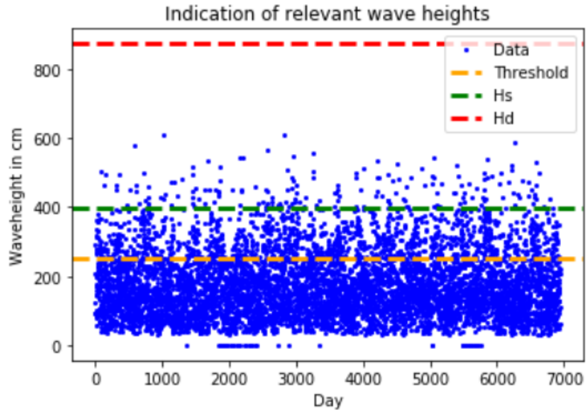


Figure 9: Indication of the relevant wave heights

The design wave height is equal to 8.74 m. However, the waves break before they the design height. The wave height is therefore be dependant of the water depth instead.

$$\gamma = 0.8$$

$$H_b = \gamma \cdot d = 0.8 \cdot d$$

$$H_d = \min \{8.74 ; 0.8 \cdot d\}$$

More elaborate calculations on the design wave height can be found in Appendix R.

Unfortunately an error has been made during these wave calculations. The calculation methods for long term and short term wave statistics are not properly used. A recommendation would be to redo these calculations. For the remainder of this thesis, it is assumed that the design wave height will be equal to the breaking wave height.

4 Functional / spatial design

The goal of this chapter is to create a functional design for the new pier of Scheveningen. First, concepts are created in Section 4.1. These concepts are rough ideas which are supported by simple hand sketches. The concepts are then worked out in more detail and are verified with the requirements, which were made in Chapter 3, in Section 4.2. If the concepts are verified, they become alternatives. These alternatives will then be evaluated in Section 4.3 and a final decision on the spatial plan is made in Section 4.4.

This chapter executes step 2, step 5 and step 6 of the main design cycle, described in Chapter 2 and Appendix C. These steps are the development of concepts, the verification and the evaluation of the spatial design.

4.1 Development of concepts

This section will show which concepts have been created for the spatial plan of the new Scheveningen Pier.

4.1.1 Used methods of concept creation

To come up with ideas, three different methods were used. These are looking at reference projects, brainstorming and using the diagnosis of the landscape analysis (Van der Velde, 2020)

Inspiration has been obtained by studying different reference projects from around the world. Other piers have been analysed, but also different and diverse structures have been researched to come up with new ideas for concepts. While looking at other projects, the following questions were asked:

- What is the goal of this project ?
- What are the surroundings of this project ?
- What different parts does this project consist of ?
- Which parts of this project could also be used for a Scheveningen Pier ?

Besides studying reference projects, brainstorming with different ideas has led to development of certain concepts as well. Systems or ideas that come to mind are used in the concepts.

The final method is the landscape analysis. In the methodology, it is stated that the analysis takes place after the development of concepts. This is to not be limited by any requirements when thinking of concepts. However, a landscape analysis has been performed in the problem exploration in Chapter 1, because this analysis can help with developing concepts. The results of the different parts of the landscape analysis are used to create concepts.

As mentioned before, multiple concepts are made. This section describes each one.

4.1.2 Spatial Concepts

In this subsection, concepts are given for entire systems, which means the whole design is taken into account. Appendix D contains multiple concepts which can be used as a subsystem, but these are not complete designs.

Concept 1: Ring dam

The first concept is made out of separate parts, which are connected. These separate parts are illustrated in Figure 10. The outer ring is the part which distinguishes this concept from other piers. This ring consists of concrete caissons, which have a double function as buildings. These buildings can contain recreation or accommodations. On the outer side of these buildings, an outer dam is placed to reduce the impact of waves arriving at the pier. On the inside of this ring, there is a wooden platform which functions as a walk way and a cycling path. If the water level on the inside of the ring is low enough, a jetty can be added to the buildings, so that people can enjoy the water. A cross-section of this outer ring is displayed in Figure 11.

The next part of this design is another ring, which is located on the inside. This is a combination of a walkway and a park. Nature is added here to give a green atmosphere to the design.

The innermost part of the design resembles a more traditional pier. A wide walkway leads from the beach to a centre platform in the middle of the design. This wide walkway offers space for pedestrians and cyclists. The traffic flows are separated with the addition of a cycling path. The sides of this walkway contains buildings as well, these could function as shops or restaurants. To make the pier fit the environment, this part takes over the same modern style and aesthetics of the boulevard.

As mentioned, a round platform is situated at the end of the walkway, in the middle of the design. These two parts are situated higher than the outer ring, the dam ring, so that a view of the sea is still possible from the centre platform and the wide walkway. Figure 12 displays the height differences between the separate parts. The centre platform is the central plaza of the design. It contains a small park in the middle and several terraces which belong to the restaurants and cafes. These restaurants and cafes are located in the lower floors of the buildings on the side of the central plaza. The reason buildings are only placed on the sides and not on the front, is so that people can still enjoy a view of the sea while walking on the pier. The upper floors of these building is reserved for apartments.

Advantages

- Separation of functions
- Possibility for cycling
- Contains nature
- Many outdoor possibilities
- Load reduction on the inner part
- Could be integrated with the boulevard

Disadvantages

- Few indoor possibilities
- Extra dam structure required
- Ring dam could have a big influence on the beach

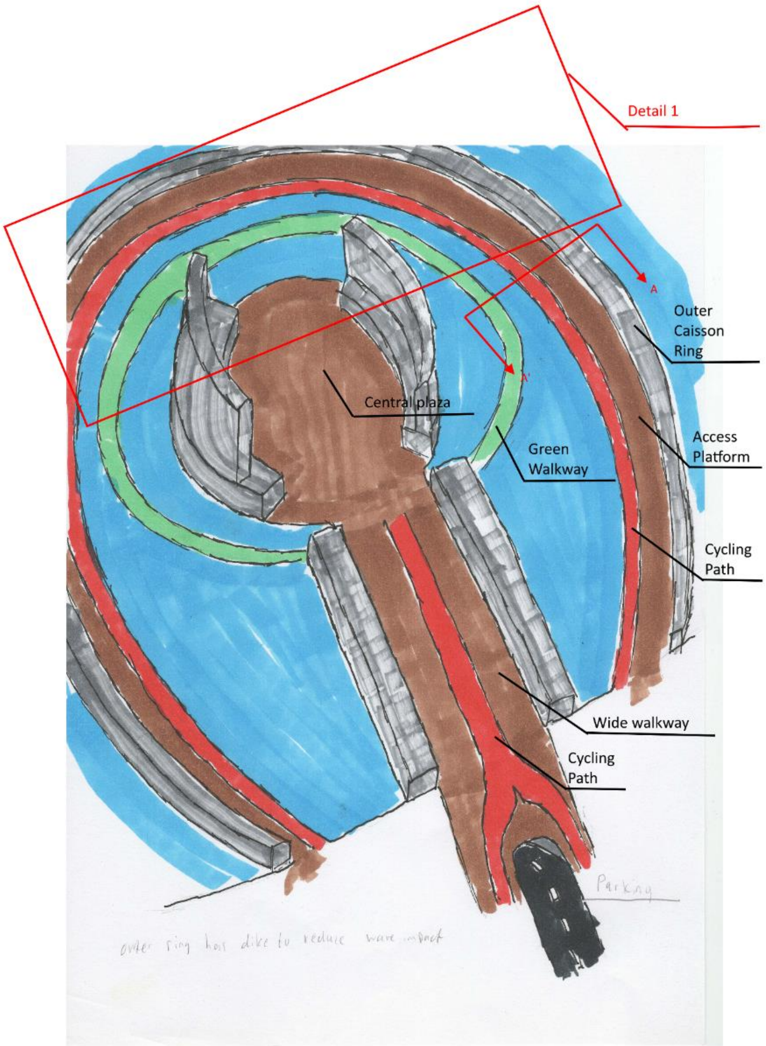


Figure 10: Sketch of concept 1

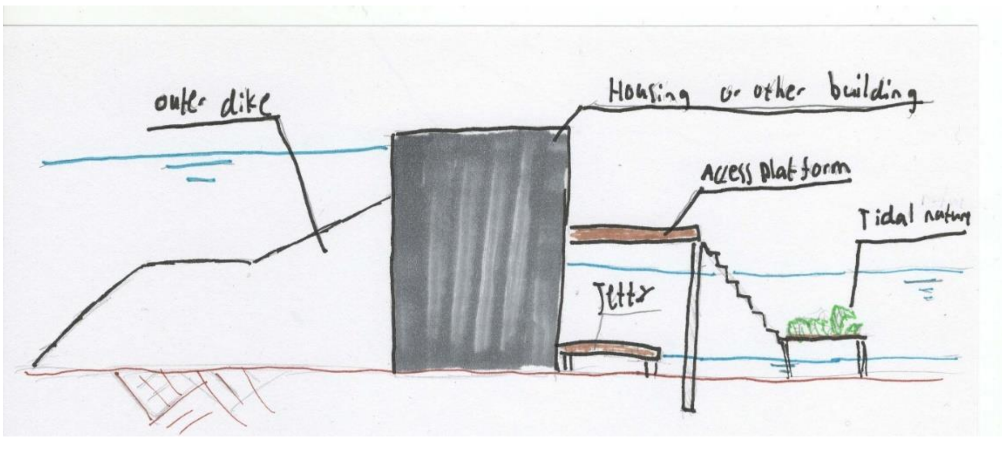


Figure 11: Cross-section of the outer ring of concept 1

Figure 12: Detail 1 impression. End of the pier of concept 1

Concept 2: Underwater caisson

This concept explores the possibility of placing a caisson, or multiple ones if necessary, in the sea which functions as a new pier. This caisson reaches the sea bed, which is estimated between at around NAP - 4 m. The insides of these caissons are used as well to accommodate different functions. A closed caisson with only the view of concrete is not a pleasant experience for future users, therefore structural glass is used to offer views to the outside. Figure 13 displays the main idea behind this concept.

The lower floors are reserved for accommodations or possibly a hotel. The middle part of the caisson is reserved for recreational purposes. Shops and different eateries are placed on this floor, on multiple levels. This part is similar to a shopping mall and is located above the average sea level. This allows for a good view of the horizon from this floor, close to the water level. However, the water level could increase and then this floor is partly below sea level. An impression of the different floors has been given in Figure 14.

The top of the caisson is used for recreation. Multiple restaurants with terraces are situated here so that people can enjoy the outdoors. Nature is added to create a park atmosphere.

A second caisson could be placed if more functions are desired. Offices, accommodation or a hotel for example would prefer an environment which is separated from the recreative touristic part and a second caisson is a good solution for this issue.

Advantages

- Separation of functions
- Contains nature
- Many indoor possibilities
- Functions are integrated with the construction
- Obvious visible landmark

Disadvantage

- Residential accommodation below sea level
- Closed environment
- Caissons have a large influence on the beach morphology

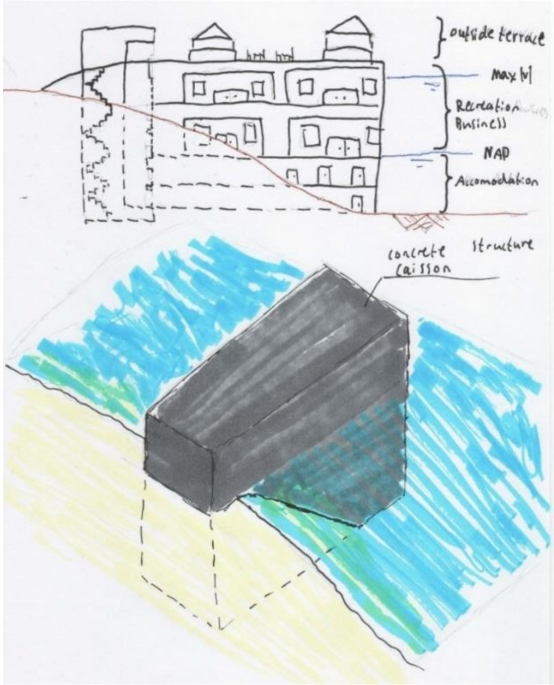


Figure 13: Sketch of concept 2

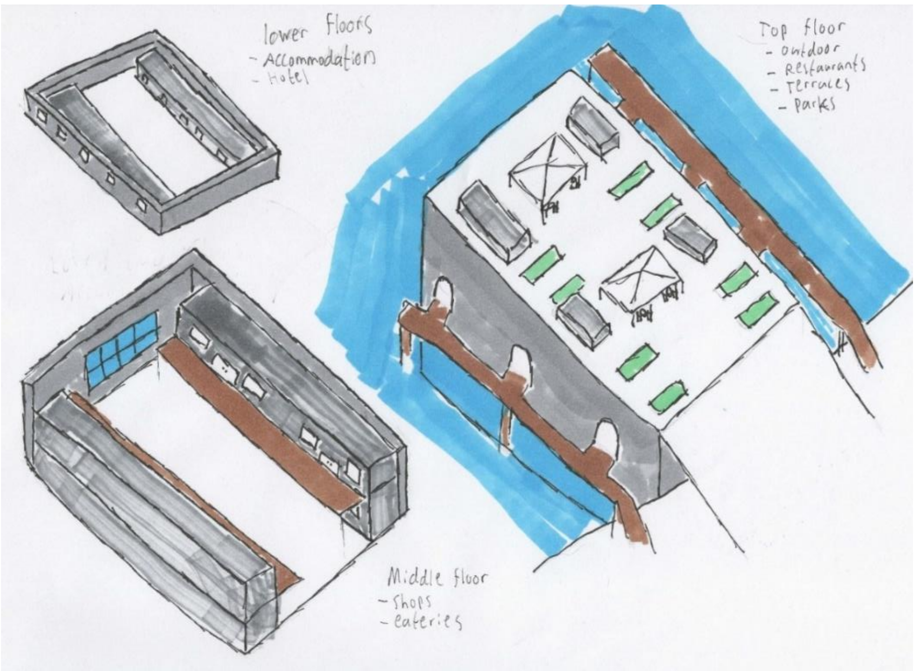


Figure 14: Different floors of concept 2

Concept 3: Multiple islands concept

An important part of the diagnosis of the landscape analysis is that most of the public areas of the pier and of the boulevard are transition areas. There are few public spaces where people can rest or stand still. This concept contains multiple islands, among which are multiple plazas which function as public spaces. All the different islands are connected with walkways. The walkways are used as transition areas, while the different platforms are used for buildings or public spaces. This causes a proper separation between transition areas and the other functions. An impression is given in Figure 15.

This concept is split in two main parts, to have a separation of the different functions. The left part of Figure 15 is more focussed on the recreation. This contains a central square, and islands for shops and eateries. A playground is also present for children. The right part of Figure 15 is for more private audiences. This includes a hotel and a business park. In the middle, accommodation is placed, so that this part is connected with the recreational part, and with the private part.

Advantages

- Good separation of functions
- Many public spaces
- Separation between transition areas and public spaces
- Possibility to expand with new islands.
- Using piles reduces impact on beach transformation

Disadvantages

- No indoor areas
- Many different structures are needed

Figure 15: Sketch of concept 3

Concept 4: Expansion of the beach

The recreational part of Scheveningen has been increasingly utilizing the beach over the course of recent history, according to the morphological historical analysis mentioned in Chapter 1. This concept tries to continue with that expansion. The current beach is often used by tourists, so placing new buildings there would remove or deteriorate this function. Therefore the beach is expanded. The middle of this new patch contains a new boulevard. This is elevated to be able to withstand the increasing water levels. This has the shape of a dike, similar to the boulevard of Scheveningen. Buildings are placed on this dike as well, to accommodate functions like shops, eateries or a hotel. The sides of this strip remains a beach for people to enjoy. Beach sport facilities are placed to stimulate activity. Figure 16 gives an impression of this concept.

Advantages

- More beach space
- Connects well with the boulevard

Disadvantages

- A new beach has to be supplemented

- The new beach will erode away over time

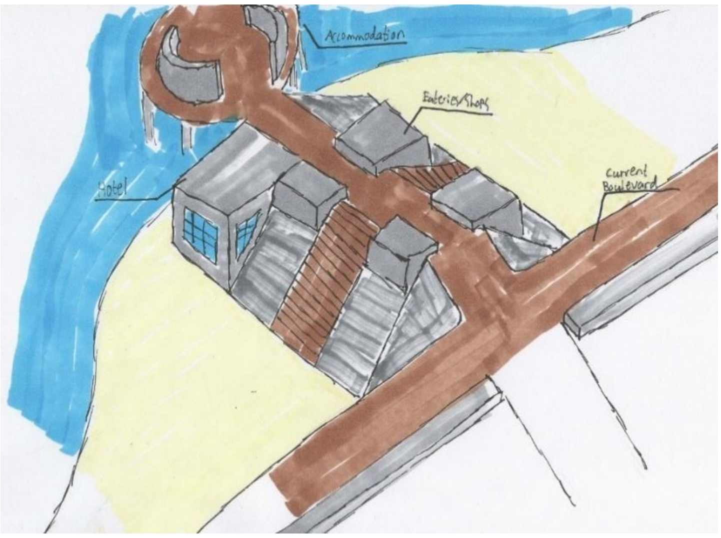


Figure 16: Sketch of concept 4

4.2 Verification of the concepts

In this section, the concepts, which were created in Section 4.1, are being verified with the requirements, which were set in Subsection 3.1.3.

Before the concepts are verified, they need to become more realistic. In Section 4.1, the concepts were very undetailed and not to scale. To be able to verify the concepts, they need to become more detailed. Every concepts has been modelled in software programme Sketchup, which makes it possible to create scale models. In these models, the different functions and transition routes need to be present as well, to so see how much space it would take.

To get a good idea of the dimensions, nearby structures and paths have been analysed in Google Maps to see what their dimensions are. These are given in Table 3.

Table 3: Rough measurements of elements in the surroundings, measured with Google Maps (Google, 2020)

Element	**Length in meters**
Width of walking space of the boulevard near the pier	25
Width of walking space of the boulevard further from the pier	10 – 35
Width of the cycling path (two ways)	5
Width of the car road (two lanes)	6
Width of the current pier	15
Length of the current pier	370
Length of a restaurant on the boulevard	10 (excl. terrace) 15 (incl. terrace)
Width of a restaurant on the boulevard	10 m
Apartment complex above the shopping mall	25 x 65

All the concepts are better illustrated in Appendix F, where sections and dimensions are also given.

4.2.1 Concept 1: Ring dam

This concept contains a pier with a central island and a dam ring surrounding it, separating the water inside of the ring with the open sea, which is outside of the ring. An overview of this concept is given in Figure 17.

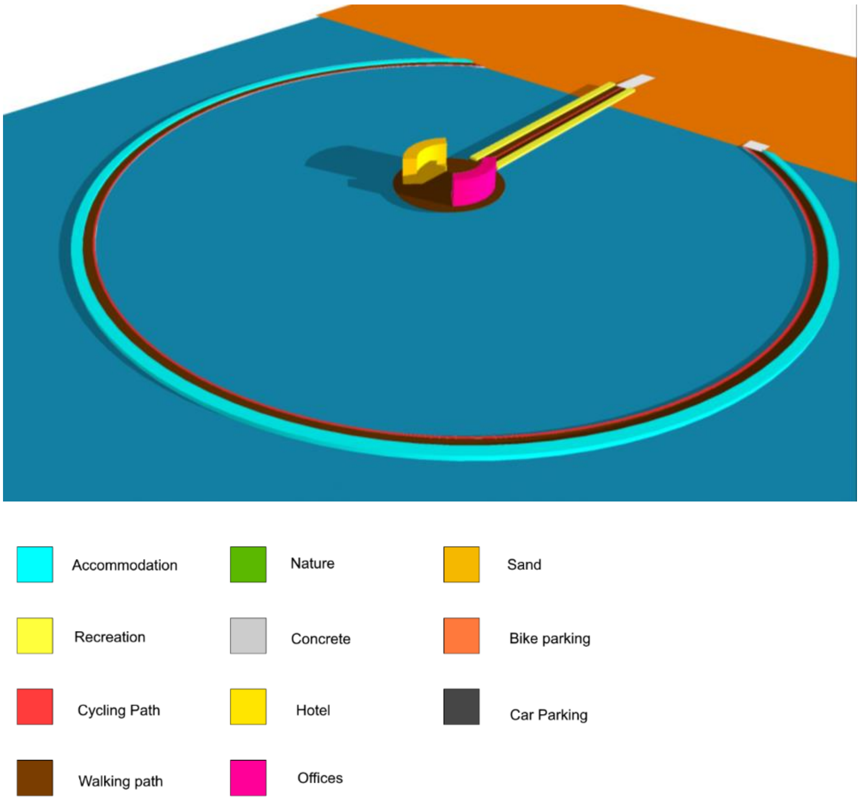


Figure 17: Concept 1, bird's eye perspective

The pier is 500 meters long and 65 meters wide, containing recreational functions on the sides. The central platform hosts a conference venue and a hotel. The outer ring, with an inner radius of 600 meters, contains accommodations, which are integrated in the ring dam.

This concept was created in the development of concepts stage of the design plan. However, when this plan was made into a model, it seemed less realistic, because this ring dam would be relatively large. Besides this, the advantages of a ring dam, namely secluding the inside from the outside does not add much benefits. It is also expected that this ring dam has a large impact on the surrounding beach. Therefore, this concept is not elaborated any further.

4.2.2 Concept 2: Underwater caisson

This concept consists of a large rectangular caisson and a circular island which is connected to it. This island is also a caisson. The caissons contain functions on the outside and the inside. The rectangular caisson is 22 meters high, 514 meters long and 64 meters wide. The circular caisson has a height of 22 meters as well. The radius is 100 meters. These dimensions have not been structurally verified yet, so therefore these dimensions can change later in the design process. Figure 18 shows an interpretation of how concept 2 looks. This concept tries to build on one of the good aspects from the previous pier and tries to improve on the lesser aspects of it. Just like the current pier, there are multiple levels which create different atmospheres, so that visitors can choose which part they would like to experience. This concept is wider than the original pier, creating more space and making it less crowded than the original pier. The other dimensions are also larger, which offers more space for recreation and other functions on this pier.

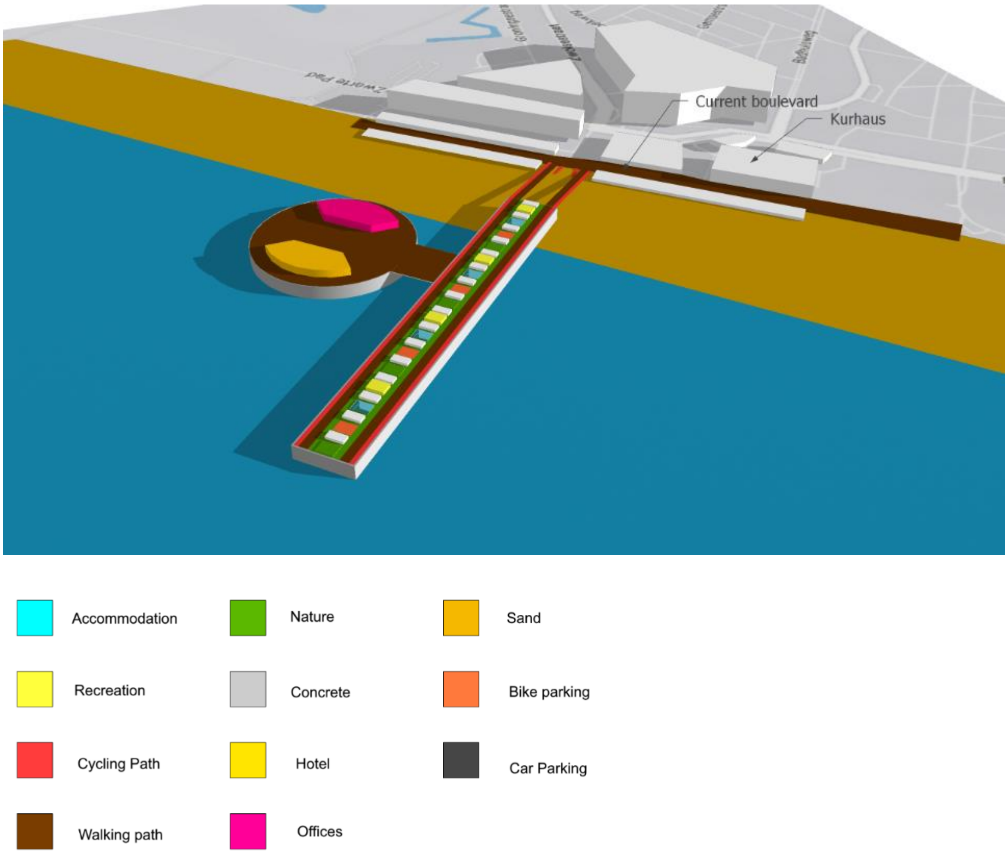


Figure 18: Concept 2, bird's-eye perspective

The top floor is located on top of both caissons, so it is located outdoors. The rectangular caissons contains a walkway and a cycling path on both sides. Nature is added on this floor to create a more tranquil environment for the people who would like to enjoy the view of the sea and the beach. Some recreational venues are placed here, but it is not as crowded as the floor below. Bike parking has been added throughout the upper floor so that people can access the inside on multiple locations, using the elevator shafts. The circular caisson contains the entrances of the conference venue and the hotel.

Before the lower floors are explained, a cross-section is shown to give a good view of how the different floors and levels in this design are oriented. Figure 19 shows a cross section of the rectangular caisson, displaying the ground level and various water levels. This section is taken around the end of the caisson, where the water depth is around 5 meters. More onshore parts have a more elevated ground level, but the caisson remains the same. Different floors can be seen, these are all connected by elevator shafts, which go from the top floors to the lower floors.

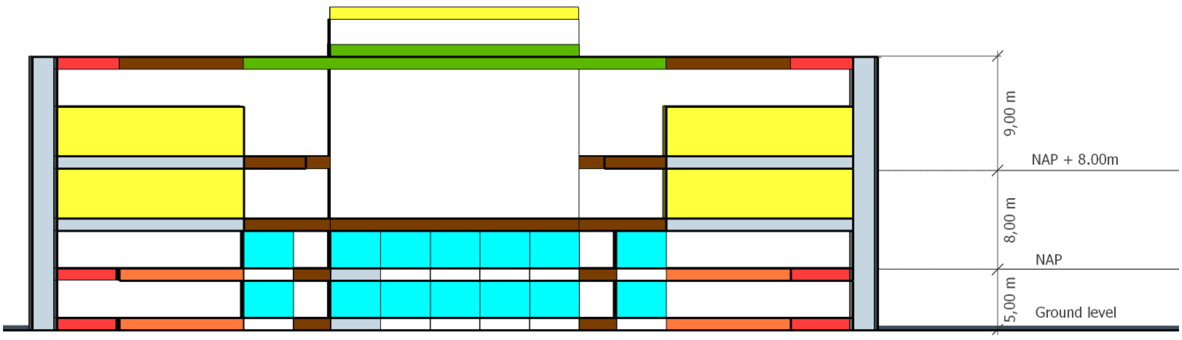


Figure 19: Concept 2, cross-section AA' with water levels displayed

The middle floor is located inside the caissons. The middle floor, which is mainly used for recreation, is situated above NAP. However, when high water levels occur, this floor can be partly below the water level. The rectangular caisson contains recreation on this floor. This floor has space for two stories of recreation. These recreations can be eateries or shops. The circular platform contains space for the hotel and the conference venue on this floor. The entrance shall be on the top floor, but most space is available on the middle floor.

The two lowest floors of the rectangular caisson contain space for accommodation. In this concept, there is space for 7168 m^2 per floor for accommodation. This can be 224 studio apartments of 32 m^2, which is how the model is currently divided. There are two floors of this which results in 448 apartments. But it would also be possible to create less apartments with a larger area. The sides of these floors contain cycling paths so that the residents of the accommodations can reach their houses without walking too large distances. There is space for residents to park their bikes. Between the houseblocks, there is a staircase and large elevator shaft. The lowest floor of the circular island does not contain anything in the current design.

4.2.3 Concept 3: Multiple islands

This concept consists of multiple islands, each containing a different function. In the current model, it looks like the platforms are floating, because a structure has not been designed for it yet. This is done in a later design stage. In contrary of the other concepts, this design can be supported by different kinds of structures. An overview of the design can be seen in Figure 20.

Since the second half of the 19th century, Scheveningen has been growing and expanding as a beach resort. This alternative continues with that growth and expansion. The city expands over the sea. This is an iconic situation and will attract more people.

In the landscape analysis of the current pier, it was mentioned that there is not a good separation between the transition areas and the functional areas. This concept has been designed to address that issue. All the different islands contain the functions and these different islands have been separated by walking paths. Therefore the transition areas and the functional areas are well separated.

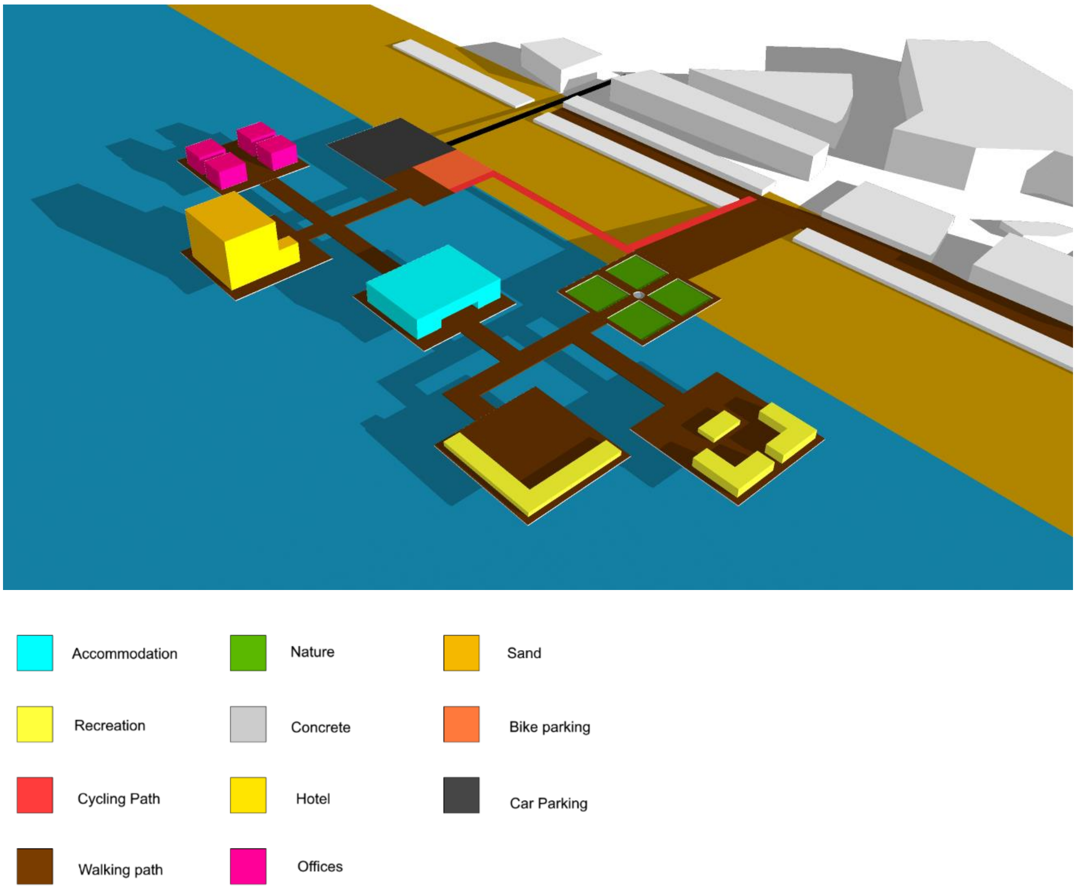


Figure 20: Concept 3, bird's-eye perspective

The whole design is around 690 meters wide and 320 meters long. The height of the platforms is to be determined in the technical design cycle, but the platforms need to stay high enough, so that they remain dry, even during extremely high water levels or waves. The different platforms are all square platforms of 100 meters wide and long. They are connected by walkways which are 20 meters wide.

The north-east part is more focussed on functions which desire more exclusivity. These platforms contain a hotel, a conference venue and accommodation. Near the beach there is a parking space for these exclusive functions.

4.2.4 Concept 4: Expansion of the beach

This concept is an extension of beach, creating something similar to the Zandmotor. In the middle, there is an extension of the boulevard, which has the same shape as a dike. On this dike there is space for recreation and a hotel. At the end of the dike, there is a circular platform to house accommodation and a conference venue. An overview of this design is given in Figure 21.

This concept was created with the thought of expanding on the beach, just like it was done throughout the history of Scheveningen. This new path would then be an extension of the boulevard, creating a better unity between the pier and the boulevard, contrary to the current situation, where the pier and the boulevard are very different.

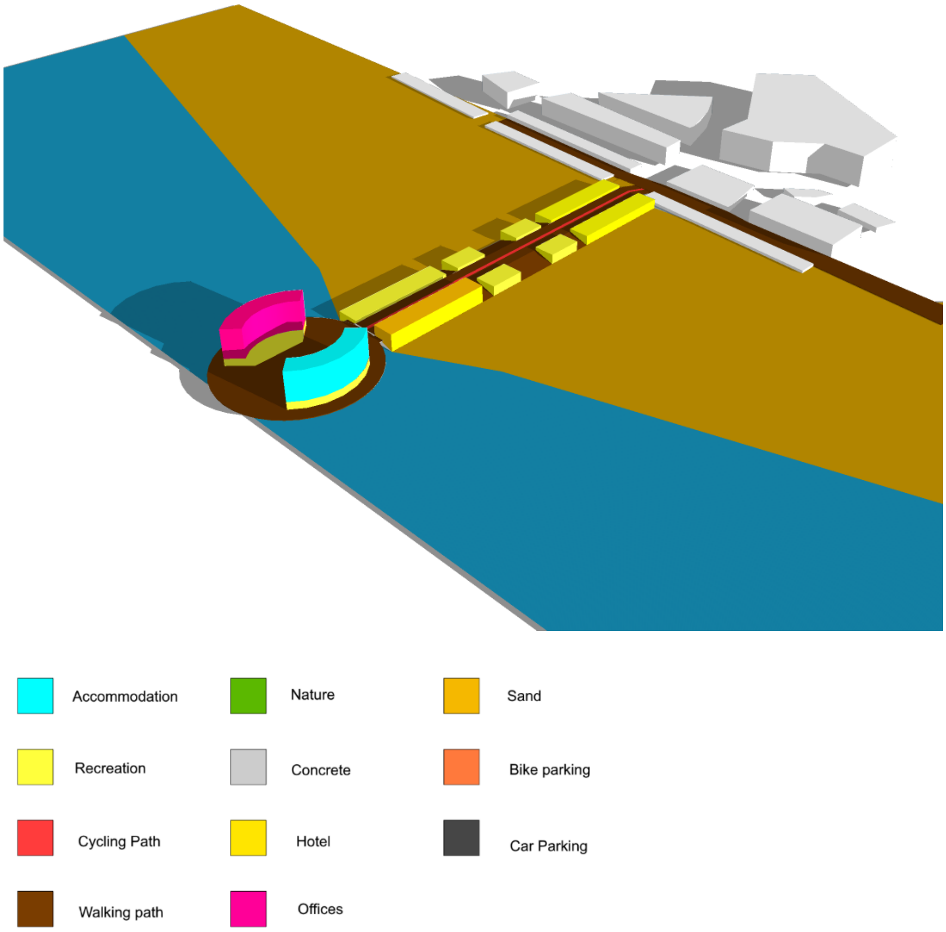


Figure 21: Concept 4, bird's-eye perspective

The dike is 10 meters high, with slopes of 1:3. The top crest is 35 meters wide, which makes space for walking and cycling paths. Buildings are implemented in the slope of the dike, which results in multi-story buildings.

4.2.5 Concept 5: Combined design

The fifth alternative, the combine design, has been designed after evaluating the previous four alternatives. The combined design is displayed in Figure 22. This concept takes the best aspects from the previous alternatives, and reduces any negative aspects. The combined design gets most of its properties from the underwater caisson and the multiple islands concepts. This can mainly be seen in the fact that the combined design consists out of different platforms and that these platforms consists out of multiple floors.

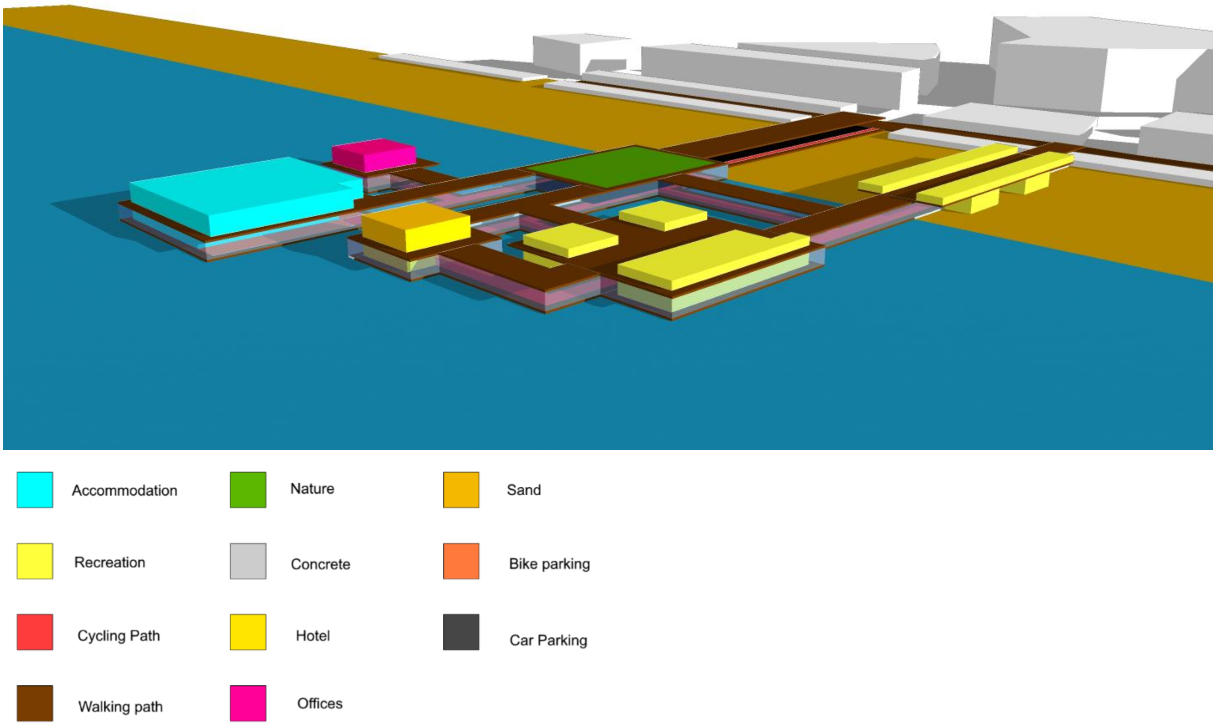


Figure 22: The combined design, bird's eye perspective

A good separation between the functions themselves, but also with the transition areas is accomplished with this alternative, because the functions are located on different platforms which are connected with transition routes. These are good properties which were taken over from the multiple islands concept. The fact that there is a large indoor area, which is the lower floor, makes sure that the pier is usable and enjoyable throughout the year, even in colder times, similar to the underwater caisson. This design has a parking lot, bike storage and cycling paths, which increases the accessibility in comparison to the other alternatives. An important lesson which is learned from the multiple islands, is that it is too wide. This would deteriorate the view of the ocean. Therefore, the combined design gets a different lay-out of the platforms, which makes it less than half as wide as the multiple islands concept. The north-east part of this alternative contains accommodation and a conference centre. There is a park in the middle of the design. The south-west side contains recreation and a hotel.

Most of the buildings which were on the top floor, continue downwards to the lower floors, which leads to extra storeys for the buildings. While the paths between the platforms on the top floor were exclusively usable for pedestrians, the lower floors include cycling paths along all the routes, and most platforms have dedicated space for people to park their bikes. The pier is also accessible by car and there are parking spots for cars below the park. The access routes on this floor goes over the beach. Figure 23 shows a plan view of this lower floor.

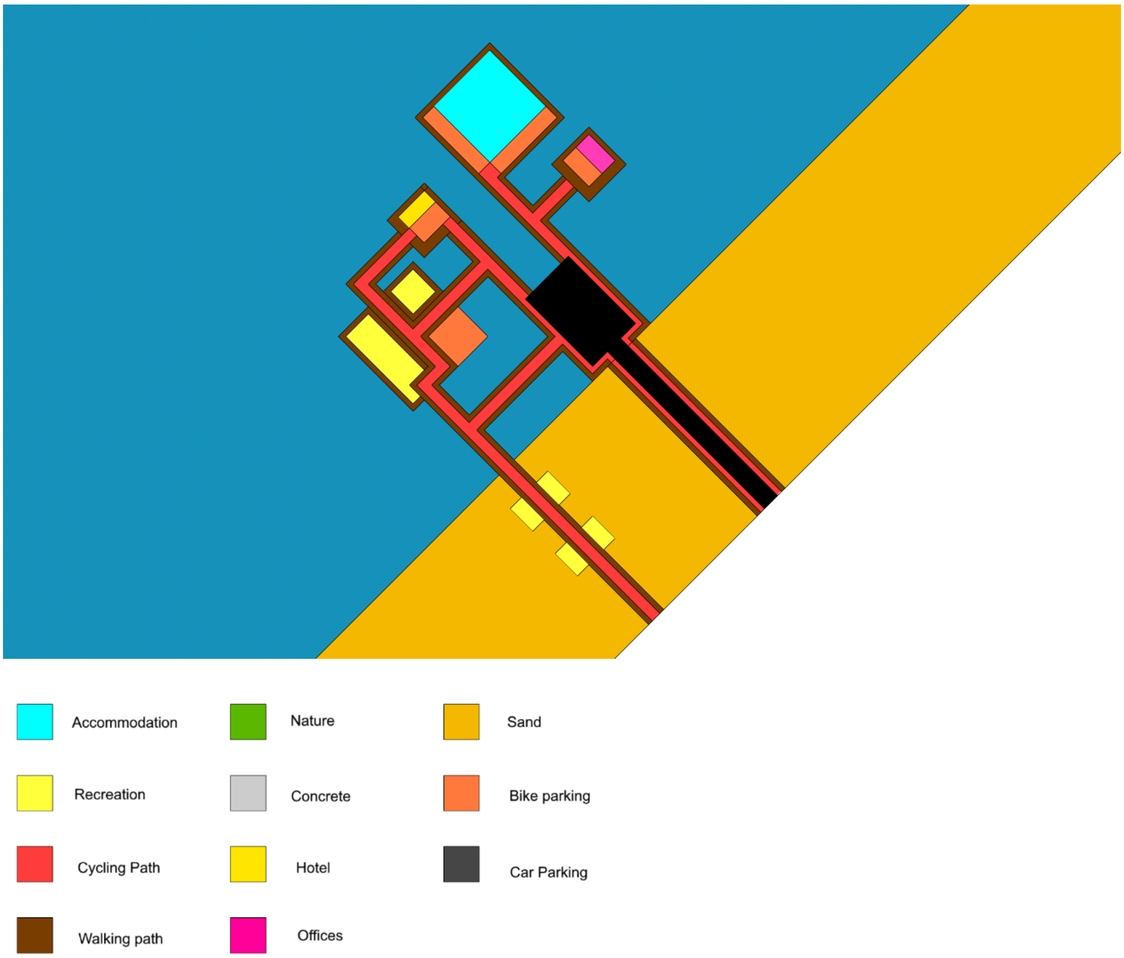


Figure 23: The combined design, plan view of the lower floor

4.2.6 Verification

In this subsection, the concepts are verified against the requirements from the functional specification from subsection 3.1.3. The results of the verification are found in Table 4. As mentioned before in Section 4.2.1, the ring dam concept has not been elaborated any further. This is because it does not suffice all the requirements. However, the other 4 concepts suffice all the requirements and are thereby deemed verified and are now alternatives.

Table 4: The requirements and if the concepts meet these requirements

Requirements	Ring dam	Underwater caisson	Multiple islands	Expansion of the beach	Combined design
• The new pier must increase the touristic value of the Scheveningen beach resort	No	Yes	Yes	Yes	Yes
• The new pier must include accommodation	Yes	Yes	Yes	Yes	Yes
• The new pier must have a hotel	Yes	Yes	Yes	Yes	Yes

• The new pier must have recreational functions	Yes	Yes	Yes	Yes	Yes
• The new pier must have space for new Conference venue	Yes	Yes	Yes	Yes	Yes
• The new pier must be accessible to everyone	Yes	Yes	Yes	Yes	Yes
• The new pier must be accessible throughout the year	Yes	Yes	Yes	Yes	Yes
• The new pier must be exploitable for the long term	Yes	Yes	Yes	Yes	Yes
• The new pier must be an improvement in comparison to the current pier	No	Yes	Yes	Yes	Yes

4.3 Evaluation with a multi-criteria analysis

The evaluation is performed with a multi-criteria analysis (MCA). Therefore, 10 different criteria were made to asses each concept. These criteria are also stated in the set of requirements and criteria from Subsection 3.1.3. They are based on the landscape analysis and the stakeholder analysis. The criteria which are used are mentioned again below:

- The new pier must have a good accessibility.
- The new pier should cause low nuisance to the environment and to the new functions on the new pier.
- The new pier should fit in with the environment and the boulevard.
- There should be a good separation between the transitional routes and the functional areas.
- The new pier should have a good view of the sea and the beach, and it should not deteriorate the view from other perspectives.
- The new Pier must be visually pleasing to its users and its surroundings.
- The new pier should interrupt the beach as little as possible.
- Nature must be damaged as little as possible.
- There must be enough space for new recreational facilities.
- A large part of the facilities should be usable throughout the year.

The remaining four alternatives receive a score for each criterion from 1-5, in which 1 is the worst score possible and 5 is the best. Therefore a maximum total score of 50 is possible in this phase of the evaluation. The scores are given relative to the other alternatives. Analysis of the alternatives support certain scores for select criteria. These analysis can be found in Appendix G. These analysis include a visual space analysis, a rough cost estimation and a traffic flow analysis. A better explanation of the criteria and the reason for the given scores are given in Appendix H. The scores have been given as accurately as possible, but in some cases assumptions have to be made. The scores are given in Table 5.

Table 5: Scores given to the alternatives for the different criteria. The minimum obtainable score for a criterion is 1, the maximum is 5. The minimum obtainable total score is 10, the maximum total score is 50.

Criterion	Underwater caisson	Multiple islands	Expansion of the beach	Combined design
1. Accessibility	4	4	2	5
2. Low nuisance	3	5	1	5
3. Blend in with the boulevard	3	3	5	3
4. Separation transitional and functional areas	2	5	2	5
5. View of the beach and sea	3	2	5	3
6. Aesthetic value	4	4	4	4
7. Continuation of the beach	1	4	3	3
8. Low damage to nature	5	5	5	5
9. Space for recreational areas	4	2	5	3
10. Usability throughout the year	5	3	3	5
Total	**34**	**37**	**35**	**41**

The next step in the MCA is to assign weight factors to the different criteria. These are multiplied with the scores. The sum of all the weight factors for the criteria must be equal to 100. For this MCA, three different perspectives were used. These are the perspective from the municipality, the client and the users. The reason why these perspectives are chosen specifically is because the municipality and the client are the two most influential stakeholders and the users are the target group of this project. The users, in this case, refer to the users of all the facilities on the pier.

The municipality values the wishes of their citizens and they want to increase the touristic value of Scheveningen. Low nuisance is a criterion which impacts the residents of The Hague. The accessibility and space for recreational areas are important to increase the touristic value of Scheveningen

The client wants to make much revenue from the pier and wants it to be an attractive spot for visitors. The criteria accessibility, space for recreational areas and usability throughout the year are therefore important to the client.

The users of the different functions of the pier want to have a good user experience. Having a good accessibility to the pier and experiencing as low nuisance improves the user experience. The tables displaying the weight factors and the resulting scores are given in Appendix H. The total sores are given in Table 6.

Table 6: Total weighed scores of the alternatives. The minimum score and the maximum obtainable score are 100 and 500 respectively.

Perspective	Underwater caisson	Multiple islands	Expansion of the beach	Combined design
Municipality	360	360	345	**415**
Client	365	340	355	**410**
Users	360	380	295	**440**

According to this analysis, the combined design scores the best. All the other designs score close to each other, considering the a range from 100 to 500. The combined design however, scores relatively high with a good score, staying above the 400 for every perspective. This was to be expected since the combined design has been designed after evaluating the other three.

4.4 Final decision on the functional/spatial design

The combined design, which is displayed in Section 4.2.5, has been chosen and is the functional and spatial design for the new Scheveningen Pier in this thesis. This design is larger than the current pier and accommodates a greater variety of functions, so that this new pier can be exploited at a higher level than the current pier, which is one of the main goals of the client. The new pier gives a boost to the beach resort of Scheveningen as well, which is important to the municipality of The Hague. The multi-criteria analysis which was performed in Subsection 4.3 also supports the choice for this alternative. The combined alternative gathers relatively high scores in comparison with the other designs. This is also because this combined design was designed to take the positive aspects of the other alternatives while trying to diminish the negative effects.

The new Scheveningen Pier is accessible with different modes of transportation. All platforms can be reached on foot or by bike, and these modes of transport are separated by being on different levels. Besides this, the pier contains a parking garage too on the lower floor.

The different platforms accommodate different functions, so that everyone has a reason to visit the new Scheveningen Pier and everyone finds their own favourite spot on the pier. Whether someone is looking for a place to stay with a nice view of the sea or if you want to spend a fun day with family or friends at the many shops, bars and restaurants on the pier, it is all possible. Even during colder weather circumstances, the pier can be used because the pier contains a lower indoor section as well.

To get to this design, multiple ideas were elaborated. They started as undetailed ideas, represented by sketches and eventually grew to real alternatives, displayed by up to scale visual models. By analysing the landscape, the surroundings and the stakeholders, more information was gathered to be able to improve the designs and to compare them. Eventually designs were combined, which resulted in the final design of the functional plan.

The structures which supports this design and keeps it safe is designed in Chapter 6.

5 Morphological design loop

The earth's surface consists mostly out of water and most of the cities in the world are located near water. 80% of the world's cities are located in coastal areas, because they are more attractive (Waterman, 2020). Recently, urban areas try to expand their cities due to a growth in population. There are different ways to expand a city, and one of these is to expand on the water. Something which is done with this thesis. When doing such an expansion, the effects on the surrounding nature should be taken into consideration. In this case, the surrounding nature is the beach of Scheveningen and the seabed located near the pier. The beach also plays a major role in protecting the Netherlands against floods, therefore the beach cannot be deteriorated much by the construction of a new pier.

In this section, the surrounding beach of Scheveningen is researched. This is done so that the design of the New Scheveningen Pier does not deteriorate the beach of Scheveningen. This chapter goes through the morphological design loop, which is a technical design loop, which is explained in Chapter 2. The failure mechanisms and the dimensioning of the elements are not treated in the morphology. The morphological design loop is meant to determine the size of the supporting structure of the pier. First the set of requirement and criteria is made, and based on these, alternatives are developed. These alternatives are then verified with models. The alternatives need to satisfy the requirements. After that, a choice is made between the alternatives. This is based how well each alternatives suffices the requirements and criteria.

The relevant boundary conditions, requirements and criteria regarding the beach morphology, which were mentioned in Chapter 3, are repeated down below.

Requirements

- The coastal defence cannot be deteriorated due to the construction of the pier.
- The beach and the sea must be safe for visitors to visit.

Criteria

- New beach area which can be used for recreation is desired.
- The beach width should not be too large so that the walking distance to the sea would be too large.
- As little erosion as possible should occur due to construction of the pier.
- The flow velocities in the sea near the coast should not decrease the swimmer safety.

A desired result is that more beach space, which could be used for recreation is being formed as a result of the pier. This cannot be at the cost of swimmer safety.

5.1 Creating alternatives

Different alternatives are developed. All these alternatives have a different impact on the beach. The impacts of the different alternatives are compared to a situation where nothing happens, the zero situation. The alternatives are mentioned below:

- Open/zero situation
- Full blockade
- Blocking at the end (similar to an emerged breakwater)
- Series of groynes

Figure 24 gives a schematisation of how these different alternatives look.

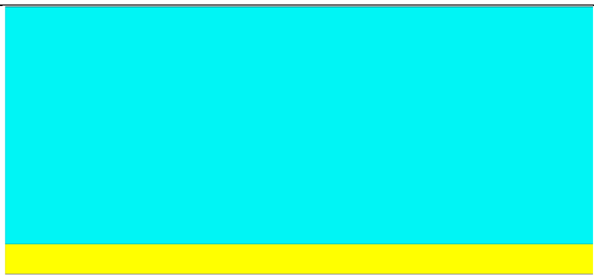


Figure 24a:Open/zero situation

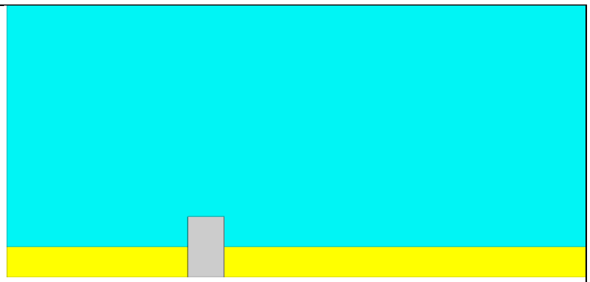


Figure 24b: Full blockade

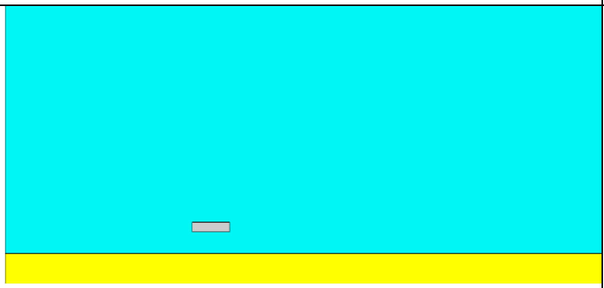


Figure 24c: Blocking at the end

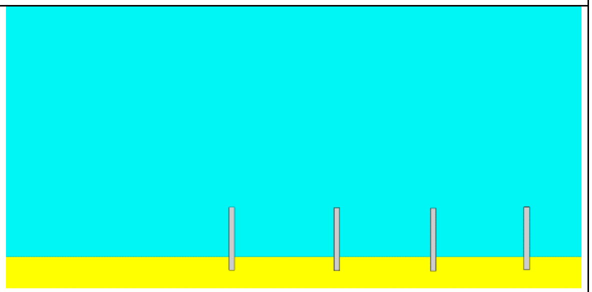


Figure 24d: Series of groynes

Figure 24: Top view of the different alternatives for the beach morphology.

These alternatives are not completely new alternatives. The functional model that is designed in Chapter 4 is still being used. These new alternatives are adaptations on this model, with the exception of the series of groynes, which are not placed underneath the functional plan. How the other three alternatives fit in with the functional plan can be seen in Figure 25 to Figure 27. In the open/zero situation it looks like the pier is floating above the sea, but in reality it is supported by a construction which blocks the flow of water and sediment as little as possible. The other two measures show a larger structure where a larger part of the flow is blocked. This structure can be acquired by placing caissons for example.

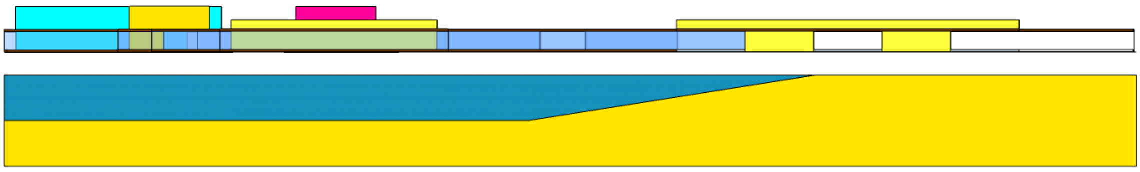


Figure 25: Side view of the new pier: Open/zero situation

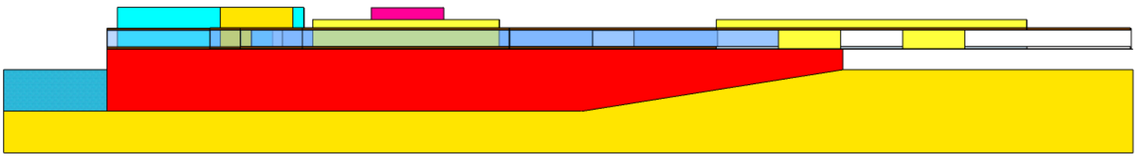


Figure 26: Side view of the new pier: Full blockade

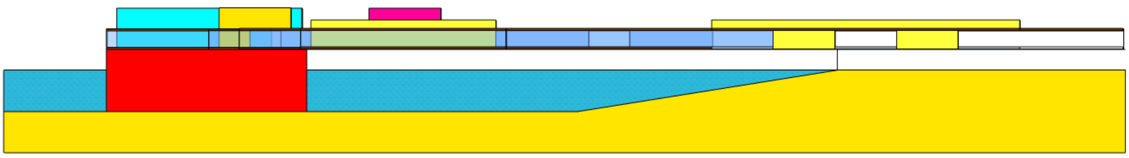


Figure 27:Side view of the new pier: Blocking at the end

Appendix I, gives the theory behind coastline changes and makes a hypothesis of what happens with the beach when these measures are being used.

5.2 Analysis on the beach morphology

The new pier is likely to influence the shape and conditions of the beach. Changes to the beach are often caused by a change in the alongshore sediment transport. A hypothesis has been made on the effect of the different alternatives on the beach. This hypothesis can be found in Appendix I.

Models have been mode to research the beach morphology. The software Delft3D has been used to make these models. Delft3D is an process based model. The main principle of these types of model is explained in Figure 28. A model for every alternative, mentioned in section 5.1, has been made.

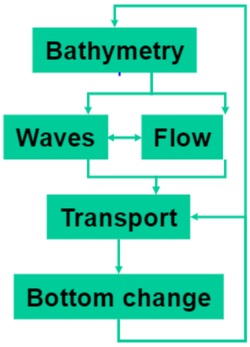


Figure 28: The principle of process based models (Luijendijk, 2019)

The most important input of the model are the bathymetry and the wave conditions. The relevant output is the erosion/sedimentation, the flow patterns and the new shape of the beach. These results of the models for the different alternatives are then compared to each other to see which alternative create the most desires or least undesired effect. The results and a more elaborate explanation of the model can be found in Appendix I.

5.3 Conclusion on beach morphology

In comparison to the zero situation, the full blockade measure is a good measure when looking at the erosion/sedimentation and also at the water depth after 80 days. The beach is regaining area, which means that there is more space for recreation on the Scheveningen beach. However, this measure does have a downside, regarding the flow velocities. According to the model, there are high flow velocities near the coast, on the left side of the blockade which does eventually go offshore. This flow near the coast was not present in the zero situation. But this flow could heavily decrease the swimmer safety in the area, so therefore the regained beach area would be less usable as recreation space.

The breakwater measure, in which only the end of the pier would block waves, shows a different effect than was originally stated in the hypothesis. The hypotheses predicted that a salient or tombolo would appear behind the breakwater, which means that there should have been sedimentation behind the breakwater. But the model shows that there is actually more erosion behind this breakwater in comparison to the zero situation. A possible explanation for this is the convergence of the alongshore flow stream. The alongshore stream enters a more narrow space when it is behind the breakwater, therefore the velocities increase, which leads to more erosion.

The series of groynes is not modelled in a small scale, since this measure is adapted over a larger scale. But this has been applied in the south of Scheveningen to preserve the coast. Unfortunately this has led to rip currents, which are very undesired since they cause both erosion and a decrease in swimmer safety (Waterman, 2020).

So the model shows that both a full blockade, a partial blockade, similar to an emerged breakwater and a series of groynes would have negative effects in comparison to the zero situation, where the flow would be blocked by the pier as little as possible. The measures either result in a decreased swimmer safety due to currents or to more erosion, which are both undesired effects.

Eventually, an open structure is chosen to continue with. This structure is as open as possible to prevent any of the negative effects, caused by one of the other measures. In the Chapter 6, this structure is designed into more detail.

The models used for the morphology were made to get an insight into the changes that could occur during certain measures at the beach. The models that were made are undetailed, this is mainly due to the fact that not much experience with the modelling software was present and that limited time is was available to finish this thesis. A recommendation would be to make a more detailed model with the actual structure of the pier from Chapter 6.

6 Structural and Hydrodynamic design loop

This chapter performs the structural and hydrodynamic design loop, which is one of the iterations of the technical design loop which is explained in Chapter 2 and Appendix C. This loop will build on the functional/spatial design from chapter 4 and the results of the morphological design loop from Chapter 5. Multiple alternatives have been made during the design process, but they do not all suffice requirements. This chapter only shows the final structural design and how this design was dimensioned.

An overview of the structural design is given in Section 6.1. The construction method of this structure is explained in Section 6.2. The relevant failure mechanisms and the governing load situations are given in Section 6.3 and Section 6.4 respectively. The dimensioning of the different elements of the structural design is done in Section 6.5 to 6.9.

6.1 Structural design of the pier

The conclusion from the morphological analysis from Chapter 5 is that the structure of the pier should be as open as possible, blocking the flow and the sediment as little as possible. A more detailed design of the structure is designed in this chapter. The pier is supported by piles to make it as open as possible. However, there are more aspects of the structure which need to be determined. The structure of the pier can be divided in the following parts:

- Foundation
- Piles
- Pier deck
- The superstructure with buildings and infrastructure

An overview of these different layers is well represented in Figure 29. The superstructure with the functions on the pier is not elaborated on in as much detail as the other layers. The relevant aspect of the functions is that it they exert a load on the deck.

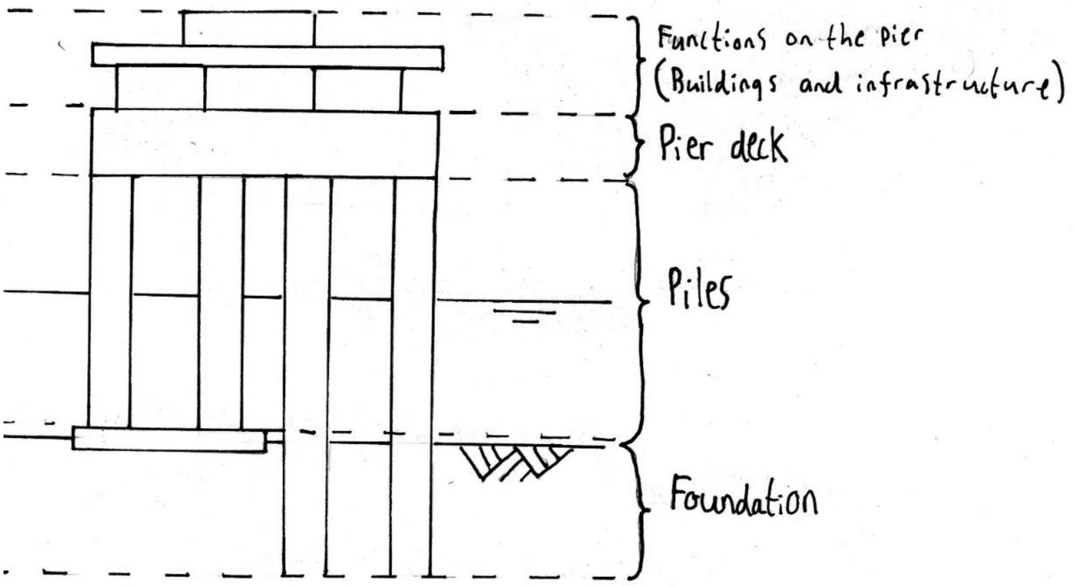


Figure 29: Schematisation of a front view of the pier, displaying the different layers of the construction

The pier consists of multiple platforms with different functions on top of the deck. This is explained in Chapter 4. This chapter only makes a structural design for one of these platforms, because the structure and the construction method can be used to the other platforms as well. The most governing platform is the platform on which the accommodation is situated. The reason for this is because it is one of the largest platforms, it contains the largest building on top of it and is located most off-shore from all the platforms. Figure 30 shows where this platform is located, relative to the entire spatial plan.

The entire platform is 100 x 100 m but most of the loads come from the building, which occupies an area of 90 x 90 m. The building contains four stories.

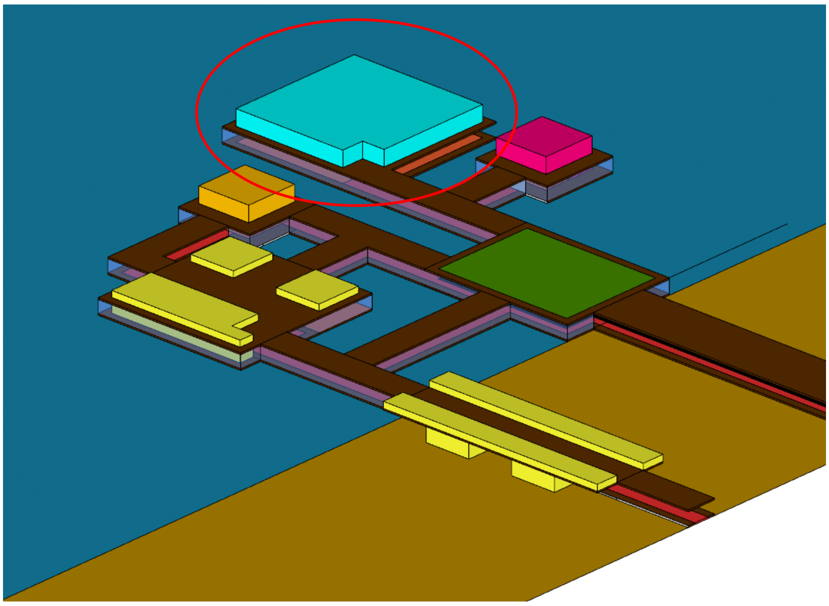


Figure 30: Birds-eye perspective of the spatial plan. The platform containing accommodation is circled in red.

Figure 31 to Figure 34 show what the structure of the accommodation platform looks like. The entire structure is supported by piles, which go into the sea bed. Concrete beams and piles are placed on these piles to transfer the load from the deck to the piles. Diagonal steel struts are added to increase the stiffness of the design.

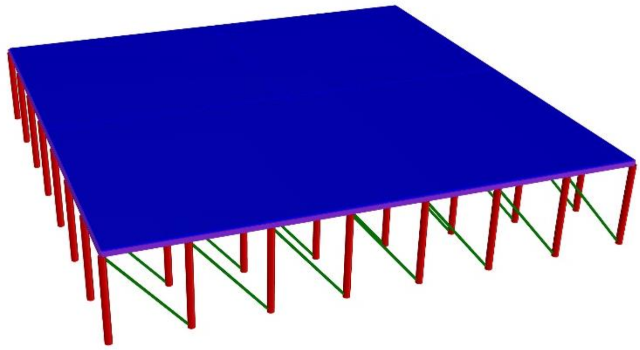


Figure 31: Birds eye perspective of the pier structure above sea bed. Units in meters.

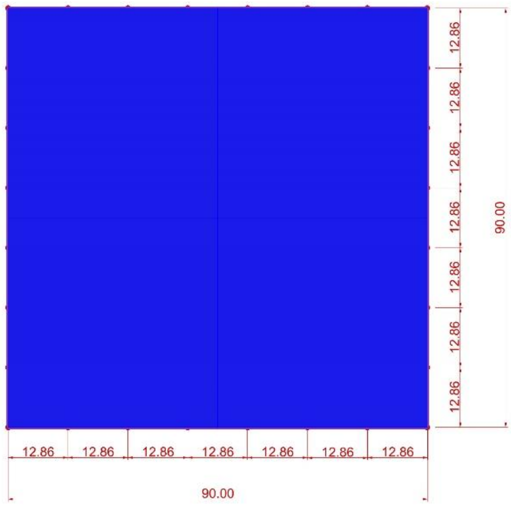


Figure 32: Top view of the pier structure above sea bed. Units in meters.

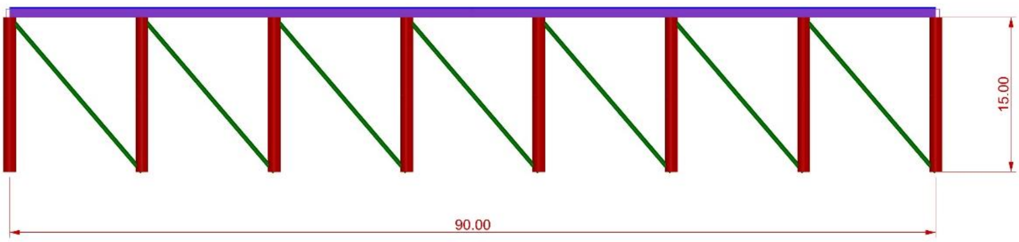


Figure 33: Front view of the pier structure above sea bed. Units in meters. Offshore side is on the left of this figure.

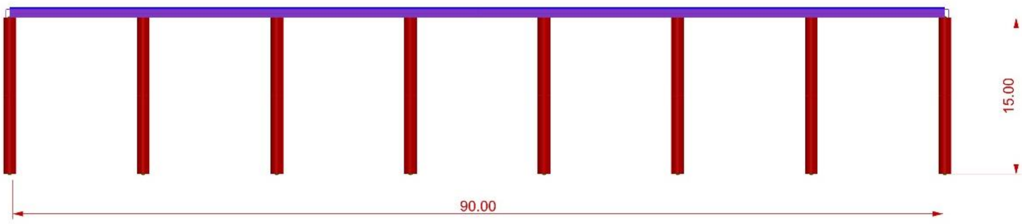


Figure 34: Side view of the pier structure above sea bed. Units in meters.

6.2 Construction method

The constructability of the structure is an important aspect of the design. Especially in the environment of this project. The pier is situated at sea for a large part and this brings complications, mostly due to waves. Three different construction methods for the foundation were considered. These are:

- Creating a dry workspace.
- Immersing elements to the bed.
- Using a jack-up barge with spud piles.

A dry workspace is difficult to construct because of the wave climate. A jack-up barge is necessary too to construct a dry workspace. The sheet pile walls, which are used for the workspace, have to resist the high loads from the waves as well. Sinking elements to the bed is a method which is only usable for a shallow foundation. However, in open water with waves it is difficult to accurately place the elements when they are submerged to the bed. Using a jack-up barge with spud piles is the best option for placing a foundation in the sea. Appendix P mentions the advantages and disadvantages of the different construction methods for the foundation.

Figure 35 shows the construction order with the use of a jack-up barge. The jack-up barge is only used for the part of the pier that is located above the sea. The part which is located on the beach, is constructed first. After that, the jack-up barge is transported and installed at the first location, which is close to the beach. The pile foundation on that location is placed. A new segment of the pier deck shall then be placed on these piles, connecting it with the previous segment. After that, the jack-up barge moves to a new location and this cycle is be repeated. So summarized:

- Jack-up barge moves to location.
- Piles are placed.
- Pier deck is placed.
- The above is repeated until the pier is complete.

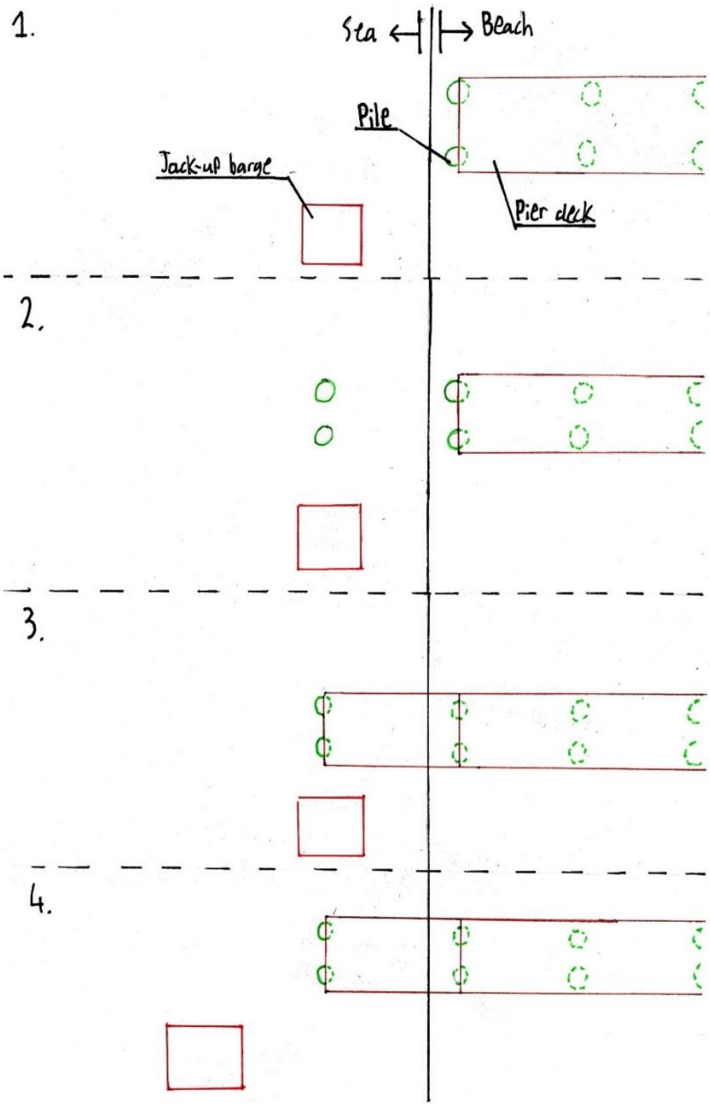


Figure 35: Construction order of the pier with the use of a jack-up barge

A jack-up barge is a platform that can be used for construction in water. Figure 36 displays the mechanisms of the jack-up barge with the spud piles. The advantage of this is that the platform stays on the desired location, and does not drift away. Especially in this project area, where there are breaking waves, a jack-up barge is useful. Construction equipment can be placed on the these platforms. From there it is possible to drill piles into the sea bed, or work on the construction of the actual deck itself. The material, like the piles, has to be transported to the jack-up barge via boats.

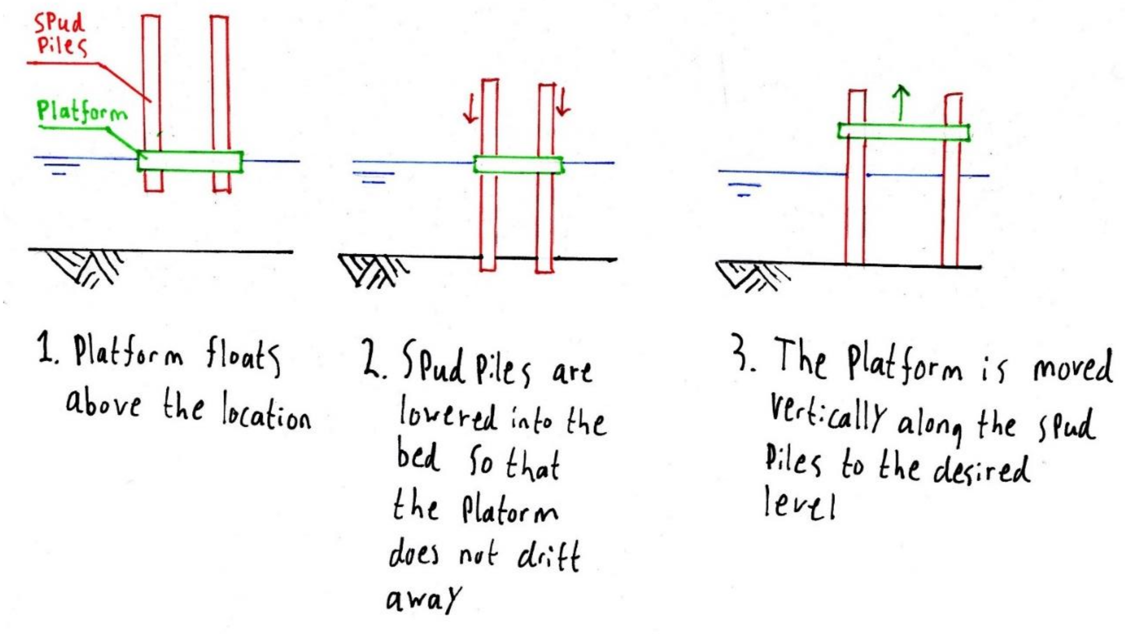


Figure 36: The mechanism of a jack-up barge with spud piles, schematised (NTS)

The piles which support the deck are being placed with Fundex piles. Figure 37 shows how these are placed from a jack-up barge. First, a hollow steel tube is drilled in the seabed. This tube is the same length as the pile, so the top reaches above the water level, which means that the inside of this hollow tube remains dry. After the tube is drilled to the desired level, the reinforcement cage is placed inside the tube, followed by the concrete mix. The equipment which is required for the placement of the steel tube, the reinforcement and the concrete mix is different. If the available space on the jack-up barge is not sufficient to contain all the equipment, multiple jack-up barges are required to place the piles.

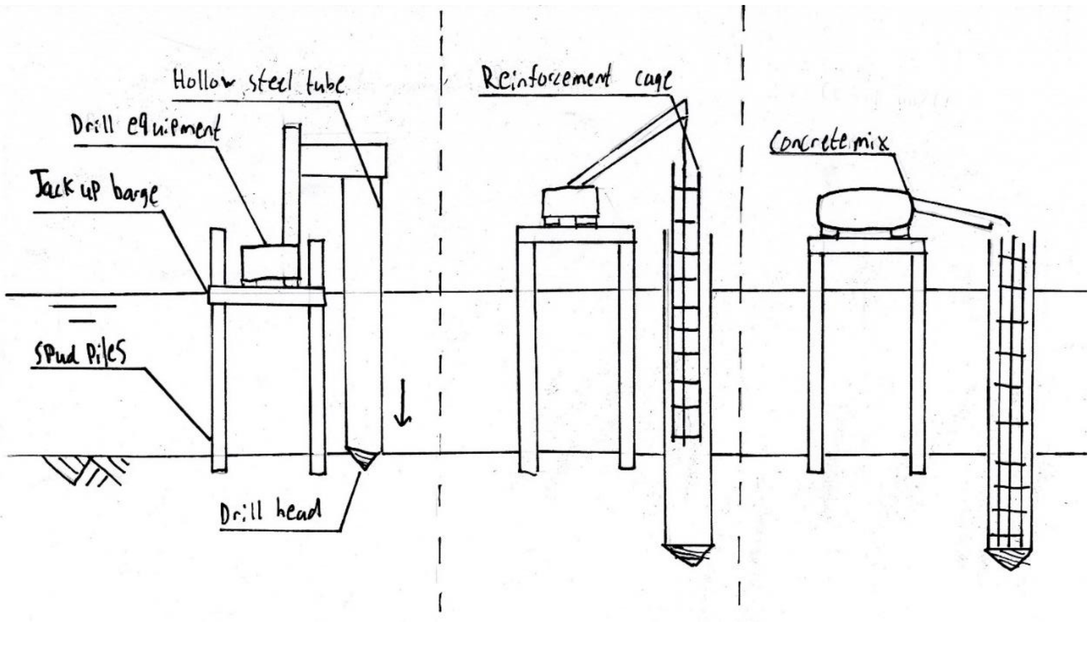


Figure 37: Placement of Fundex piles in the sea bed, with a jack-up barge on spud piles (NTS)

The deck is placed from the deck itself. This can be seen in Figure 38. First, the equipment is transported to the end of the currently placed deck. The equipment contains the building materials but also the machinery required to place the deck. This can be supplied from the land. When the equipment is at the location, the beams are placed between the current deck and the piles which have been placed before. At last the plates are placed. This cycle is continued for the rest of the pier.

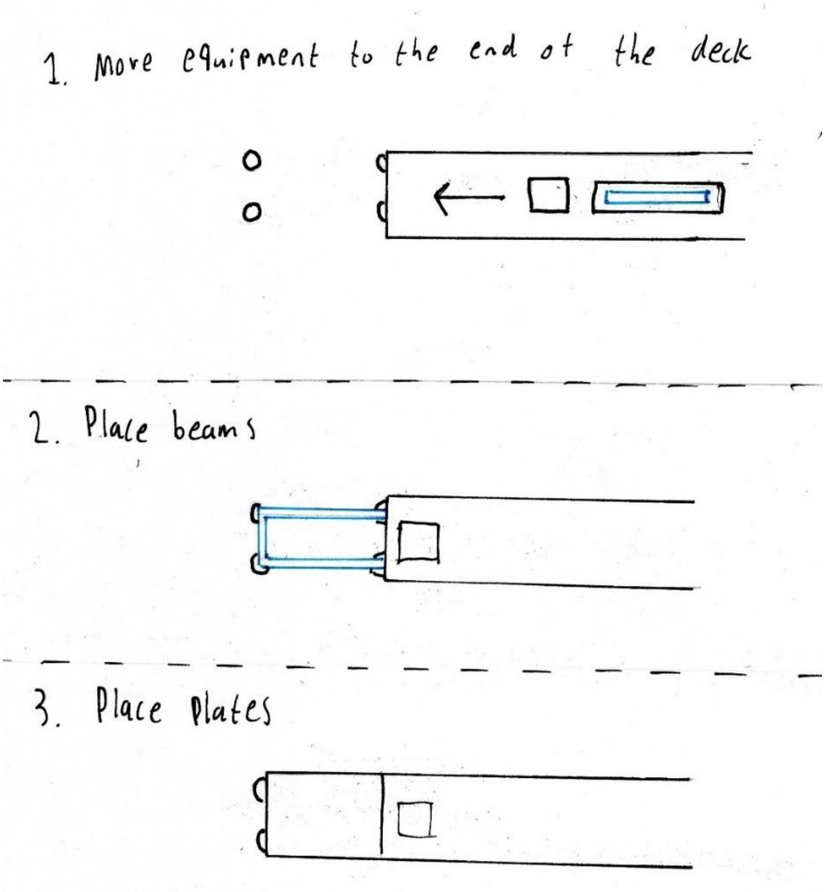


Figure 38: Construction of the deck, from the deck (NTS).

A construction sequence, where the construction method is summarised, can be found in Appendix S.

6.3 Fault tree of the failure mechanisms

The total allowable yearly failure probability of the pier is 10^{-7}, as stated in Chapter 3. This is in the ultimate limit state according to the Eurocode (Nederlands Normalisatie-instituut, 1990). This means that only structural failure is taken into account. Other ways of failure or other limit states are outside the scope of this thesis.

For the structural verification of the pier, a semi-probabilistic method of level I is used, which is also used by the Eurocodes (Nederlands Normalisatie-instituut, 1990). This section explains this method and derive the required safety factors and design values for the loads and resistance of the pier.

There are multiple elements of the pier that can fail and there are multiple failure mechanisms as well. The sum of the failure probabilities must be equal or lower than the total maximum allowable failure probability of the pier. To clearly visualise the failure probabilities for the different elements and mechanisms, a fault tree has been made. Multiple assumptions are made with this fault tree to reduce its complexity:

- All the failure mechanisms are mutually exclusive from each other.
- All the deck beams are fully dependant of each other.
- All the deck slabs are fully dependant of each other

- All the columns are fully dependant of each other.

The independency of the failure mechanisms means that the total failure probability is the sum of different failure probabilities. The dependency of the different elements means that the total failure probability of the system of those elements is leading on the most governing element. This means, for example, that all the columns fail when the most governing column fails. This also accounts for the slabs and the deck beams.

The structure of the pier consists of two main parts, these are the columns on which the pier is being supported and the deck of the pier itself. The pier can either fail at the columns or at the deck, this is displayed in Figure 39. It also shows that failure of the pier can be caused by failure in the serviceability limit state or other modes of failure. However, in this thesis, only the structural failure in the ultimate limit state is being calculated.

The columns and the deck have different failure mechanisms as well. The fault tree of the columns and the deck are given in Figure 40 and Figure 41 respectively.

For each failure mechanism, the reliability factor, β, has been determined. This is needed to calculate the safety factors in Subsection 6.4.1. The relation between β and the failure probability is given with the following formula.

$$\beta = \Phi^{-1}(1 - p_f)$$

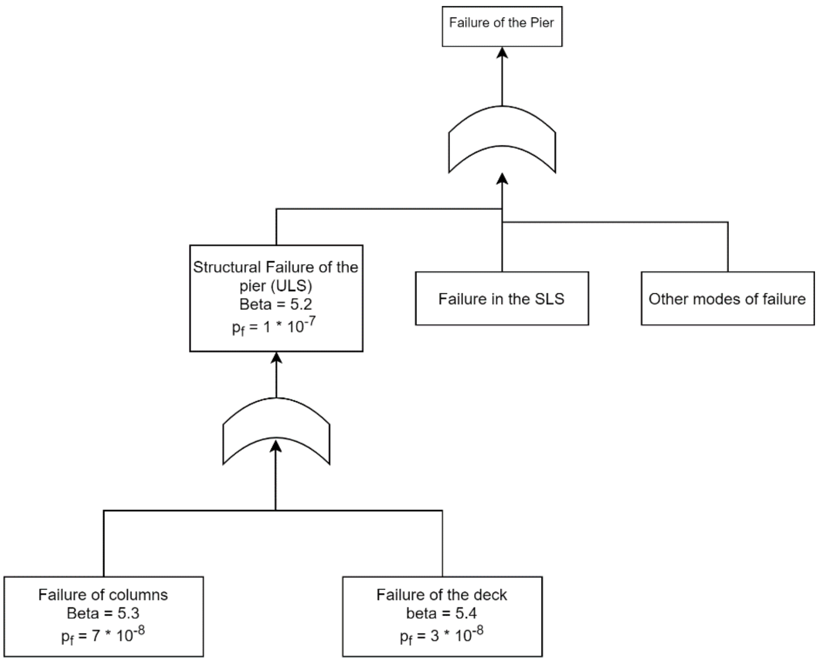


Figure 39: Fault tree of the pier

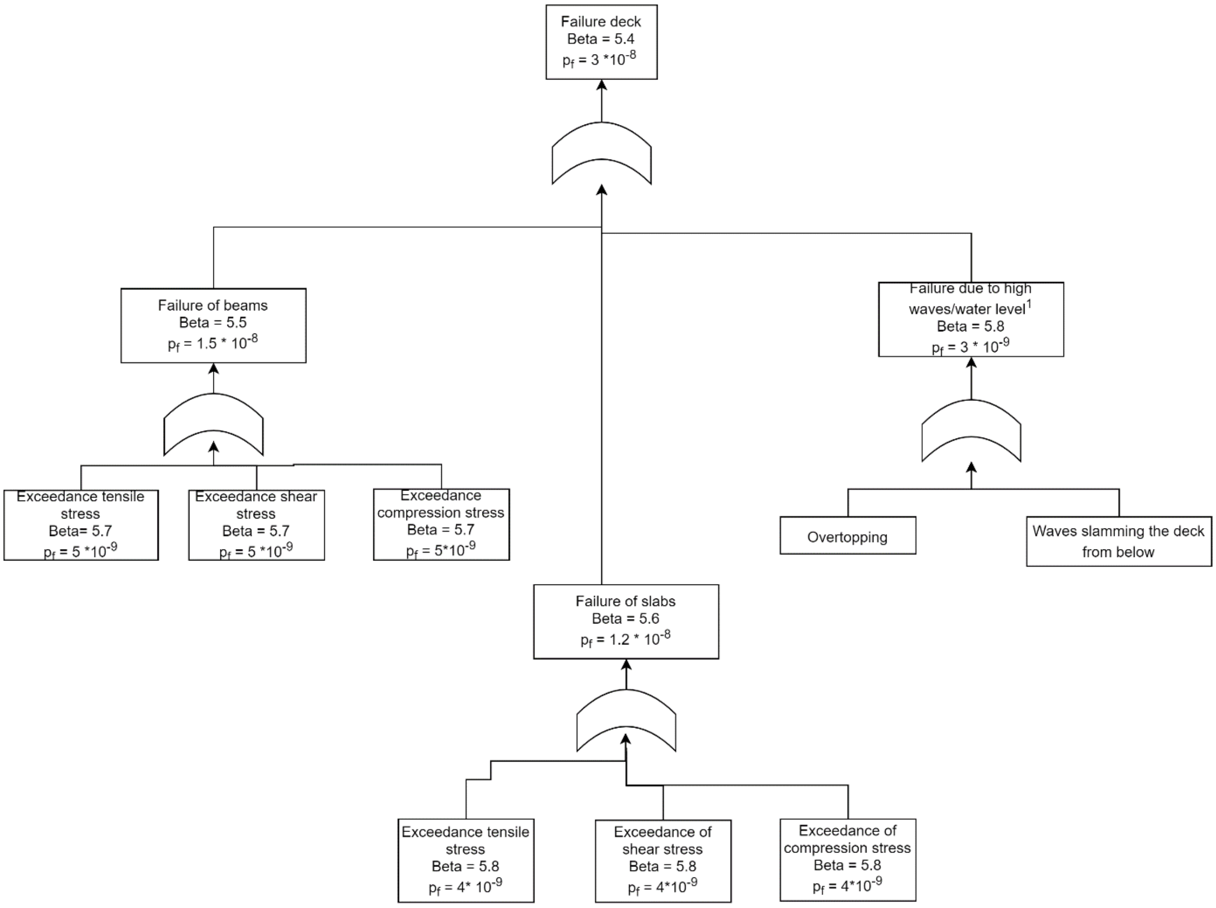


Figure 40: Fault tree of the deck

[1]:Into depth calculations for the failure of the deck due to high waves\water level are not performed. In this thesis, this failure mechanism does not occur when the waves do not reach the deck.

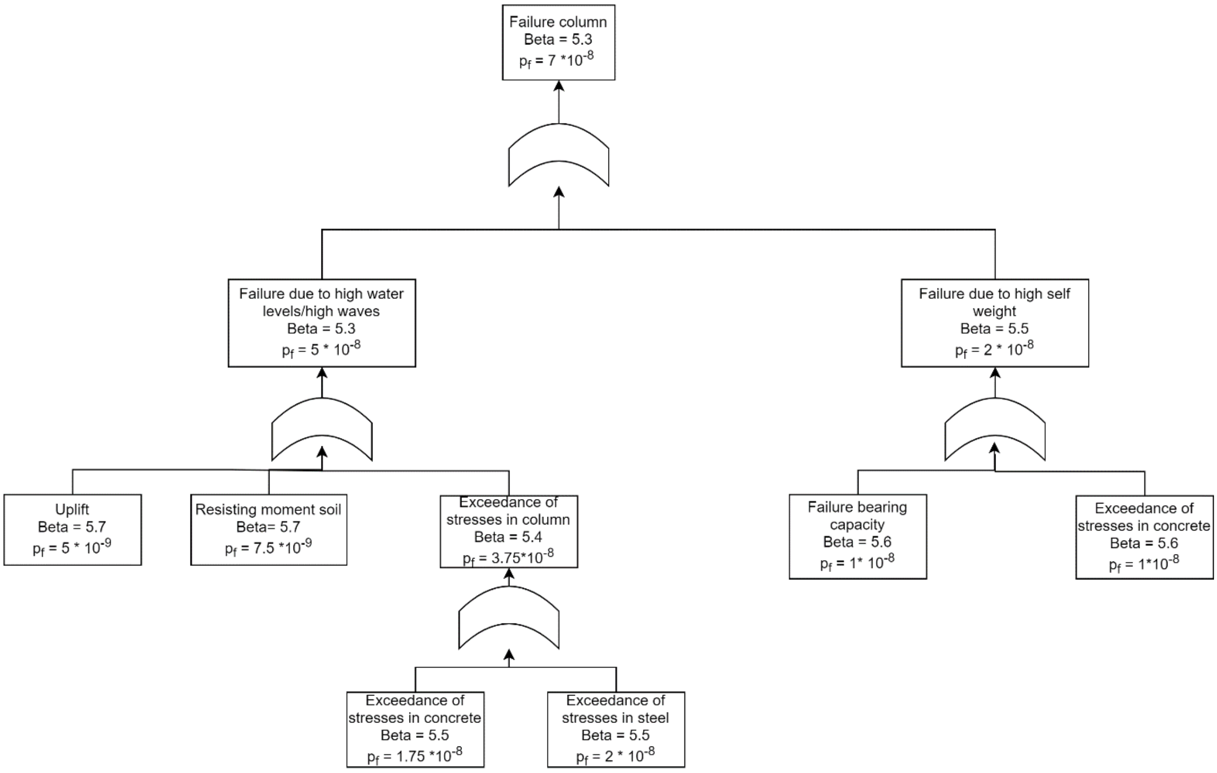


Figure 41: Fault tree of the columns

6.4 Determining the governing load situations

The different failure mechanisms occur at different load situations. This section determines the governing load situations and which safety factors are used for each situation.

6.4.1 Derivation of the safety factors and design values

A semi probabilistic check of level I makes use of safety factors, noted as γ. The full calculations of the safety factors are given in Appendix R. Following is a summary of how these safety factors are derived.

The following equation must be true for a structure to be structurally safe (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017).

$$\frac{R_k}{\gamma_R} > \gamma_s \cdot S_k$$

$$\gamma = \text{Safety factors}$$

$$R_k = \text{Characteristic Resistance}$$

$$S_k = \text{Characteristic load}$$

The characteristic value of a load is the value with a 5% chance of exceedance and the characteristic value of the resistance is the value with a 95 % chance of exceedance.

The safety factors can be determined with the characteristic value and the design value.

$$\gamma_m = \frac{R_k}{r^*}$$

$$\gamma_E = \frac{e^*}{E_k}$$

The design value depends on the distribution, the α value and the reliability index β. The α value depends on the type of load or resistance. Different values for α are given in Table 7. The design values for a normally distributed load are given in the following equation.

$$r^* = \mu_R(1 - \alpha_R \beta V_R) = \text{Design point if normally distributed}$$

$$s^* = \mu_s(1 - \alpha_s \beta V_s)$$

$$V = \frac{\sigma}{\mu}$$

Table 7: Standardized α values for structures according to the Eurocode (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017)

Variable	$\boldsymbol{\alpha}$
Dominant strength parameter	0.80
Remaining strength parameter	0.32
Dominant load parameter	-0.70
Remaining load parameter	-0.28

The design water level and design wave height are determined with a different method than the method mentioned above. The exact calculations of the safety factors are given in Appendix R.

6.4.2 Governing load situations

For each governing load scenario, the safety factors have been determined. In each scenario, the relevant loads and resistances have been classified into dominant or remaining loads or strengths. This determines the α value. The β values are from the fault trees from Section 6.3. With this, the safety factors for all the loads and resistances can be determined for each different governing scenario. The results can be seen in Table 8 and Table 9. The calculations for the safety factors can be found in Appendix R.

Table 8: Safety factors for the loads of different load scenarios for the column

Load	Type	α	β	γ_G
Uplift				
Self-weight	Dominant strength	0.80	5.7	0.54
Water level	Dominant load	-0.70	5.7	1.5
Resisting moment soil				
Waves	Dominant load	-0.70	5.7	1.5
Wind	Remaining load	-0.28	5.7	1.3
Exceedance of stress in column due to lateral load (Tensile stress)				
Self-weight	Remaining strength	0.32	5.5	0.82
Concrete	Dominant strength	0.80	5.5	1.5
Steel	Dominant strength	0.80	5.5	1.3
Waves	Dominant load	-0.70	5.5	1.5
Wind	Remaining load	-0.28	5.5	1.3
Exceedance bearing capacity (Permanent load dominant)				
Self-weight	Dominant load	-0.70	5.6	1.4

Concrete	Dominant strength	0.80	5.6	1.5
Waves	Absent	-	-	-
Variable floor/roof load	Remaining load	-0.28	5.6	1.3
Wind	Absent	-	-	-
Exceedance bearing capacity (Variable load dominant)				
Self-weight	Remaining load	-0.28	5.6	1.2
Concrete	Dominant strength	0.80	5.6	1.5
Waves	Absent	-	-	-
Variable floor/roof load	Dominant load	-0.7	5.6	1.8
Wind	-	-	-	-
Exceedance compression stress column (Permanent-weight dominant)				
Self-weight	Dominant load	-0.70	5.6	1.4
Concrete	Dominant strength	0.80	5.6	1.5
Waves	Remaining load	-0.28	5.6	1.1
Variable floor/roof load	Remaining load	-0.28	5.6	1.3
Wind	Remaining load	-0.28	5.6	1.3
Exceedance compression stress column (Variable-load dominant)				
Self-weight	Remaining load	-0.28	5.6	1.2
Concrete	Dominant strength	0.80	5.6	1.5
Waves	Remaining load	-0.28	5.6	1.1
Variable floor/roof load	Dominant load	-0.7	5.6	1.8
Wind	Remaining load	-0.28	5.6	1.3

Table 9: Safety factors for the loads of different load scenarios for the deck

Load	Type	α	β	γ_G
Tensile / shear stress in the deck beams				
Self-weight	Dominant load	-0.7	5.7	1.4
Concrete	Dominant strength	0.80	5.7	1.5
Steel	Dominant strength	0.80	5.7	1.3
Variable floor/roof load	Remaining load	-0.28	5.7	1.3
Compression in the beam (Wind governing)				
Self-weight	Remaining load	-0.28	5.7	1.2
Concrete	Dominant strength	0.80	5.7	1.5
Variable floor/roof load	Remaining load	-0.28	5.7	1.3
Waves	Remaining load	-0.28	5.7	1.1
Wind	Dominant load	-0.7	5.7	1.8
Deck plates				
Self-weight	Dominant load	-0.7	5.8	1.4
Concrete	Dominant strength	0.80	5.8	1.5
Steel	Dominant strength	0.80	5.8	1.3
Variable floor/roof load	Remaining load	-0.28	5.8	1.3
Flooding				
Water level	Dominant load	-0.70	5.8	1.5

6.5 Strength checks with Matrixframe

A Matrixframe model is made to perform the strength checks of the structure. A two dimensional model is made which represents one row of columns of a platform of the pier. These are connected with each other with horizontal beams at the top, and with diagonal struts. The horizontal beams are also a part of the deck of the pier. The geometry of the model is seen in Figure 42. Vertical gridline A is located most off-shore and gridline H is located most onshore. Horizontal gridline 1 is at the bed level and gridline 2 is at the top of the piles. Horizontal gridline 0 is located at 0.65 times embedded depth of the pile and at the bottom the piles have a fixed support. This is the same assumption as Blum’s method. The connections between the columns, beams and struts are hinged.

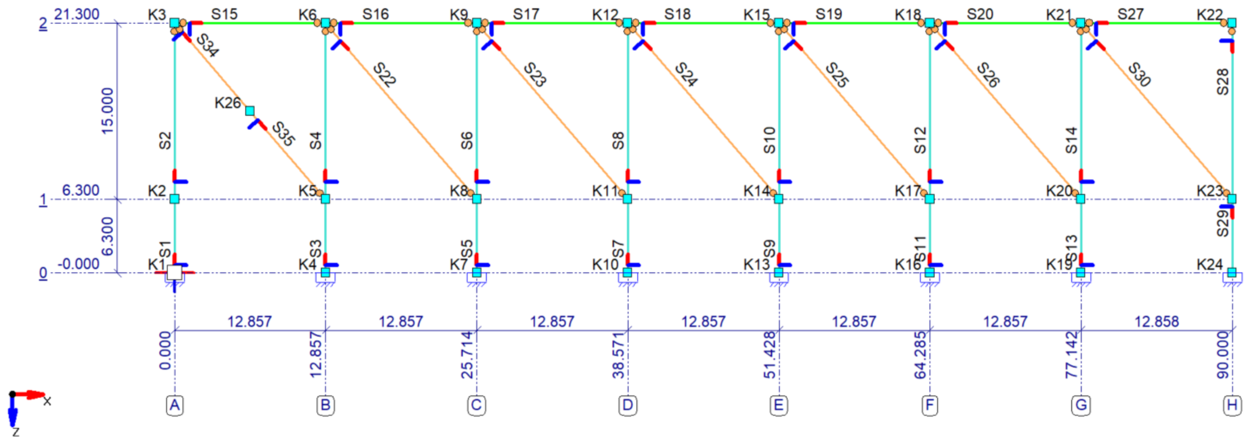


Figure 42: Geometry of the Matrixframe model. Distances in m.

Different loads were entered in Matrixframe. These loads are:

- Wave force
- Wind force
- Flow force
- Permanent load from above
- Variable load from above
- Self-weight of all the elements

Different load combinations were made to test different failure mechanisms. These load combinations LC.1 to LC. 7 are described in Table 10 and Table 11. These load combinations are derived from Table 8 and Table 9 from subsection 6.4.1.

Table 10: Load combinations for the column

Load combinations	Governing situation
LC. 1	Tensile stress / shear force / lateral soil resistance
LC. 2	Bearing capacity (permanent is dominant)
LC. 3	Bearing capacity (variable load is dominant)
LC. 4	Compression stress (permanent load is dominant)
LC. 5	Compression stress (variable load is dominant)

Table 11: Load combinations for the beams

Load combinations	Governing situation
LC. 6	Tension stress/shear force in the beams
LC. 7	Compression stress in the beams

Matrixframe has performed a linear elastic calculation. With this, it can calculate the internal forces, the deflections and the stresses in the beams and columns. These results are used to perform the unity checks for the beams and the piles.

Appendix K contains the strength calculations, performed in this section, in more detail. The results and visualisations of the loads can also be found in Appendix K.

6.6 Dimensioning of the pier deck

This section describes how the deck is dimensioned. This includes the height of the deck and the dimensioning of the different elements of the deck, which are the plates and the beams.

6.6.1 Height of the pier deck

The deck of the pier must be placed at a height, so that it does not get submerged during high water and that waves are not able to hit the deck or get on top of the deck. Therefore the minimum total height needs to be equal to the sum of the design water level and around $\frac{2}{3}$ of the wave height.

According to the fault tree from Figure 40 from Section 6.3, the reliability factor for the failure mechanism of the water reaching the deck is 5.8. The water level and the wave height are the dominant loads in this scenario. In these extreme situations, the crest of the waves reach to a level of NAP + 10.88m. The deck is be placed at NAP + 11m. This means that the pier is situated at 15 meters above the bed level. The different levels in this calculation can be found in Figure 43. A more elaborate calculation of the height of the pier deck is given in Appendix L.

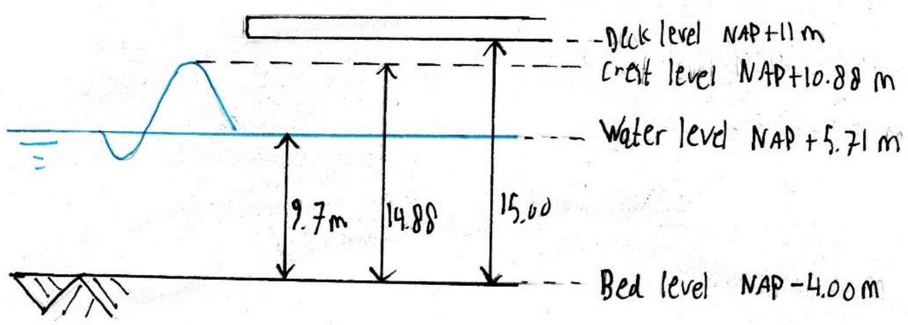


Figure 43: Water, crest - and deck level (NTS).

6.6.2 Deck dimensioning

The deck of the pier is what is between the piles and the superstructure on top of the pier. This is illustrated in Figure 29 in Section 6.1. The top of the deck consists of plates where people can walk on and on which the buildings can be placed. The deck is carried by a frame of beams. This frame increase the stiffness of the entire deck and it transfers the loads from the deck to the piles which support the deck. As mentioned before, the governing structure is the one which contains the accommodation. The building which houses the accommodation has been worked out further in Appendix J, and the loads it exerts are given there as well.

The most common material to use for beams in a pier is concrete. However, it should be treated such that concrete rot cannot occur.

The critical load situation for the deck elements is during the usage period, when the both the permanent and the variable loads from above are acting on the deck. The wind load from the building also transfers the loads to the deck, so a large wind load is part of the critical load situation. The governing loads are given in Figure 44.

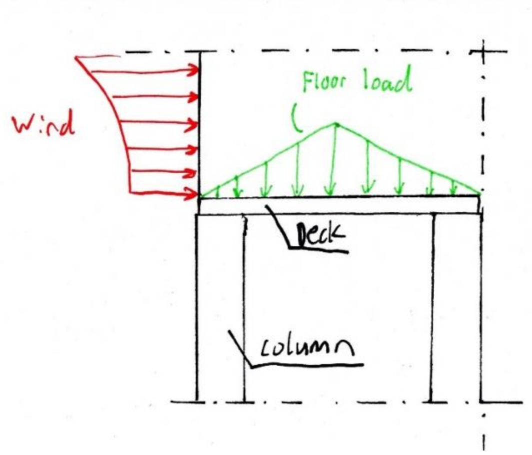


Figure 44: Critical load situation for the deck calculations

The chosen alternative contains one layer of both vertical and horizontal beams. There are 64 piles in total for the accommodation platform. The beams divide the total square of the accommodation from 90 x 90m into 49 squares of 12.86 x 12.86 m. Each beam also has a length of 12.86m. This is illustrated in Figure 45. The columns, which support the building, should be situated above the piles which go to the seabed. The plate of the deck therefore only has to carry the area load caused by the ground floor. The columns are directly supported by the piles and do not cause an extra bending moment on the beam structure.

The deck plates are made of concrete and contain tensile reinforcement bars. The beams are rectangular concrete beams with tensile reinforcement and shear reinforcement. Strength calculations have been performed to verify the dimensions of the deck. The elements are calculated on the following failure mechanisms:

- Exceedance of the compression resistance
- Exceedance of the tensile resistance
- Exceedance of the shear resistance
- Exceedance of the maximum allowable deflection.

The calculations and the exact dimensions for the deck can be found in Appendix L. The dimensions of the deck and the beams are illustrated in Figure 46. The reinforcement is well illustrated in Figure 47 to Figure 49.

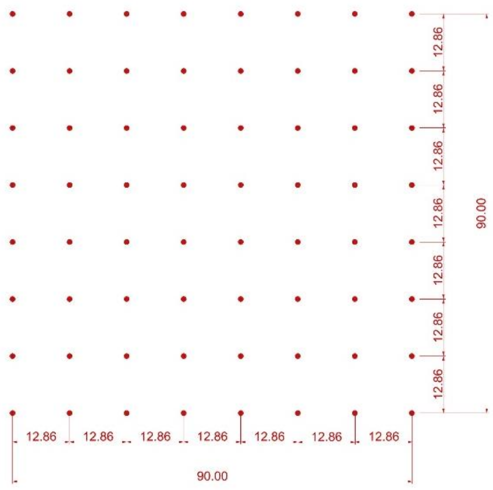


Figure 45a: Columns

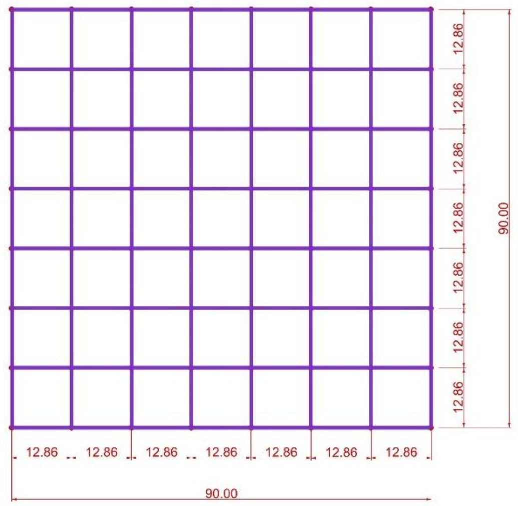


Figure 45b: Beams

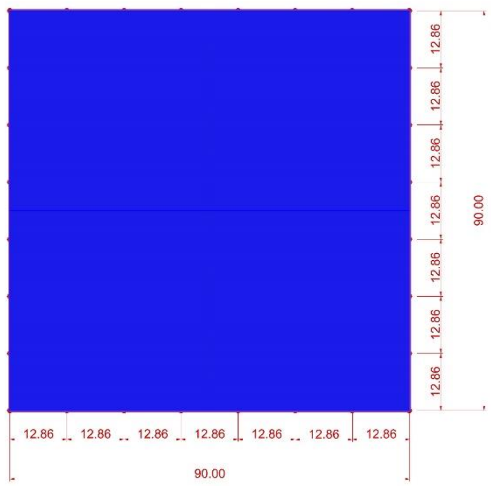


Figure 45c: Plates

Figure 45: Plan views of the different layers of the 1 beam layer alternative. Units in meters.

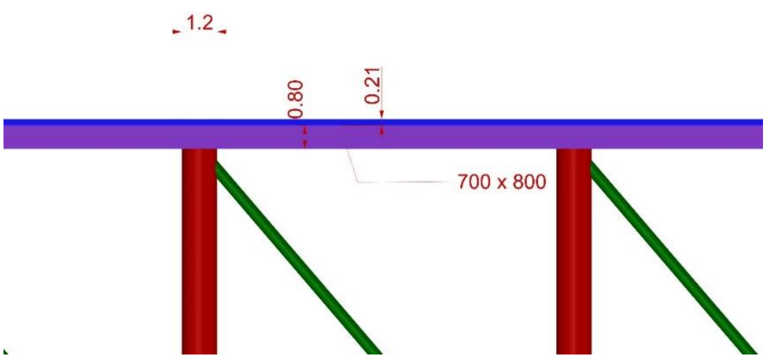


Figure 46: Side view of the structure, zoomed in on the deck.

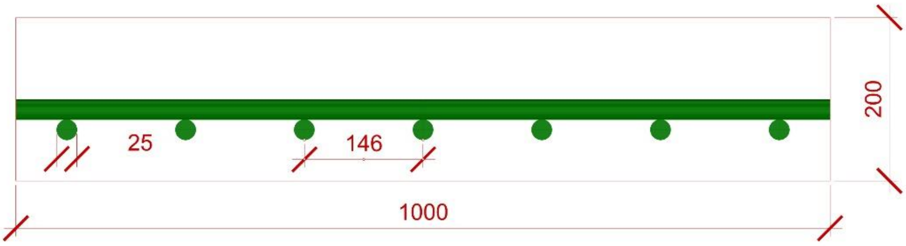


Figure 47: Cross section of 1m width of a deck plate. Units in mm.

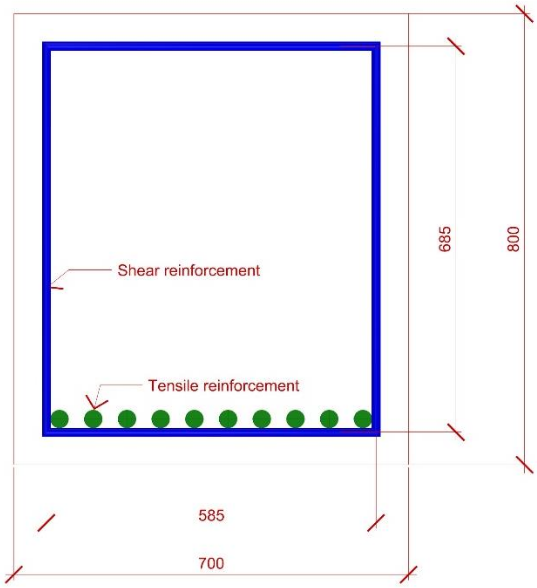


Figure 48a: Cross section deck beam. Units in mm.

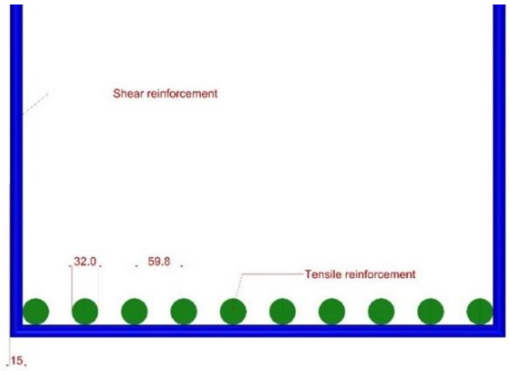


Figure 48b: Cross section deck beam zoomed in. Units in mm.

Figure 48: Cross section of the deck beam.

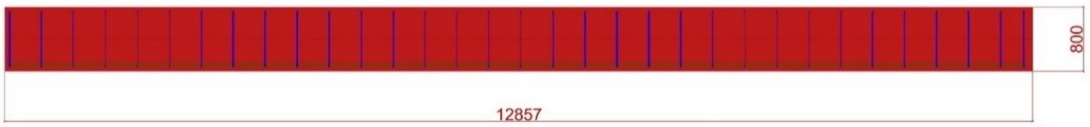


Figure 49a: Longitudinal section of the deck beam, zoomed in. Units in mm.

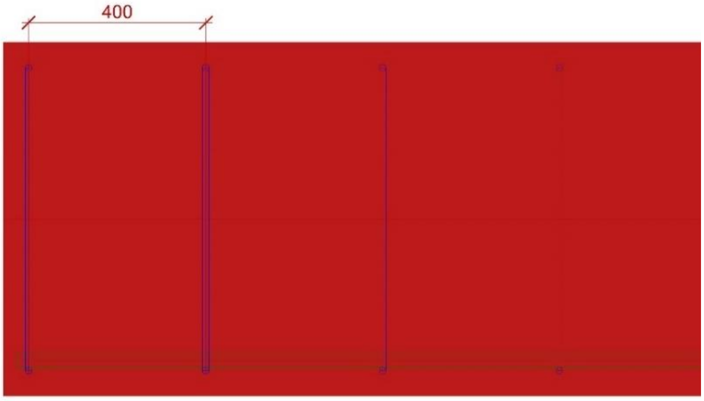


Figure 49b: Longitudinal section of the deck beam, zoomed in. Units in mm.

Figure 49: Longitudinal section of the deck beam.

6.7 Dimensioning of the piles and the pile foundation

The new pier will be supported on piles. These piles are also a part of the foundation. This section explains why the a pile foundation is chosen and how these piles are dimensioned.

6.7.1 Selection of the type of foundation

The conclusion of the morphological design cycle from Chapter 5 states that the structure of the pier should be as open as possible, which means that the flow of water and sediment should be blocked as little as possible. The best way to do this is by supporting the deck with piles. However, the foundation can be different. Two different alternatives have been created for a pile support. Sketches of these alternatives can be seen in Figure 50. In both cases, it shows the pier deck is being supported by piles, and these piles go to at least close to the sea bed. But what happens at the sea bed is different for both alternatives. In the first alternative, the piles are standing on a shallow foundation and on the right, the piles continue into the seabed, so that these piles become a part of the foundations as well.

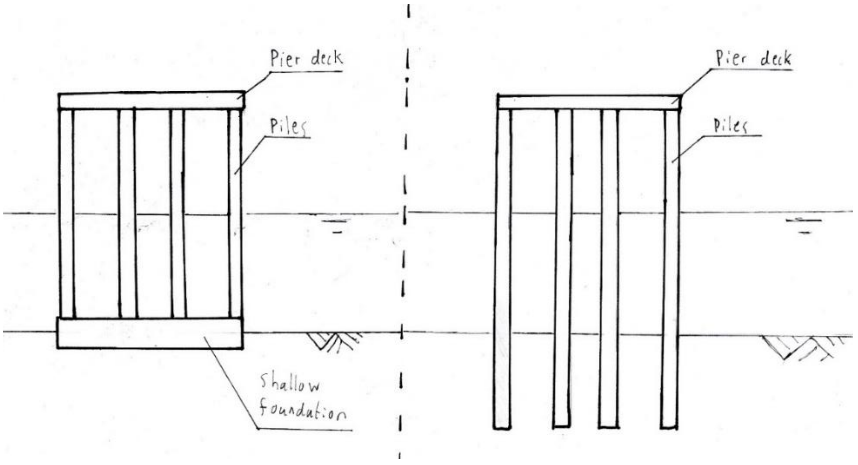


Figure 50: Schematisation of piles on a shallow foundation (left) and a pile foundation (right).

After evaluating both the alternatives for the foundation, there has been chosen for a pile foundation. This is mainly due to the fact that a pile foundation is easier to construct in this project area with breaking waves. These piles have and embedded depth of 9.7 meters. This can be seen in Figure 51.

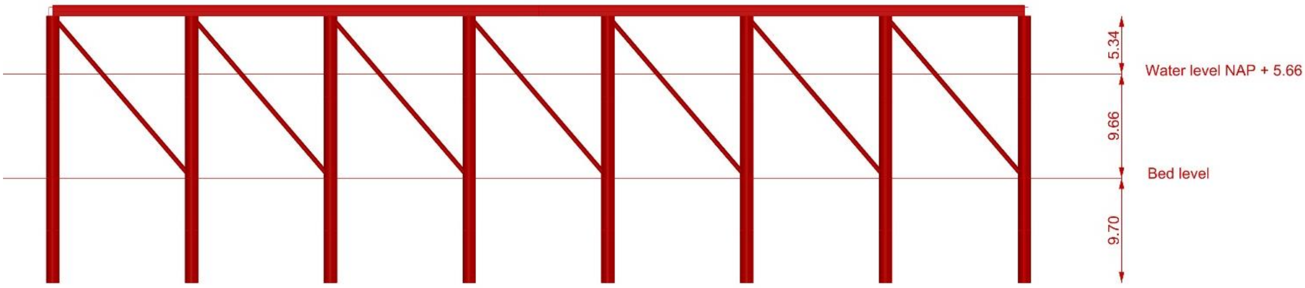


Figure 51: Front view of the pier structure, with indicated elevations.

The dimensioning of the piles is treated in Subsection 6.7.2 and the construction method of this pier is be treated in Section 6.2.

6.7.2 Pile dimensioning

After the deck has been dimensioned, the piles can be dimensioned. The piles which are located most off-shore are placed in the largest water depth and receive the most heavy loads from the sea. These can therefore be seen as the governing piles and because of this, these are dimensioned in this section.

The pier is almost 500 meters long, while going 300 meters into the sea. The bottom at that location is at NAP – 4m. The water level is assumed at NAP + 5.66m, as mentioned in subsection 3.2.3. This results in an extreme water depth of 9.66. The significant wave height H_{m0} at the location is over 8 meters, but the waves break before they arrive at the coast. $H = 0.8 \cdot d = 7.728$ m

The piles are cylindrical and made out of concrete. Therefore parameters of the dimensioning are the diameter and the length of the piles.

The most relevant forces which are working on the piles are:

- Wave force
- Flow force
- Wind force
- Soil pressure
- Load from the deck
- Self-weight of the pile

The wave force, flow force and wind force are the lateral forces which work above the sea bed. The soil pressure is the force resisting against horizontal translation or rotation of the pile. The hydrostatic water pressure is also a lateral force working on the pile, but these are close to equal on both sides of the pile, and therefore the hydrostatic water pressure does not exert a resultant force. Because of this, the hydrostatic pressure is not taken into account in the calculations. The load from the deck and the self-weight of the pile are the axial forces working on the pile.

The piles have been tested against the following failure mechanisms:

- Resisting moment of the soil with the method of Blum
- Deflection at the top
- Bearing resistance of the soil with the method of Prandtl and the method of Koppejan
- Compressive strength of the pile

- Buckling of the pile
- Tensile reinforcement of the pile
- Shear reinforcement of the pile
- Uplift of the Fundex tube

The critical load situations depend on which failure mechanism is being tested. Appendix N gives the load situations for each mechanism.

The calculations have been performed in Python, so that it is possible to perform these calculations for different scenarios and dimensions. The Python scripts calculates the loads and the resistances of the piles.

The total length of the piles is 24.7 meters, of which 9.7 meter is embedded in the seabed. These piles have a circular cross section with a diameter of 1.2m and are made of concrete. Both shear and tensile reinforcement is added to the columns. The reinforcement in illustrated in Figure 52 and Figure 53.

Appendix M explains how the wave load, the wind load and the flow load are calculated. In Appendix N, the calculations for the resistances for different failure mechanisms of the pile are given. The exact dimensions of the pile and the reinforcement can also be found in Appendix N.

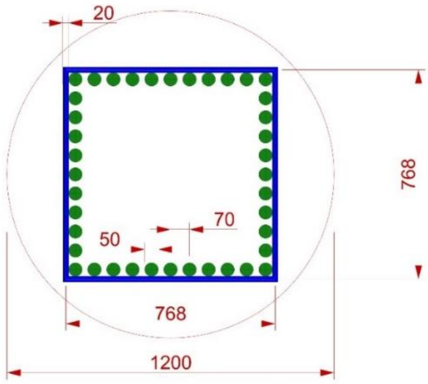


Figure 52: Cross section of the pile. Units in mm.

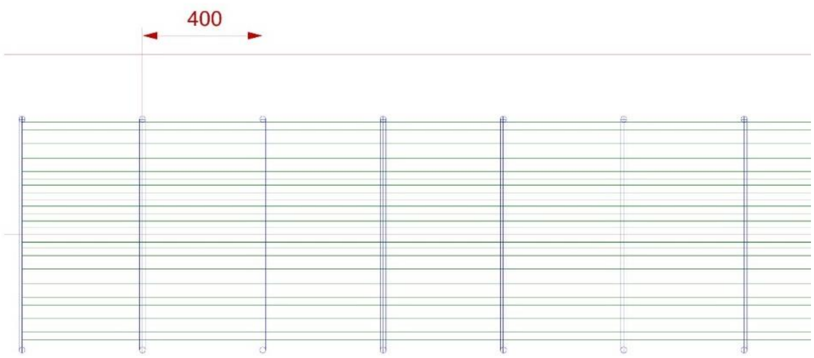


Figure 53: Zoomed in longitudinal section of the pile. Units in mm.

6.8 Dimensioning of the diagonal struts

Diagonal struts are added to the structure of the pier. These have been added in one of the iterations of the structural design loop. The struts increase the stiffness of the entire structure. Without these struts, the piles of the structure are not able to resist the lateral loads, which are mainly caused by waves.

The struts go from the top of the column to the bed level of an adjacent column. Matrixframe was used to determine which orientation of the struts would be more effective. A simplified model is made in Matrixframe, which can be found in Appendix O.

From the results of the analysis from Appendix O, it can be concluded that the struts reduce the deflection and the internal forces in the column. The only disadvantageous about these struts is that an additional axial tensile force occurs in the column. However, the reduction in the deflection and the bending moment still makes the struts a good addition. The struts themselves are mainly loaded in compression because they are hinged on both sides. Only self-weight of the struts causes a bending moment and a shear force. The profile of the struts are HP220x57. The maximum stress that occurs in the struts according to the Matrixframe model is 66.67 MPa. So it would be sufficient enough to make these struts out of S235 steel.

6.9 Bed protection

The piles of the pier influence the flow of the water. At the bed, the flow around a pile can be a factor 1.2 larger than the unobstructed flow (Schiereck, 2012). The bed protection needs to be designed so it can resist this flow.

$$u_{cr} = 1.2 \cdot 1.5 = 1.8 \text{ m/s}$$

The stone diameter can be determined with the shields formula.

$$d_{n50} = \frac{u_{cr}^2}{\Psi_c \cdot \Delta \cdot C^2}$$

$$\Psi_c = 0.03$$

$$\Delta = 1.65$$

$$C = 18 \cdot \log\left(\frac{12 \cdot R}{k_r}\right)$$

$$R \approx \text{depth}$$

$$k_r = 2 \cdot d_{n50}$$

The stone size occurs on both sides of the equation. Because of this, the equation has to be performed iteratively. An initial assumption needs to be made for the roughness, C. When the stone size is calculated, the assumption for C needs to be verified. The eventual values are given in Table 12.

Table 12: Values for the bed protection

C	$64.01 \sqrt{\text{m}}/\text{s}$
d_{n50}	0.016 m

7 Presentation of the Final design

This chapter presents the final design of the new Scheveningen Pier. Figure 54 shows which functions and types of facilities are placed on each platform. The pier consists of multiple platforms with accommodations, recreation, a hotel, a park and a conference centre. All the platforms are reachable on foot or by bike. The top layer of the pier is an outdoor section, and the lower floor is an indoor section, which also contains access routes for bikes. The new pier is supported by a concrete pile construction, which is designed to resist the loads coming from the sea and from the superstructure of the pier.

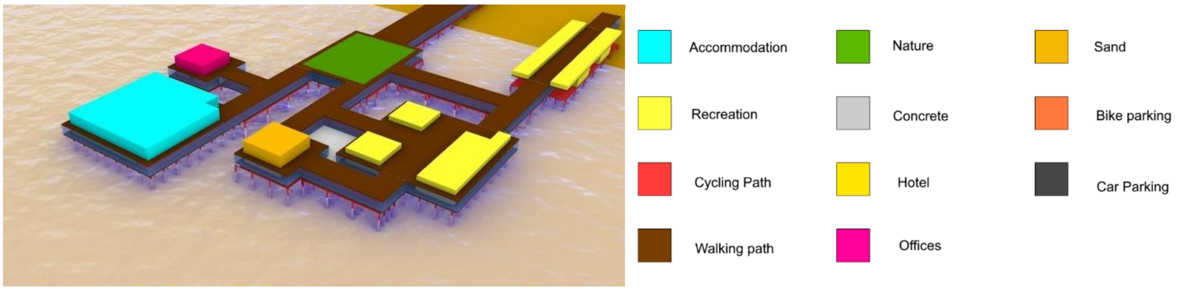


Figure 54: Birds-eye perspective of the final design with the functions highlighted in their respective colours.

The model has been improved to get a more realistic view of what the pier looks like. Figure 55 to Figure 60 give an impression of the new Scheveningen Pier. The buildings have a modern look, so that it matches with the boulevard of Scheveningen. Trees and grass are planted to create a more green environment on the pier, similar to a park. The walkways of this new pier are wider than its predecessor, so that it is less likely to get crowded on the walkways of this new pier. The trees, benches, windows and the park in the model were gathered via the Sketchup 3D warehouse (Trimble, 2021).

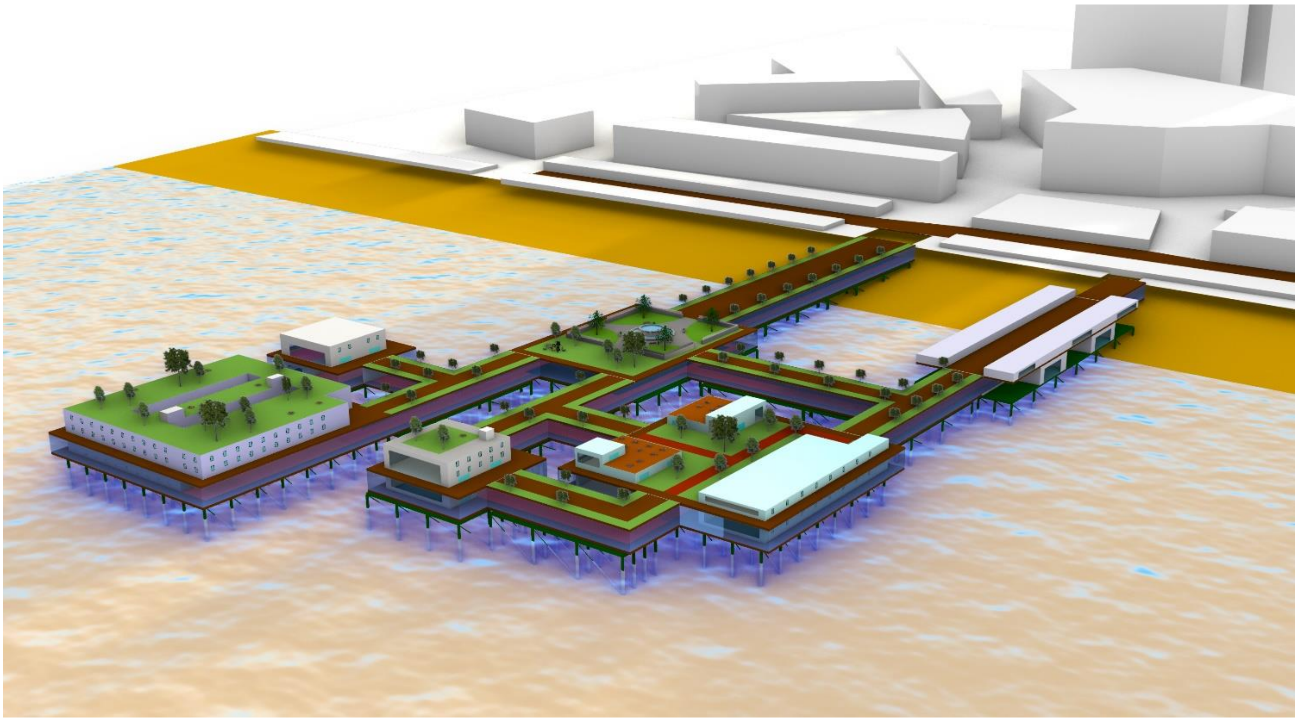


Figure 55: Birds-eye perspective of the final design

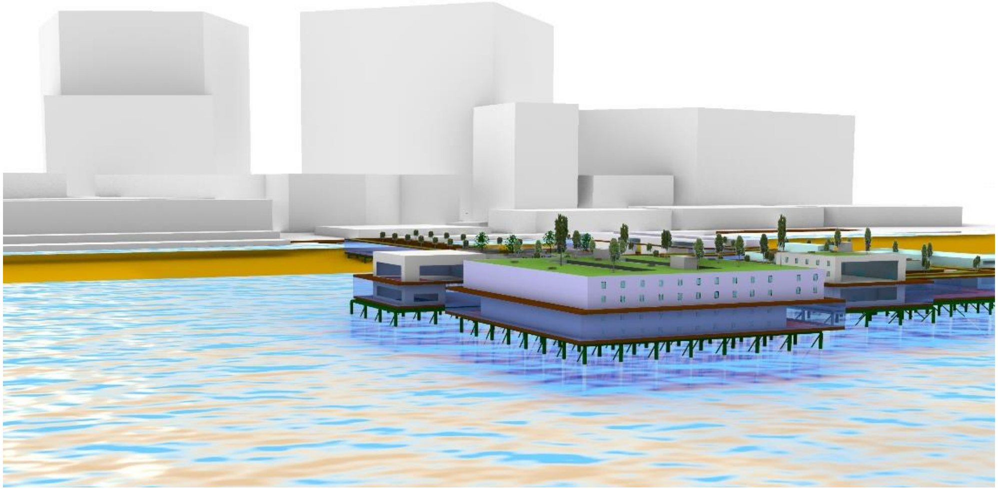


Figure 56: 3D view of the final design, perspective from the sea

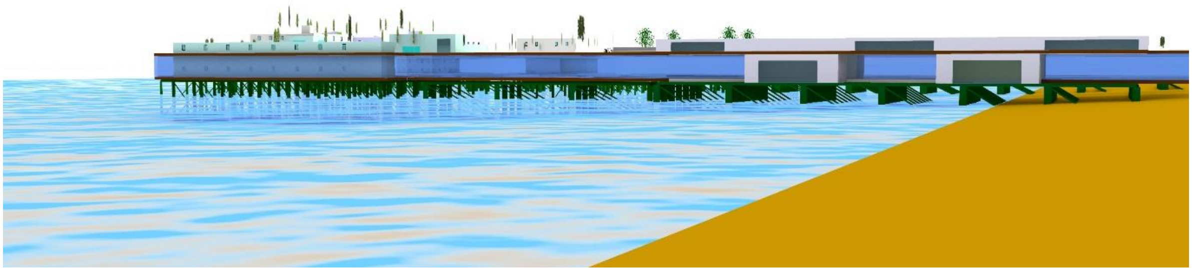


Figure 57: 3D view of the final design, perspective from the beach

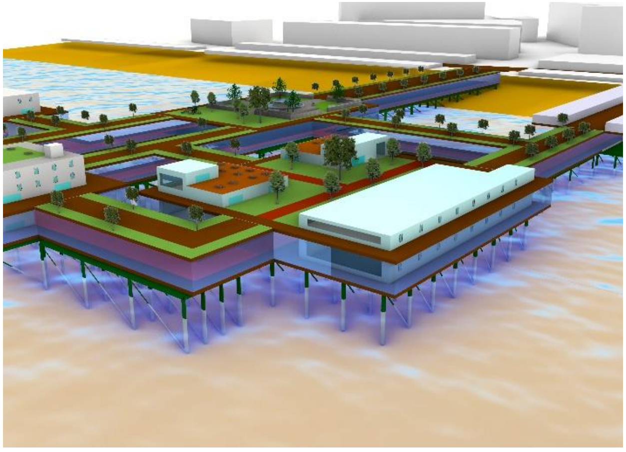


Figure 58: Birds-eye perspective on the recreation platform

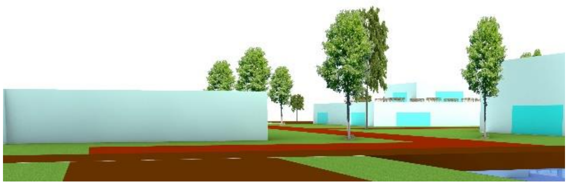


Figure 59: Perspective of the top deck of the recreation platform

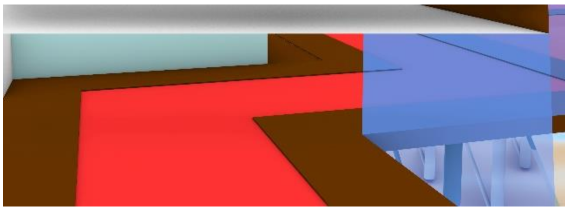


Figure 60: Perspective of the lower deck of the recreation platform

8 Discussion and Recommendations

The goal of this thesis is to create an integral design for a new Scheveningen Pier. One which is larger than the current pier and can exploit at a higher level than the current pier. A design method, which combines the engineering design method and the spatial design method has been used to achieve this goal.

The result of this thesis accomplishes the goal which has been set. The new design has a spatial plan which suffices the requirements and scores high on the criteria which have been set in this thesis. Besides that, the pier has been structurally verified with structural calculations and morphological models. However, these calculations and model lack detail. Recommendations regarding the structural calculations would be to make a structural three dimensional model, instead of a two dimensional model, and to perform a full-probabilistic analysis instead of a semi-probabilistic analysis. Besides this, more research should be done on the boundary conditions, to make these more accurate. These changes make the calculations more reliable. A recommendation regarding the morphological models would be to add more detail to these models. This includes having a more accurate bathymetry, a more accurate wave climate and to add the piles of the current design in the model as well to see what impact they have on the beach.

The calculations are only performed for the ultimate limit state (ULS). But for the design to be complete, calculations regarding other modes of failure should be performed as well. This thesis mainly focusses on structural failure, while other modes of failure can be governing for this new design as well.

There are also recommendations regarding the design method of this thesis, which has been described in Chapter 2 and Appendix C. Because of the limited time for this thesis, the entire design method has been completed once. The technical design cycle has been completed twice, once for the morphologic design and once of the hydrodynamic and the structural design. A recommendation is to revise certain design steps and repeat these steps. Knowledge from the technical design cycle can change the verification and the evaluation of the spatial concepts. For example, the conclusion of the morphological design cycle, from Chapter 5, is that the pier should have a construction which is as open as possible. This is an argument to discard the ring dam, the underwater caisson and the expansion of the beach in Chapter 4. Instead of discarding these alternatives, it is possible to alter them so that they can be verified. It is also possible to create new alternatives, but these alternatives should then all be supported by piles, since that is the most efficient structure for a pier in this situation, according to Chapters 5 and 6.

If the design process would be repeated entirely, changes to the design method are recommended. In the design method which has been used in this thesis, the technical design cycle is too isolated and occurs too late in the design method. Information gathered in this technical design cycle is useful for the earlier design steps. Therefore it is recommended to implement select design steps from the technical design cycle into the design method, instead of having a separate technical design cycle. This should result in a more efficient design cycle where the knowledge about the construction and the morphology is known more early on during the design cycle. More realistic alternatives can be made in an earlier stage and less iterations of the design loop are required.

A flowchart of this new design method with the above mentioned changes can be found in Appendix U. In this new method, the structural and the spatial alternatives are designed simultaneously. The evaluation is performed on alternatives which are spatially, structurally and morphologically verified. Performing the different types of verifications on all the different alternatives takes much time. Because of this it has not been possible to perform this design method in this thesis. It is recommended to develop design steps, like the structural verifications, into a more automated process, so that time can be saved on this design method. This also makes it possible to analyse and research more alternatives into more depth, which leads to a more accurate evaluation of the alternatives.

Appendix A Landscape analysis

This appendix shows the landscape analysis. This analysis has been performed to get an idea of the setting and the location of the project of the new Scheveningen Pier.

Function analysis of Scheveningen

Scheveningen is divided into different districts. The division of the districts is given in Google maps (Google, 2020). There are many districts with much nature in Scheveningen. Most of these consist out of woods for the majority of the area. The district of Oostduinen is located north-east of the current location of the Pier of Scheveningen. This contains much nature as well, but this is mostly represented by a large dune landscape. Then there is the Vissershaven district which occupied by a harbour. Most other districts are mostly occupied with accommodations. The distinction of the different districts and their functions have been clarified in Figure 61.

Figure 61: Large scale function analysis of Scheveningen

A function analysis has also been done on a smaller scale. This can be seen in Figure 62. Near the pier there is an area with recreation and accommodation combined. Most of the buildings there have recreational functions in the lower floors and accommodation built on top of that. Along the beach, there are many restaurants as well. Along the beach, many restaurants and shops are present. Besides these locations, the area offers most of its space to accommodation.

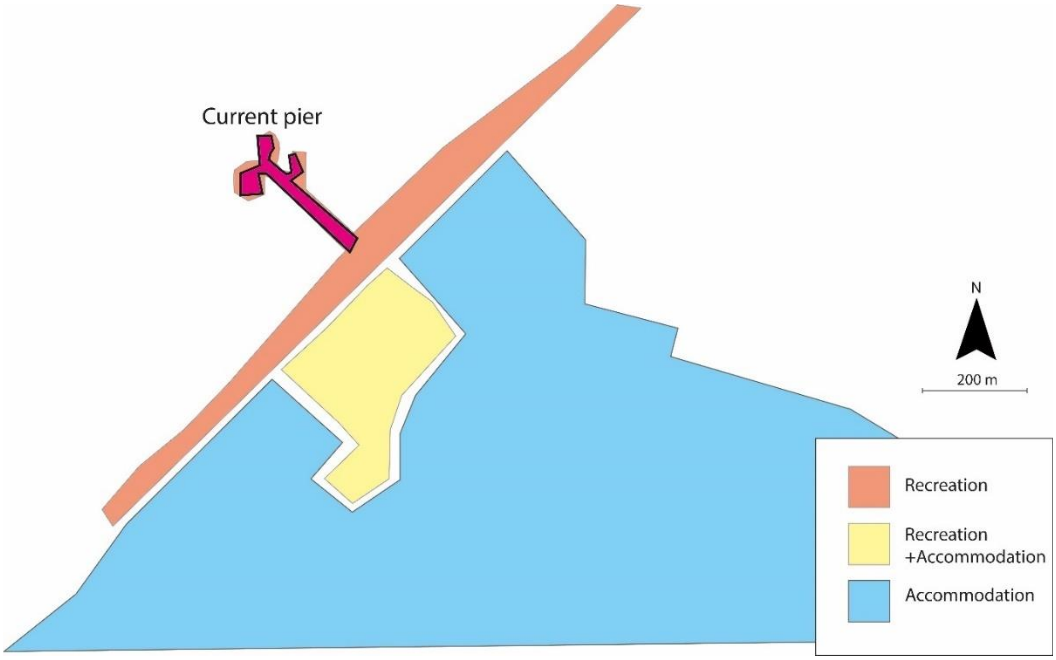


Figure 62: Small scale function analysis of Scheveningen

Function analysis of the Pier

The pier is elevated above the beach, giving an overview of the beach and the sea to the visitors of the pier. The pier consists of two levels. Figure 63 and Figure 64 give a rough schematization of the different levels. The lower level is located inside. This consists of shops, bars and restaurants. Most of these are located on the north side of this floor, while the south part is being used for the walking space. Many tables, chairs and benches are on this floor for visitors to enjoy their food or drinks. The top floor is outdoors. This floor contains several bars and restaurants as well, but less than the lower floor. There is much empty space on this floor for people to walk on or enjoy the view.

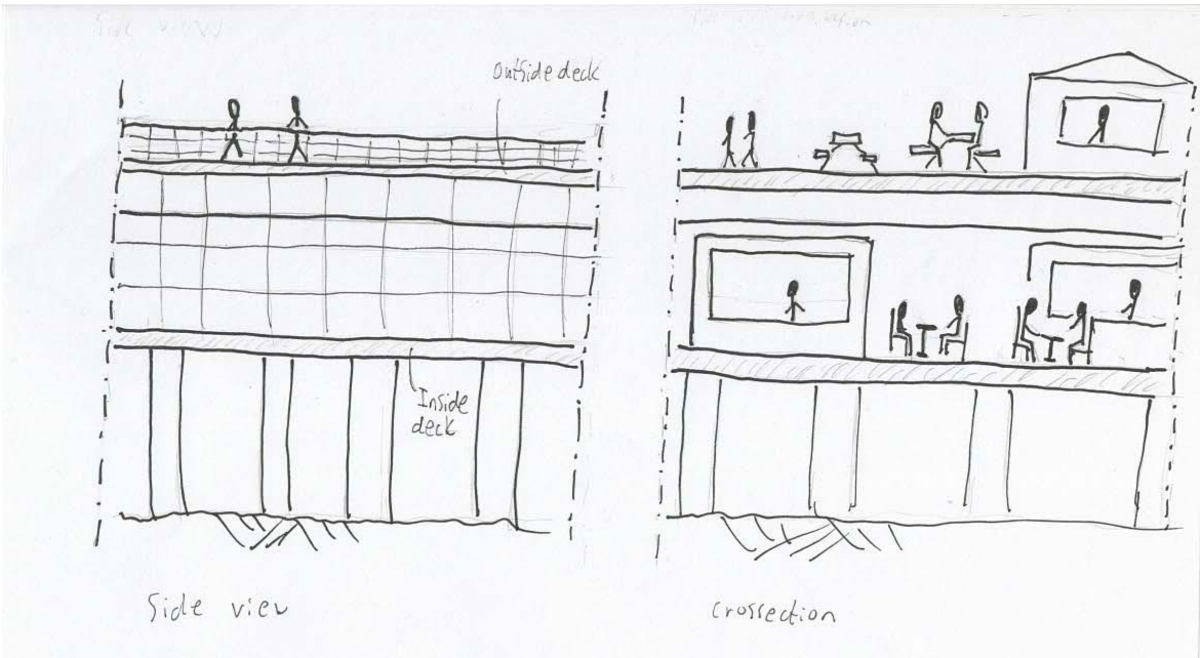


Figure 63: Sketch of the side view and a section of a segment of the pier. NTS

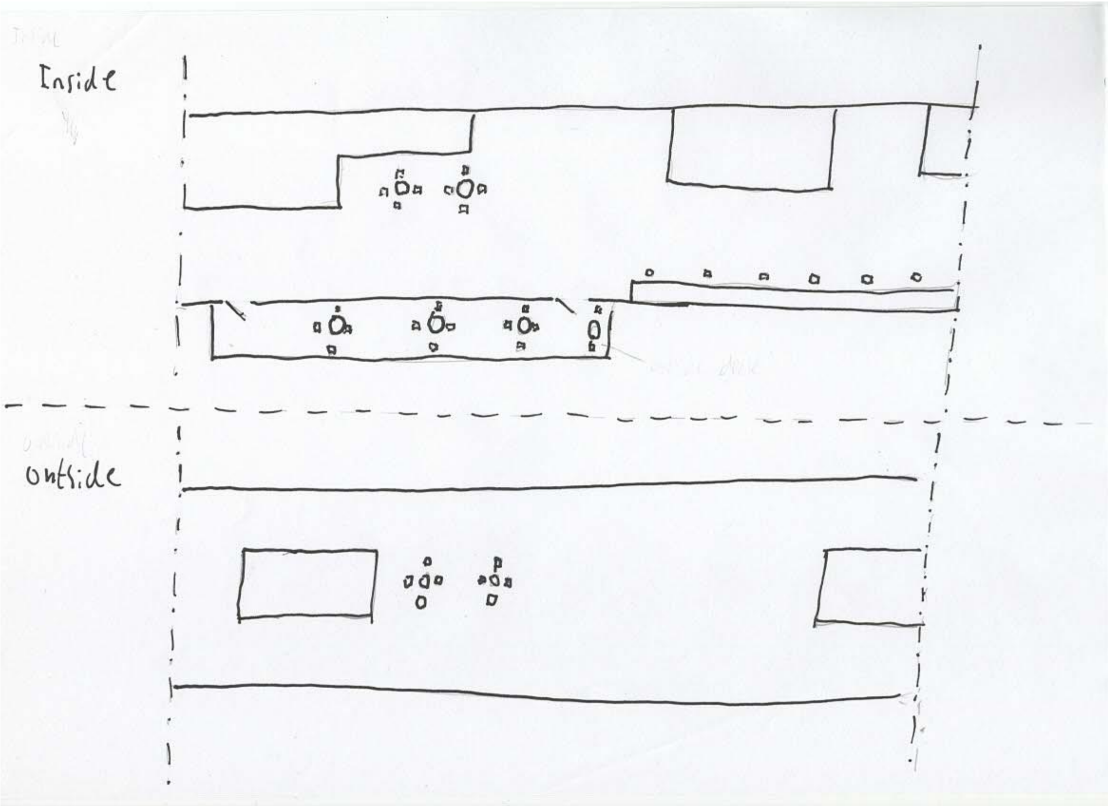


Figure 64: Sketch of the plan views of segments of the outside and inside deck of the pier. NTS

The different floors give different atmospheres. The lower floor is more crowded, in a way that there are more restaurants and shops there. If people are not using the restaurants or shops, they keep moving in the walking path. The upper floor shows a different kind of atmosphere. The many open spaces give a relatively less crowded feeling than the lower floor. If visitors are not using the facilities like the bars or the restaurants, they are not often only walk along the path. The top floor gives a good view of the sea and the beach, therefore people like to stand along the railing to enjoy the beautiful view. This difference in atmosphere makes the pier attractive to a diverse kind of audience. Along the pier there are staircases to switch between these two floors which is a good opportunity to have a chance of scenery if desirable by the visitor.

At the far seaside of the pier, there are multiple artificial islands which have different functions on it as well. The northern islands have recreational facilities. These are a Ferris wheel and a tower from where people can bungee jump or use the zipline. The southern island contains another restaurant. This one can be rented for parties, weddings and business meetings.

Appendix B shows photos taken at the Pier, to get a good impression of the atmosphere.

Function analysis of the of the boulevard

The boulevard of Scheveningen starts near the pier and continues south until it reaches the harbour. The boulevard can be accessed by foot, by bike or by car and these traffic flows are well separated by different lanes. However, near the current pier, the location is only accessible by foot. The part for pedestrians is wide and offers space for a large number of people. A variety of beach clubs, restaurants and other eateries are situated at the boulevard. The density of these facilities increases at the boulevard near the pier.

The boulevard can be used by tourists to enjoy the different facilities, or to take a walk or ride to enjoy the view of the beach and the sea. The colours and the furniture used on the boulevard gives it a modern look. Overall people stay in transition in the boulevard.

Infrastructure analysis

The location of the Pier is accessible on foot, by bike or by car. Two parking garages are in the vicinity of the pier. One of these is an underground parking garage at the boulevard. Via public transport the Pier is accessible by tram. A tram stop is located closely to the pier. It has trams coming in from The Hague and from Delft. Tourists who come from outside this region can take the tram from any of the train stations of The Hague. According to google, the trams arrive around every 8 minutes. So overall, there are many available options to reach the pier on a large infrastructure level. Figure 65 shows the infrastructure system of Scheveningen.

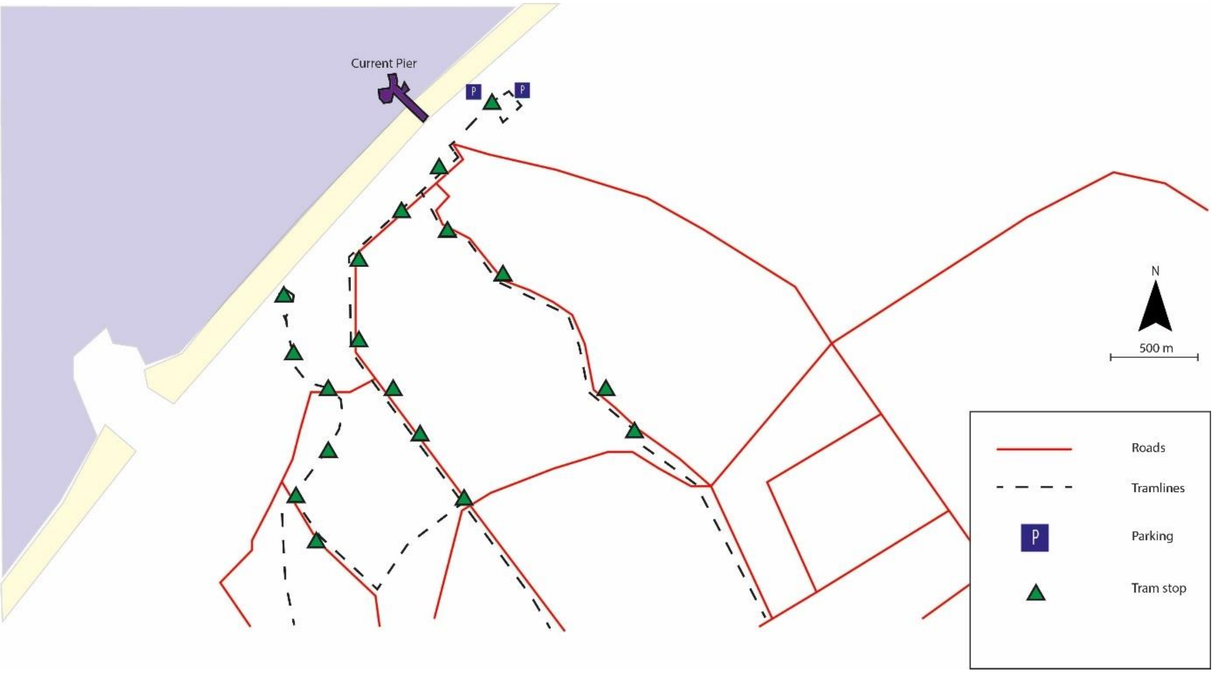


Figure 65: Large scale infrastructure analysis of Scheveningen

Historical morphological analysis

In the historical morphological analysis, old maps are reviewed to see how the landscape has changed over time.

The first relevant time period is the early landscape period. A map of this period is given in Figure 66. This is a map of Scheveningen in 1815. Nothing has been built at the coast yet, with the exception of a few buildings which are connected to the city of The Hague with a road. In this time, the area is covered by a dune landscape.

Figure 66: Map of Scheveningen from 1815 (Kadaster, 2020)

According to old maps (Kadaster, 2020), Scheveningen started to expand in the second half of the 19th century. This era is called the cultural period, where civilisation is starting to be built. Figure 67 shows this time period. More buildings and roads were built in this period. The map does not exactly show wat type of functions were built, but it does display some important buildings. It can be seen that a hotel, bath house, racetrack and a beach pavilion are present in this era. Therefore it can be concluded that the recreational functions of Scheveningen have been around since the cultural time period.

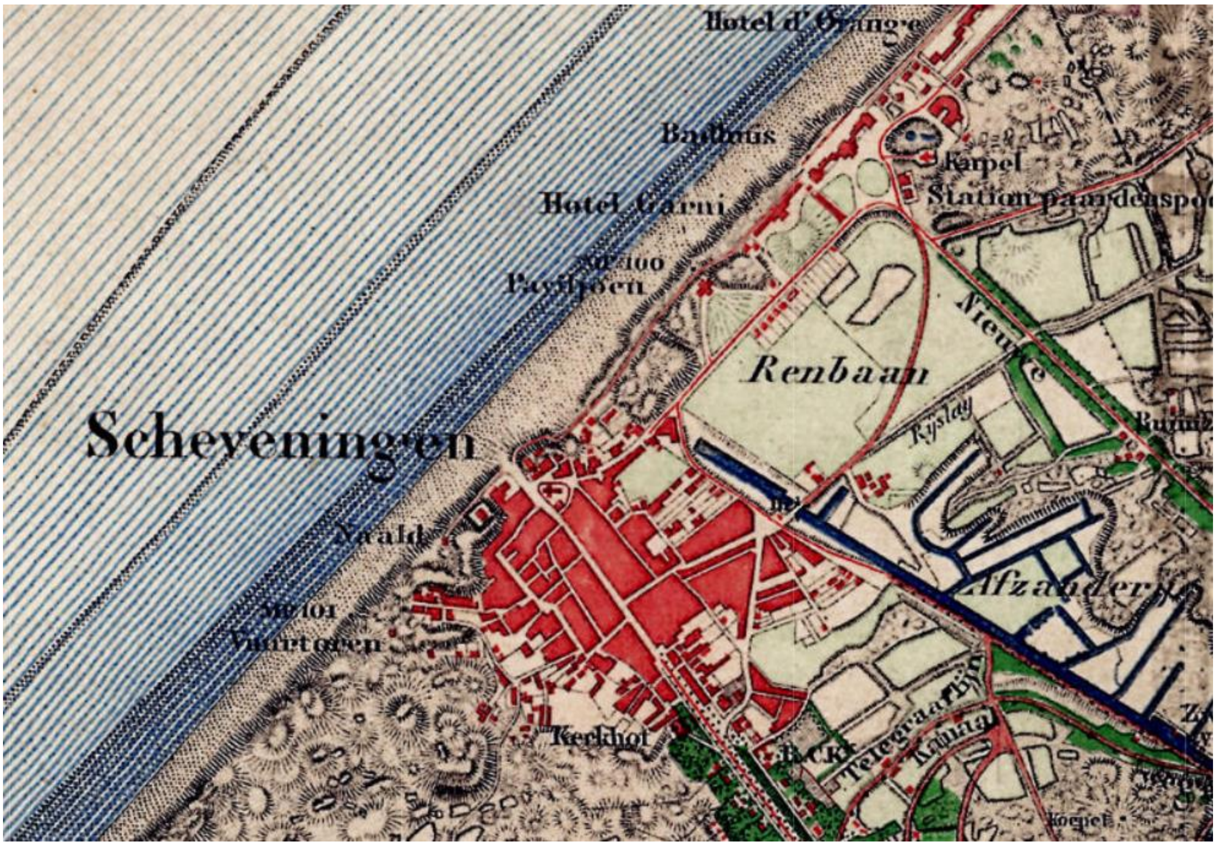


Figure 67: Map of Scheveningen from 1876 (Kadaster, 2020)

The next relevant time period is the urban period. In this period, Scheveningen starts to look more like a city. This started happening in the beginning of the 20th century, shown in Figure 68. The pier can be seen in this time period. More houses have been built and Scheveningen has expanded even more. Another notable aspect is that Scheveningen started building more near the coast as well.

The final period is the present. The changes are similar to the last one. The city has expanded more, growing more towards the city centre of The Hague. More recreational functions have been added to the beach, like the boulevard and beach clubs. This is displayed in Figure 69.

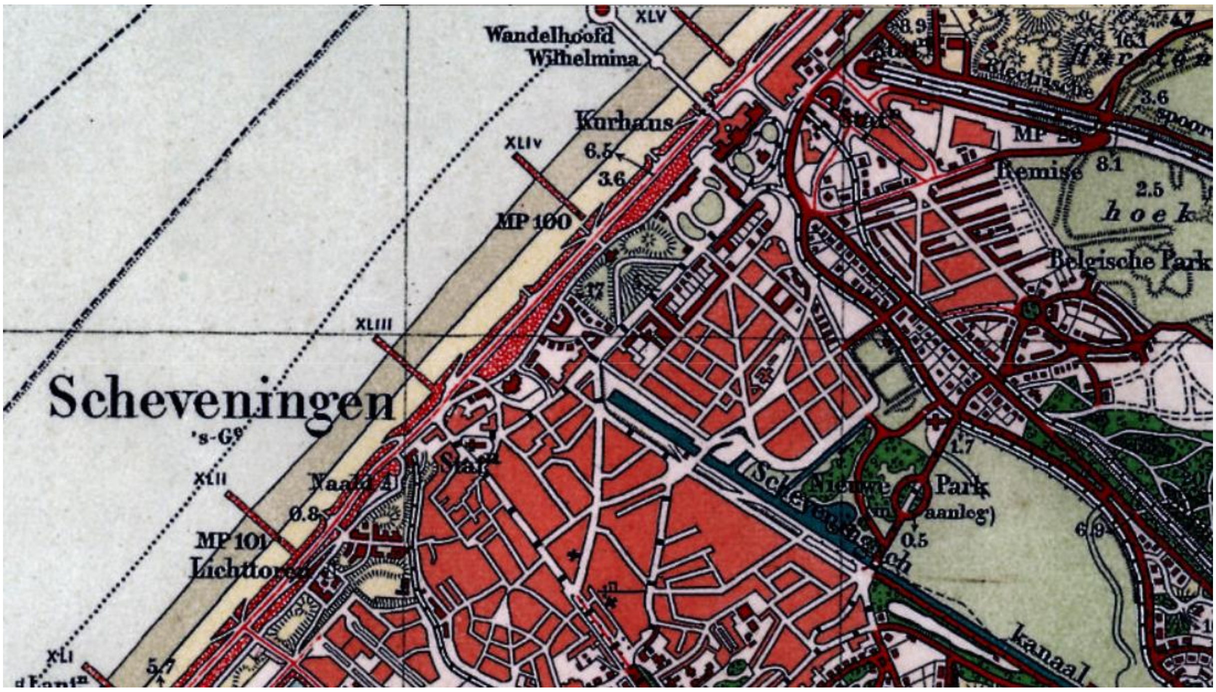


Figure 68: Map of Scheveningen from 1924 (Kadaster, 2020)

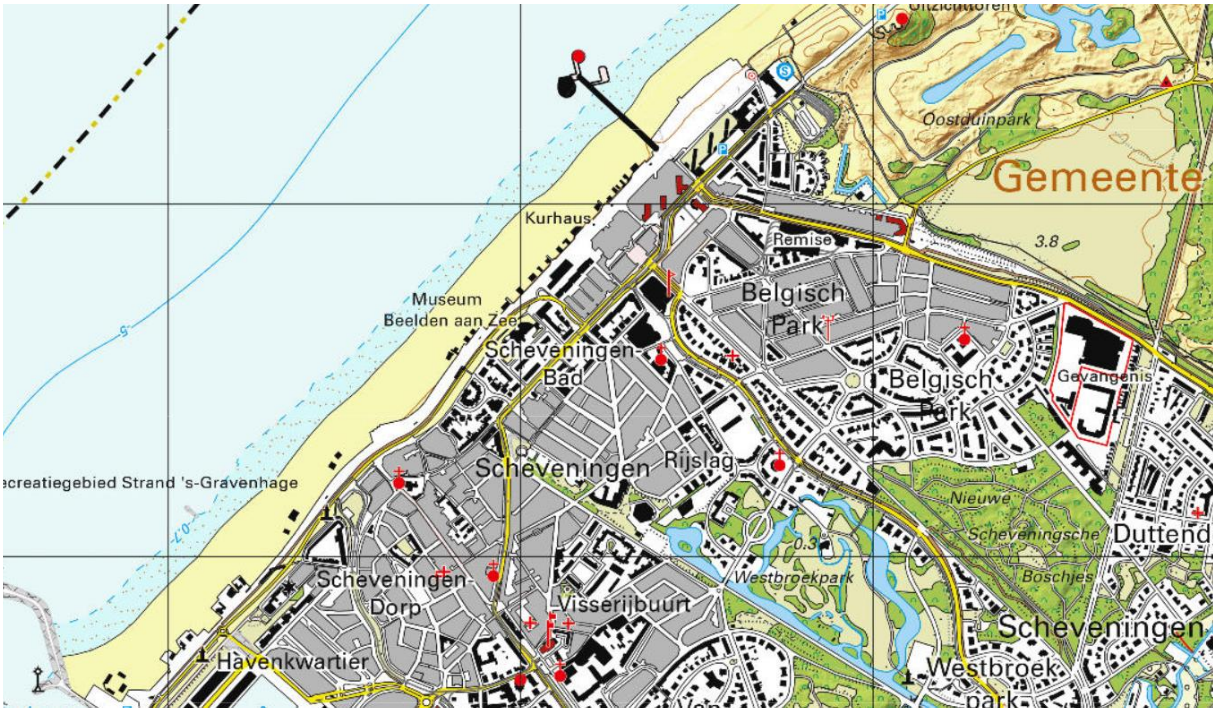


Figure 69: Map of Scheveningen from 2019 (Kadaster, 2020)

Visual space

The visual space analysis looks at the characteristic aspects from the surroundings. Usually this is done from a person's perspective or from a birds-eye perspective. These are outstanding parts of the surroundings. For this analysis, a drone video has been watched which goes from the harbour to the pier, to get a good birds- eye perspective (Oudshoorn, 2017).

- Lighthouse
- Boulevard
- Different elevations of the boulevard
- Beach clubs
- Dike structure
- Kurhaus
- The pier
- Ferris wheel
- Bungee tower

The visual space analysis looks at how the design is experienced when people visit the location. To analyse this better, the location has been visited. Photos from this visit can be seen in Appendix B. The pier is an icon and can be seen from a large distance. A good view from the beach into the horizon is possible for a large part.

Diagnosis of the landscape analysis

Back in the day Scheveningen was covered by a dune landscape. When the city was built, it started as a recreational area. Since then it started expanding over the years, becoming a part of the big city The Hague. Over the years the recreational area has expanded as well, making more use of the beach.

The pier is located in a recreational area which is surrounded by a large area of nature and accommodation. Therefore people from the surrounding area and from far like to visit the pier to have a good time. A new pier should keep this function or improve on it. The Pier itself offers a large variability of different functions. This variability should be remained in a new design regarding the recreational part of the pier. The pier does contain many walkways. People stay in transition a lot. People are able to stand still and enjoy the view, but there are not many public spaces on the pier for this function.

However, the current pier and the boulevard are not aligned. The pier has a rather different style and atmosphere in comparison to the boulevard. The boulevard is more wide and more modern, while the pier is relatively small and has outdated aesthetics.

Appendix B Photos taken at the location

These photos were taken at the current Scheveningen Pier and its surroundings. This has been done to research the atmosphere of the current pier and its surroundings.

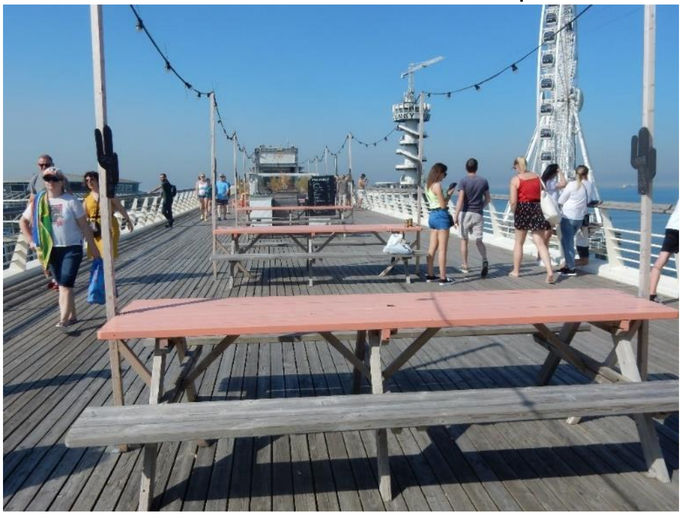


Figure 70: View of the top floor of the pier.

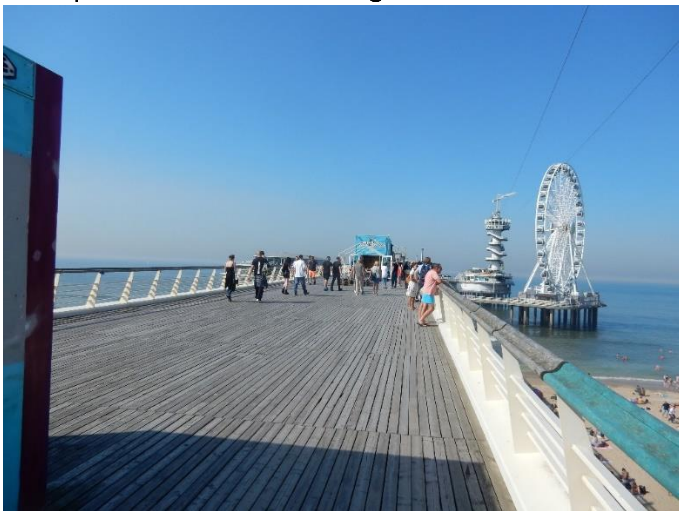


Figure 71:View of the top floor of the pier.

Figure 72: View of the beach and the boulevard from the top floor of the pier.

Figure 73: View of the beach and the sea from the top floor of the pier.

Figure 74: Restaurants at the lower floor of the pier.

Figure 75: Seats at the lower floor of the pier.

Figure 76: A view of the pier from the right side of the boulevard.

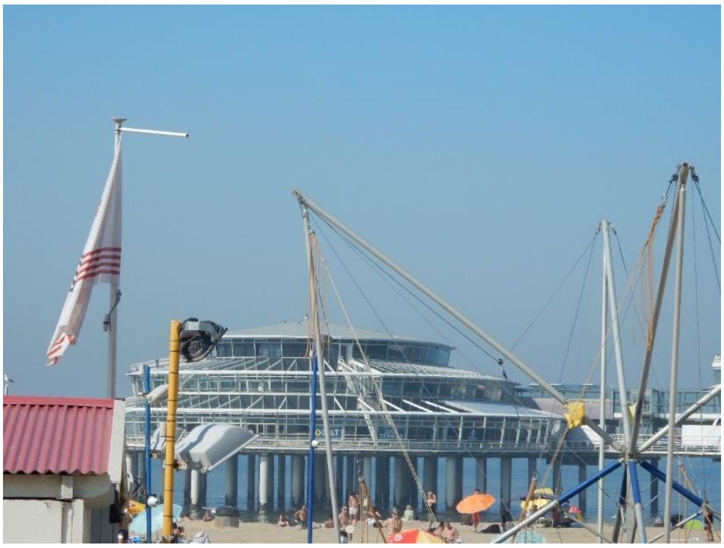


Figure 77: A view of one of the islands, located at the end of the pier.

Appendix C Explanation of the methodology

This appendix gives a more elaborate explanation of the methodology which was mentioned in Chapter 2.

1. **Problem exploration**

The design process starts with exploration of the problem of the current situation. This step results in a problem statement. The goal of the new design for the pier of Scheveningen is to solve this problem. This step should not result in concrete requirements yet, this is done in a later stage. However, the order of certain hydraulic boundary conditions can already be determined since this could be useful while developing concepts. Doing this without that knowledge can lead to unnecessary iterations of the concepts.

In this step, a landscape analysis has been performed as well. For this analysis, the current pier and the surroundings of it are analysed. This is done on different scales and for different systems. The largest scale is the entire city of The Hague, while the smallest scale is the neighbourhood of the pier, like the boulevard, the beach, the sea and the surrounding streets. The systems that are analysed are the buildings, the infrastructure and the public spaces. For this analysis, the site has been visited and photos were taken to get a good impression. A useful tool which is used in this part is drawing on maps of the surrounding. Drawing a map of each system individually helps to distinguish the systems from each other, which makes it more clear to analyse them individually.

. A landscape analysis answers the following questions (Van der Velde, 2020):

- What is particular about the form and configuration of the area?
- What kind of land-uses are present and how are the accessed or connected?
- What are the spatial- visual characteristics of the area?
- What functions shaped the surroundings as it is?

2. **Development of concepts**

The second step is where this integrated design method (Voorendt, 2017) differs from the engineering approach (Roozenburg & Eekels, 1995). In the engineering approach, criteria and requirements are already set, and they impact the concepts made in the synthesis phase. However, in the integrated design method, only the goal, acquired from the exploration of the problem, is used to create concepts. They do not have to suffice any requirements yet, this comes in a later design step. The focus here is that the concepts are designed to accomplish the goal of this project. It is of importance to already integrate the different functions and subfunctions in the design. So the technical design and the functional plan should already be combined while developing concepts.

For this phase, the creative part of the mind should be stimulated. Brainstorm sessions can help to achieve this. Hand sketches are made to give shape to new ideas. Reference projects with the same or similar goals have been researched to see how other designers over the world have accomplished these goals. Different visions should be used to come up with different designs.

The product of this step is multiple concepts which are verified and evaluated in the later design steps.

In following design steps, more knowledge becomes available. This knowledge could lead to different concepts, therefore this step should be revisited often to process new knowledge into the design.

3. **Analysis**

There are multiple and different kinds of analysis that are performed in this design step. This design step explains these analysis.

Stakeholders*:* This analysis looks at the people and organisations that have influence, interest or both with this project. It is of importance to know what they value. The different stakeholders are listed. Their relation with the project is and how much influence they can exert is researched.

Technical analysis*:* The previously mentioned analysis are more related to the functional aspect of the design. An analysis of the technical aspects is required as well. In this design step it, the boundary conditions do not need to be known exactly, it is more important to know the order of magnitude of certain conditions, to come up with an integrated design. The boundary conditions are worked out in more detail in the technical design cycle.

4. **Functional specification**

This step follows from the analysis of the previous step. The functional specification uses the results from the analysis and turns these into clear requirements, criteria and boundary conditions. The requirements have to be met by the design, while criteria are used to evaluate the design. Boundary conditions are given by the surroundings and should be taken into account when the design is made.

As stated before, there are two different design loops within this design method. The first one focusses more on the design in general, while the second loop goes more into depth on the technical and structural aspects. Because of this, the requirements are categorised into two different categories: The functional requirements and technical requirements.

Requirements can come from the client or other stakeholders. The criteria are often derived from the desires of the stakeholders. The boundary conditions are mostly given by the soil, the sea, laws and regulations.

The product of this step is a clear overview of all the requirements, criteria, boundary conditions.

5. **Verification of concepts**

This step elaborates on the concepts of the second design step, the development of concepts. These concepts are being verified with the requirements and the boundary conditions obtained from the functional specification. The concepts have to suffice the requirements. In the integrated design method of Voorendt (2017), there are two types of verification. These are the functional and the structural verification.

For the functional verification, a functional design is made for each concept to verify if it is feasible and if enough space is available. This functional design adds more detail to the rough concepts.

These are aspects which the functional plan of a new pier includes:

- Access routes to the pier.
- A lay-out of the different functions on the pier.

A structural verification is performed in the technical and structural design cycle of this method. This is because the structural verification is relatively detailed in comparison to the functional verification. It would take much time to do this for multiple concepts. Therefore this is done when an alternative has been chosen already.

During this step, it could occur that the concepts need to be changed, to be able to suffice the requirements and resist boundary conditions. Therefore an iterative loop between this step, and the second step, the development of concepts, is added to this design method.

At the end of this step, the concepts become alternatives. They have been verified and are thereby deemed realistic.

6. **Evaluation**

The alternatives that remain after the fifth step are compared with each other during the evaluation step. Common methods of evaluation are a multi-criteria analysis (MCA) and a cost-benefit analysis. However, the latter one is rather complicated. For the construction, it is relatively simple to make a cost estimation. But for the other functions like accommodation, recreation and business, this is more complicated because it is uncertain how much the building would cost or what revenue it could generate. This would also be outside of the scope of this project.

A multi-criteria analysis is used instead to evaluate the different alternatives. Therefore, the criteria from the functional specification are necessary. The criteria are given a weight. This weight determines the importance of each criterion. The analysis could be executed with different visions, because each vision would differ in the amount of value they have to a criterion, which means that the weights are different for each vision as well. After the weight has been assigned, each alternative gets a score for each criterion. Combined with the weights, this results in a final score. This score can be a helpful tool to make a decision between the alternatives. It also shows which alternative is more preferable for each vision.

The following step in the design method, used in this thesis, is the technical and structural design. This is relatively detailed in comparison to the previous steps. Therefore it seems time-inefficient to do this for multiple designs, given the time-span of this thesis. It has been decided to make a decision for an alternative after the sixth step already. Instead of choosing an alternative, it is also possible to combine the alternatives. Using the bests aspects from multiple alternatives to get a better combined version.

7. **Technical Design cycle**

The technical design cycle is the main focus of this thesis. Until this step, a functional – and spatial design has been determined with step 1 until step 6 in the combined design method. This separate design loop focusses on the structure, the construction and the protection of the new pier. It is possible that during this loop, new information becomes available and alterations need to be made to the functional plan so that it complies better with the structure.

Just as the main design loop, the technical design loop needs a goal and a problem definition as well. This is more definable than the goal of the main design loop. The goal of the structure in this case is to offer safety from the various loads that come from the sea.

Requirements, criteria and boundary conditions have been determined in the functional specification of the main design loop. But in the case of the technical design loop, the requirements are already used to make different alternatives for the structure of the pier. Various options can be made for the structure, the foundation, construction method and the materials. These different alternatives are then compared with each other in a separate evaluation, which can help to choose the best alternative.

This alternative is then calculated further, so that the required elements can be dimensioned properly. This step contains many technical calculations. In step 5, it was mentioned that the design method described by Voorendt (2017) has a verification with a functional and a structural verification. This part of the technical design loop is the same as the structural verification from Voorendt's(2017) method.

The structural elements need to be calculated to see if the structure of the concept can be realised. For this the loads and the resistance need to be calculated. Besides the structure itself, the construction method should be verified as well. If there is no feasible way to build the concept, it is not feasible at all.

First, all the possible failure mechanisms that could occur at the structure need to be determined. These need to be calculated. If, during this thesis, this costs too much time, only the most governing failure mechanisms are calculated into more detail.

There are multiple aspects to take into account when dimensioning a structure in the sea. Unfortunately, not all these aspects can be taken into account during this thesis. Therefore choices have been made for which aspects are calculated into more depth. These are:

- Morphologic design
- Hydrodynamic design
- Structural design.

With the morphology, there is looked at the impact the new construction has on the beach. The hydrodynamics looks at the different loads from the water on the construction and the construction mechanics looks at the different elements of the pier. They are structurally safe, which means that the stresses and the deflections do not exceed any critical values.

The technical design cycle which is being performed in this step is performed at least two times, once for the morphologic design and once for the hydrodynamic and structural design.

At the beginning of this thesis, there are various unknowns of the technical design loop. For example, the goal, the requirements and the types of failure mechanisms of the construction depend on the functional and spatial plan as well.

8. **Integration of subsystems**

In this step, the structural and the spatial design are combined in a visual model, to be able to present the entire new pier as a whole. More detail is added to the subsystems of the total design.

9. **Validation**

The validation is the final step of the design method. This step checks whether the set of requirements is correct. Therefore this step is connected to previous steps as well. Whether this leads to significant changes in this thesis depends on the available time. Unfortunately, in the time given it is not possible to come up with a perfect design which is worked out in full detail. If it is not possible to implement the required changes in this thesis, they are discussed in the discussion and recommendations.

Appendix D Concepts for subsystems

This appendix displays ideas which were made for certain subsystems. These concepts can be used for multiple designs which were mentioned in Section 4.1.2.

Seaside platform

This circle-shaped platform is located at the end of a long walkway and is located above the sea. In the centre, a park is located to add nature to the design. The rest of the square contains terraces for restaurants or bars for people to enjoy. These bars and restaurants are located on the sides. Apartments and hotels are built on top of these. A walkway around the circle-shaped platform is built so that people can take a walk and enjoy the view of the beach and sea. A sketch is given in Figure 78.

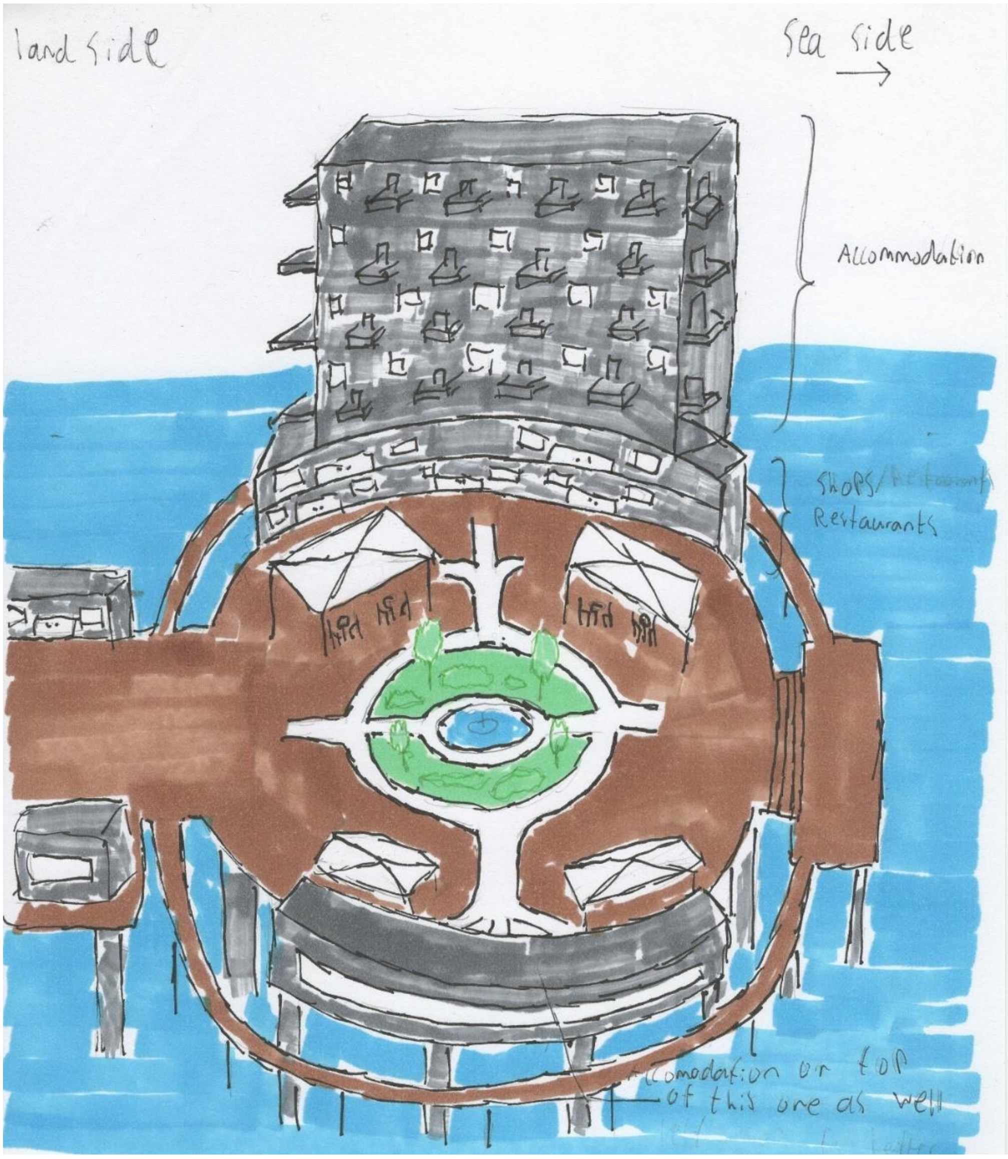


Figure 78: Sketch of seaside platform

Green walkway

To add more nature to the design, a green walkway can be added to the design. This is a walkway with nature. A good example of this is the High Line in New York City, which is an elevated walkway with nature, which is located in the middle of the city. A concept of this is given in Figure 79: Rendering of the High Line in New York City Figure 79.

Figure 79: Rendering of the High Line in New York City (High Line, 2020)

Appendix E Elaboration on the stakeholders

In this appendix, the different stakeholders which are involved in this project are analysed. This is an addition to the stakeholder analysis, which has been performed in subsection 3.1.1. This stakeholder analysis has been performed to analyse which parties are involved and what their role is in this project.

The most important stakeholders are mentioned below.

- The client (De Pier BV, Kondor Wessels, Danzep BV)
- The municipality of The Hague
- The local residents
- Local businesses
- The new residents of the Pier
- New businesses of the pier
- Tourists
- New Hotel on the pier
- New Conference centre
- Environmental organisations

To get a good overview of them, they are sorted into different groups. The different groups are distinguished by their power and their interest in the project. The different groups are well displayed in Figure 80.

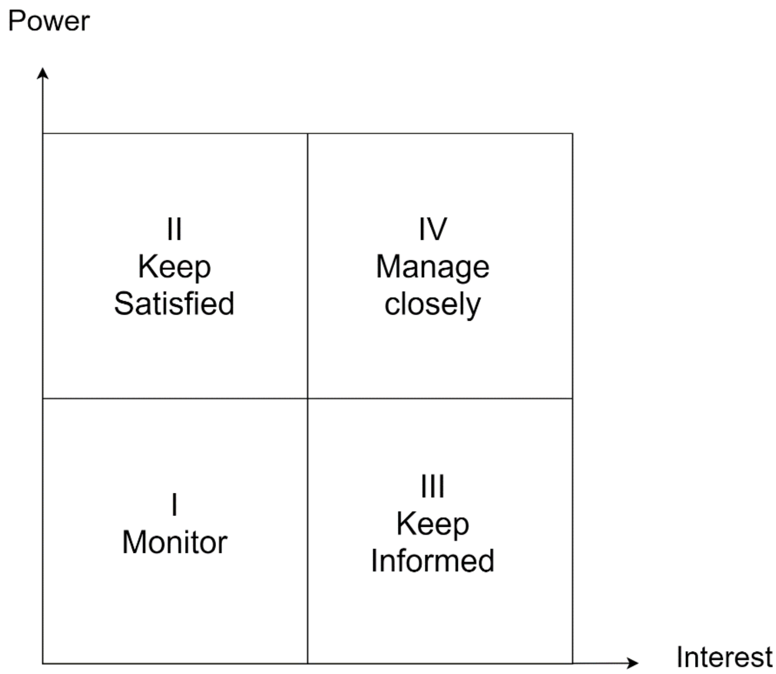


Figure 80: Stakeholder Matrix

The stakeholder matrix shows how different stakeholder should be treated. This depends on their power and interest. Of course it is important to take all the wishes of all the stakeholders into account, but separating the stakeholders in different groups can help to prioritize certain stakeholders and wishes. Figure 81 shows where the different stakeholders are located in this matrix.

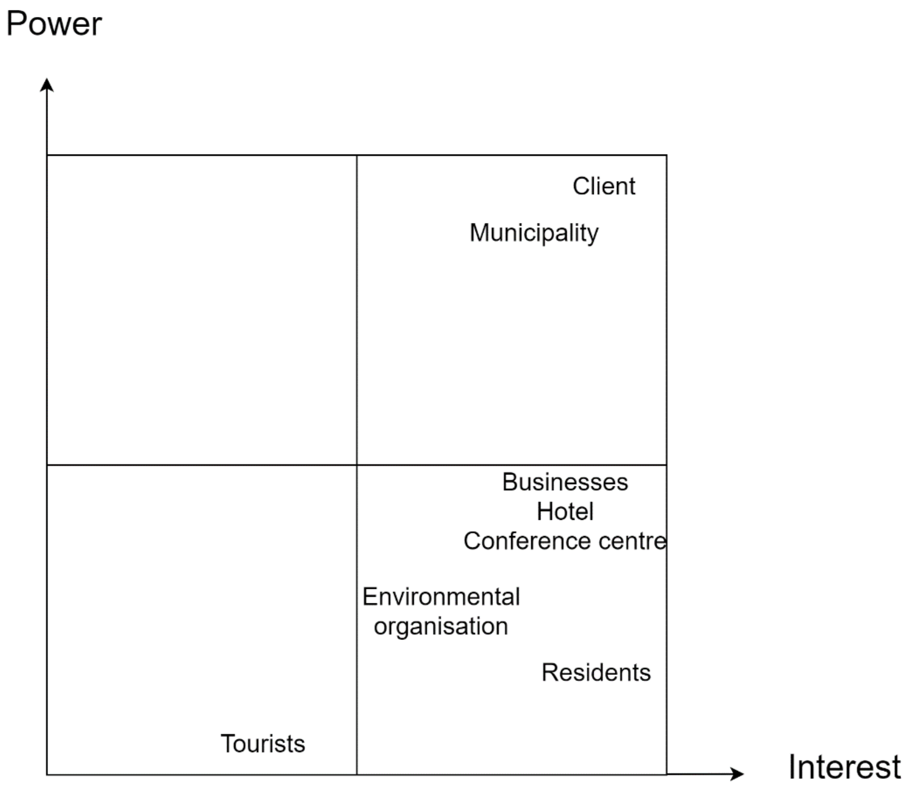


Figure 81: Positions of the stakeholders in the stakeholder matrix

Underneath follows a small explanation of each stakeholder, what their interests are and what power they have in this project.

The client

After going bankrupt, the Pier was bought by De Pier BV, consisting out of Konder Wessels Vastgoed and Danzep BV. Kondor Wessels is a real estate company. Danzep BV, also known as Xcentric Hotels BV, owns multiple hotel chains, develops buildings into new hotels and they currently have a venue at the end of the pier. Since they own the new pier, they have much power and interest, because they need to approve the plans in the end. They want to build a pier which creates high revenue for them. This pier should last for the long term. The project should be cost-effective and should be built as fast as possible. On their website, Kondor Wessels also mentions that they want a large diversity of functions on the new Pier, which can be exploited for the long term. However, a long term plan is currently still under construction (Kondor Wessels Vastgoed, 2020). They want the new pier to be an improvement of the current pier.

The municipality of The Hague

Scheveningen is located in The Hague, therefore the municipality of The Hague has much interest and power in this project. They need to approve the plans of the new pier before it can be realised. They see the seaside resort of Scheveningen as an important part of The Hague. It creates business - and employment opportunities for the local residents. Therefore it is a good economic stimulus for the city. The main interest of the municipality in this project is to increase the attractiveness of the seaside resort of Scheveningen.

In 2018, an agreement was written between the municipality of The Hague and De Pier BV, about the second phase of the plan for the pier (Revis, 2018). The first phase was about refurbishing the current pier and is currently still in effect. The second phase is about the development of a new pier. The municipality mentions the following parts of their vision:

- The municipality wants to invest in the touristic sector to increase the position of Scheveningen as a seaside resort.
- The goal of the second phase to is to have a long term exploitation at a high level. The definition of a long term and a high level is not mentioned. However, it is hereby assumed that this is relative to phase one, which lasted for 5 years and exploited at the current level.
- The new project should have public support.
- The new project should be a semi-public space for everyone.
- The outside spaces should be attractive for tourists, residents and entrepreneurs
- The new pier should be an icon

The municipality also has a programme called: De kust gezond. This programme is made to increase the area of Scheveningen Bad, which is also where the pier is located. The government is investing 25 million euros to make the area more attractive to residents, entrepreneurs and visitors all year long (Den Haag, 2020).

A Dutch news website mentions the plans of the municipality and the and the new owners of the pier to demolish the old pier and construct a new one (Navis, 2020). This new pier should include accommodations, a hotel and a congress centre. Plans are made to construct a modern version of the old pier, which was located in front of the Kurhaus.

Local Residents

The local residents is the group of residents that live near the project site. These stakeholders do not want to experience any nuisance from the project. During both the construction and the usage period. They can use their influence complicate the design – or the construction process if the design results in too much nuisance to their home environment. Therefore it is important to listen to this group and to cooperate with them, to come to a design which is acceptable for everyone.

Local Businesses

These are the businesses that are located near the project site. These are mostly located at the boulevard, at the beach or near the current pier. They consist mostly out of shops and eateries. They would value an attractive location so that their businesses will become more attractive as well.

New residents of the pier

The new pier of Scheveningen contains new accommodations as well, and the new residents of these new accommodations have certain values. Taking these values into account helps make these new accommodations more attractive to new residents. They would like a safe home environment where they can have privacy and rest. The new homes should be accessible and they should not experience nuisance from tourists during the usage period.

New businesses on the pier

The new pier offers room for new business like shops and eateries. They want to make revenue. Being accessible and attractive to tourists helps with achieving their goal.

Tourists

In this thesis, the tourists are the group of people who visit the beach for a single day and who do not make use of the hotel. They do not have much power in this project, but they are an important target group for this project. The goal is to make them want to visit the new pier and the touristic area of Scheveningen. Therefore it is necessary to take their values into account. The tourists want to have a good time when visiting the pier and they want to have various recreational possibilities and a good accessibility to the pier. At last they want to enjoy and feel the atmosphere of the beach and the sea.

New Hotel on the pier

There is a new hotel on the pier. The owners of this hotel and the hotel guests have different values. First of all, the new hotel should be attractive and accessible for its guests. They also want a tranquil environment, separated from the tourists. The hotel does not have much power, but they do have much interest in the project, since their business are on the new pier.

New conference centre on the pier

The new pier contains space for a new conference centre. They do not want to experience nuisance from the tourists and they want to be accessible to the people who make use of the conference centre.

Environmental organisations

The project site is located at sea and is located near a dune landscape. This new pier should not deteriorate these natural environments during the construction and during the usage period.

Appendix F Visualisation of the spatial design concepts

This appendix shows the different alternatives which were designed in Section 4.2. The alternatives are shown from different perspectives.

F.1 Concept 1: Ring dam

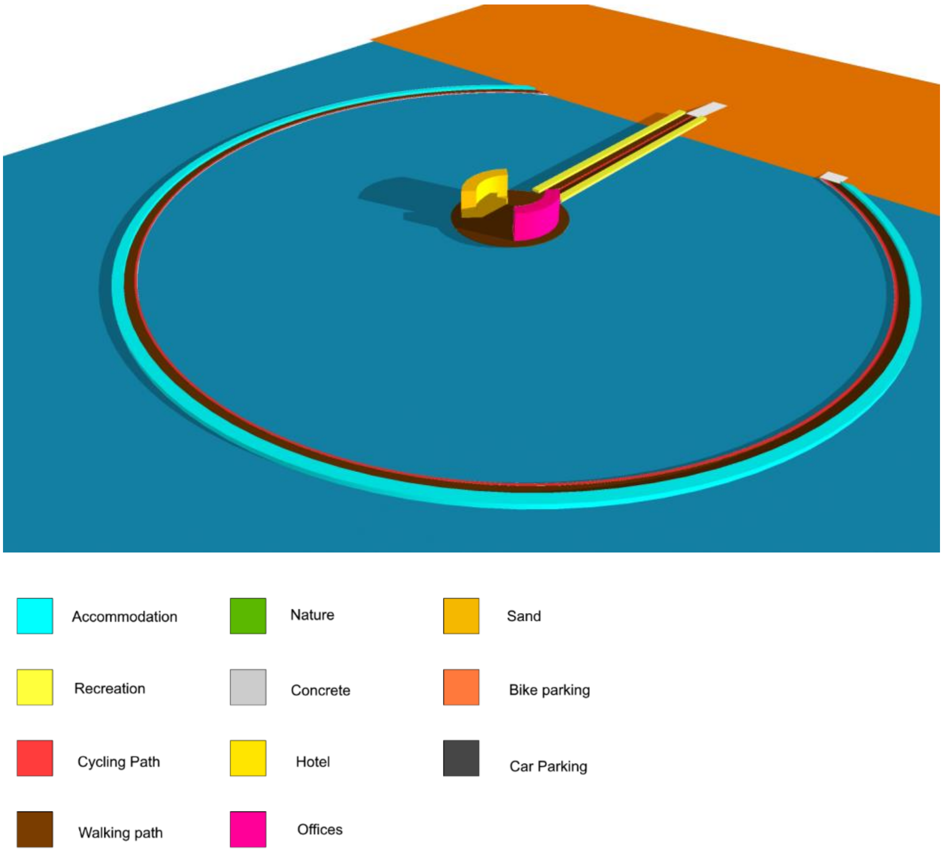


Figure 82: Concept 1, bird's eye perspective

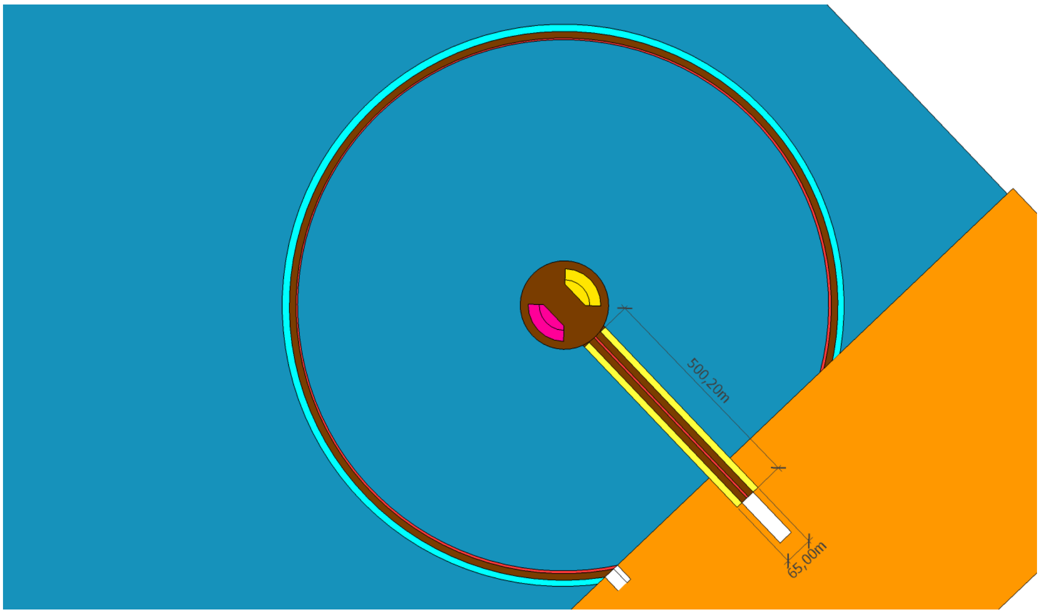


Figure 83: Concept 1, plan view

F.2 Concept 2: Underwater caisson

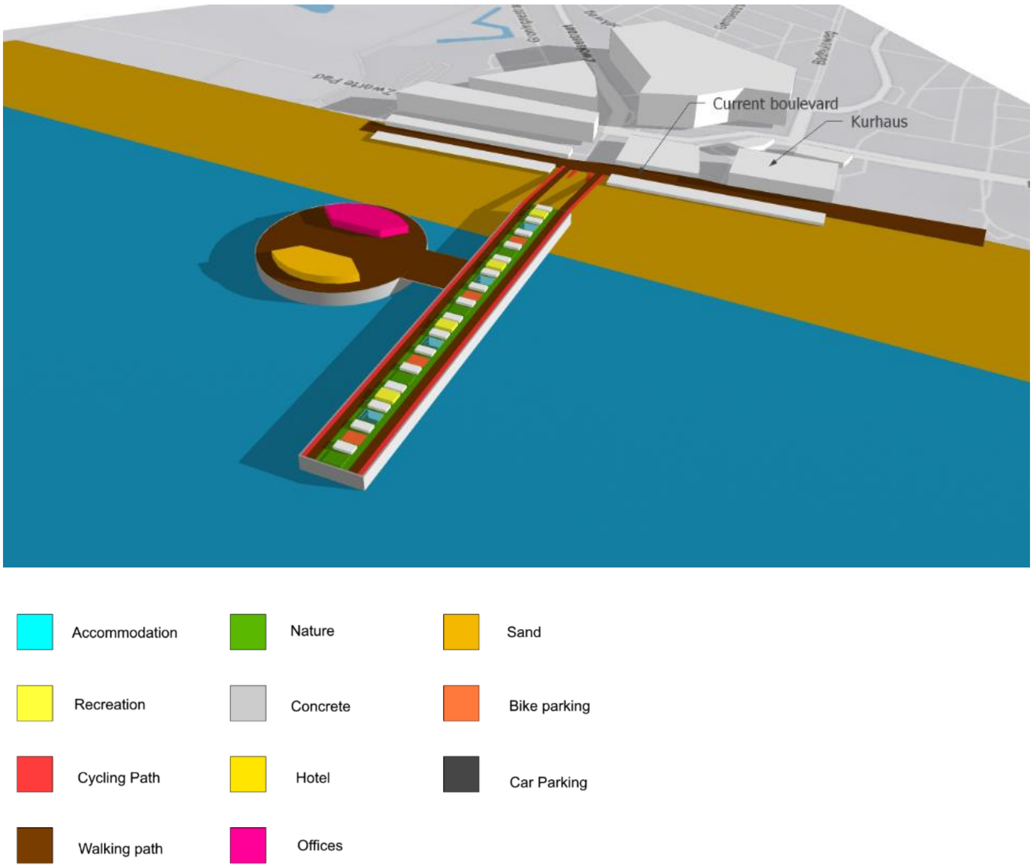


Figure 84: Concept 2, bird's-eye perspective

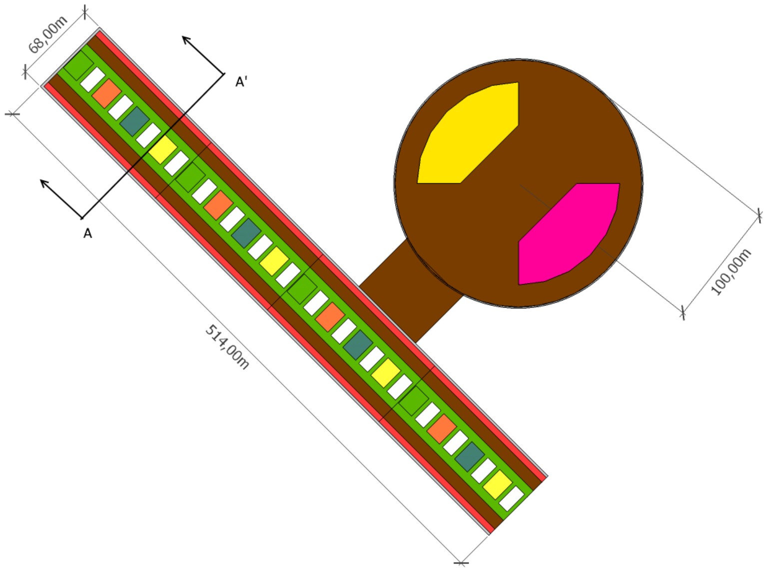


Figure 85: Concept 2, plan view of the top floor with dimensions

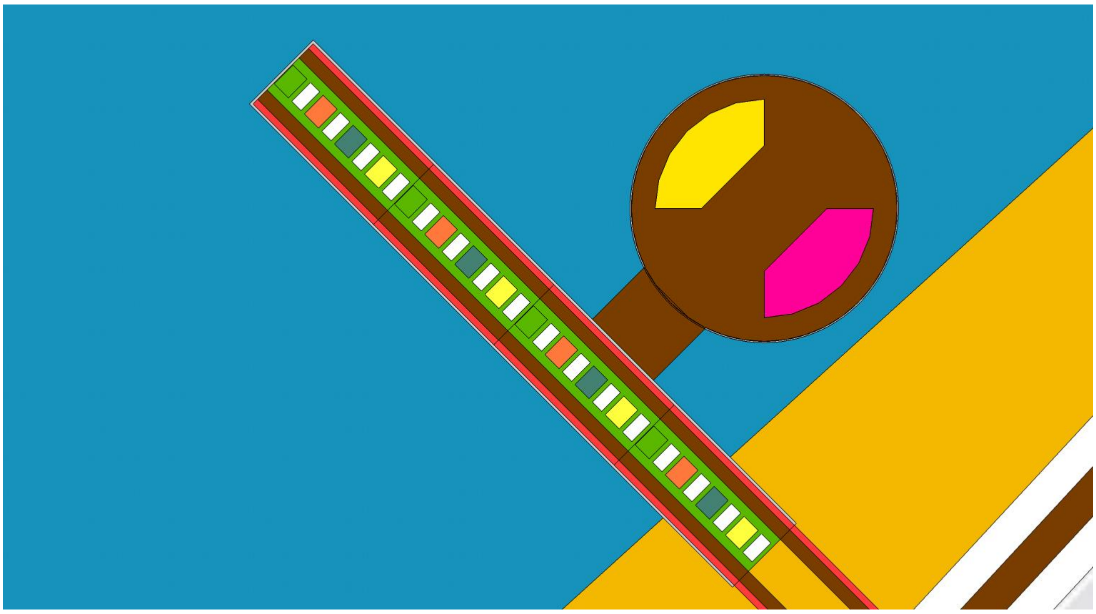


Figure 86: Concept 2, plan view of the top floor in relative to the beach

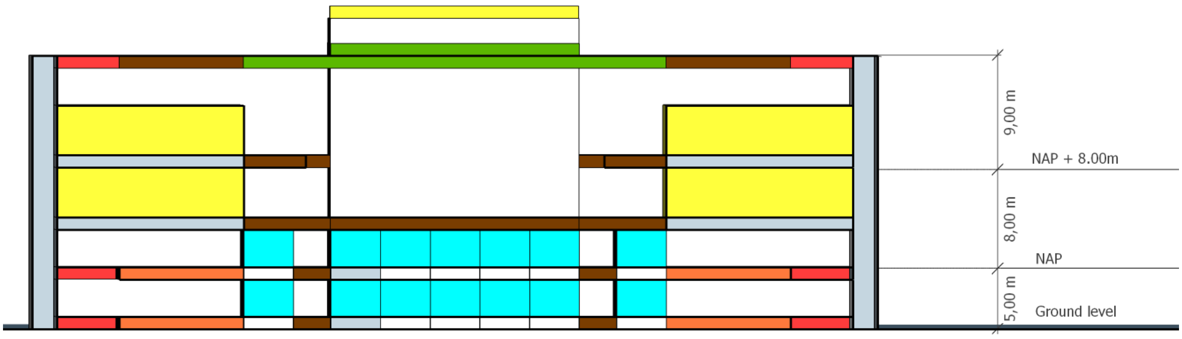


Figure 87: Concept 2, cross-section AA' with water levels displayed

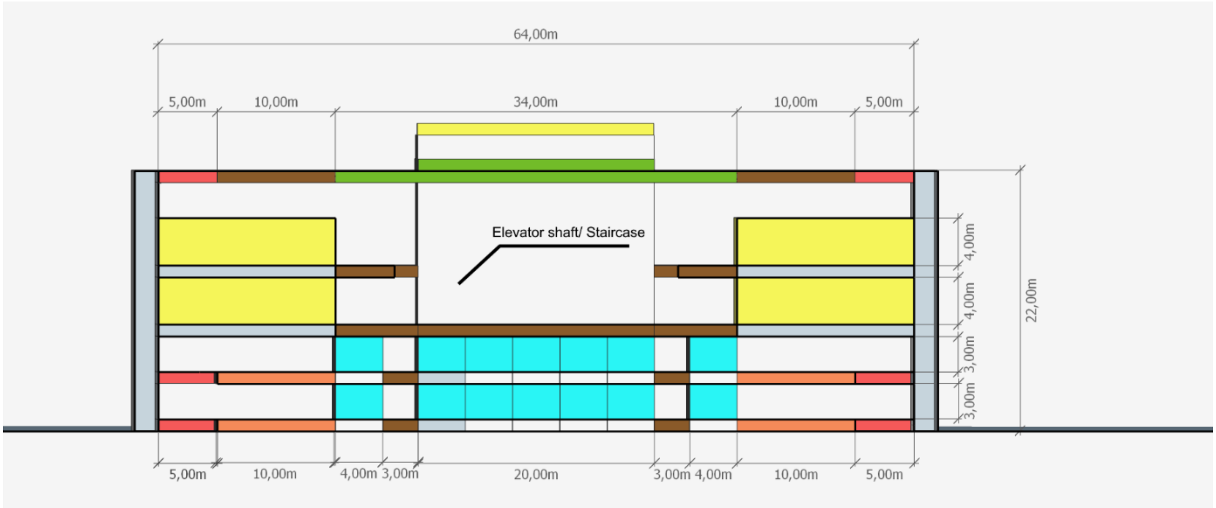


Figure 88: Concept 2, cross-section AA' with dimensions displayed

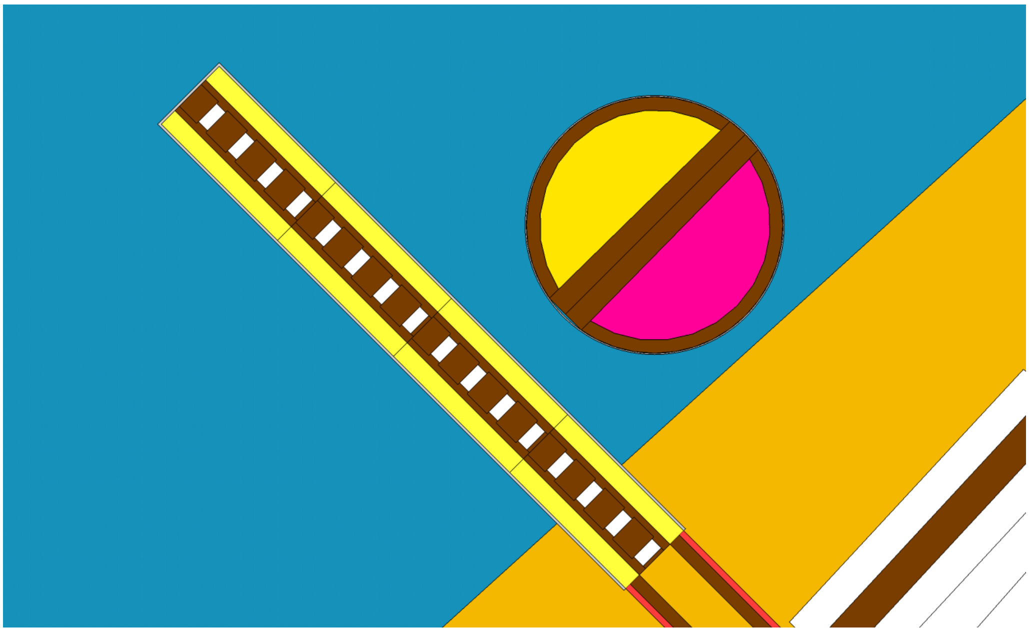


Figure 89: Concept 2, plan view of the middle floor

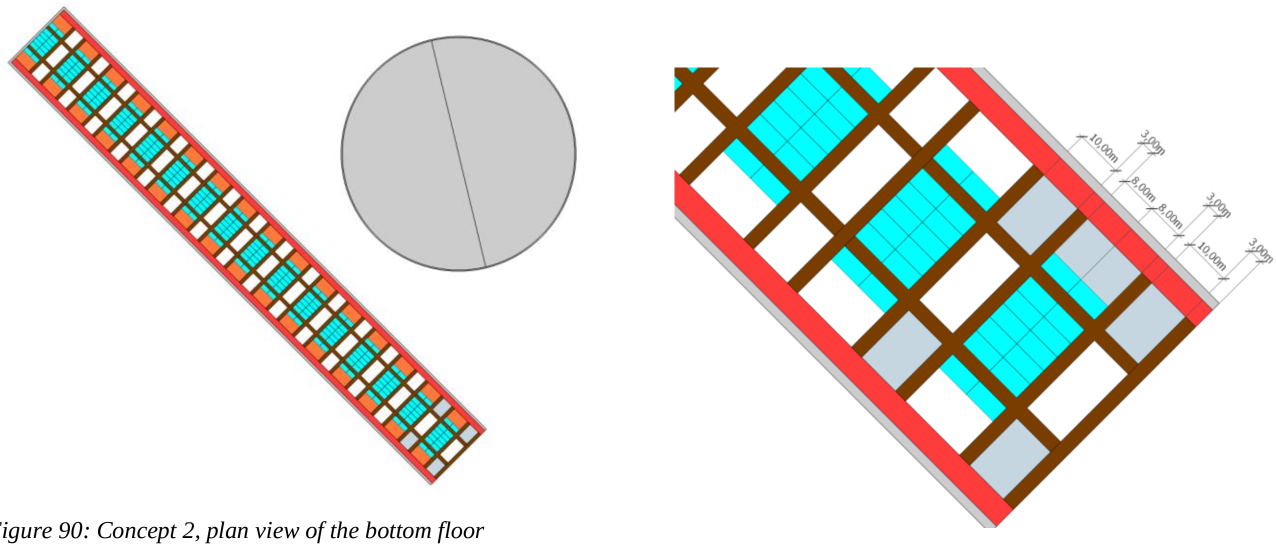


Figure 90: Concept 2, plan view of the bottom floor

F.3 Concept 3: Multiple islands

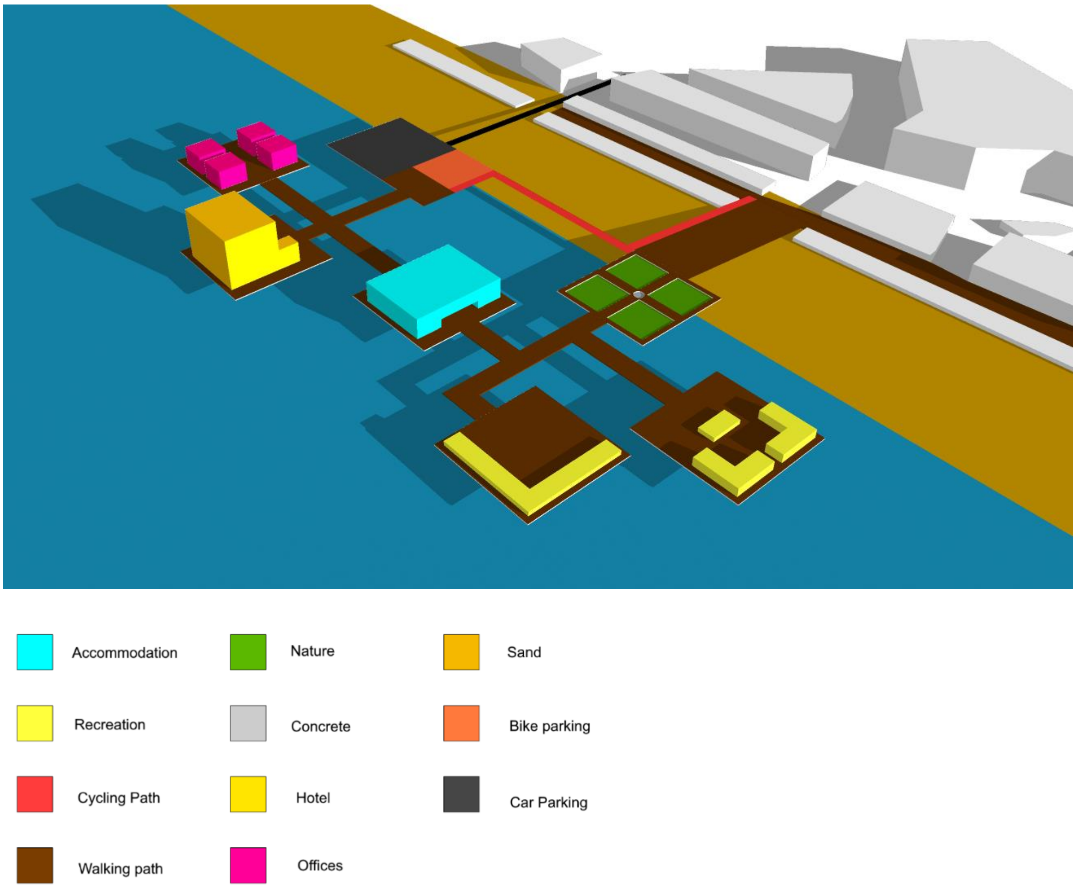


Figure 91: Concept 3, bird's-eye perspective

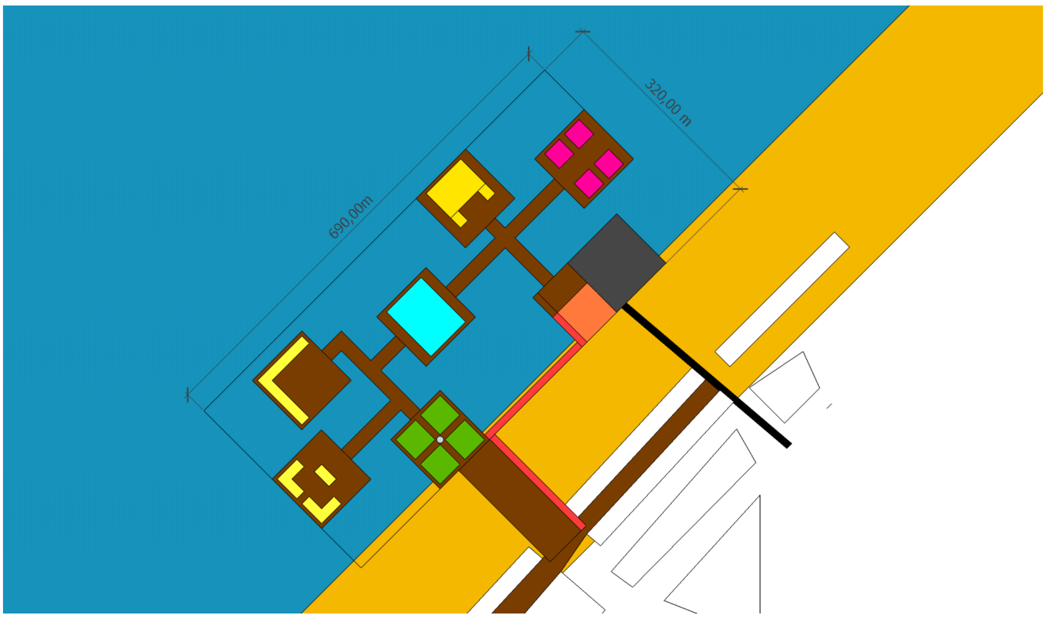


Figure 92: Concept 3, plan view

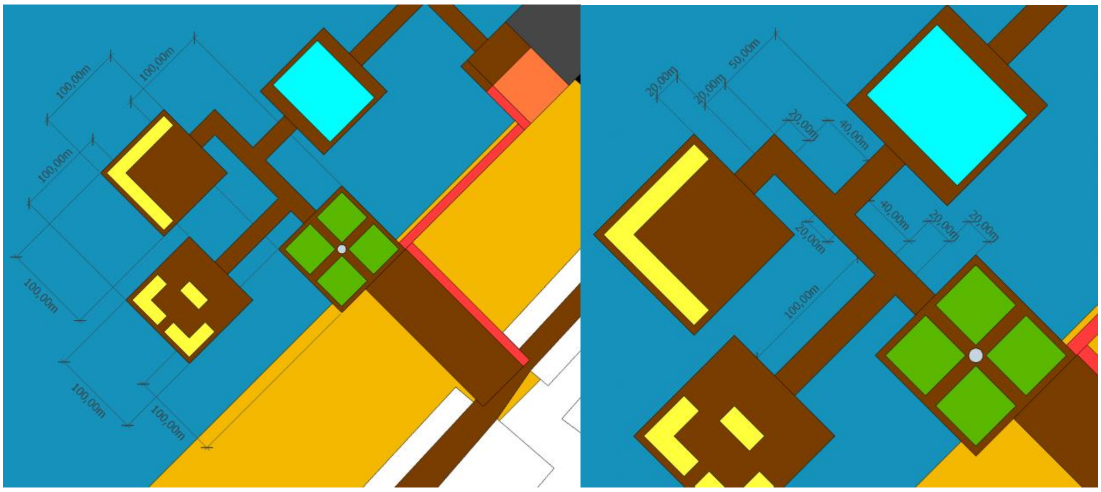


Figure 93: Concept 3, plan view of the left, recreational section of the design

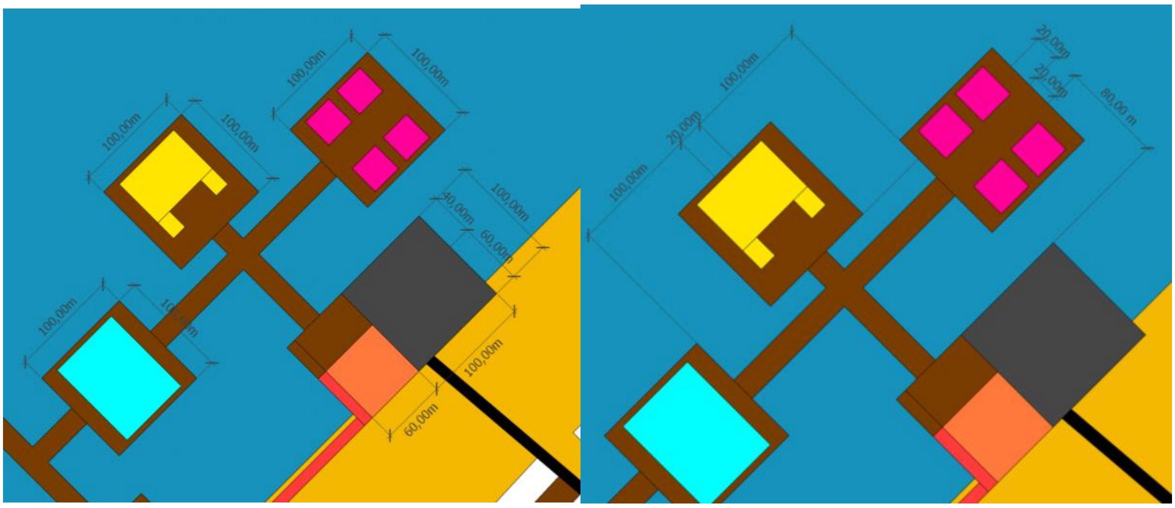


Figure 94: Concept 3, plan view of the right section of the design

F.4 Concept 4: Expansion on the beach

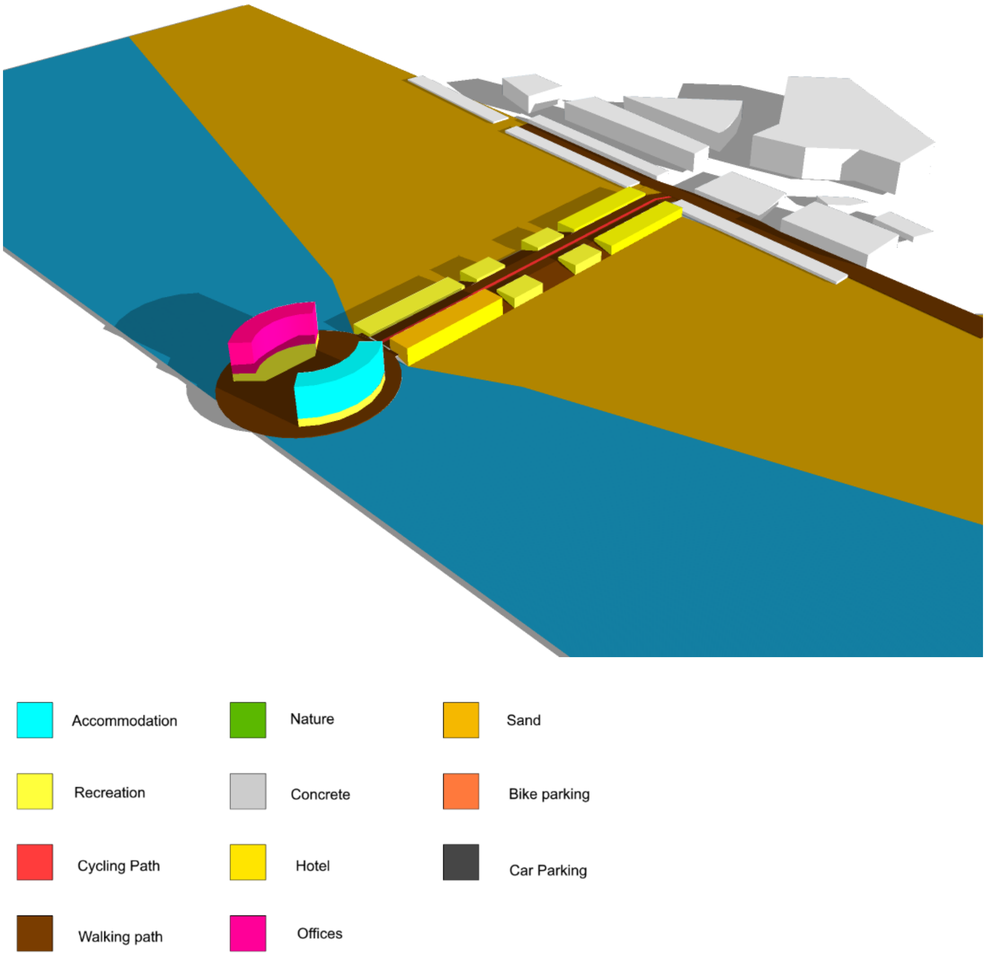


Figure 95: Concept 4, bird's-eye perspective

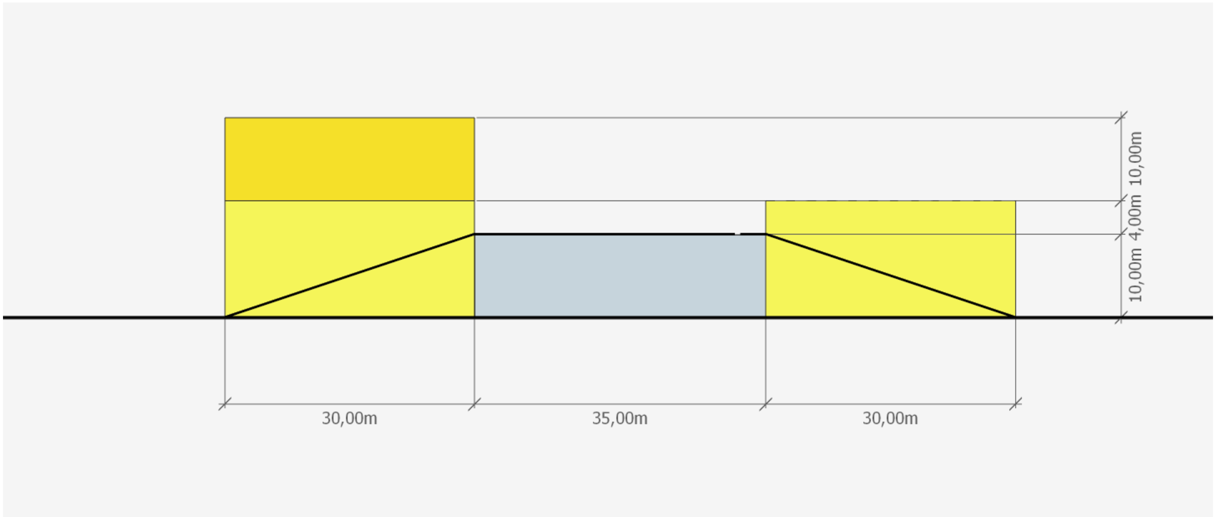


Figure 96: Concept 4, cross-section AA'

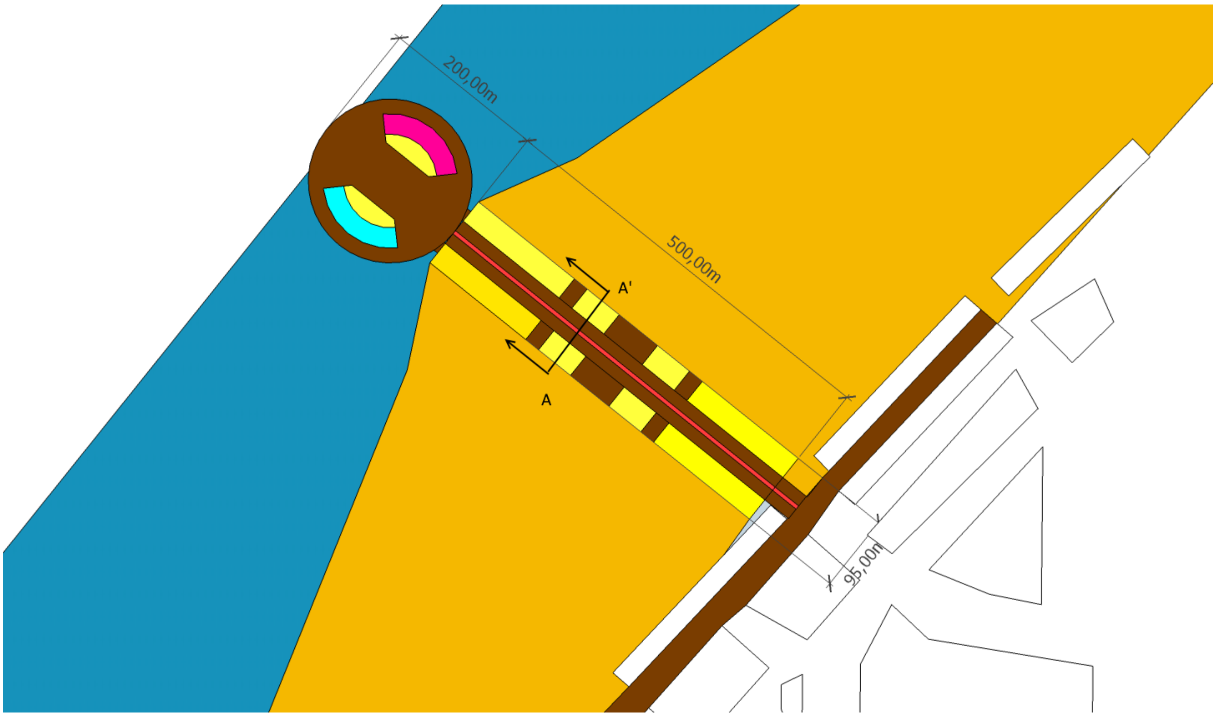


Figure 97: Concept 4, bird's-eye perspective

F.5 Concept 5: Combined design

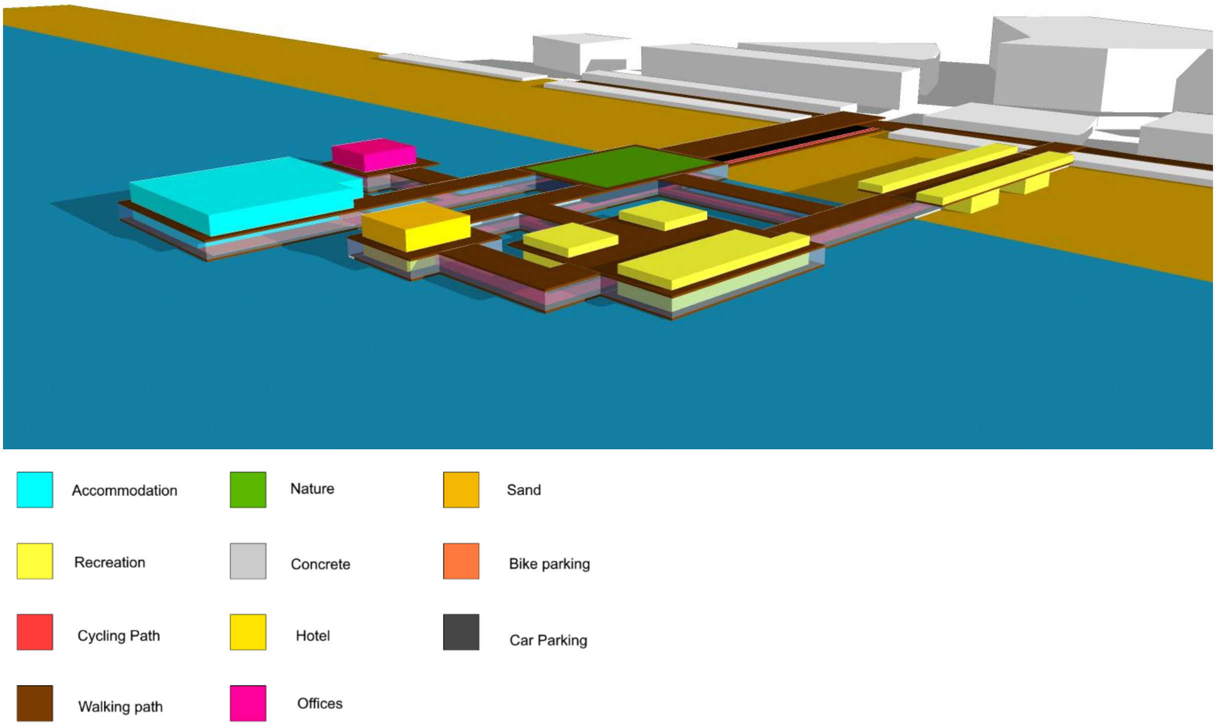


Figure 98: The combined design, bird's eye perspective

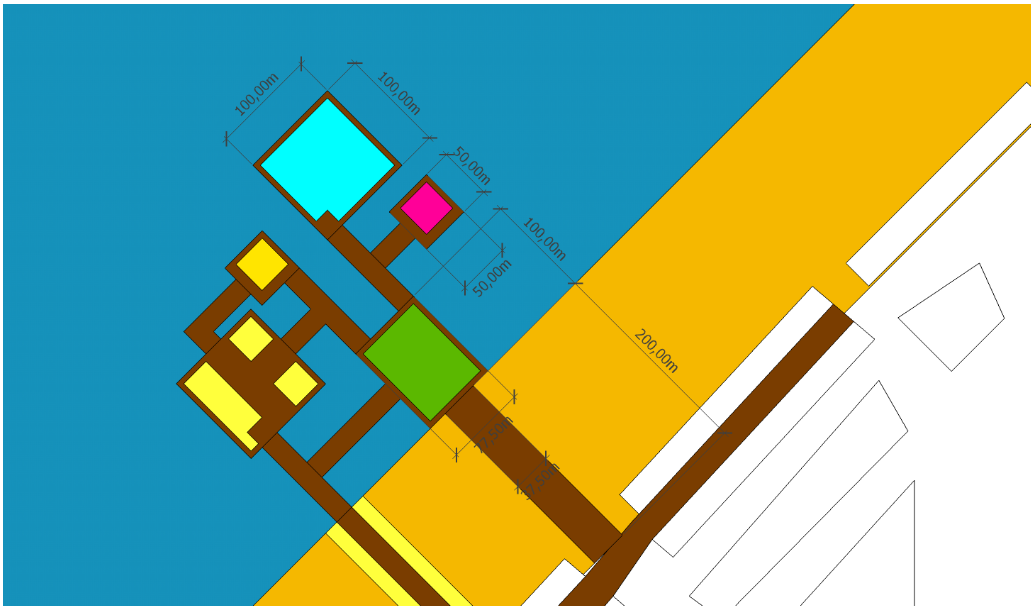


Figure 99: The combined design, plan view of the right side

Figure 100: The combined design, plan view of the left side

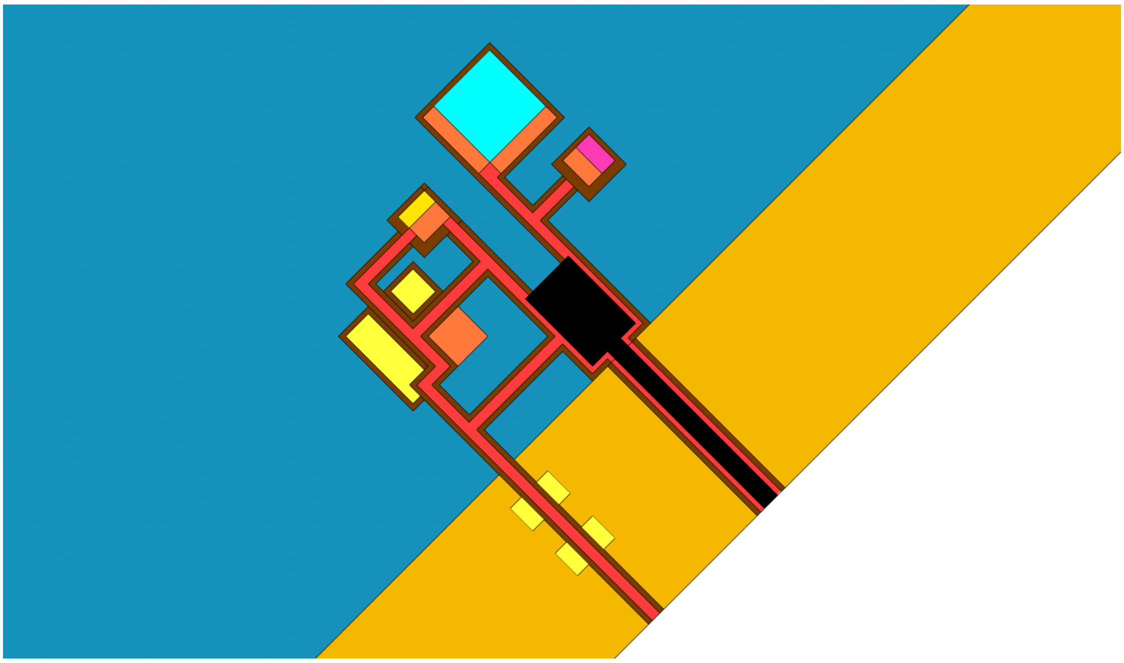


Figure 101: The combined design, plan view of the lower floor

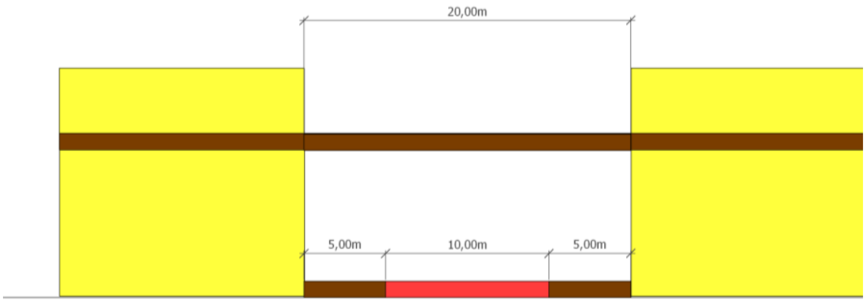


Figure 102: The combined design, section AA'

Figure 103: The combined design, section BB'

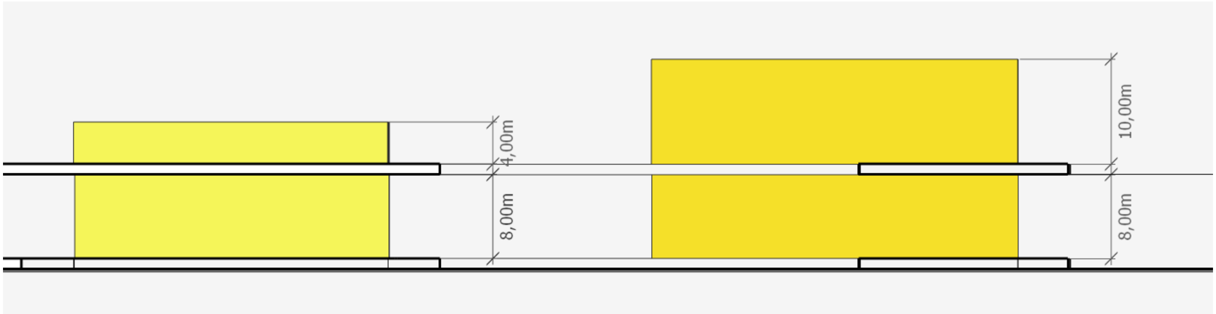


Figure 104: The combined design, section CC'

Appendix G Analysis of the spatial alternatives

This appendix shows three different analysis which were performed on the spatial alternatives which were created in Section 4.2. These are a visual space, cost and traffic flow analysis. These analysis are used to evaluate and compare the different alternatives to each other.

G.1: Visual space analysis

A visual space analysis is performed on the different alternatives which were treated in Section 4.2. These are the underwater caisson, the multiple islands, , the expansion on the beach and the combined design. Alternative 1, the ring dam, was already discarded after the verification in Section 2.3 and is therefore not analysed. In this visual space analysis, the experience is analysed. In the verification, most figures of the concepts are sections, plan views or birds-eye perspectives. These do not properly show the impression of what it is like to walk near or on the new design. This is important because a large new pier could impact the experience of the beach. Perspective images have been made in Sketchup, which in this case mainly focusses on the view of the beach and the sea.

Alternative 2: Underwater caisson

Figure 105 shows different perspectives which are looking at the pier. These are taken from the beach and the boulevard. On the one side, there is a clear view of the sea and the horizon. This is because the rectangular caisson is relatively long and thin. On the other side of the pier, the view is blocked by the large circular caisson. This is because this is a large island, located closely to the shoreline.

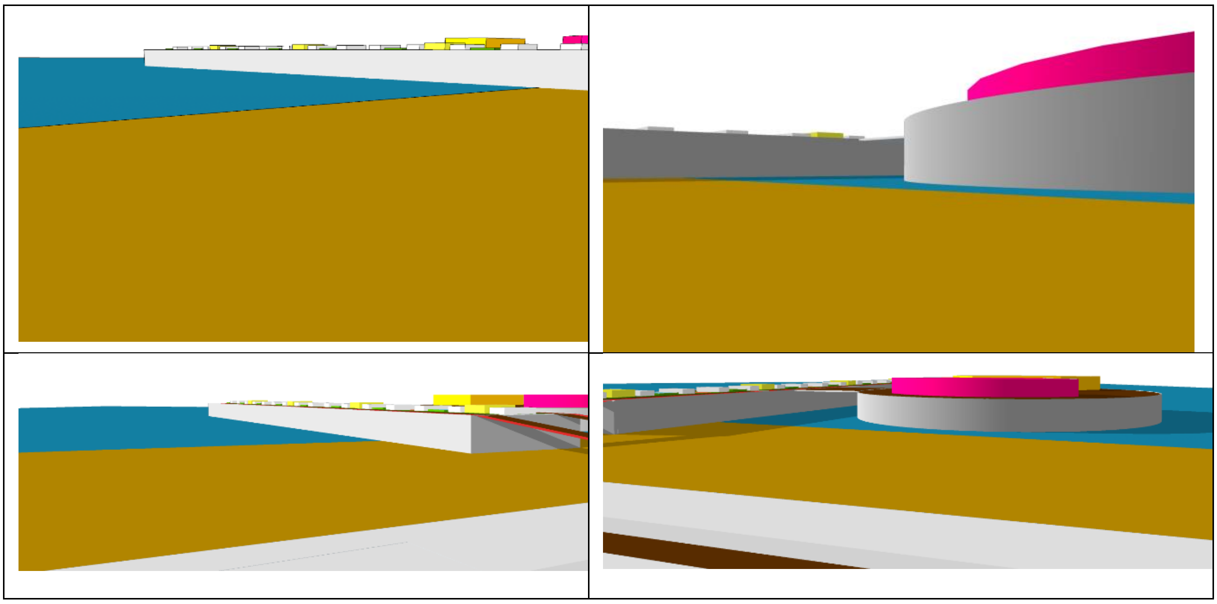


Figure 105: Visual impressions of the underwater caisson

Alternative 3: Multiple islands

This alternative is relatively wide. On the platforms itself, a good view of the sea and the beach is possible. However, when located on the beach, this concepts deteriorates a good view of the horizon for a large portion of the beach. The tall buildings which are placed on top of these platforms increase the deterioration of the horizon as well. This can be seen in Figure 106.

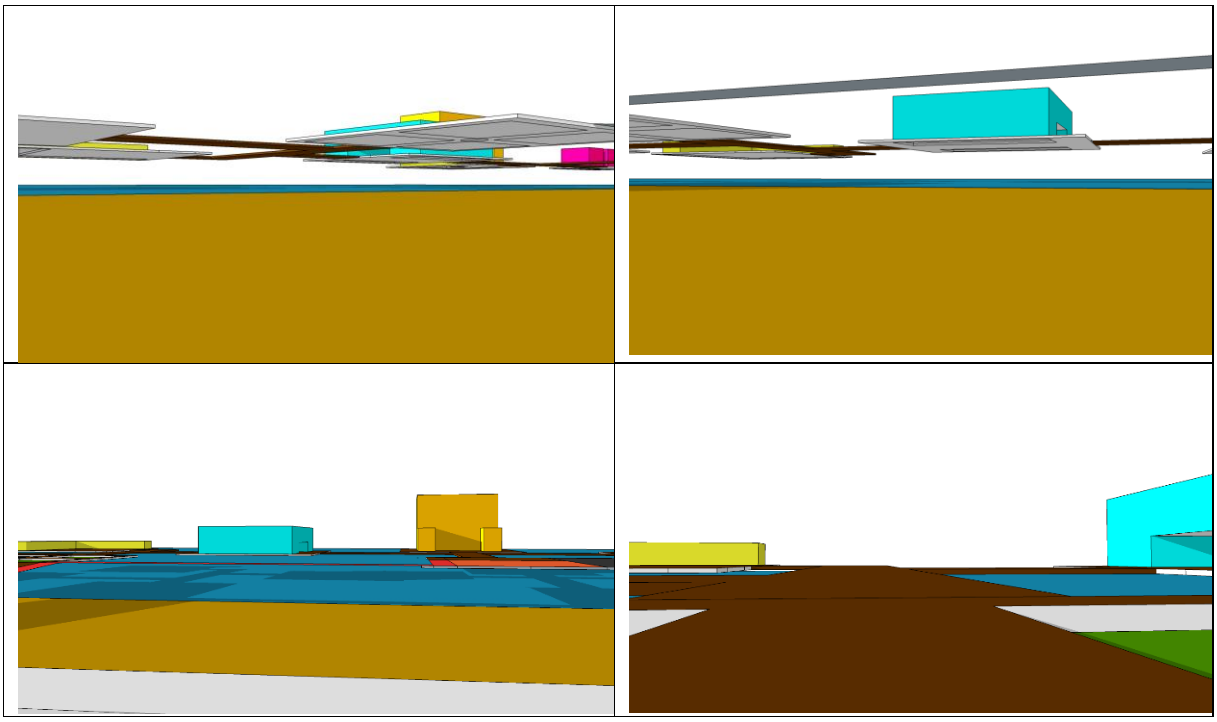


Figure 106: Visual impressions of the multiple islands alternative

Alternative 4: Extension of the beach

This alternative would have the best view of all the current alternatives. This is a relatively long and thin design, which results in less blocking of the view of the horizon. But besides that, the extension of the beach results in only a very little part of the design being located at sea. This creates a good vision of the sea and the horizon. This is well represented in Figure 107. The only downside would be the tall buildings which are placed on the platform at the end of the design.

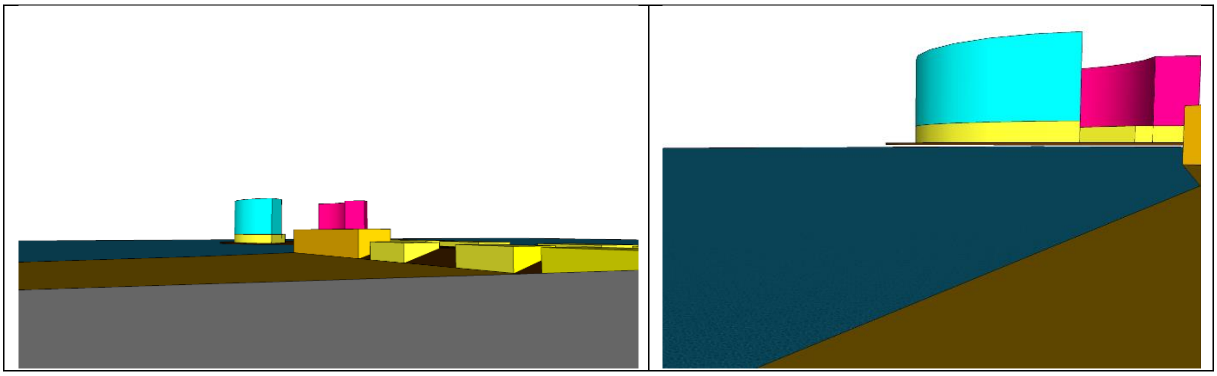


Figure 107: Visual impressions of the expansion of the beach

Alternative 5: Combined design

The combined design is mainly a combination from the underwater caisson and the multiple islands alternative, which results in multiple islands with multiple levels. The main issue from the underwater caisson, regarding this visual space analysis, was the large circular island which was too close to the shore. The main issues from the multiple islands alternative was that the entire concept was too wide, deteriorating the view from a large part of the beach and that some of the buildings were too tall. The combined design has been designed in such a way that these issues are partly solved. The lay-out of these islands is made is such a way that this alternative is relative long and not wide. A part of the islands has been made smaller and they are located more offshore. Being a smaller design creates an overall better view from the beach. With the exception, of the area between the design. On top of these changes, the buildings on this alternative are also lower than the ones from the multiple islands alternative.

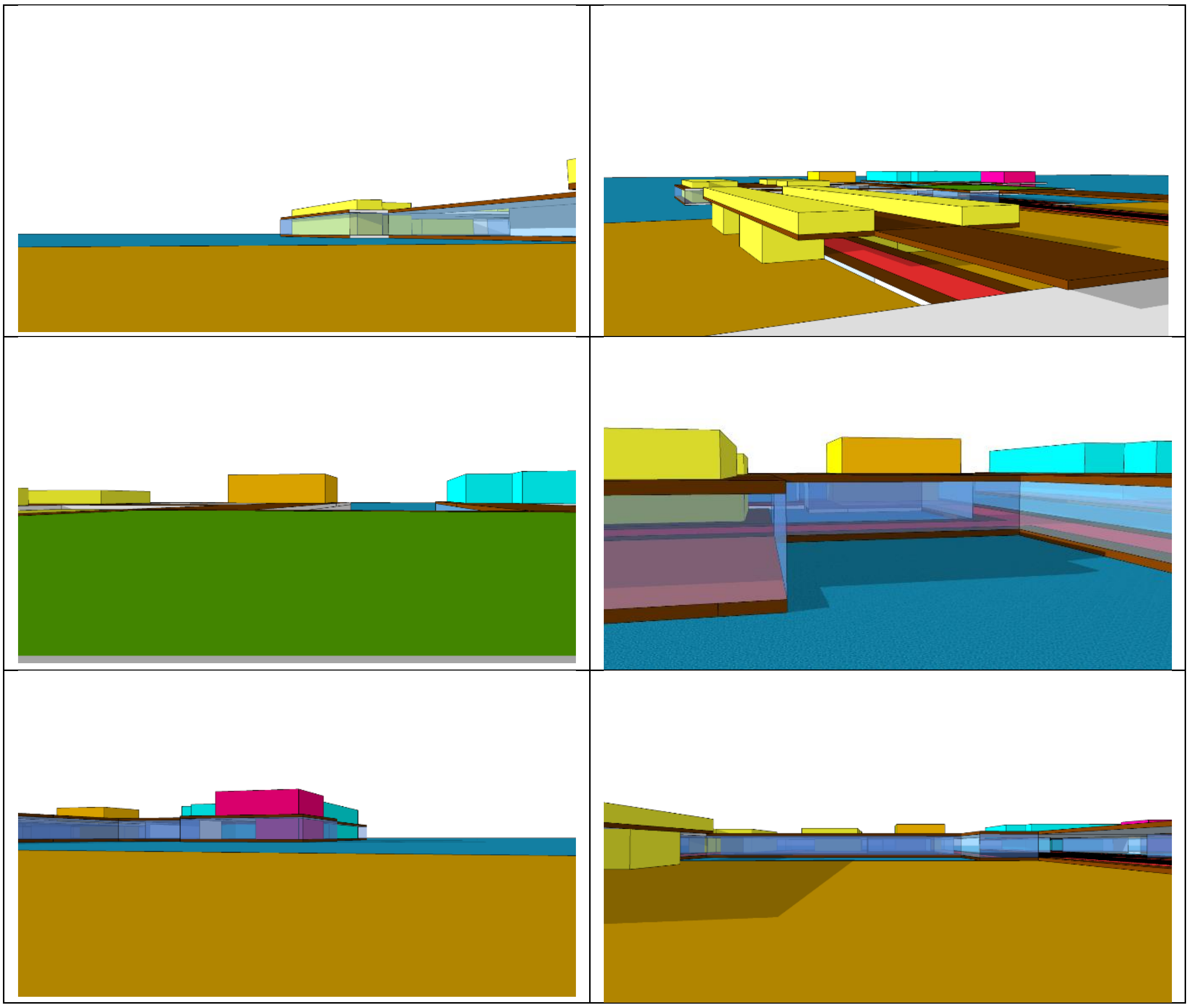


Figure 108: Visual impressions of the combined design

G.2: Cost estimation

An aspect of a project which always plays an important role in the decision making is the costs of the project. Four different designs passed the verification in Chapter 4 and were evaluated, so that an decision can be made between the different alternatives. This appendix gives an estimation of the costs of these alternatives. This is a rough estimation, since calculating the costs of the functional plan is not one of the main focusses of this thesis.

The approach of this estimation is done as follows. First the areas of all the different functions is determined for all the alternatives. This would include different floors of a building as well. These numbers are found in Table 13.

Table 13: Area of the functions for the alternatives of the functional plan. The recreation includes: buildings for recreation. Accommodation includes only the buildings which are solely used for accommodation, which means that hotels are excluded from this.

Function	**The underwater caisson Area in m^2**	**The multiple islands concept, Area in m^2**	**The expansion of the beach, Area in m^2**	**The combined design, Area in m^2**
Recreation	41,328	9,750	75,000	18,900
Accommodation	14,336	25,900	27,050	27,000

Conference venue	17,760	18,000	27,050	3,675
Hotel	17,760	35,000	27,000	3,500
Parking	0	10,000	0	5,175
Bike parking	11,440	3,600	0	5,163
Pedestrian routes	74,403	64,981	39,161	49,500
Cycling routes	11,637	4,335	2,689	13,125
Car routes	0	3,105	0	3,585

Every function then gets a unit cost, which is found Table 14. This is the amount of money that 1 m^2 would cost. A list from the course Integraal Ontwerpen (TU Delft) gives multiple numbers to estimate unit costs for certain functions. These were used to come up with unit costs for this cost estimation as well. Not all the numbers of the functions on the pier are given, so the following assumptions were made:

- All the accommodations are apartments
- The costs for a conference venue are the same as a hotel
- The cycling -and car routes and parking are a 20 cm thick asphalt layer
- Pedestrian routes have the same cost as a 10 cm thick asphalt layer

Table 14: Unit costs and revenue per functions.

Function	**Unit cost** $[€/\mathbf{m^2}]$
Recreation	400
Accommodation	790
Conference venue	1200
Hotel	1200
Parking	60
Bike parking	60
Pedestrian routes	30
Cycling routes	60
Car routes	60

The numbers from Table 13 and Table 14 are then multiplied with each other to get the estimated cost for the alternatives, which can be found in Table 15. This shows that the combined alternative costs the least. This is most likely due to the fact that the hotel and the conference venue on this alternative is smaller than the rest, which would impact the costs. The hotels on the other alternatives were rather big and this was kept in mind when designing the combined alternative.

Table 15: Total estimated costs for each alternative

	The underwater caisson	**The multiple islands concept**	**The expansion of the beach**	**Combined alternative**
Total revenue	€ 74,097,350.00	€ 91,172,830.00	€ 117,565,670.00	€40,607,880.00

These are some points which should be kept in mind when looking at this cost estimation:

- The revenues are not taken into account
- The construction has not been supported for any of the alternatives yet when this cost estimation was made. This probably contributes largely to the costs of the alternatives.

G.3: Traffic flow analysis

This appendix displays the different traffic routes of the new pier for the different alternatives of the pier.

Alternative 2: The underwater caisson

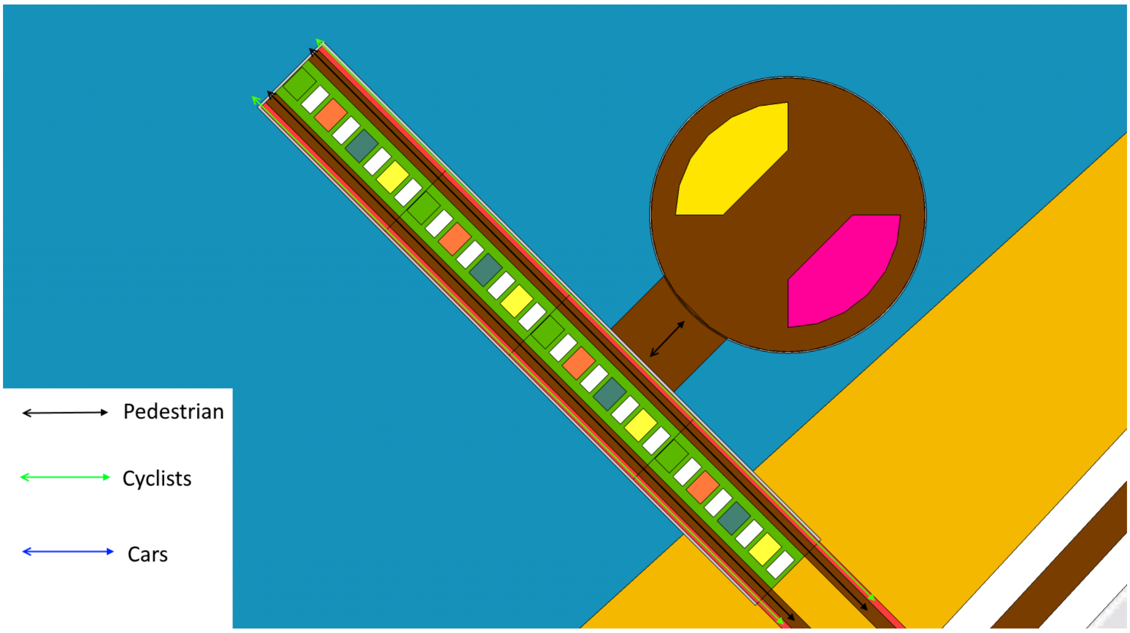


Figure 109: Traffic routes of the underwater caisson, top floor

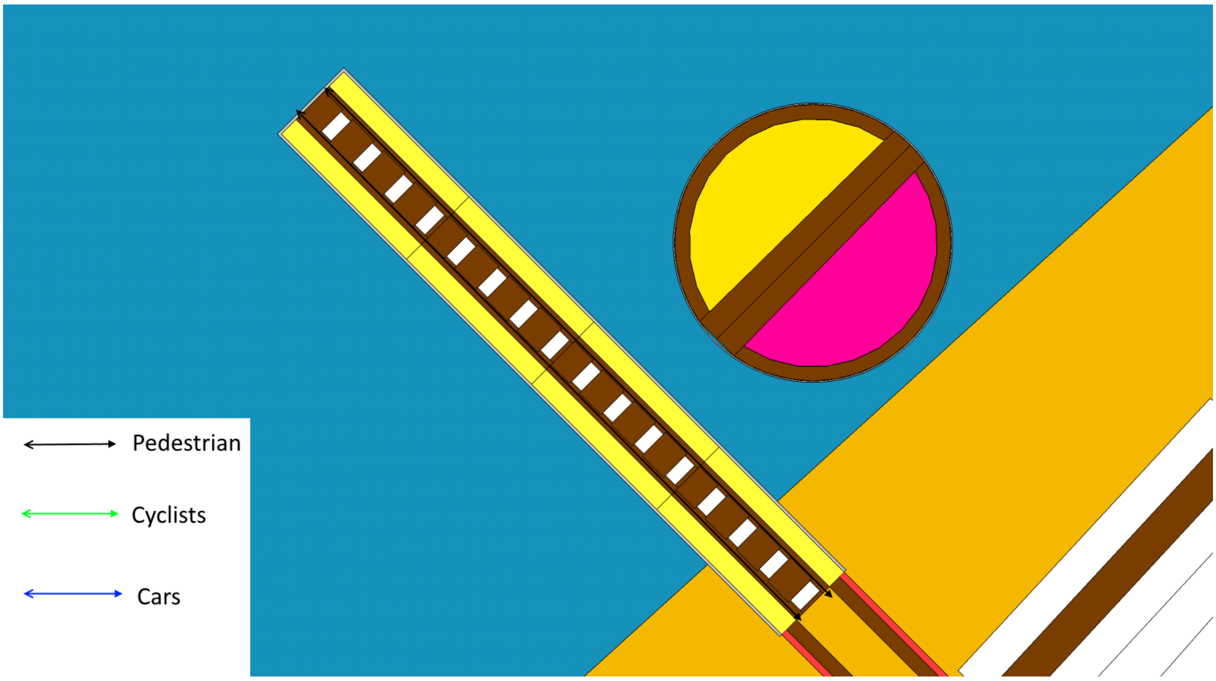


Figure 110: Traffic routes of the underwater caisson, middle floor

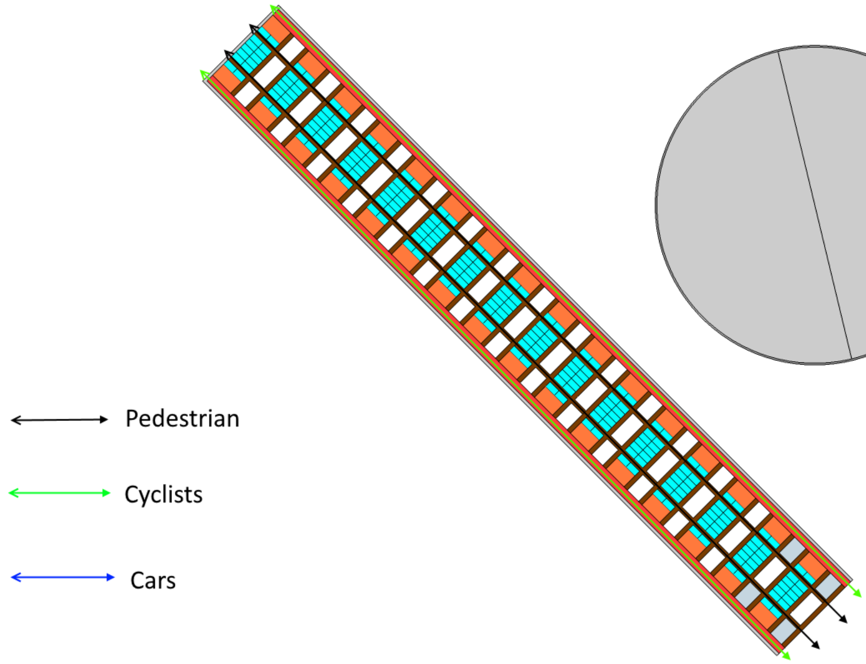


Figure 111: Traffic routes of the underwater caisson, bottom floor

Alternative 3: The multiple islands concept

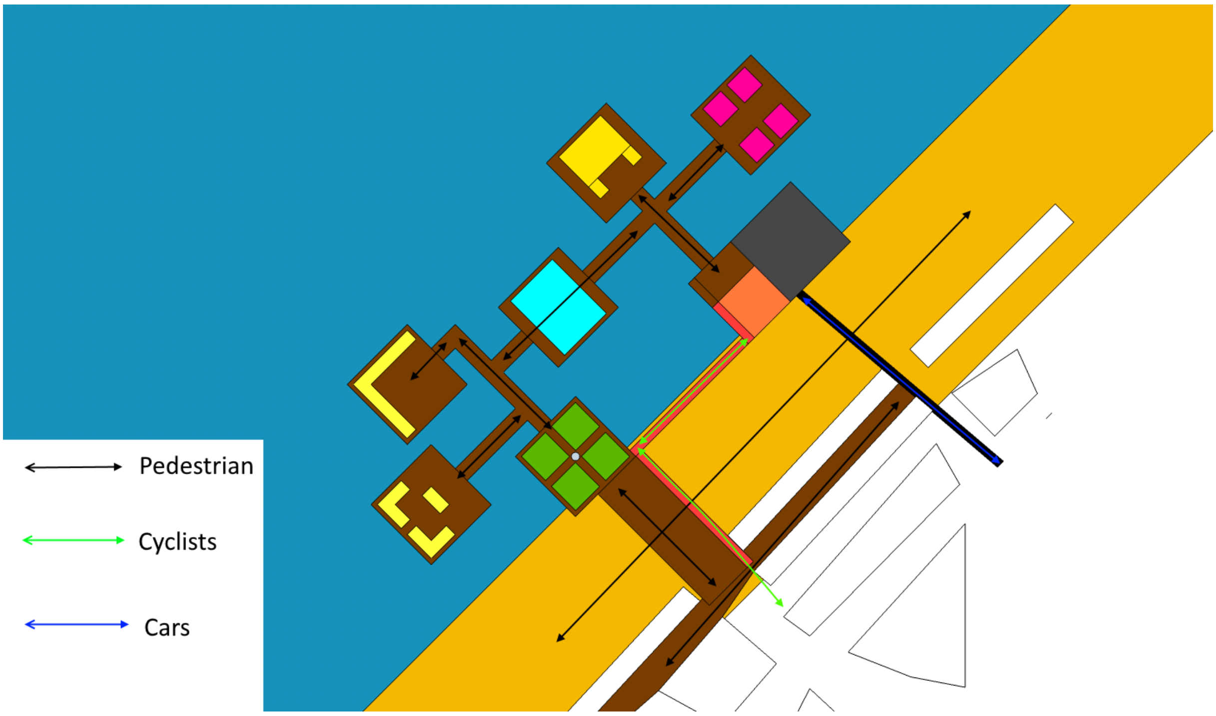


Figure 112: Traffic routes of the multiple islands concept

Alternative 4: The expansion of the beach

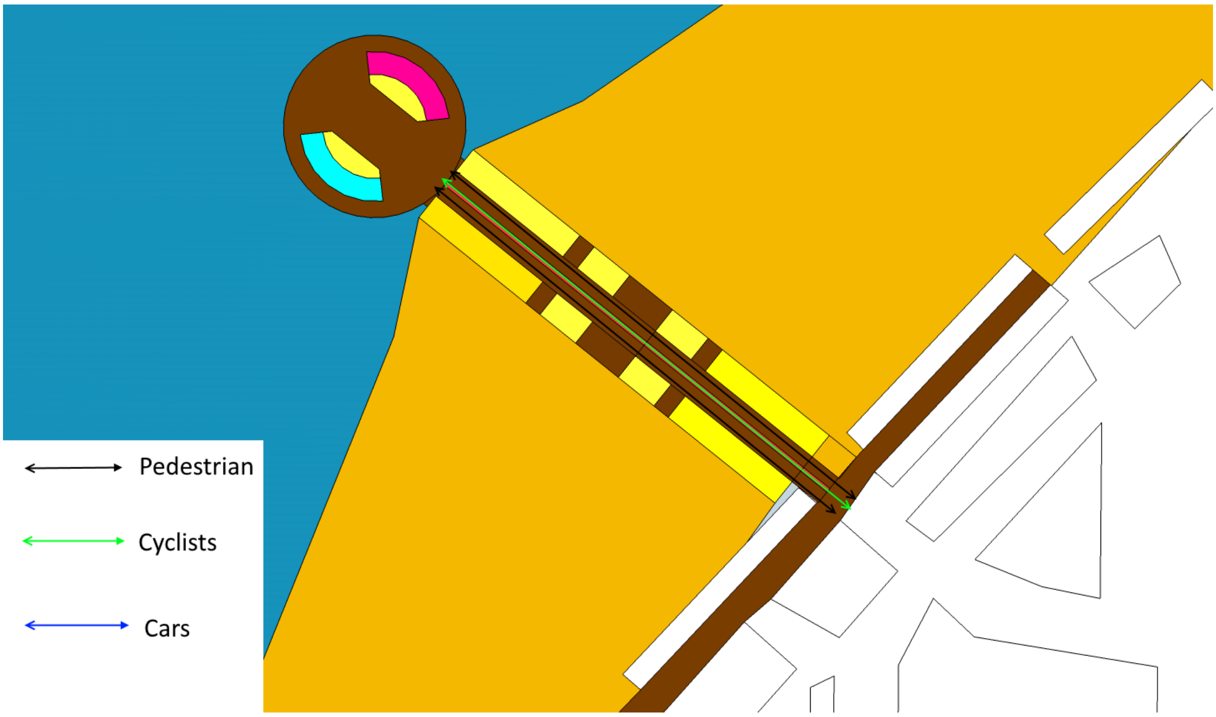


Figure 113: Traffic routes of the expansion of the beach

Alternative 5: The combined design

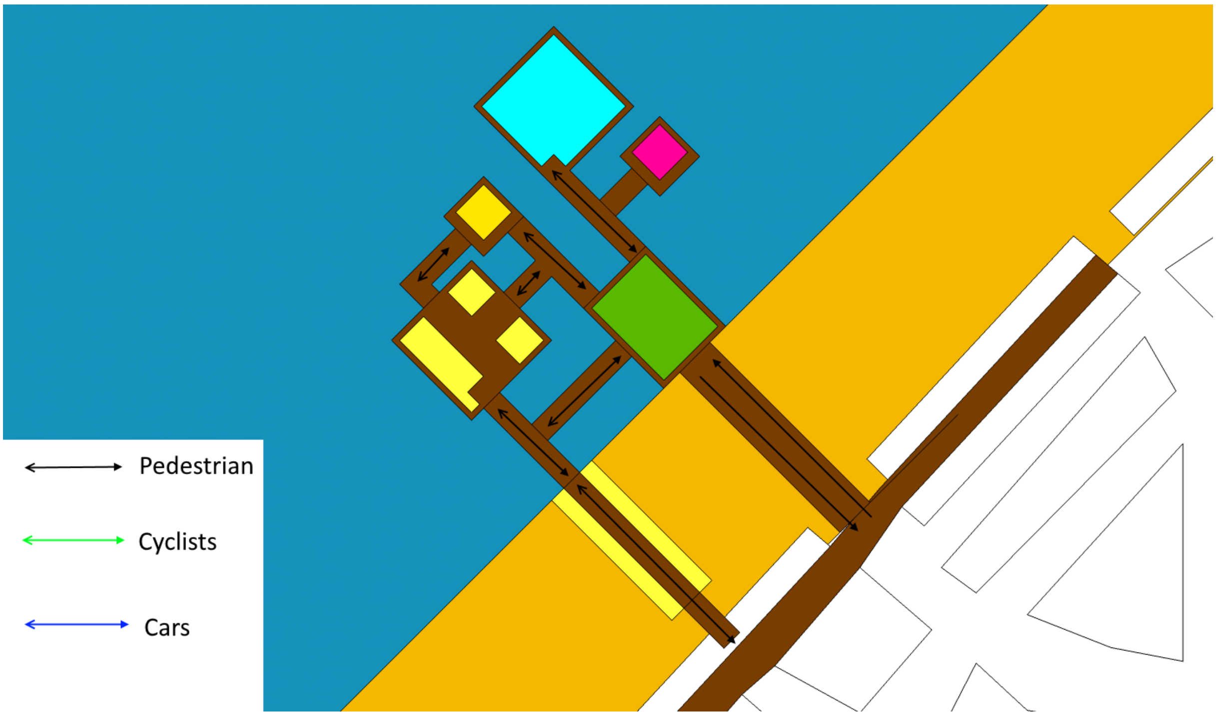


Figure 114: Traffic routes of the combined design, top floor

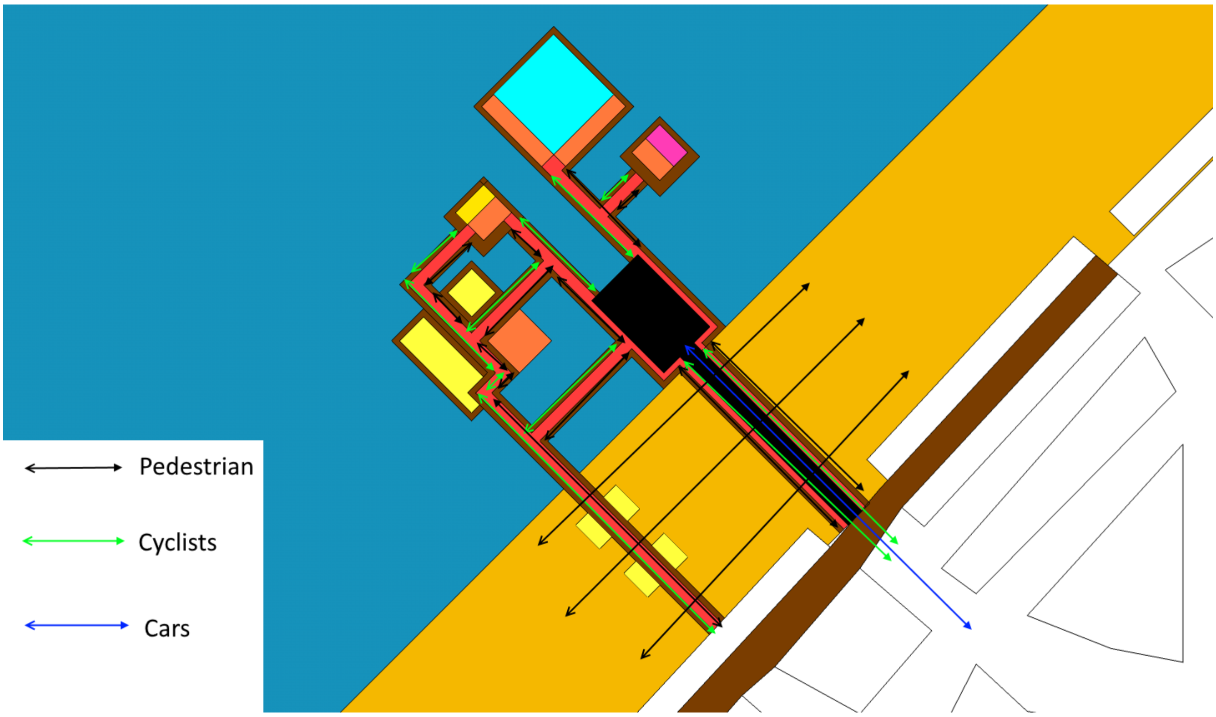


Figure 115: Traffic routes of the combined design, bottom floor

Appendix H Elaboration on the multi-criteria analysis

This appendix provides more information on the various criteria, scores and weights of the multi-criteria analysis of the spatial design alternatives, which was performed in Section 4.3. The multi-criteria analysis has been performed to support the decision for a functional/spatial design.

H.1 Explanation of the criteria and the reasoning for the scores

Down below is an explanation of each criterion and the reasoning for the scores given to the alternatives.

Accessibility: The new pier and its different functions should be accessible. Accessibility is also a requirement which was used in the verification. However, in the verification it is asked if the design is accessible. In the evaluation it is asked how well the design is accessible. Things to measure the accessibility are the parking spaces, the transition routes and the different kinds of mobility which can be used on the pier like walking, cycling and by car.

Scores on accessibility: The underwater caisson and the multiple islands alternative both offer different solutions to improve the accessibility of the design. The underwater caisson has cycling lanes across the entire design and offers much space to park these. The multiple islands alternative has a separate island to accommodate parking for both bikes and cars. However, the combined design method has both these solutions. It has cycling paths to every platform and has space to park cars and bikes, and is therefore granted the maximum score. The expansion of the beach has a single cycling path but does not offer much space to park bikes and it does not offer space to park cars either.

Low nuisance: There are many residents and other exclusive functions on - and close to the new pier. The recreational functions on the pier should cause low nuisance to the more exclusive functions. This can be reached by a good separation of the functions.

Scores on low nuisance: The expansion of the beach scores the worst on this criterion. The exclusive functions are at the end of the design. Therefore people have to walk or cycle along the tourists to reach their homes, hotel or conference centre. Besides that, these functions are also closely located to the recreational functions. The underwater caisson scores better, since the functions are more separated by different floors, however, they are still close to each other. The multiple islands alternative and the combined design are given the best scores, since the more exclusive functions are separated from the recreation completely and they have their own access routes.

Blend in with the boulevard: The diagnosis of the landscape analysis stated, among other things, that the current pier does not connect with the style of the new boulevard. Being a part of a larger Scheveningen is an important criteria of a new pier. The boulevard is wide and can be accessed by foot, bike or car, while the current pier was small and only accessible by foot.

Score on blend in with the boulevard: All alternatives were designed to be more compatible with the current boulevard. This was done by creating wider walkways, which are also found at the boulevard. Therefore none of the alternatives score below average on this criterion. The expansion of the beach, however, stands out. This alternative is a real extension of the boulevard, trying to extend the beach and adapting the shape of a dike, similar to the boulevard.

Separation between transition areas and functional areas: From the landscape analysis it became clear that the current pier consists mainly out of transitional areas, and that these are not well separated from the areas where people stand still or areas where other functions are located. This criterion judges whether these functions are well separated.

Score on separation between transition areas and functional areas: Both the underwater caisson and the expansion of the beach have their recreational functions close to their transition paths and are therefore given a low score. The multiple islands alternative and the combined design were especially designed to separate the functional areas from the transitional areas and it fulfils this function properly by having the transition routes between the different islands.

View of the beach and sea: When people visit Scheveningen, they want to visit the beach and enjoy the view of the horizon. This criterion judges whether a good view of the sea and beach is possible with the new design in place. It should not block too much of the view. An analysis for this criterion has been performed in Appendix G.

Score on view of the beach and sea: The expansion of the beach scores the best on this, because it blocks the view of the horizon the least. The multiple islands alternative scores the worst. This alternative is very wide and therefore blocks a large part of the horizon. The combined design has learned from this mistake and has a different lay-out of the islands, making it the total design less wide.

Aesthetic value: The aesthetic value of a design is a very subjective criterion. It describes how well the design looks and appeals to people.

Score on aesthetic value: As stated in the description of this criterion, this is a very subjective criterion and therefore it is difficult to assign a score. The reasoning which is used is that all the designs are good looking, but none of them excel in this criterion.

Continuation of the beach: This criterion looks at the beach surrounding the design. There is continuation if it is still possible to walk along the beach without being interrupted by the new pier.

Score on the continuation of the beach: The underwater caisson completely blocks the beach with its concrete walls. The multiple islands alternative does currently not have a structure, but it can be possible to still walk underneath it. The expansion of the beach offers the possibility to walk over the dike. The combined design does not block the beach, but a cycling road and a car road cross the beach. But these roads can be crossed by pedestrians.

Low damage to nature: There are various areas in the surroundings of Scheveningen containing much nature. These should be as less interrupted as possible.

Score on low damage to nature: None of the alternatives have a relative impact on the surroundings areas which contain much nature. Therefore all are given the maximum score on this criterion.

Space for recreation: The municipality and the client, who are both important stakeholders want to have many recreational possibilities. This leads to an increase of the touristic value of Scheveningen and to more revenue for the owners to the owners of the pier, who rent the space to new businesses.

Score on space for recreation: This score is solely based on Table 13, *which displays the area per function and can be found in Appendix G. The area for recreation on the current pier is estimated to be below 10,000* m^2. *The expansion of the beach simply has the most area available and therefore receives the highest score.*

Usability throughout the year: This criterion looks at if a design can be used throughout the year. Many spaces or often usable in the summer, however in the winter only indoor spaces are used because of the decreasing temperatures.

Score on usability throughout the year: Most functions on all the alternative are located inside buildings, so therefore no alternative gets a bad score. However, the underwater caisson and the combined design are more located inside and has inside transition areas, which makes it more attractive during colder periods.

H.2 Weighed score tables for the different perspectives

Table 16 to Table 18 show the weight factors and the weighed scores of the different alternatives which were evaluated in the multi-criteria analysis in Subsection 4.3. Each table represents a different perspective.

Table 16: Weight factors for the municipality's perspective. The minimum total score for an alternative is 100, the maximum is 500.

Criterion	Weight	Underwater caisson	Multiple islands	Expansion of the beach	Combined design
1. Accessibility	15	60	60	30	75
2. Low nuisance	15	45	75	15	75
3. Blend in with the boulevard	10	30	30	50	30
4. Separation transitional and functional areas	5	10	25	10	25
5. View of the beach and sea	10	30	20	50	30
6. Aesthetic value	5	20	20	20	20
7. Continuation of the beach	5	5	20	15	15
8. Low damage to nature	10	50	50	50	50
9. Space for recreational areas	15	60	30	75	45
10. Usability throughout the year	10	50	30	30	50
Total	**100**	**360**	**360**	**345**	**415**

Table 17: Weight factors for the client's perspective. The minimum total score for an alternative is 100, the maximum is 500.

Criterion	Weight	Underwater caisson	Multiple islands	Expansion of the beach	Combined design
1. Accessibility	15	60	60	30	75
2. Low nuisance	5	15	25	5	25
3. Blend in with the boulevard	5	15	15	25	15
4. Separation transitional and functional areas	10	20	50	20	50
5. View of the beach and sea	10	30	20	50	30
6. Aesthetic value	10	40	40	40	40

7. Continuation of the beach	5	5	20	15	15
8. Low damage to nature	5	25	25	25	25
9. Space for recreational areas	20	80	40	100	60
10. Usability throughout the year	15	75	45	45	75
Total	**100**	**365**	**340**	**355**	**410**

Table 18: Weight factors for the users' perspective. The minimum total score for an alternative is 100, the maximum is 500

Criterion	Weight	Underwater caisson	Multiple islands	Expansion of the beach	Combined design
1. Accessibility	20	80	80	40	100
2. Low nuisance	20	60	100	20	100
3. Blend in with the boulevard	5	15	15	25	15
4. Separation transitional and functional areas	5	10	25	10	25
5. View of the beach and sea	10	30	20	50	30
6. Aesthetic value	10	40	40	40	40
7. Continuation of the beach	5	5	20	15	15
8. Low damage to nature	5	25	25	25	25
9. Space for recreational areas	5	20	10	25	15
10. Usability throughout the year	15	75	45	45	75
Total	**100**	**360**	**380**	**295**	**440**

Appendix I Beach Morphology

This appendix gives the hypothesis of the changers of the beach, caused by different measures. After that it explains the Delft3D models which were used to predict the changes over the beach. These models were used to analyse the beach morphology in Chapter 5.

I.1 Hypotheses of the different measures

Most of the changes on to the beach are long term structural changes. These changes are often caused by a change in the alongshore sediment transport. This type of sediment transport is caused by a combination of the alongshore current and the stirring up of sediment. Both of these are caused by waves. A well-known formula to determine the sediment transport is the CERC formulation which can be written as:

$$S = \frac{K}{16(s-1)(1-p)} \sqrt{\frac{g}{\gamma}} \sin\left(2\phi_b H_b^{2.5}\right)$$

Where

$S =$ The deposited volume of sediment transported $[\mathrm{m}^3/\mathrm{s}]$

$K =$ Coefficient $[-]$

$s =$ Relative density of the sediment $[-]$

$p =$ Porosity $[-]$

$g =$ Gravitational constant $[\mathrm{m}/\mathrm{s}^2]$

$\gamma =$ Breaker index (assumed at 0,78) $[-]$

$\phi_b =$ Wave angle of incidence [°]

$H_b =$ Breaking wave height [m]

In the project area it is assumed that the relative density and the porosity is close to uniform. This means that the sediment transport is directly correlated with the angle of incidence of the waves and the breaking wave height:

$$S \propto H_{s,b}^{2,5} \cdot \sin(2\phi_b)$$

This relation leads to the $S - \phi$ curve, which shows the relation between the alongshore sediment transport and the angle of incidence between the waves. This is displayed in Figure 116. It shows that the alongshore sediment transport is at a maximum when the angle of incidence is equal to 45 °. In reality this would be at 42 °. The sediment transport is equal to zero when waves are exactly parallel or exactly perpendicular to the shore.

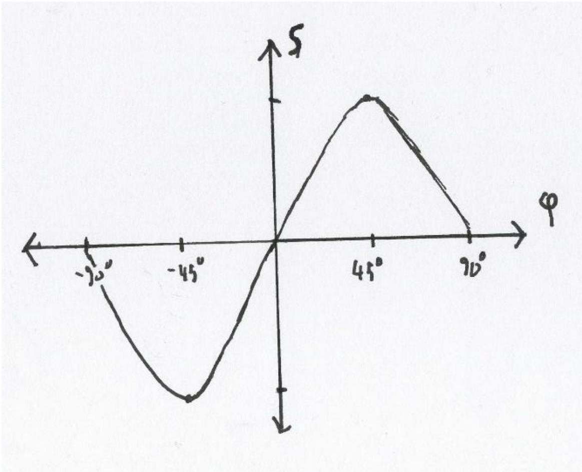


Figure 116: S-ϕ curve

The alongshore sediment transport itself does not lead to a changing coastline. This happens when there is a change in sediment transport. This can be explained by continuity. In this case it is assumed that the height of the coastal profile, d, remains constant along the coast and does not change with any coastline changes. Figure 117 looks at a segment of the coast with a length of Δx. The coastline change in this box is denoted as Δy. Sediment transport goes in and out of this box. The difference between the transport that goes in and out can be written as $\frac{\partial S_x}{\partial x} \cdot \Delta x$.

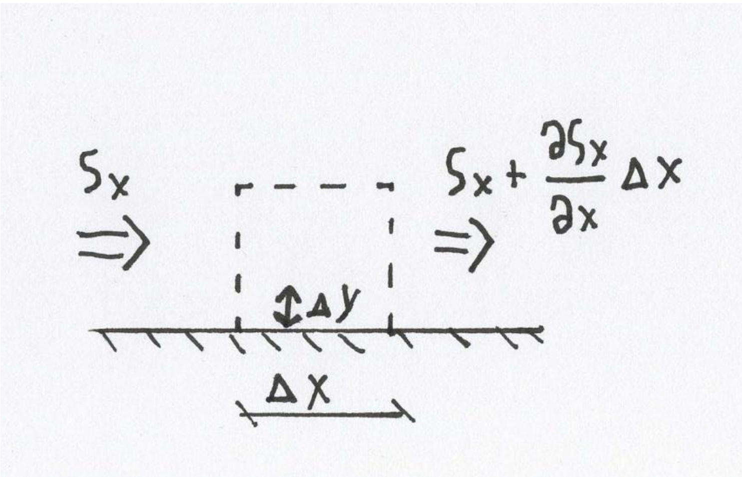


Figure 117: Sediment transport for a single segment of the beach

If looked at a single timestep, Δt, the volume of the sediment that stays in the box can be determined. This would be equal to $-\frac{\partial S_x}{\partial x} \cdot \Delta x \cdot \Delta t$. The sediment that is accumulated by the change of the coastline in this timestep can be written as $\Delta x \Delta y d$. The sediment that is accumulated by the change in sediment transport is equal to the sediment which is used for the change of the coastline. This leads to:

$$\frac{\partial y}{\partial t} = -\frac{1}{d}\frac{\partial S_x}{\partial x}$$

This means that the change is shoreline is correlated to the change in sediment transport. An increase in transport would lead to erosion and a retreat in the shoreline. A decrease in transport would lead sedimentation and to the shoreline moving in the offshore direction.

With this knowledge, it is possible to make an initial hypothesis of what will happen with the shoreline of Scheveningen for multiple alternatives of the construction.

Piles

Using piles to support the new pier would have the least impact. The alongshore sediment transport can continue mostly and is not disturbed. The sediment transport remains constant, like in Figure 118. This means that there is no significant change in the shoreline. The most important issue that could occur here are scour holes.

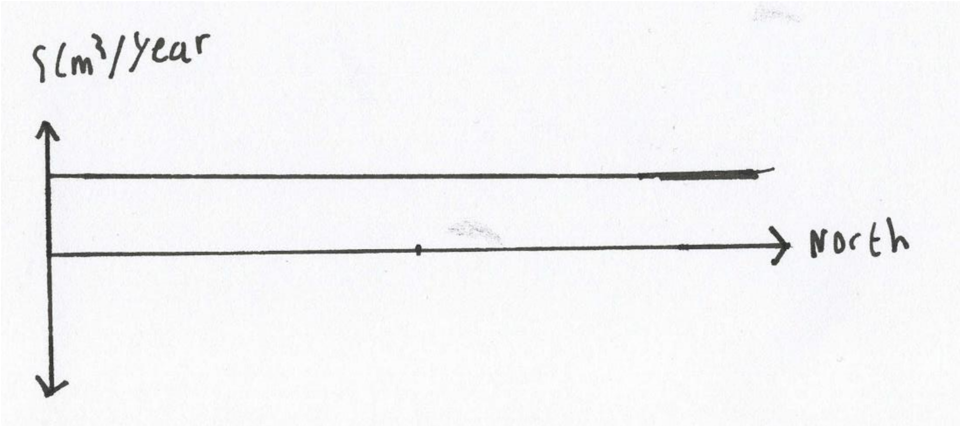


Figure 118: Sediment transport with a pile foundation

- Piles would not significantly impact the littoral drift.
- Main issue is scour holes

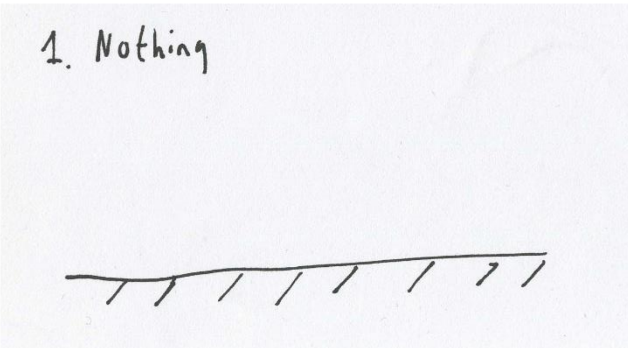


Figure 119: Coastline change of pile foundation

Full blockade

When the pier is supported by a closed construction, the sediment transport is blocked. Examples of this type of construction are a caisson or an impermeable shore normal breakwater. To make a hypothesis for this type of solution, an assumption is made which is schematised in Figure 120. The waves arrive from two directions, which are $45\ °$ and $-45\ °$ from the coast. This creates two shadow zones near the breakwater. The shadow zones are zones which cannot be reached by the waves, because they are blocked by the blockade.

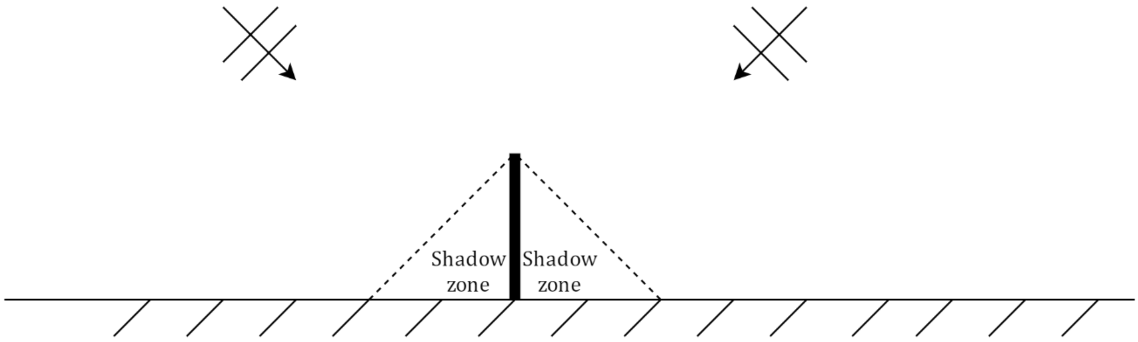


Figure 120: Schematisation of the waves and the blockade

At the areas which are not affected by the blockade, there is a net littoral drift towards the north, which is towards the right in Figure 120. First the south side of the blockade is viewed. The first change on that side occurs when the southern shadow zone is reached. The waves from the south are not impacted, but there are no or less waves from the north, which means that the transport towards the north increases. But at the pier itself, the transport has to be equal to zero, because it is completely blocked there. So on the south side of the pier, the transport first increases and the decrease towards zero.

North of the pier, there is a shadow zone blocking waves which come from the south, but not the waves from the north. So in the shadow zone, this transport towards the south increases while going more north. When out of the shadow zone, the sediment transport returns to the original situation. Figure 121 shows a diagram displaying these changes in sediment transport and how this leads to a change in the coastline. Accretion occurs near the pier and erosion occurs further away from it.

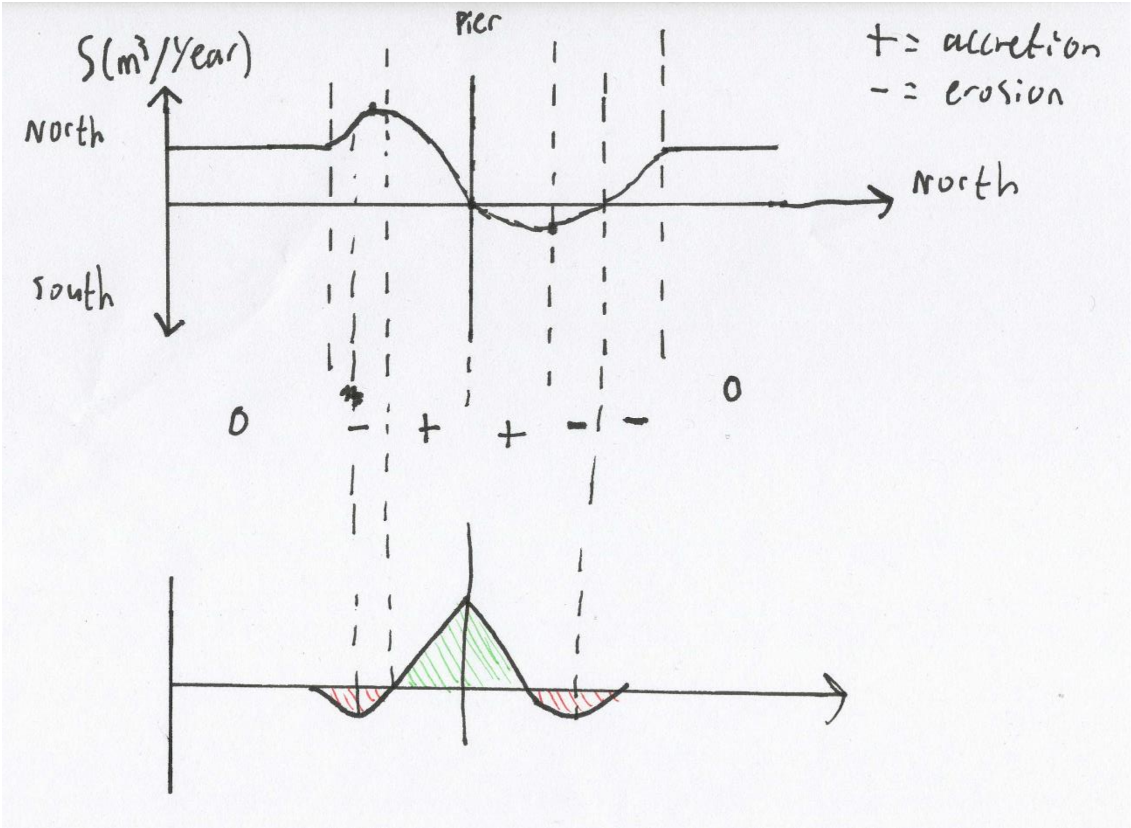


Figure 121: Sediment transport with the blockade and its impact on the coastline

- The pier could be lowered since walls would block the waves hitting the bottom of the structure
- Would lead to sedimentation close to the structure
- Erosion further away from the structure
- Depending on the sizes, a rip – current could occur between the pier and the harbour, which would be bad for swimmer safety and the beach.

Emerged breakwater

A shore parallel emerged breakwater creates a shadow on the onshore side of itself. The sediment transport would decrease and this causes sedimentation in the shadow zone. Outside the shadow zone, the sediment transport will increase again which leads to erosion outside the shadow zones. This could result in a tombolo or a salient, displayed in Figure 122.

- Could result in tombolo or salient
- Cause erosion next to it.

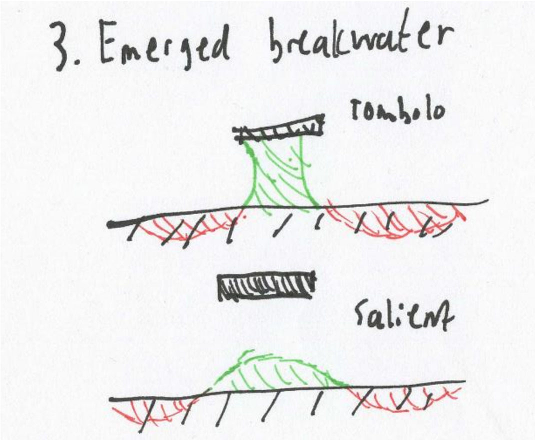


Figure 122: Erosion and sedimentation effects of an emerged breakwater

Series of groynes

A series of smaller groynes would counter any erosion effects for the area for which these groynes are located, since the sediment transport is trapped in between. However, this method was tried before to preserve the beach, south of the Scheveningen harbour (Waterman, 2020). But then the undesired effects or rip currents occurred. These are strong flows which are directed off-shore. This causes erosion and this endangers swimmer safety.

- Creates many shadow zones and leads to accretion. However, this leads to many rip currents between the groynes, which causes erosion and is bad for swimmer safety.
- This was tried in the south of Scheveningen harbour, but it did not work because of the rip currents (Waterman,2020)

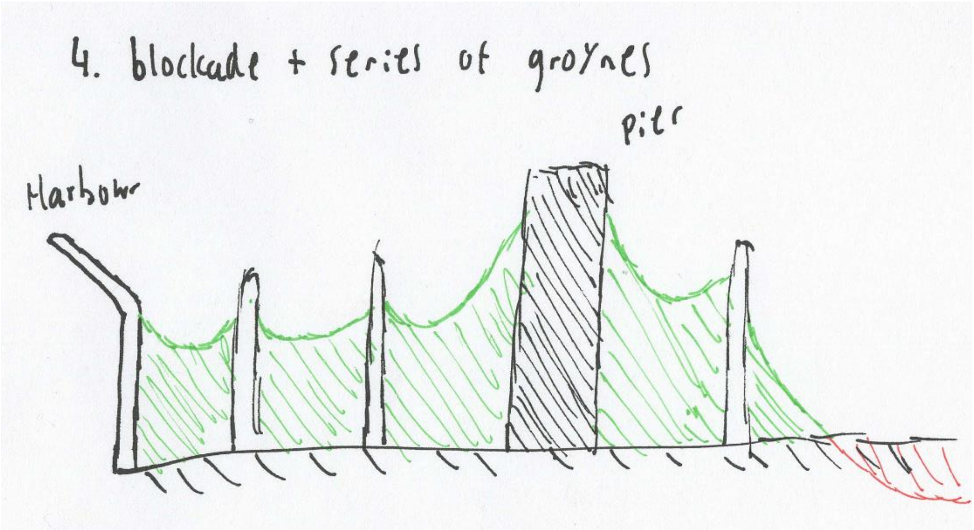


Figure 123: Erosion and sedimentation effects of an emerged breakwater

I.2 Beach model in Delft3D

Multiple models with Delft3D have been made to research the beach near the pier. This section explains how the model was set up and it discusses the results obtained from it.

Goal of the beach model

The goal of the beach model is to research the impact of different alternatives for a pier on the surrounding beach. A desired result is that more beach space, which could be used for recreation is being formed as a result of the pier. This cannot be at the cost of swimmer safety. Important results from the model are the erosion and sedimentation that occur in the project area, the water depth and the flow velocities.

The situations that are being tested are

- Zero situation
  - Nothing is added besides the original bathymetry.
- Full blocking
  - A set of dry points functions as pier. This pier is 500 m long and 300 meters wide. No water or sediment is able to go through this.
- Blocking at the end (similar to an emerged breakwater)
  - A block is placed at the end of the pier, but between the beach and this block, water and sediment can be transported.
- Series of groynes
  - A series of groynes is placed over the beach.

Software selection

There are multiple software which are able to model beach changes. These have also been used in the course Coastal Dynamics 2. Table 19 shows which software is available and gives comments about what type of model they are. The final column tells whether this software could be used for this research.

Table 19: Comparison of available coastal software

Model	Comments	Valid
Delft3D	• Area model • Good scale and time	Yes
Finel2D	• Area model • Takes probabilities into account. • Good scale and time • Area model	Yes
Xbeach	• Small scale • Storm erosion model • Does not model structural beach changes	No
AeoLis	• Is meant for aeolian transport • Aeolian transport model	No
ASMITA	• Models tidal basins	No
UNIBEST	• Large scale model • Line model	No

As Table 19 states, there are two possible software choices, which are Delft3D and Finel2D. The reason that the other three are invalid is because their main focus is on either storm erosion, aeolian transport or tidal basins or the scale is too large.

So both Delft3D and Finel2D are appropriate models to analyse the impact of a pier on the beach. Because the interface of Delft3D is more user friendly and because it is easier to create own models with Delft3D, it has been chosen to use this software to make a model.

Model size and bathymetry

The first steps of creating a model in Delft3D, is making a grid of the project area and modelling the bathymetry.

The first model that was made had a grid of 15 km beach, going 5 km offshore. This model started 2 km south of the harbour, until 10 km north of the location of the pier. The results of this model showed changes in the coastline, but it was also clear that most of these changes occurred closer to the pier itself. Therefore it was concluded that a smaller model was needed as well.

So after a large scale model, a smaller model was made. This had a grid of 4.5 km beach, going 2 km offshore. This model starts at the northern breakwater of the harbour and goes 4.5 km north from there. This model shows the impact more close to the pier.

Figure 124 and Figure 125 show the grid and the bathymetry of the large and small model respectively. Figure 126 and Figure 127 show the location of the harbour groynes (only for the large scale model) and of the pier. Figure 128 and Figure 129.

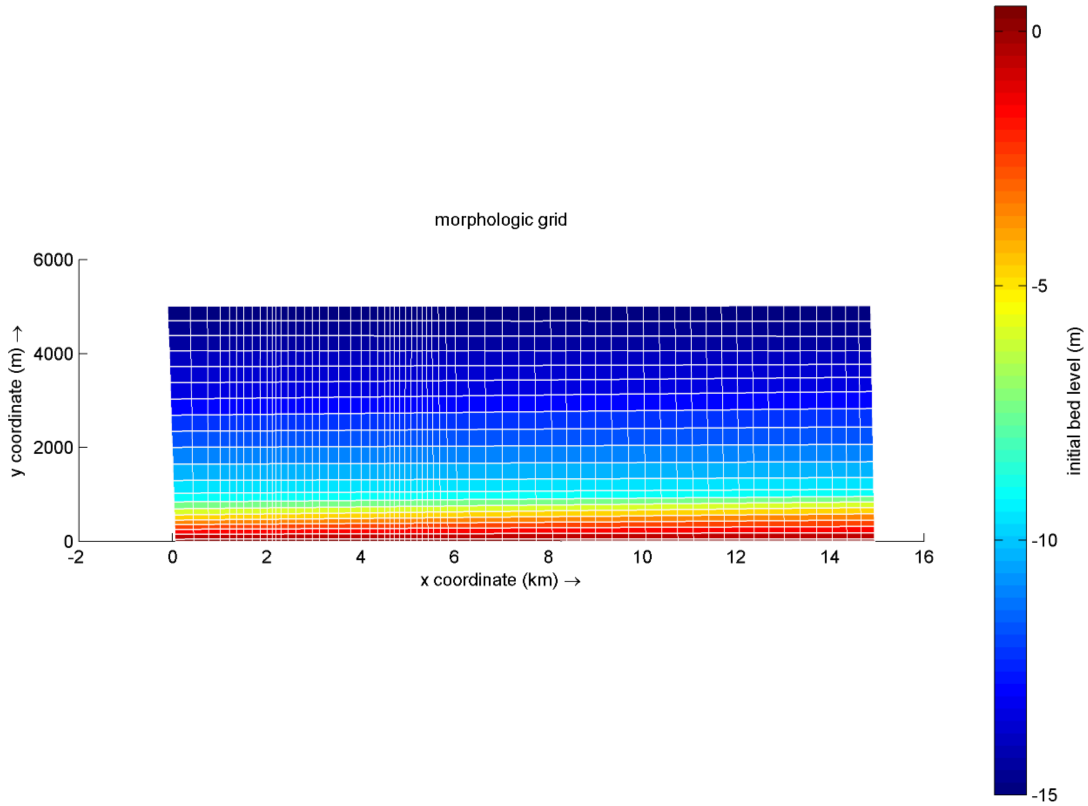


Figure 124: Grid and bathymetry of the large scale model

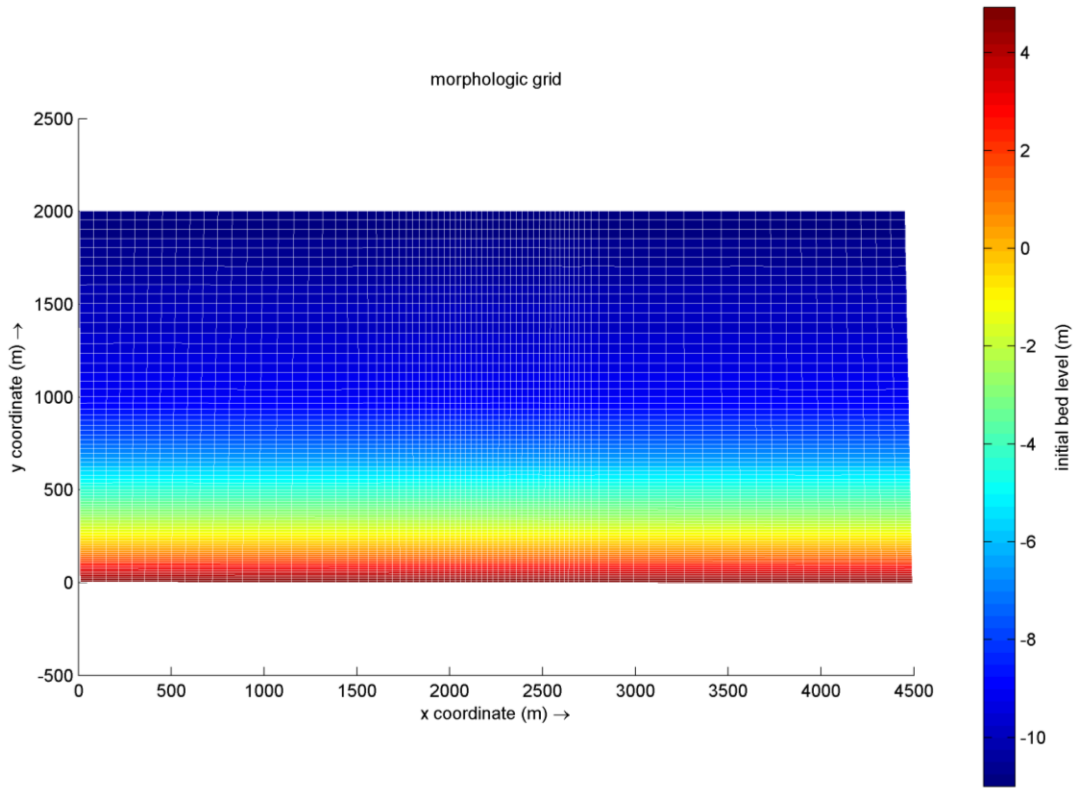


Figure 125: Grid and bathymetry of the small scale model

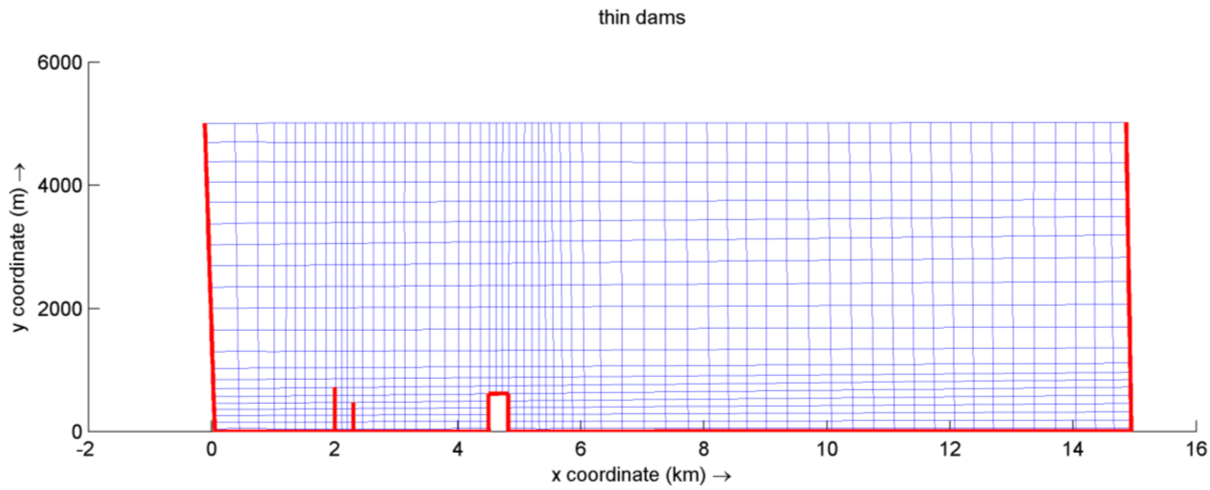


Figure 126: The locations of the harbour groynes (located around x = 2 km) and the pier (located around x = 4.5 km)

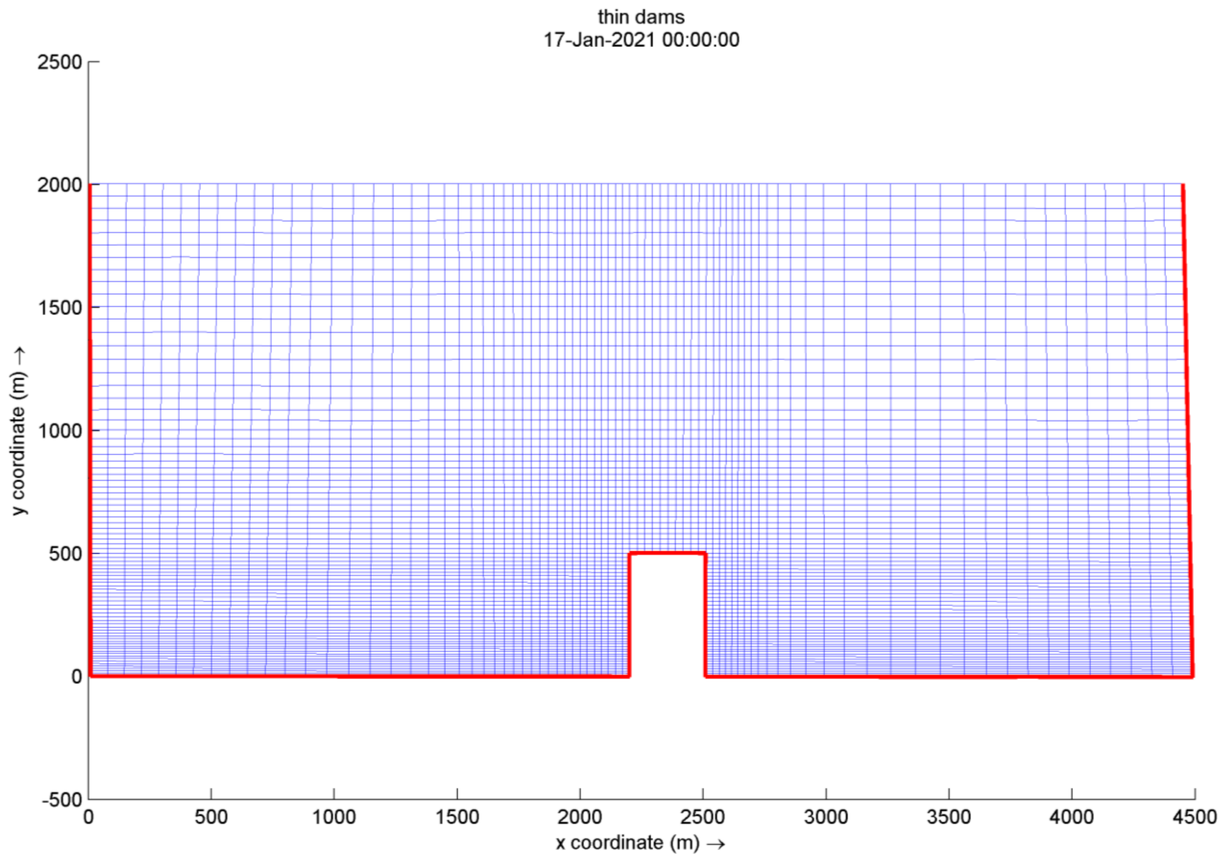


Figure 127: Location of the pier in the small scale model, located at around x = 2500 m

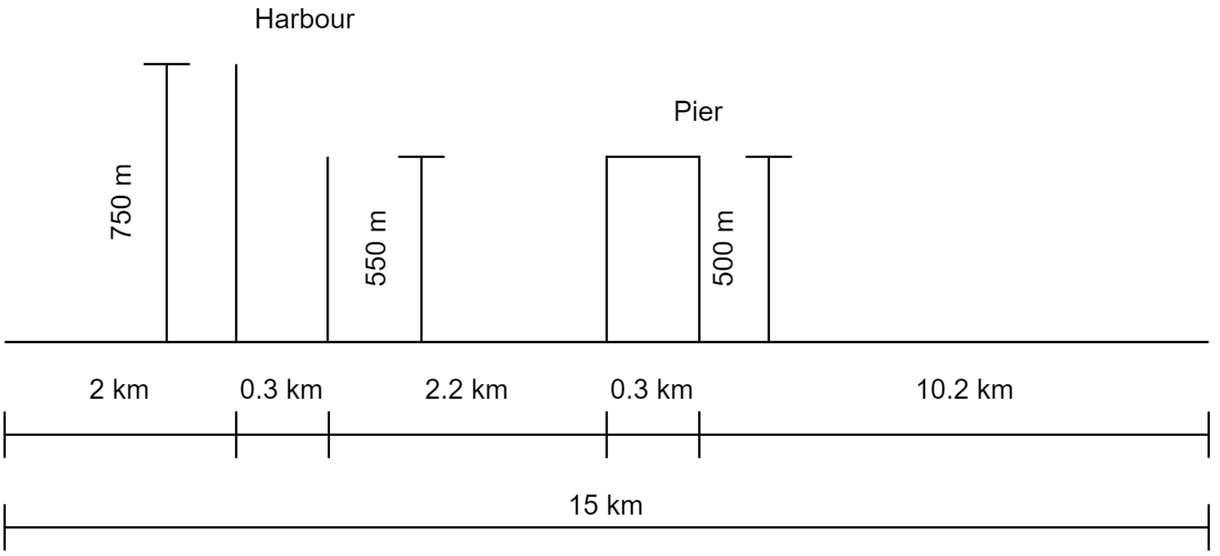


Figure 128: Schematisation of the large scale model (NTS)

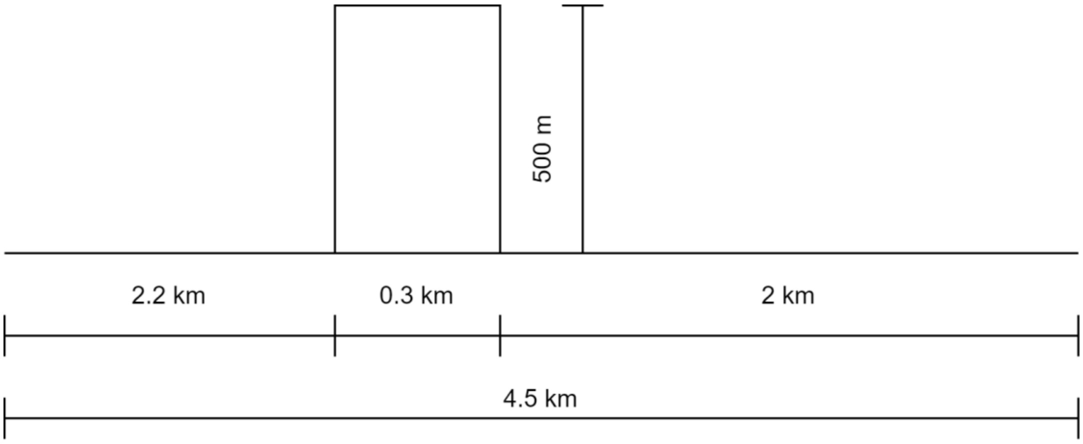


Figure 129: Schematisation of the small scale model (NTS)

Conditions

Delft3D requires input variables. These can be found in Table 20 and Table 21

Table 20: Variables which are the same over all the models

Variable	Value
Simulation time	2 days
Time step	10 min
Dry bed density	$1600\ \mathrm{kg/m^3}$
Specific density	$2600\ \mathrm{kg/m^3}$

Table 21:Variables which are different for each model

Variable	Large scale model	Small scale model
Waved direction (relative to north)	$45\ °$	$-45\ °$
H_{sig}	4 m	1.7 m
T_p	10 s	5 s
Morphological Factor	40	40

The morphological factor extrapolates the morphological changes, which makes it possible to predict the changes for a longer time than the simulation time. Waves coming in from 45 or -45 degrees have been chosen because these waves result in the most sediment transport, according to the $s - \phi$ curve, and therefore the most change in sediment transport, so that the changes in bed are well visible.

Results

Large scale model

The first model that was created was the large scale model. The erosion and sedimentation of the different measures. Unfortunately, there are not any significant differences between the situations. It is visible in Figure 130 that more sedimentation occurs with a full blockade, but no other significant changes are observed in comparison to the zero situation.

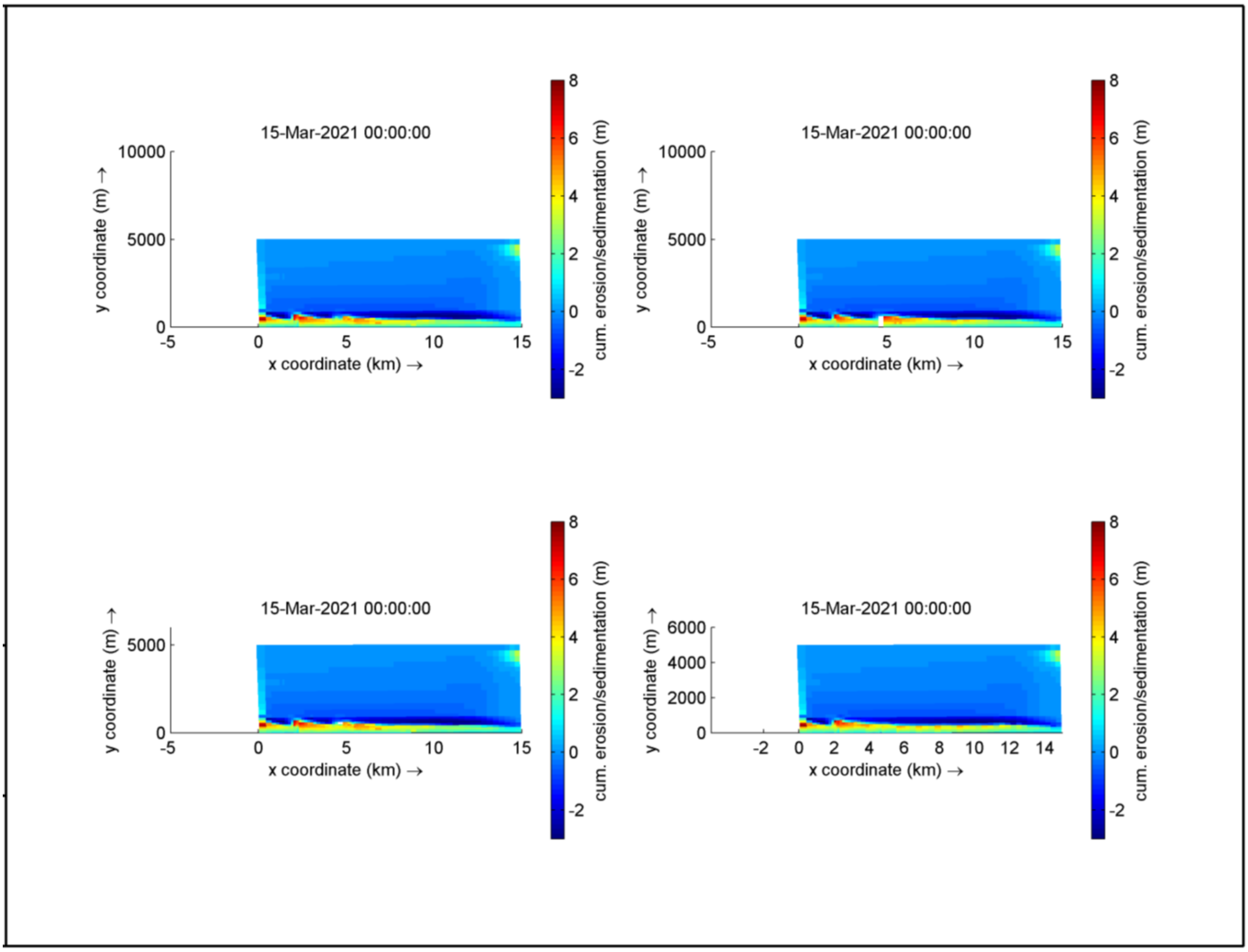


Figure 130: Erosion and sedimentation of the different measures. Top left: 0-situation, top right: full-blockade, bottom left: emerged breakwater, bottom right: groynes on multiple locations on the beach

Small scale model

Because the large scale model did not show any significant results, a smaller scale model was made to see the effects of the measures in more detail. This scale is too small to test the effect of a series of groynes, because this measure would be placed in a larger area. In this model, the waves come from the top left corner. The initial simulation time is 2 days, but a morphological factor of 40 is applied, so the results of the morphology are for a simulation of 80 days instead. Figure 131 to Figure 133 show the erosion and sedimentation on the coast. Figure 134 to Figure 136 show the water depth and Figure 137 to Figure 139 show the depth averaged flow velocity in the new situation.

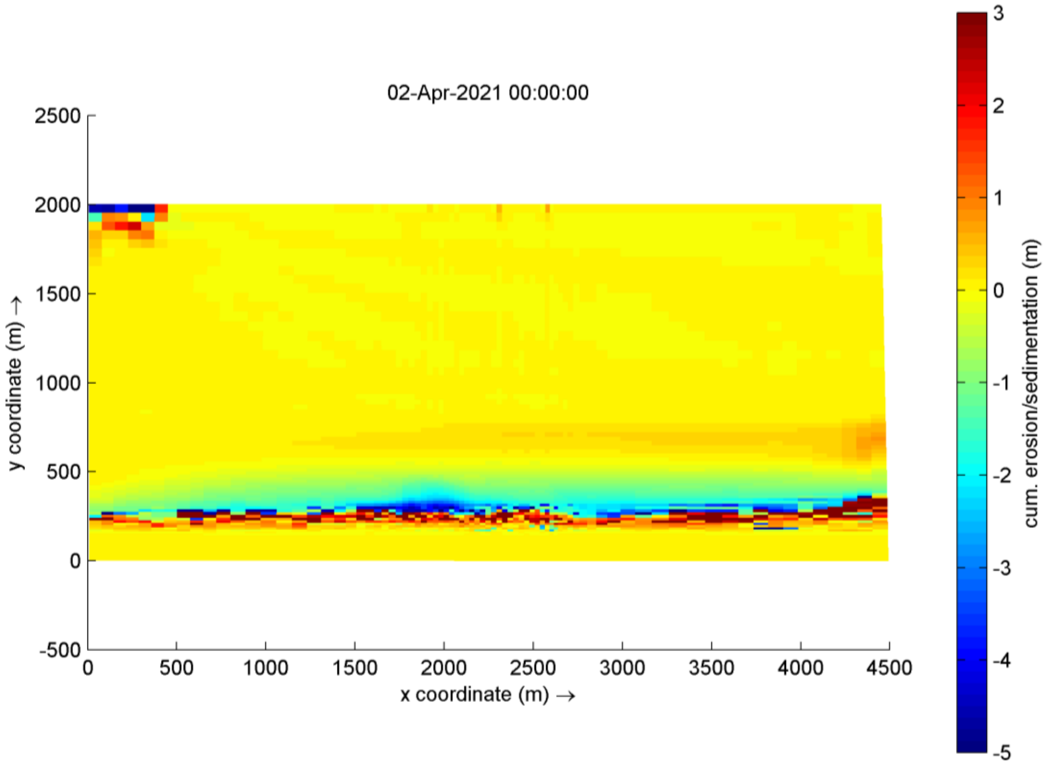


Figure 131: Cumulative erosion and sedimentation of the zero situation for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s,\ Dir = -45\ °$*, morphological factor = 40.*

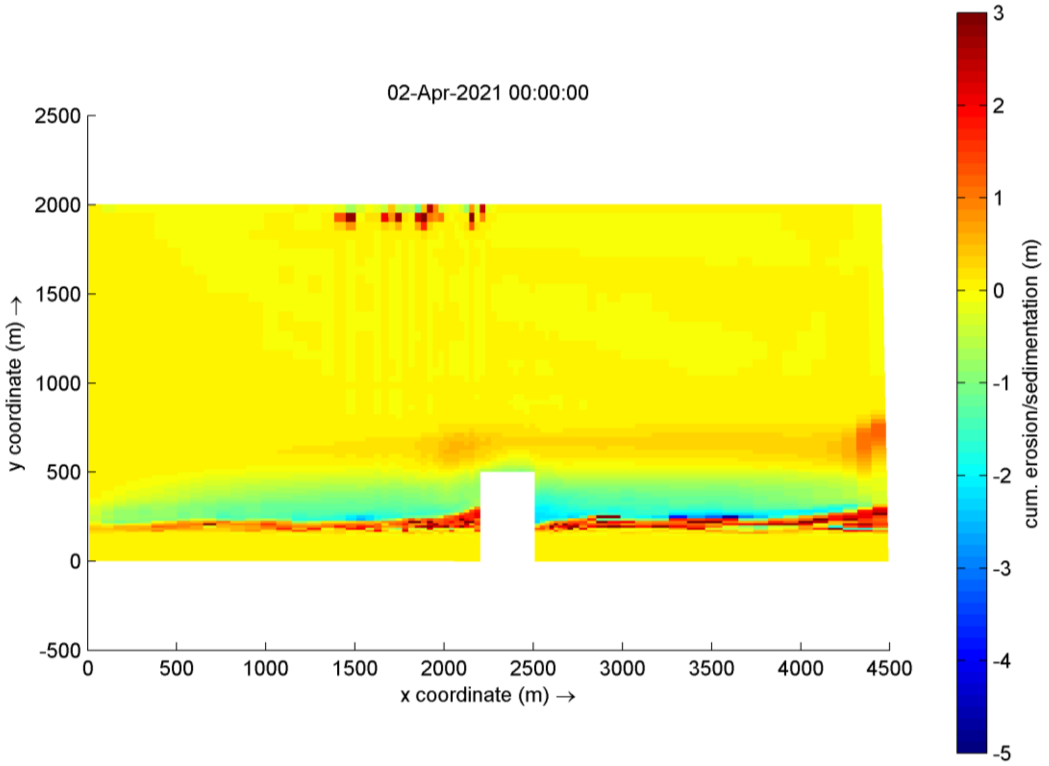


Figure 132: Cumulative erosion and sedimentation of the full blockade measure for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s,\ Dir = -45\ °$, *morphological factor = 40.*

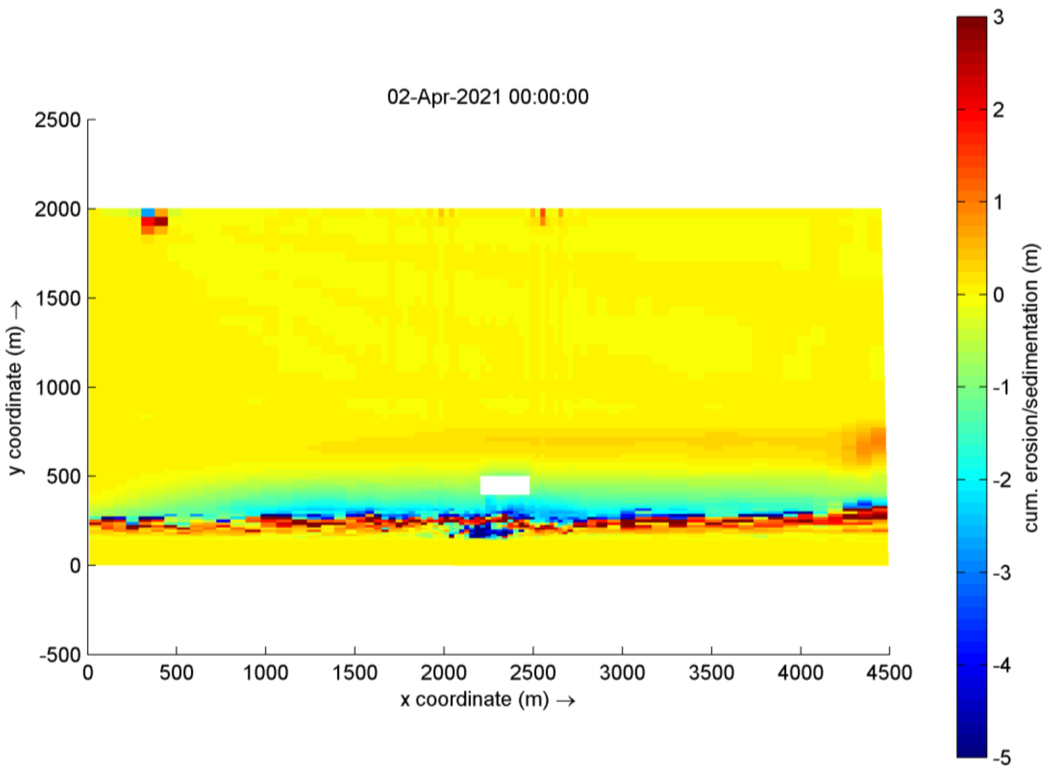


Figure 133: Cumulative erosion and sedimentation of the emerged breakwater measure for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s,\ Dir = -45\ °$, *morphological factor = 40.*

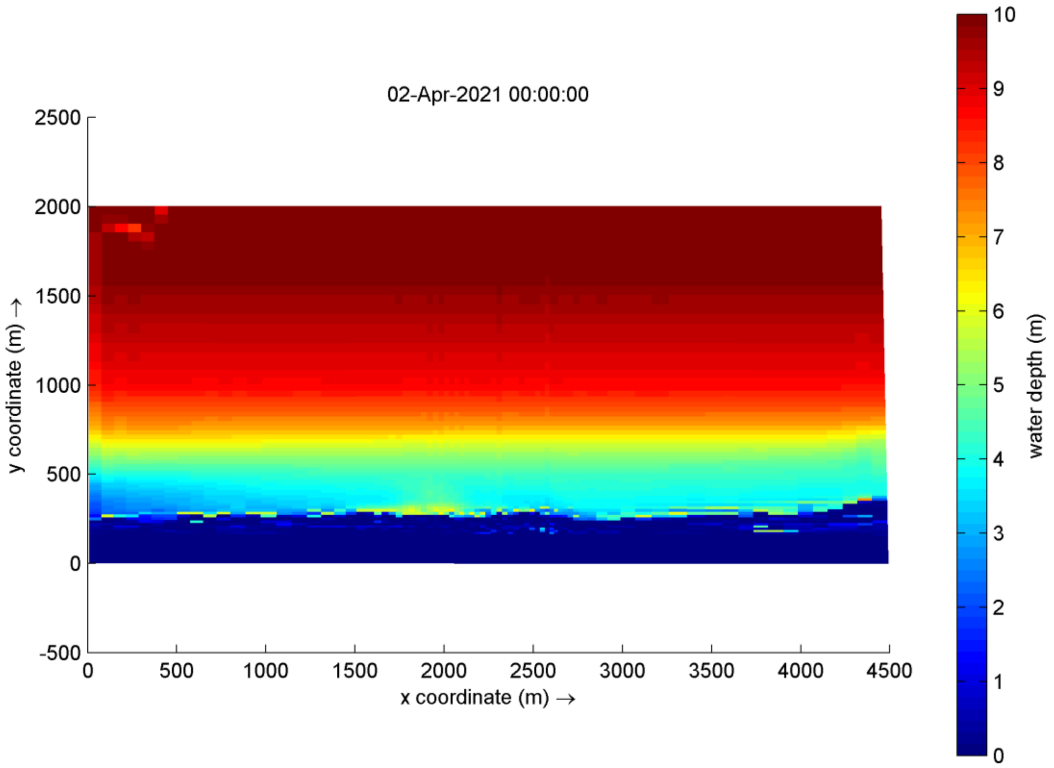


Figure 134: Water depth of the zero situation for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s,\ Dir = -45\ °$, *morphological factor = 40.*

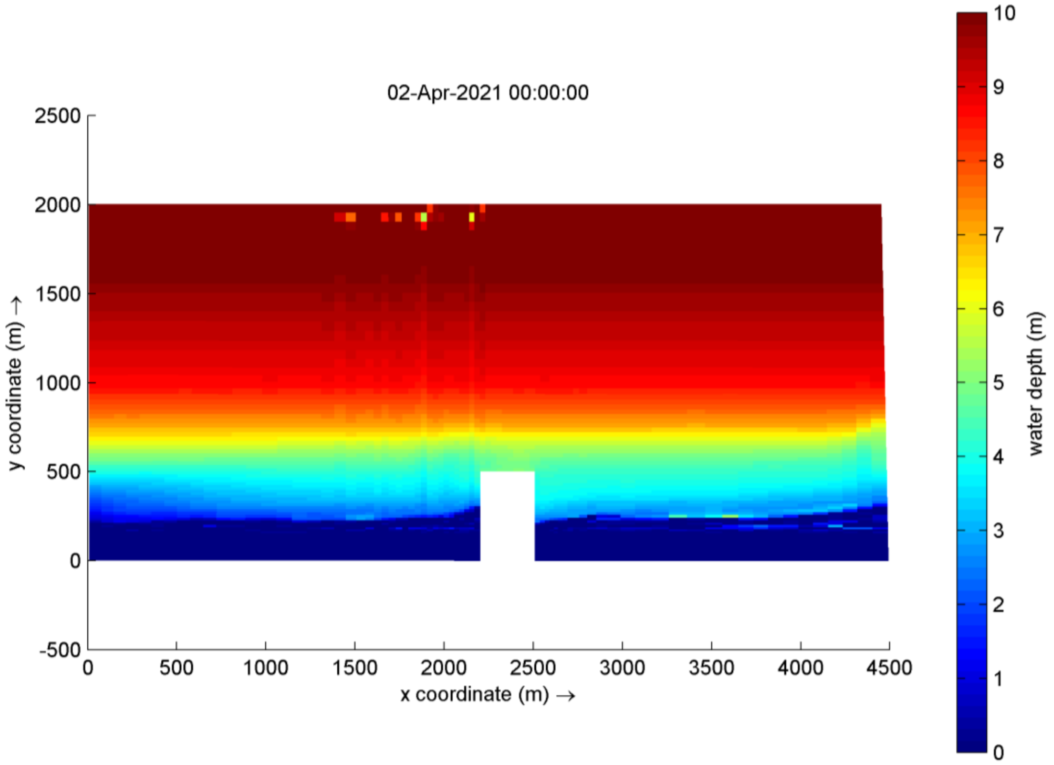


Figure 135: Water depth of the full blockade measure for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s,\ Dir = -45\ °$, *morphological factor = 40.*

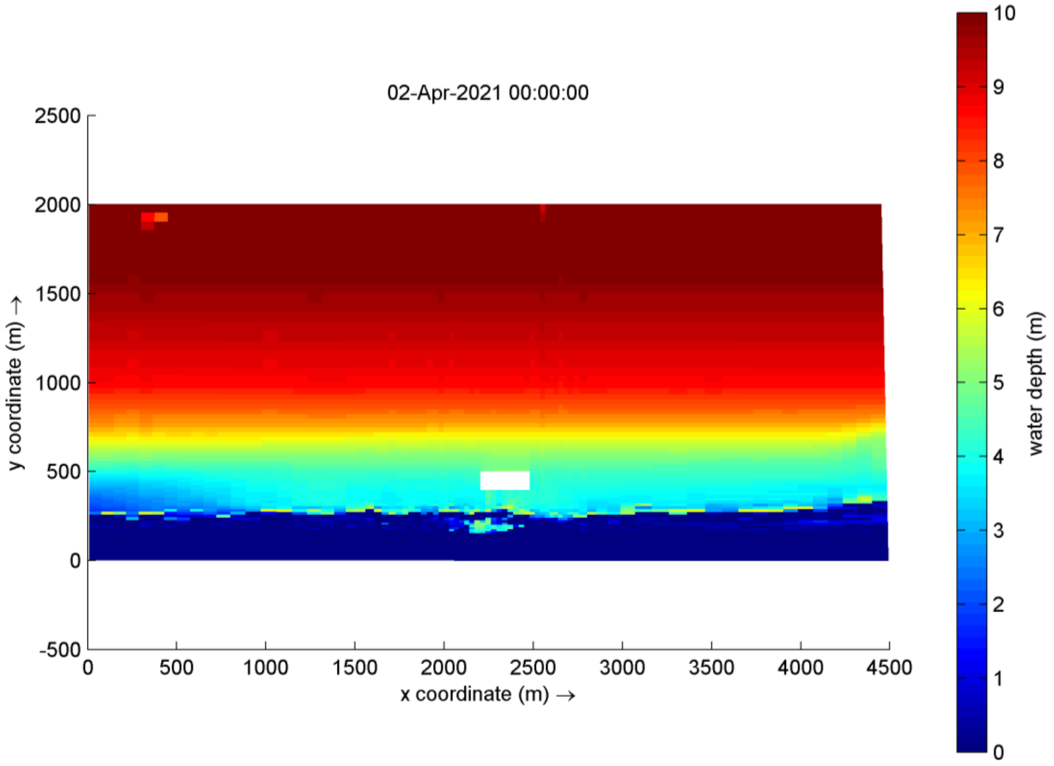


Figure 136: Water depth of the emerged breakwater measure for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s$, $Dir = -45\ °$, *morphological factor = 40.*

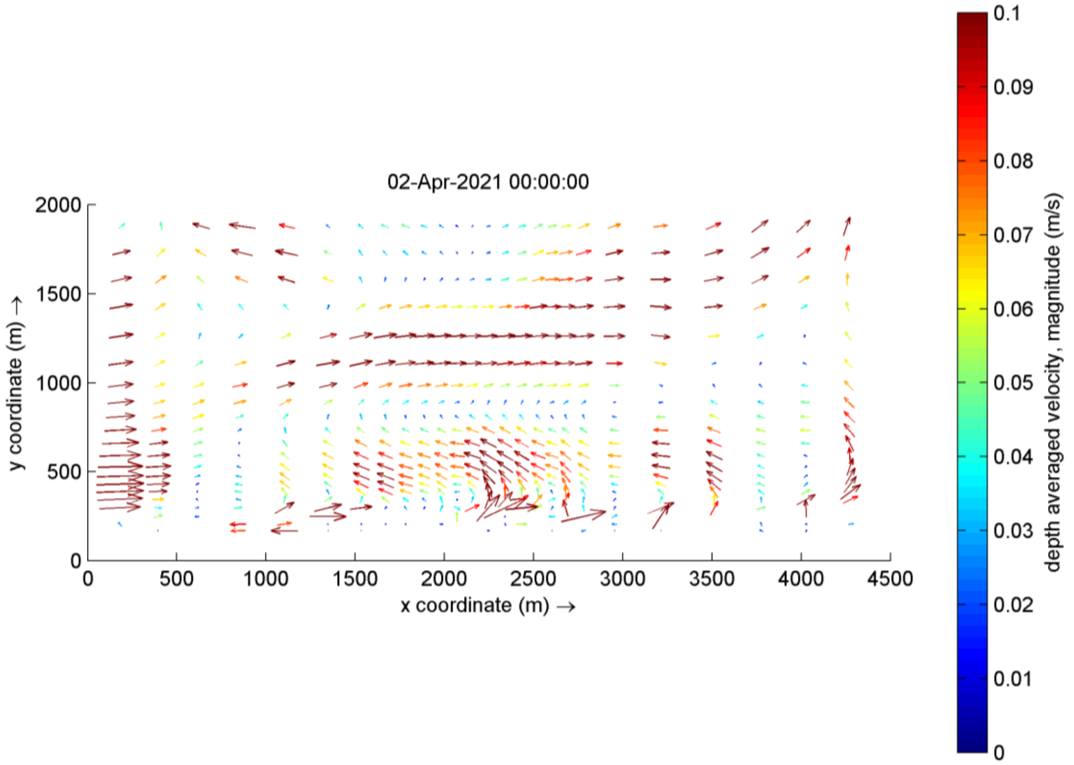


Figure 137: Depth averaged flow velocity of the zero situation for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s$, $Dir = -45\ °$, *morphological factor = 40.*

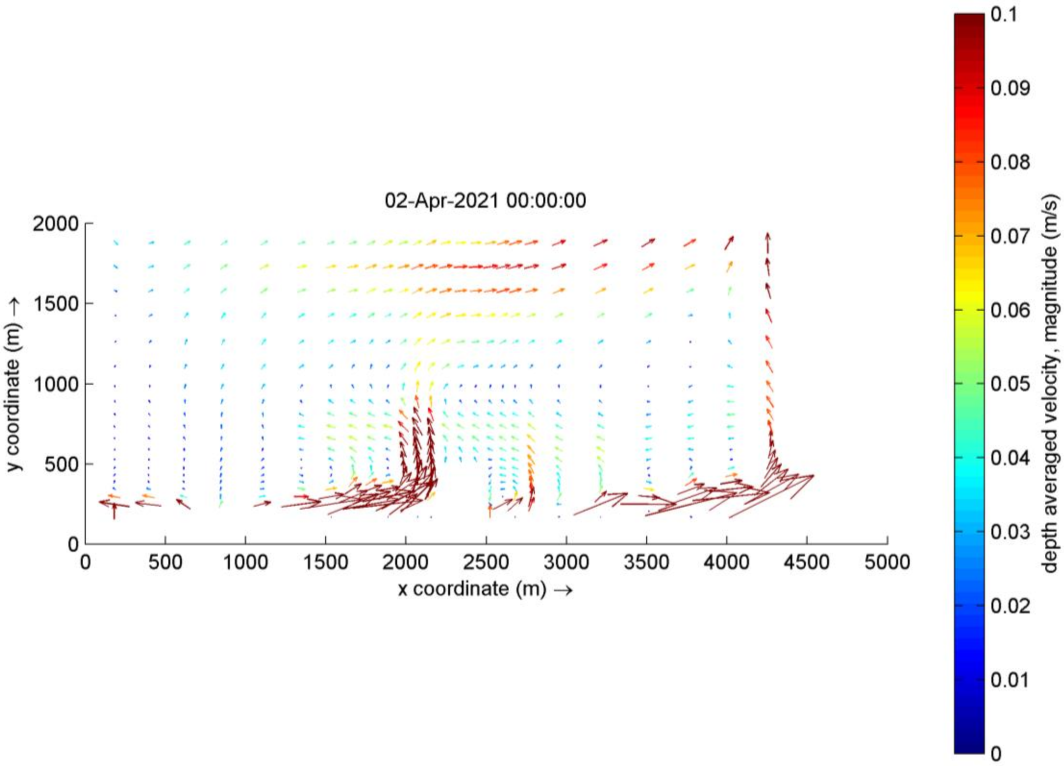


Figure 138: Depth averaged flow velocity of the full blockade measure for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s,\ Dir = -45\ °,$ *morphological factor = 40.*

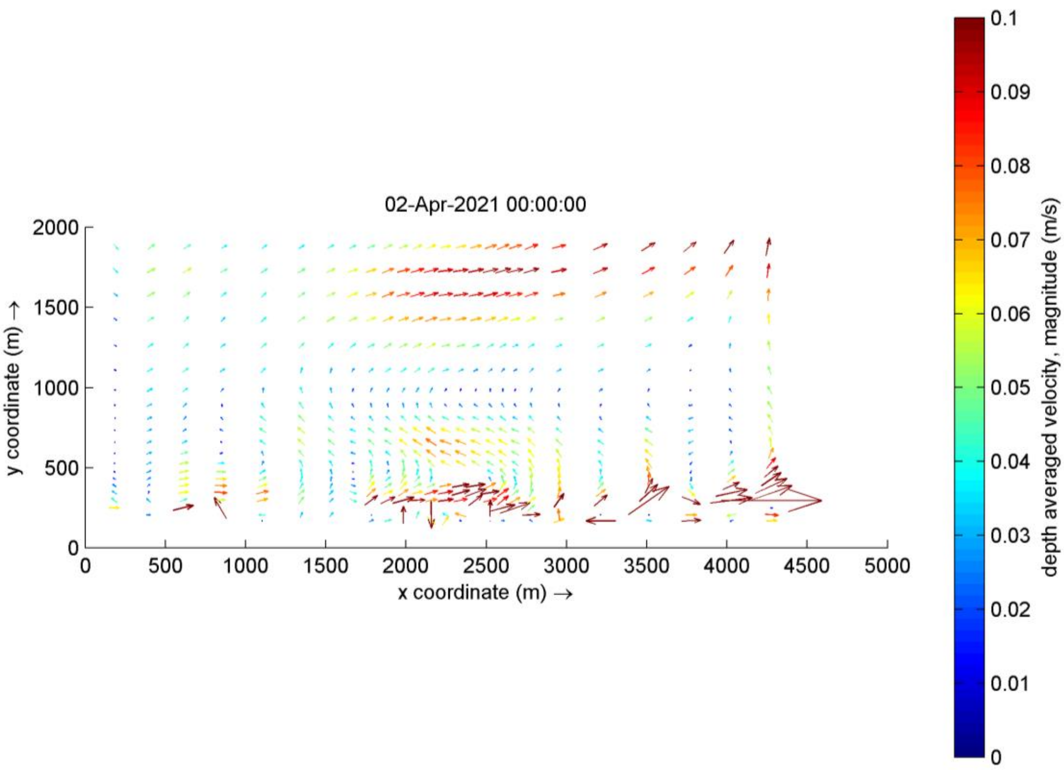


Figure 139: Depth averaged flow velocity of the emerged breakwater measure for the small scale model after 80 days. $H_{sig} = 1.7\ m, T_p = 5s,\ Dir = -45\ °,$ *morphological factor = 40.*

Appendix J Accommodation building

For the platform containing accommodation, a rough estimate needs to be made of the loads. Most of these loads come from the buildings itself. To do this, a basic layout has been made for the accommodation building. The loads, which are calculated in this appendix, are used in Chapter 6 to perform strength calculations on the structure of the pier.

A model has been made with Google Sketchup, these can be seen in Figure 140 to Figure 142. Each residence is contains 2 floors of around 50 m^2, so 100 m^2 per residence. There are 88 residences in total. The accommodation on the lower two floors occupy less space in total, to make space for bike parking. The red blocks indicate elevators or staircases. The building is estimated at 16 meters tall.

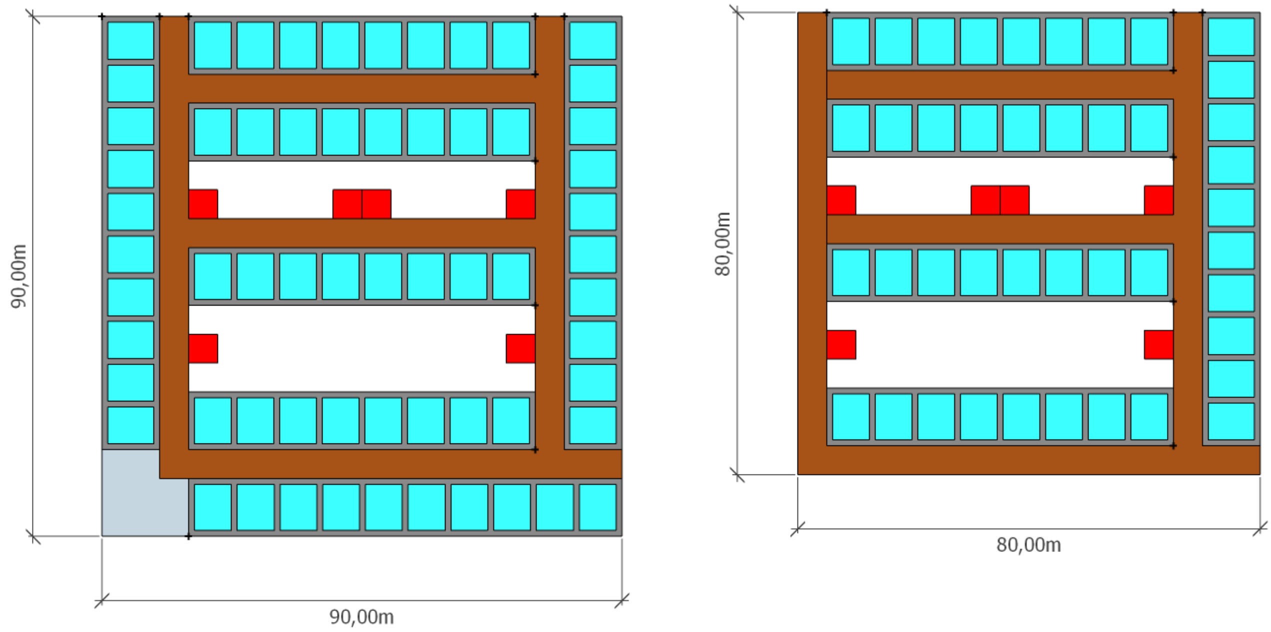


Figure 140: Floor plan of the top two floors (left) and the lower two floors (right)

Figure 141: Front view of the accommodation building

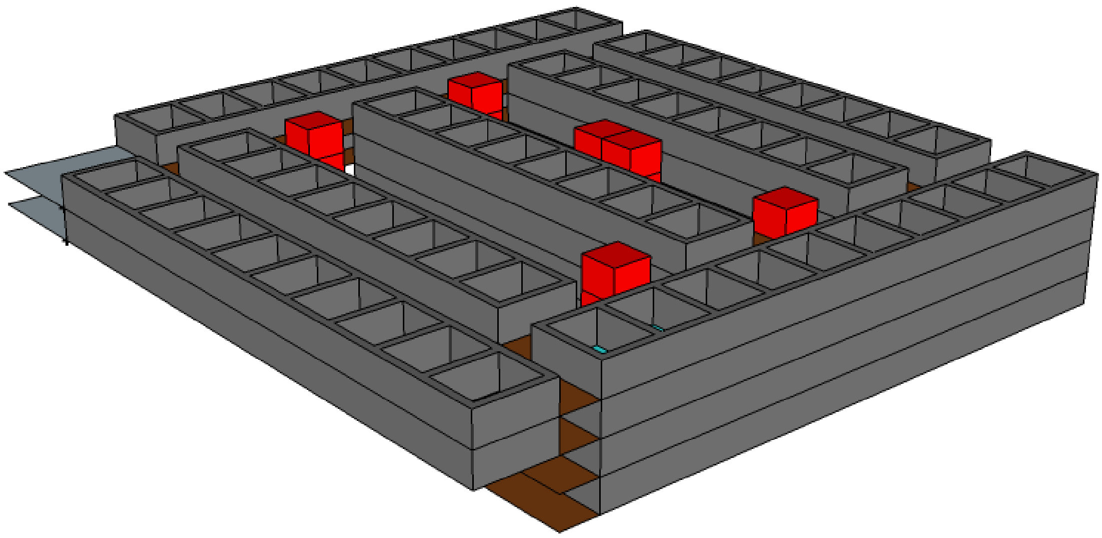


Figure 142: 3D view of the accommodation building, roof and outer walls are hidden

Loads

Table 22 and Table 23 show loads from the Quick Reference (Soons, van Raaij, & Wagemans, 2014) for different functions from a building. These values are used to calculate the load from the building.

Table 22: Variable loads for different functions (Soons, van Raaij, & Wagemans, 2014)

Load	**Distributed load (kN/m^2)**
Floor loads	
Restaurants	4
Shops	4
Traffic	2
Residential	1.75
Conference centre	4
Roof load	**1**

Table 23: Permanent loads (Soons, van Raaij, & Wagemans, 2014)

Load	**Distributed load (kN/m^2)**
Timber floor + beams	0.3
Flat roof + beams	0.36
Insulation and roofing	0.10 – 0.20

The lowest floor exerts an area load directly on the deck. The other floors support on columns of the building, so the upper floors only exert point loads on the deck, via the columns of the building. To reduce loads on the beams of the deck, the columns of the building should be aligned with the piles supporting the deck, which means that the building is supported by 64 columns in total, which are located above the piles of the pier which go to the sea bed.

From Table 22 and Table 23, the area loads was determined for both the roof and the floor. The area load for the roof consists of variable roof load, a flat roof with beams and insulation and roofing

$$Q_{roof,permanent} = 0.36 + 0.2 = 0.56 \text{ kN/m}^2$$

$$Q_{roof,variable} = 1 \text{ kN/m}^2$$

$$Q_{roof} = 1 + 0.36 + 0.2 \approx 1.6 \text{ kN/m}^2$$

The floor load consists of variable floor load for a residential area and a timber floor with beams.

$$Q_{floor,permanent} = 0.3 \text{ kN/m}^2$$

$$Q_{floor,variable} = 1.75 \text{ kN/m}^2$$

$$Q_{floor} = 1.75 + 0.3 \approx 2.1 \text{ kN/m}^2$$

To make the calculation less complicated, it is assumed that the floor load and the roof load act on the full are of 90 x 90. In reality the load covers less, since not the entire area is covered by apartments.

The lower floor exerts an area load on the deck immediately. The three floors above and the roof are carried by columns of the building, which directly transfer the load from to the piles below. The spacing between the columns should be the same as the piles, which is 12.86 m This means that one inner pile carries the load from an area of 12.86 x 12.86 m. Besides the load from the floors and the roof, the self-weight of the column is also taken into account.

$$F_{column} = 12.86^2 \cdot \left(Q_{roof} + 3 \cdot Q_{floor}\right) + \gamma_{concrete} \cdot h \cdot A_{column} = 1350 \text{ kN}$$

$$F_{column,permanent} = 12.86^2 \cdot \left(Q_{roof,permanent} + 3 \cdot Q_{floor,permanent}\right) + \gamma_{concrete} \cdot h \cdot A_{column} = 317 \text{ kN}$$

$$F_{column,variable} = 12.86^2 \cdot \left(Q_{roof,variable} + 3 \cdot Q_{floor,variable}\right) = 1033.62 \text{ kN}$$

$$\gamma_{concrete} = 24 \text{ kN/m}^3$$

For columns of a diameter of 0.5 and a 4 storey building of 16 meters tall , the load on the column is 1350 kN.

Appendix K Strength Calculations with Matrixframe model

A Matrixframe model was made to perform the strength checks of the structure. The results of these strength checks are used to perform a structural verification of the construction of the pier which has been designed in Chapter 6.

A two dimensional model was made which represents one row of columns of a platform of the pier. These are connected with each other with horizontal beams at the top, and with diagonal struts. The horizontal beams are also a part of the deck of the pier. The geometry of the model is seen in Figure 143. Vertical gridline A is located most off-shore and gridline H is located most onshore. Horizontal gridline 1 is at the bed level and gridline 2 is at the top of the piles. Horizontal gridline 0 is located at 0.65 of the embedded depth of the pile and at the bottom the piles have a fixed support. This is the same assumption as Blum's method. The connections between the columns, beams and struts are hinged.

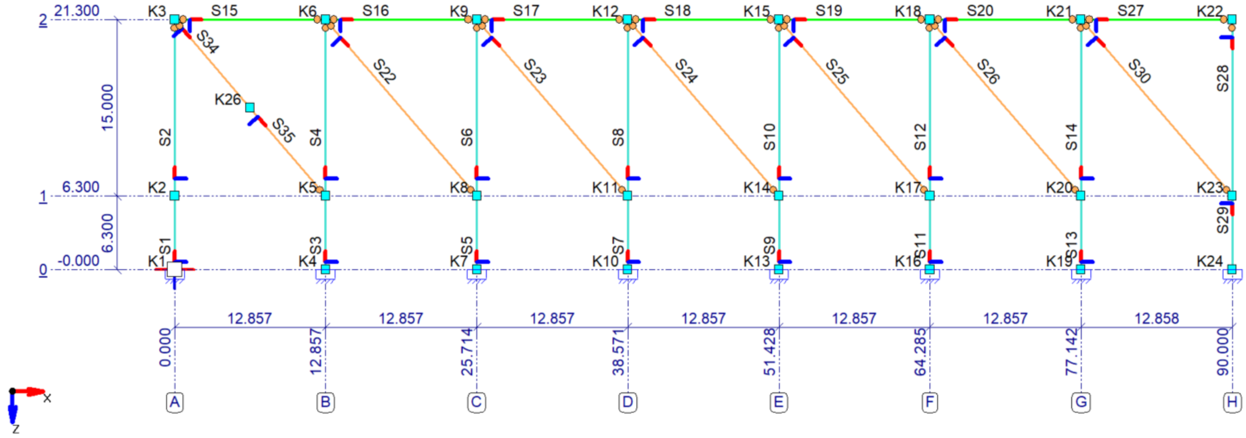


Figure 143: Geometry of the Matrixframe model. Distances in m.

Different loads were entered in Matrixframe. These are L1 to L5. The schematisations of these loads can be seen in Figure 149 to Figure 153.

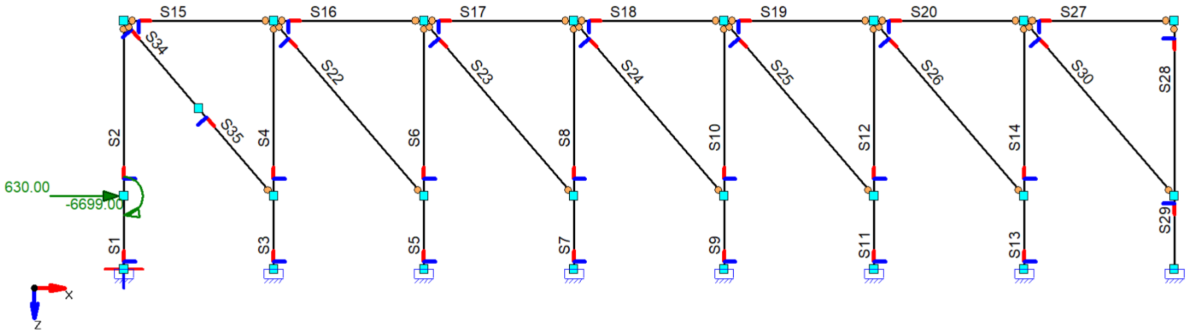


Figure 144: L1: Lateral loads (wave, wind and flow) acting on the pile. Moved to the bed. Units in kN and kNm

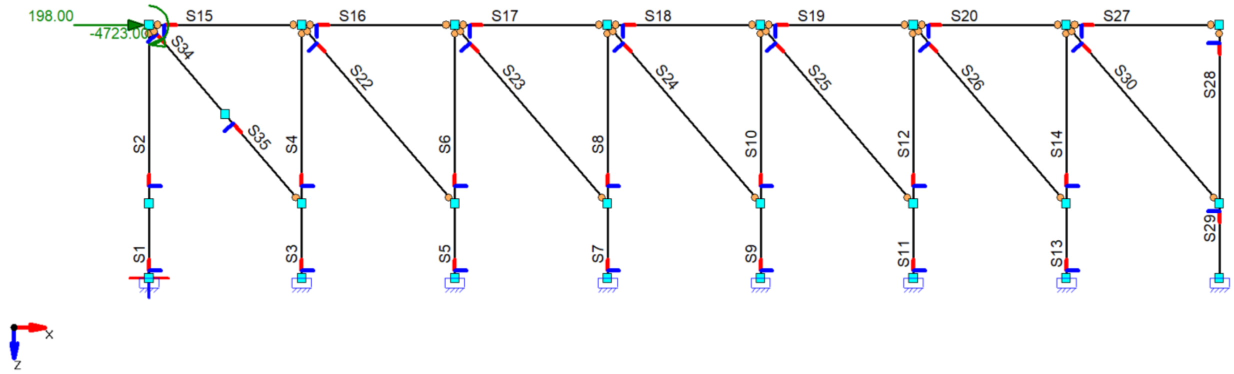


Figure 145: L2: Wind force acting on the building. Moved to the deck level. Units in kN and kNm

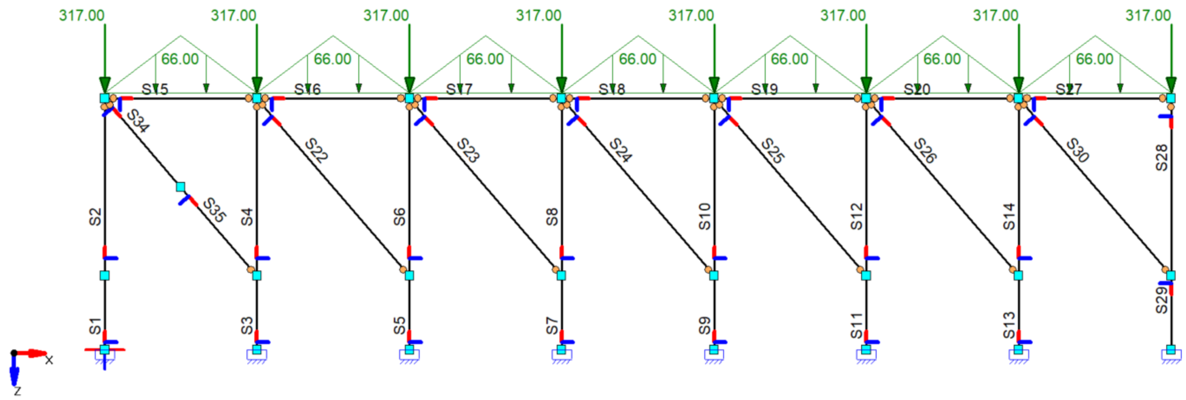


Figure 146: L3: Permanent load from the building and the deck plates. Units in kN and kN/m

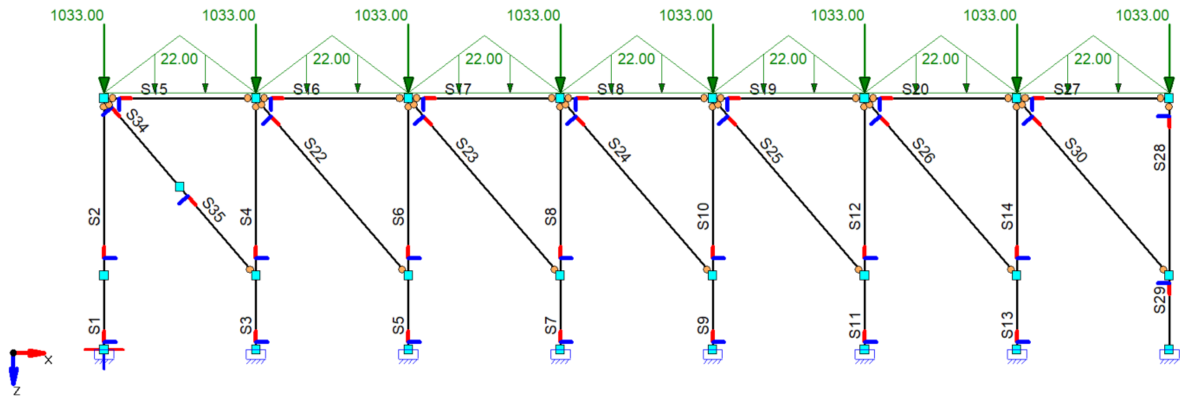


Figure 147: L4: Variable load from the building and the deck plates. Units in kN and kN/m

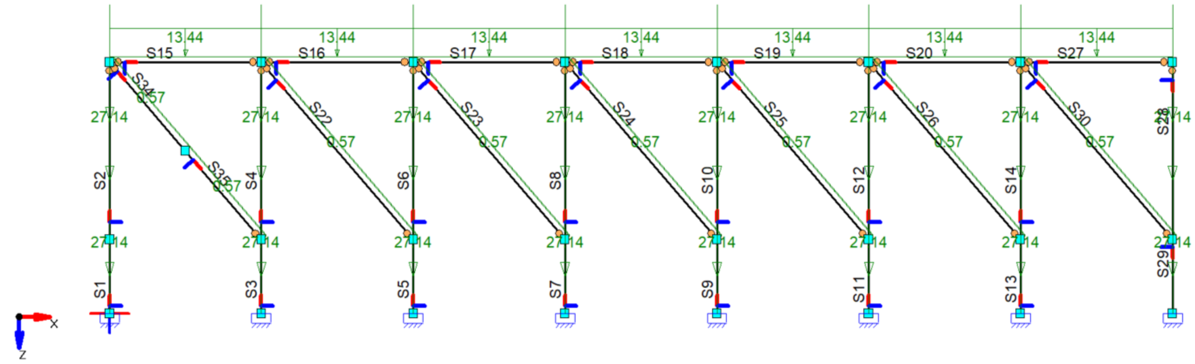


Figure 148: L5: Self-weight of the columns and the deck beams. Units in kN/m

Different load combinations were made to test different failure mechanisms. These load combinations LC.1 to LC. 7 are described in Table 24 and Table 18. The safety factors for each load for each load combination can be found in Table 19. These values are derived from the safety factors from Table 8 and Table 9 from subsection 6.4.1.

Table 24: Load combinations for the column

Load combinations	**Governing situation**
LC. 1	Tensile stress / shear force / lateral soil resistance
LC. 2	Bearing capacity (permanent is dominant)
LC. 3	Bearing capacity (variable load is dominant)
LC. 4	Compression stress (permanent load is dominant)
LC. 5	Compression stress (variable load is dominant)

Table 25: Load combinations for the beams

Load combinations	**Governing situation**
LC. 6	Tension stress/shear force in the beams
LC. 7	Compression stress in the beams

Table 26: Safety factors for the different loads cases

Load	LC.1	LC. 2	LC. 3	LC. 4	LC. 5	LC. 6	LC.7
L1	1.00	-	-	1.00	1.00	-	1.00
L2	1.30	-	-	1.30	1.30	-	1.80
L3	0.82	1.40	1.20	1.40	1.20	1.40	1.20
L4	-	1.30	1.80	1.30	1.80	1.30	1.30
L5	0.82	1.40	1.20	1.40	1.20	1.40	1.20

Matrixframe then performs a linear elastic calculation. With this, it can calculate the internal forces, the deflections and the stresses in the beams and columns. The results are summarised in Table 27 and Table 28. These results were used to perform the unity checks for the beams and the piles.

Table 27: Maximum values of the deflections, forces and stresses in the piles according to Matrixframe

	Value	**Load Combination**
Maximum deflection [mm]	31.3	Fu.C.1
Maximum normal force [kN]	-3924	Fu.C.5
Maximum shear force [kN]	323	Fu.C.1

Maximum bending moment [kNm]	4631	Fu.C.5
Maximum vertical support reaction [kN]	3924	Fu.C.5
Maximum horizontal support reaction [kN]	323	Fu.C.1
Maximum bending moment in support [kN]	4119	Fu.C.1
Maximum compression stress [Mpa]	-30.05	Fu.C.5

Table 28: Maximum values of the deflections, forces and stresses in the deck beams according to Matrixframe

	Value	**Load Combination**
Maximum deflection [mm]	33.5	Fu.C. 6
Maximum normal force [kN]	-551	Fu.C.7
Maximum shear force [kN]	510	Fu.C.6
Maximum bending moment [kNm]	2055	Fu.C.6
Maximum compression stress [kPa]	-28.37	Fu.C.6

Appendix L Structural calculations of the pier deck

This appendix shows how the structural calculations were performed for the pier deck. Besides the calculations, the dimensions of the pier deck are also given in this appendix. The pier deck consists of deck plates and deck beams.

L.1 Deck plates

An area load is working on the plates from above. This loads comes from the buildings on top of the deck. For this specific platform it is the accommodation. The plates have been dimensioned first. These are schematised as a beam supported on two sides, with a width of 1 meter. The plates have to carry a floor load and their own self weight. This schematisation is displayed in Figure 149 and Figure 150.

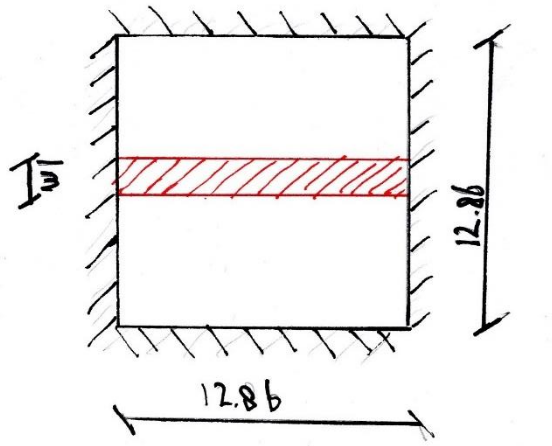


Figure 149: Square deck plate. The red section is the part that is modelled. This is section is 1 meter wide.

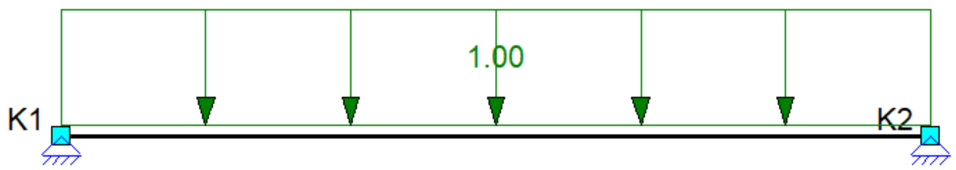


Figure 150: Schematisation of the plates.

$$q = 1 \cdot Q_{floor}$$

$$V_{\max} = \frac{1}{2} \cdot q \cdot l = 63.62 \text{ kN}$$

$$M_{\max} = \frac{1}{8} \cdot q \cdot l^2 = 204.55 \text{ kNm}$$

$$\sigma_c = \frac{M_{\max} \cdot \frac{1}{2} \cdot h}{I_{zz}} = 30683.08 \text{ Kpa}$$

$$w_{\max} = \frac{5}{384} \frac{q \cdot l^4}{EI} = 0.1428 \text{ m}$$

Compression stress

The concrete that is used is C50/60. This has a characteristic strength of 50 MPa. A material factor for the concrete, γ_c, needs to be applied to account for safety.

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{50}{1.5} = 33 \text{ Mpa}$$

$$u.c_{compression} = \frac{\sigma_c}{f_{cd}}$$

Tensile stress

The tensile force, taken by the reinforcement should compensate for the bending moment. The arm between the tensile reinforcement and the centre of the compression stress is denoted as z_s.

$$M_{Rd} = \frac{f_s}{\gamma_s} \cdot A_s \cdot z_s = 237.88 \text{ kNm}$$

$$f_s = 500 \text{ MPa}$$

$$\gamma_s = 1.3$$

$$z_s = 0.9 \cdot h \ (quick\ reference)$$

The unity check for the tensile reinforcement is given as follows:

$$u.c_{tensile\ reinforcement} = \frac{M_{Ed}}{M_{Rd}}$$

Shear stress

The resistance for shear-force of concrete according to the Eurocode is given in the following formula (Molenaar & Voorendt, 2020).

$$V_{Rd,c} = \frac{\left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] A_{section}}{1000} = 211.79\ kN$$

$$C_{Rd,c} = 0.12$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (d\ in\ mm)$$

$$\rho_1 = \frac{A_{tensile\ reinforcement}}{A_{concrete}} \leq 0.02$$

$$f_{ck} = compressive\ cylinder\ strength$$

$$k_1 = 0.15$$

$$\sigma_{cp} = compressive\ stress \leq 0.2 \cdot f_{cd} = 0$$

This leads to a unity check of

$$u.c_{shear} = \frac{V_{Ed}}{V_{Rd,c}}$$

Deflection

The maximum allowable deflection is dependant of the length of the structure.

$$w_{allowed} = 0.004 \cdot 90 = 0.36 \text{ m}$$

$$u.c_{deflection} = \frac{w_{\max}}{w_{allowed}}$$

Dimensions of the deck plates

Table 29 shows the parameters which were chosen for the plates of the deck. Table 30 shows the unity checks for these parameters. A section of 1 meter width of a deck plate can be seen in Figure 151.

Table 29: Parameters for the deck plates for preliminary calculations

Concrete	
Height	200 mm
Concrete strength class	$C50/60$
Tensile reinforcement	
Diameter	25 mm
Number of bars	$7/\text{m } width$
Reinforcement steel type	B500B
Shear reinforcement not required	

Table 30: Unity checks for the plates of the deck

Compression stress	0.92
Tensile reinforcement	0.86
Shear stress	0.30
Deflection	0.40

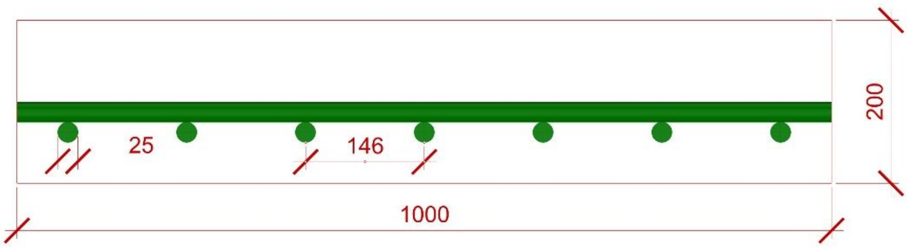


Figure 151: Section of the deck plate showing the reinforcement

L.2 Deck beams

Figure 152 shows how the deck plates transfer their load to the beams which support them on the side. The load is divided equally among the beams, since the plates are square. Every beam carries a quarter of the load from each plate it connects to, so if it is connected to two plates, the total load on that beam equals the load working on half a plate. The load working on the beam is triangular. The peak is this line load is called q_{beam}. The load from the plate is the floor load and the weight of the plate.

$$\frac{1}{2} q_{beam} \cdot a = \frac{1}{2} \cdot a^2 \cdot Q_{plate}$$

$$q_{beam} = a \cdot Q_{plate}$$

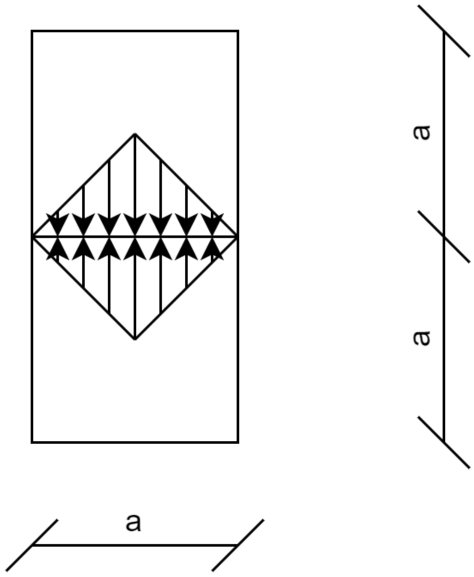


Figure 152: Top view of two square plates and the beams on which they support. Every plate transfers ¼th of its area load to the beam

A schematisation of an entire beam of 90 meter is given in Figure 153. As mentioned before, the columns of the building should be placed above the piles of the pier, therefore they do not cause internal bending moments in the beam.

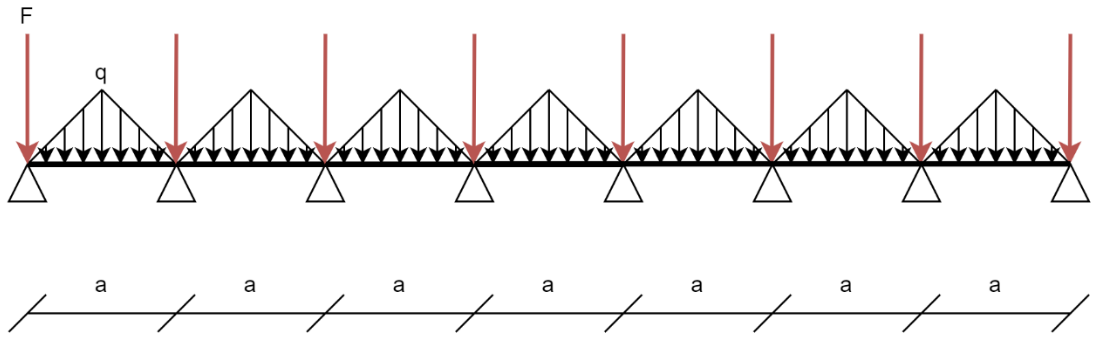


Figure 153: Schematisation of the load on 7 beams. The supports represent the piles on which the beams are placed.

Compression stress

$$f_{cd} = \frac{50}{1.5} = 33 \text{ Mpa}$$

$$u.c_{compression} = \frac{\sigma_c}{f_{cd}}$$

Tensile Reinforcement

The tensile force, taken by the reinforcement should compensate for the bending moment. The arm between the tensile reinforcement and the centre of the compression stress is denoted as z_s

$$M_{Rd} = \frac{f_s}{\gamma_s} \cdot A_s \cdot z_s = 2227 \text{ kNm}$$

$$f_s = 500 \text{ Mpa}$$

$$\gamma_s = 1.3$$

$$z_s = 0.9 \cdot d \ (quick\ reference)$$

The unity check for the tensile reinforcement is given as follows:

$$u.c_{tensile\ reinforcement} = \frac{M_{Ed}}{M_{Rd}}$$

Shear without reinforcement

The resistance for shear-force of concrete is given in the following formula (Molenaar & Voorendt, 2020).

$$V_{Rd,c} = \frac{\left[C_{Rd,c} \cdot k \cdot \left(100 \cdot \rho_1 \cdot \frac{f_{ck}}{\gamma_c}\right)^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] A_{section}}{1000} \ [kN]$$

$$C_{Rd,c} = 0.12$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (d \text{ in mm})$$

$$\rho_1 = \frac{A_{tensile\ reinforcement}}{A_{concrete}} \leq 0.02$$

$$f_{ck} = compressive\ cylinder\ strength$$

$$\gamma_c = 1.5$$

$$k_1 = 0.15$$

$$\sigma_{cp} = compressive\ stress \leq 0.2 \cdot f_{cd}$$

This leads to a unity check of

$$u.c_{shear} = \frac{V_{Ed}}{V_{Rd,c}}$$

If these requirement is not met, shear reinforcement must be added in the form of stirrups

Shear with reinforcement

When the concrete itself is not strong enough to resist the maximum shear force, reinforcement is required. Shear reinforcement consists of concrete stirrups. The resistance is calculated based on the truss model (Molenaar & Voorendt, 2020).

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot \frac{f_{ywd}}{\gamma_s} \cdot \cot(\theta)$$

$$A_{sw} = cross\text{-}sectional\ area\ stirrups = \frac{1}{2} d_{stirrup}^2$$

$$s = distance\ between\ stirrups$$

$$z = arm\ of\ internal\ leverage$$

$$\theta = angle\ concrete\ compression\ strutt$$

$$21.8\ ° \leq \theta \leq 45\ °$$

With a maximum resistance of.

$$V_{Rd,max} = \frac{\alpha_{cw} \cdot D \cdot z \cdot v_1 \cdot \frac{f_{ck}}{\gamma_c}}{\cot(\theta) + \tan(\theta)}$$

$$\alpha_{cw} = 1\ (non\ pre\text{-}stressed)$$

$$v_1 = v = 0.6\left(1 - \frac{f_{ck}}{250}\right)$$

$$\gamma_c = 1.5$$

The total shear force resistance is the minimum of these values.

$$V_{Rd} = \min\left(V_{Rd,s}, V_{Rd,max}\right) = 611.75 \text{ kN}$$

This gives the following unity check for shear resistance with reinforcement

$$u.c_{shear\ reinforcement} = \frac{V_{Ed}}{\min\ (V_{Rd,s}, V_{Rd,\max})} = \frac{V_{Ed}}{611.74}$$

Deflection

$$w_{allowed} = 0.004 \cdot 90 = 0.36 \text{ m}$$

Dimensions of the deck beams

Table 31 shows the parameters for the beam of the deck, for the preliminary design. Table 32 shows the calculate unity checks. Figure 154 displays the sections of the deck beams

Table 31: Parameters for the deck beams for preliminary calculations

Concrete	
Height	800 mm
Width	700 mm
Concrete strength class	$C50/60$
Tensile reinforcement	
Diameter	32 mm
Number of bars	10
Reinforcement steel type	B500B
Shear reinforcement	
Stirrup diameter	15 mm

Spacing stirrups	400 mm
Reinforcement steel class	B500B

Table 32: Unity checks for the beams of the deck

Compression stress	0.85
Tensile reinforcement	0.92
Shear reinforcement	0.83
Deflection	0.093

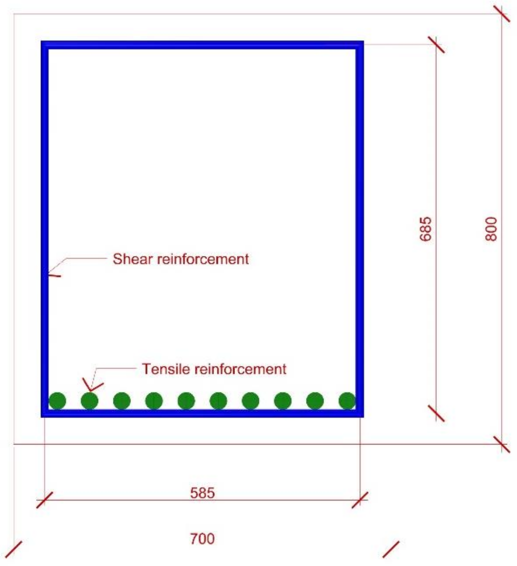


Figure 154a: Cross section of the deck beam

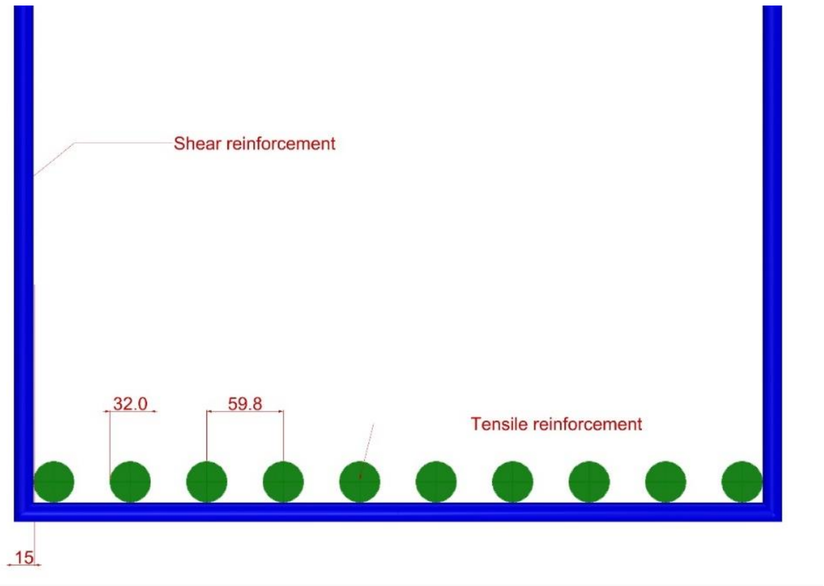


Figure 154b: Zoomed in cross section of the deck beam

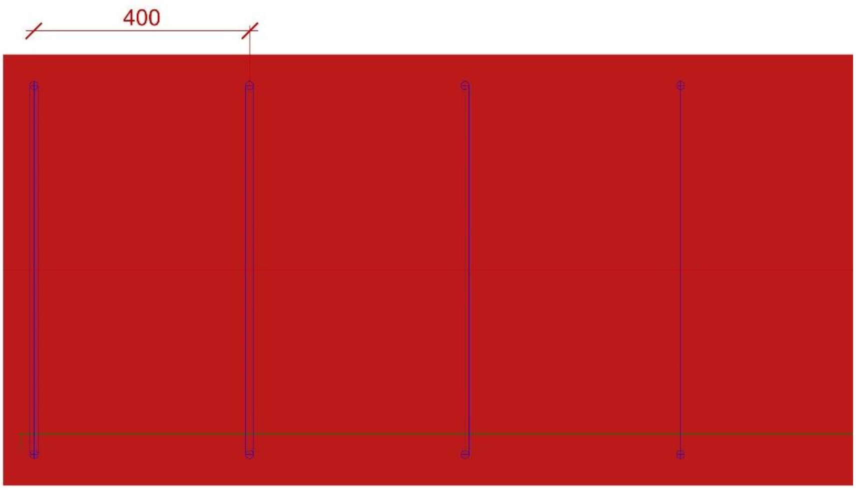


Figure 154c: Longitudinal section of the deck beam, zoomed in, displaying the spacing of the stirrups

L.3 Height of the pier Deck

The deck of the pier must be placed at a height, so that it does not get submerged during high water and that waves are not be able to hit the deck or get on top of the deck. Therefore the total height of the deck needs to be higher than the sum of the design water level and around $\frac{2}{3}$ of the wave height.

According to the fault tree from Figure 40 from Section 6.3, the reliability factor for the failure mechanism of flooding is 5.8. The water level and the wave height are the dominant loads in this scenario. This leads to the following design water level, h_d, and design water depth, d_d.

$$\alpha_s = -0.70$$

$$\beta = 5.8$$

$$h_d = NAP + 2.586 \cdot \exp(-0.138 \cdot \alpha_s \cdot \beta) + 1.178 = NAP + 5.71 \text{ m}$$

$$d_d = 9.7 \text{ m}$$

For this level, the maximum wave height can be calculated. The waves are breaking at this water level.

$$\text{H} = \min\{8.74\,; 0.8 \cdot d\} = \min\{8.74; 7.76\} = 7.76 \text{ m}$$

The level of the crest can be calculated as follows.

$$h_{crest} = h_{water\ level} + \frac{2}{3}H = NAP + 5.71 + 5.17 = NAP + 10.88 \text{ m}$$

The deck is placed at NAP + 11m. This means that the pier is situated at 15 meters above the bed level. The different levels in this calculation can be found in Figure 43.

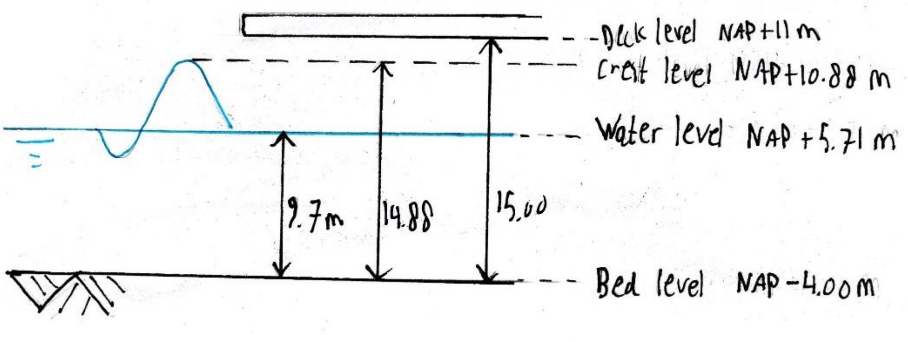


Figure 155: Water, crest - and deck level (NTS).

Appendix M Calculations of the lateral loads on the piles

This appendix shows the formulas which are used for the calculation of the lateral loads working on the pier. These are the wave load, wind load and the flow load. The loads are required to perform the structural calculations on the pier.

The forces act on the piles of the pier, and are moved to the seabed to simplify following calculations. A couple moment is added to compensate this movement. These loads can be seen in Table 33.

Table 33: Lateral forces and moments working on the pile. Moved to the sea bed. Pile diameter is 1.2m and the water level is at NAP + 5.66m.

	Force	Moment
Wave load	615 kN	6596 kNm
Wind load	3.08 kN	40.21 kNm
Flow load	11.74 kN	56.69 kNm

The wind load also exerts a force on the building on top of the pier deck. This force has been moved to the pier deck to simplify following calculations. The loads are given in Table 34.

Table 34: Lateral forces and moments working on the building. Moved to the pier deck. Building height is 20 m and the deck is at is at NAP + 11.00m.

	Force	Moment
Wind load	197.90 kN	4722.90 kNm

Relevant boundary conditions and dimensions for these calculations are mentioned in Table 35. For the water level, a reliability factor of 5.7 was used. This is the highest reliability factor for the failure mechanisms for which these loads play a role in the calculations.

Table 35: Relevant boundary conditions and dimensions for the lateral load calculations. $\beta = 5.7$.

Water level	NAP + 5.66 m
Ground level	NAP – 4.00 m
Wave height	7.73 m
Pile height (above the bed)	15 m
Pile diameter	1.2 m

M.1 Wave load

Waves can exert loads on the pile. The piles are assumed to be slender. There is a difference in breaking waves and non-breaking waves. Waves break when:

$$\frac{H}{d} \geq 0.78$$

Non-breaking waves on slender structures

Non breaking waves exert an inertia force and a drag force on the piles. The following formulas show how these forces can be calculated. These formulas have been derived from Morison’s formula. They are taken from the manual hydraulic structures (Molenaar & Voorendt, 2020).

$$F_{\max} = F_I + F_D$$

$$F_I = C_I \cdot K_I \cdot H \cdot \rho \cdot g \cdot \frac{\pi \cdot D^2}{4}$$

$$F_D = C_D \cdot K_D \cdot H^2 \cdot \frac{1}{2} \rho \cdot g \ \cdot D$$

The bending moment at the seabed, caused by these wave forces can be calculated with the following formulas. The S-factors determine where these forces occur.

$$M_{\max} = F_I \cdot d \cdot S_I + F_D \cdot d \cdot S_D$$

The factors C_D, K_I, K_D, S_D and S_I are given in the Shore Protection Manual (Coastal Engineering Research Center, 1984). The highest values of these factors were taken to be on the safe side of these calculations.

$$C_I \approx 2.0 \ (given)$$

$$C_D \approx 1.2 \ (given)$$

$$K_I = \ 0.5$$

$$k_D = 0.95$$

$$S_D = 1.1$$

$$S_I = 1.0$$

Breaking waves on slender structures

When the waves are too high, relatively to the depth, they start breaking. This happens at $\frac{H}{d} \geq 0.8$. The horizontal velocities in a breaking wave are relatively large in comparison to the acceleration, and therefore the inertia force can be neglected and the forces caused by the waves only contain a drag component. The following formula is also derived from Morison's Formula

$$F_{max} = F_{Drag} = C_D^* K_D H^2 \frac{1}{2} \ \rho \ g \ D$$

$$M_{max} = F_D \cdot d \ \ S_D$$

For breaking waves in shallow water:

$$C_D^* \approx 1.75$$

$$K_D \approx 1.0$$

$$S_D \approx 1.11 = \text{Factor for resultant drag Force}$$

$$H = H_b = \text{Wave height of breaking wave}$$

$$d = d_b = \text{Breaking wave depth}$$

$$D = \text{Diameter}$$

Breaking waves result in a larger force than non-breaking waves. This has been tested by using both formulas. For this, a water level of NAP + 5.66 m is used. This is the water level that occurs with a reliability factor of 5.7, as calculated in subsection 3.2.3. The results can be seen in Table 36.

Table 36: Wave forces for non-breaking waves and breaking waves. $D = 1.2m$ and $d = 9.66m$

	Wave height	Force	Bending moment

Non Breaking	7.72m	486 kN	5077 kNm
Breaking	$0.8 \cdot d = 7.73$m	615 kN	6596 kNm

In subsection 3.4.4, the design wave height was calculated at 8.74m. But this is higher that the breaking wave height. So the most governing wave attacks come from breaking waves and result in a force of 615 kN and a bending moment of 6596 kNm.

The force is moved to the seabed. This makes it easier to combine all the lateral forces in the end. To compensate for this movement, a couple moment of 6596 kNm needs to be added as well. Figure 156 shows how the wind load is moved to the bed. The wave load is shifted in the same way.

M.2 Wind load

The wind exerts a load on the piles and on the building on top of the piles.

The following formula is from the Technische Grondslagen voor Bouwconstructies (Nederlands Normalisatie-instituut, 1990) and has been used in the former building code NEN6702. This formula is sufficiently accurate for a conceptual design.

$$p_{rep} = C_{\dim} \cdot C_{index} \cdot C_{eq} \cdot \phi_1 \cdot p_w$$

If $h < 50$ and $\frac{h}{b} < 5$, the formula can be simplified, but for slender piles, $\frac{h}{b} < 5$ does not count. But for the preliminary design stage, these formulas are used.

$$p_{rep} = C_{\dim} \cdot C_{index} \cdot p_w$$

$$p_w = \left(1 + 7 \cdot I(z)\right) \cdot \frac{1}{2} \cdot \rho \cdot v_w^2$$

$$I(z) = \frac{k}{\ln\left(\frac{z-d}{z_0}\right)}$$

$$v_w(z) = 2.5 \cdot u^* \cdot \ln\left(\frac{z-d}{z_0}\right)$$

Several of these factors depend on the location the pier in the Netherlands and the type of area in which the pier is built. The pier is located in sector II in an open area. This leads to the following values

$$\rho = \rho_{air} = 1.25 \frac{kg}{m^3}$$

$$u^* = friction\ velocity = 2.30$$

$$z_0 = 0.2$$

$$d = reference\ height = 0$$

$$k = 1.0$$

For very narrow structures:

$$C_{\dim} \approx 1$$

The index factor is used for buildings and is not relevant for the piles of the pier, therefore:

$$C_{index} \approx 1$$

This results in:

$$p_{prep} = p_w$$

The pile, above the water level, has been divided in segments of 1 meter high. For each segment, the pressure has been calculated. This works on an area of $1m$ high and has the width of the pile diameter D. The wind is working on the pile above the water level. Similar to the wave force, the wind force is moved to the bottom of the bed. An additional couple moment needs to be added to compensate for the shift in the force. The shift in the force is schematised in Figure 156. This is done to make upcoming calculations for the pile dimensioning more simple.

$$F(z) = p_w(z) \cdot 1 \cdot D$$

$$M_{bed}(z) = F(z) \cdot (d_{water} + z)$$

The pile is around 5 meters above the water level.

$$F_{tot;bed} = \sum_{k=1}^{N=5} F(z)$$

$$M_{tot;bed} = \sum_{k=1}^{N=5} M_{bed}(z)$$

The load that is acting on the building is moved to the level of the deck. Each row of columns carries a section of 12.857 m width of the building. The building starts at a height of 5.34 above the water level in extreme situations.

For the building the following formulas are used.

$$F(z) = p_w(z + 5.34) \cdot 1 \cdot 12.857$$

$$M_{deck}(z) = F(z) \cdot z$$

The building has a height of around 16 meters

$$F_{tot;bed} = \sum_{k=1}^{N=16} F(z)$$

$$M_{tot;deck} = \sum_{k=1}^{N=16} M_{deck}(z)$$

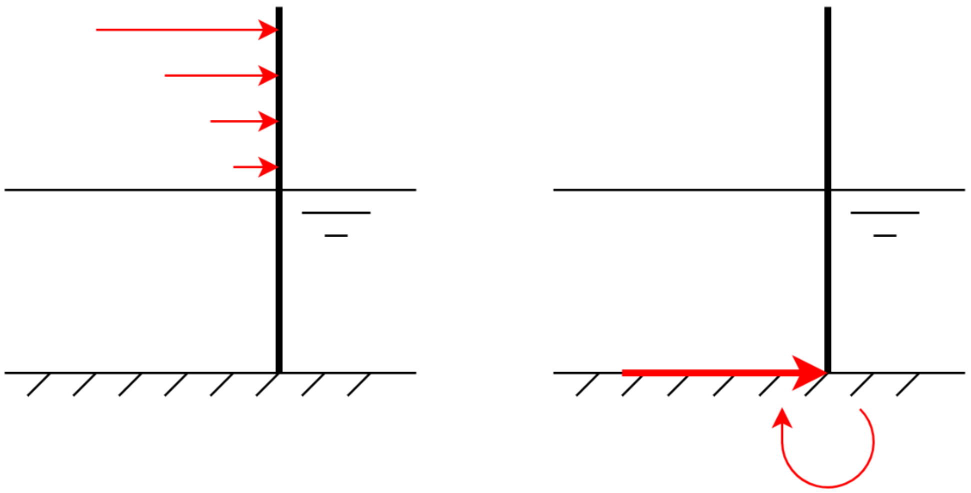


Figure 156: Shift of the wind force to the sea bed. A couple moment needs to be added

Just like the load on the piles, the wind load on the building is also moved and an additional couple is added. This is moved to the deck of the pier and not to the seabed.

	Force	Moment
Pile	3.08 kN	40.21 kNm
Building	197.90 kN	4722.90 kNm

M.3 Flow load

Rijkswaterstaat has a database which has information on the flow velocities, among others. Unfortunately, this is not available for every location. The location which is most close to the Scheveningen is at Ijmuiden.

Figure 157: Location of Ijmuiden (Rijkswaterstaat, 2020)

Over the last 28 days. The maximum total flow velocity near the coast is just below 1.5 m/s. The direction of this flow is mostly alongshore, since the direction are mostly north or south. For calculating the flow load, a velocity of 1.5 m/s is used.

If there is a straight approaching flow, there is no lift, only drag. The following formula is an empirical formula for the drag force caused by flow.

$$F_D = \frac{1}{2}\rho u^2 (C_D + C_D')A$$

$$\rho = density\ water = 1000\ \mathrm{kg/m^3}$$

$$u = flow\ velocity$$

$$C_D = static\ drag\ coefficient$$

$$C_D' = dynamic\ draf\ coefficient$$

$$A = D \cdot d$$

The static drag force mainly depends on the shape of the structure and the flow surrounding it. The latter is expressed with the Reynolds number.

$$R_e = \frac{u \cdot D}{v} \quad (see\ graph\ 12-1)$$

$$u = flow\ velocity$$

$$D = diameter\ (for\ cylinders)$$

$$v = kinmetic\ viscosity = 10^{-6}\ \mathrm{m/s}$$

For a pile diameter between 1m and 2m, the Reynolds number varies between $1.5 \cdot 10^6$ and $3.0 \cdot 10^6$. Figure 158 shows the relation between the Reynolds number and the static drag coefficient. Unfortunately the calculated Reynolds numbers are too far on the right for this graph. The maximum value for C_D is estimated at 0.6

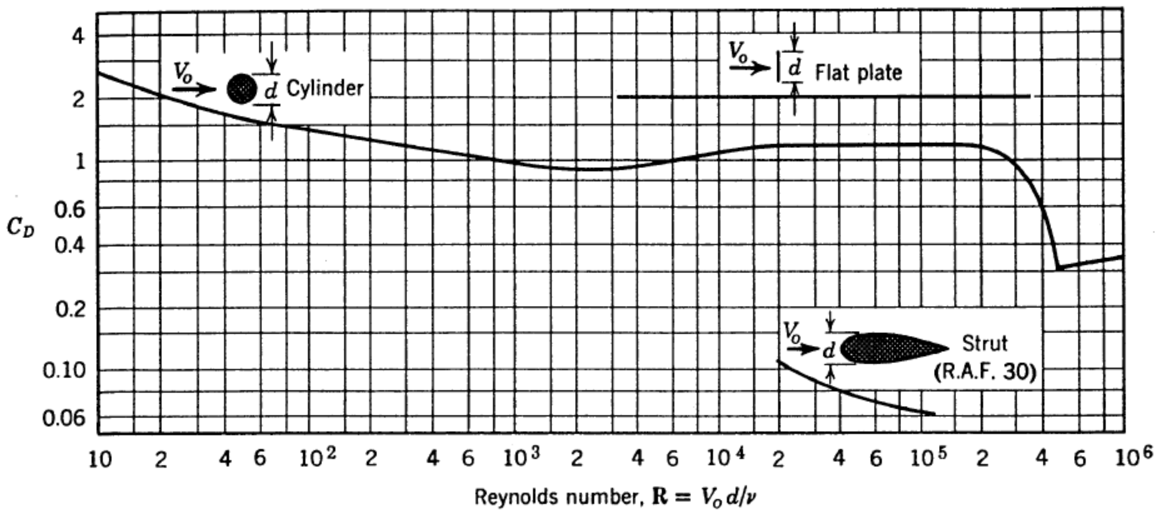


Figure 158: C_D depending on the Reynolds number (Molenaar & Voorendt, 2020)

The dynamics part of the drag force is caused by fluctuations over time. A dynamic calculation needs to be performed to calculate the eigenfrequency of the structure and to determine whether resonance could occur or not. In this preliminary design, it is assumed that resonance does not occur. This would mean that:

$$C_D' = 0.1\ to\ 0.5 \cdot C_D$$

The largest force would occur if:

$$C_D' = 0.5\ C_D = 0.3$$

In the design for the new pier, $D = 2\text{m}$, $d = 9.66$ and $u = 1.5\ \text{m/s}$. It is assumed that the velocity profile is uniform and that the location of the force is at half the water depth.

This results in:

$$F_{flow} = 11.74\ \text{kN}$$

$$M_{flow} = 56.69\ \text{kNm}$$

Appendix N Calculations for the pile dimensioning

This appendix shows the structural calculations for the piles and the dimensions of the piles. The resistances of the most governing failure mechanisms are calculated and then a unity check is performed to structurally verify the piles.

The most relevant forces are schematised below in Figure 159 to Figure 166. The soil pressures are resisting forces. The hydrostatic pressure works on both sides of the pile and is therefore not taken into account.

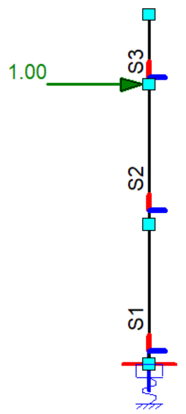


Figure 159: Wave force

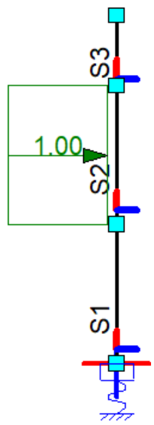


Figure 160: Drag by flow

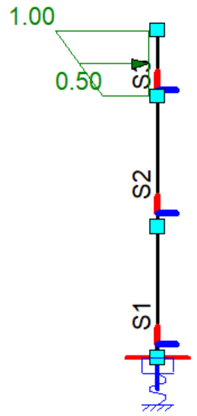


Figure 161: Wind force

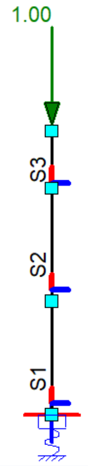


Figure 162: Load from the deck

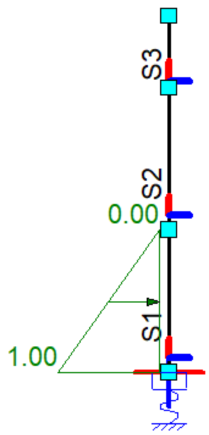


Figure 163: Soil pressure

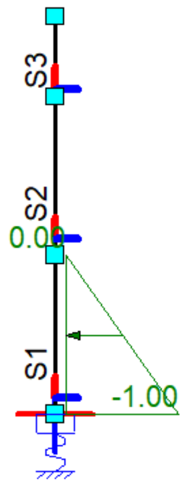


Figure 164: Soil pressure

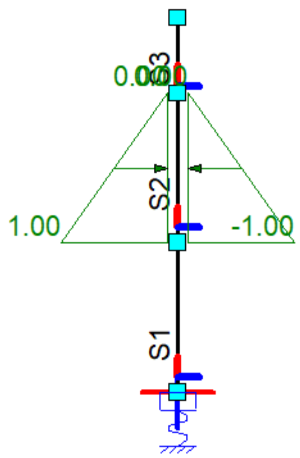


Figure 165: Hydrostatic pressure

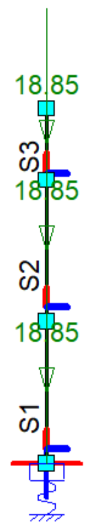


Figure 166: Self weight

N.1 Blum's method (lateral load)

Blum's method is used to calculate the resistance of the piles against lateral loading. The most critical load situation for the failure mechanisms related to lateral forces occur with high water levels and high waves. The deck has not been placed yet in this situation, so that there is no additional compression stress. This increases the total tensile stresses from the bending moments caused by the lateral loads. The loads which work on the pile in this calculation are the wind, wave and flow load. A schematisation of this critical load situation is given in Figure 167.

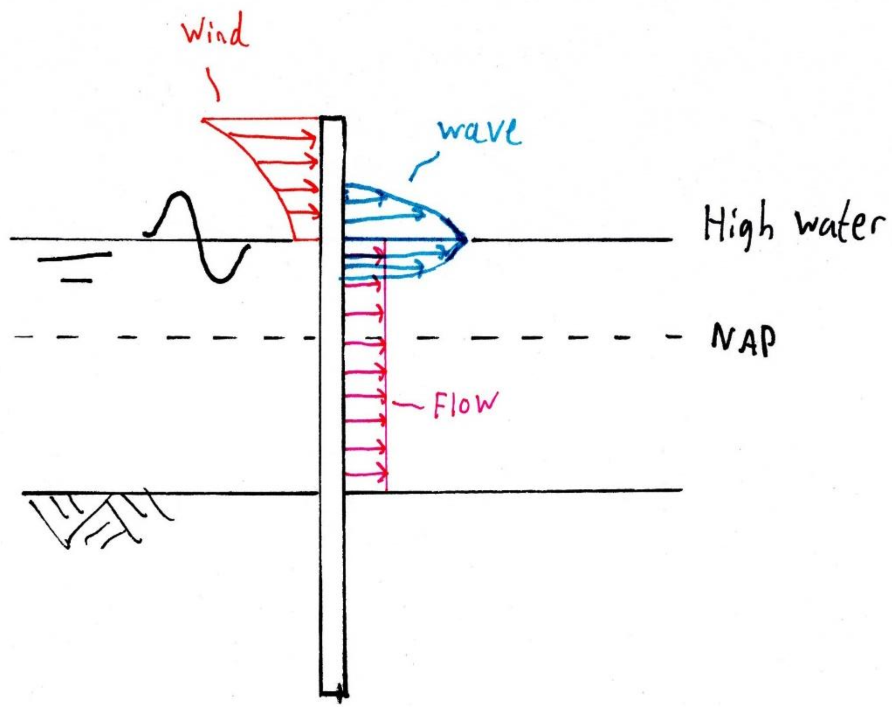


Figure 167: Schematisation of the critical load situation for the lateral loads (NTS)

In Blum's method, the soil offers the resistance against the pile moving horizontally and against rotating. This is passive soil stress, because the soil is being pushed in this case. This resistance comes from two forces, R_1 and R_2, where the first one originates from the soil wedges next to the soil behind the pile, which are pyramid shaped, and the latter one represents the resistance from the soil behind the pile. Figure 168 gives a schematisation of Blum's method.

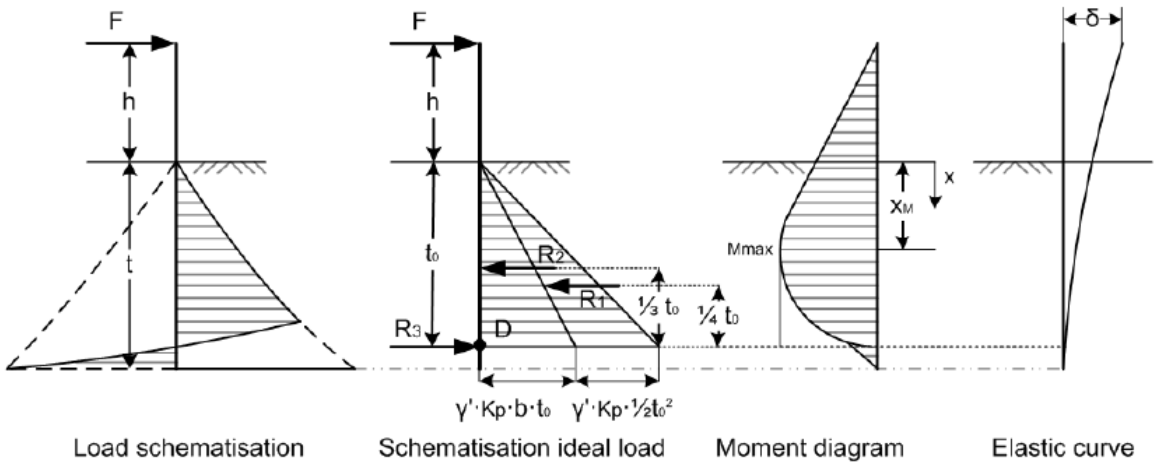


Figure 168: Blum's schematisation (Molenaar & Voorendt, 2020)

The length of the pile which is under the soil, which is called the embedded depth, can be calculated from the moment equilibrium around the theoretical bottom of the pile. In reality, the pile needs to be longer. The relation between the theoretical depth and the actual depth of the pile is given with:

$$t = 1.2 \cdot t_0$$

$$t = \text{actual embedded depth of the pile}$$

$$t_0 = \text{theoretical embedded depth of the pile}$$

The lateral forces which are coming from above the bed, mostly from the sea, were calculated in Appendix M**.** To make the calculations for the moment equilibrium more simple, all these loads have been combined into one horizontal force, located at ground level, and an additional couple which compensates for the displacement of the horizontal forces.

$$F \cdot (h + t_0) \quad => \quad F_{ground\ level} \cdot t_0 + M_{ground\ level}$$

This is entered in Blum's formula to calculate the bending moment balance around the point of a depth of t_0.

$$F_{ground\ level} \cdot t_0 + M_{bed} = R_1 \cdot \frac{1}{4} \cdot t_0 + R_2 \cdot \frac{1}{3} \cdot t_0$$

Where:

$$R_1 = \frac{1}{6} \cdot \gamma' \cdot K_p \cdot t_0^3$$

$$R_2 = \frac{1}{2} \cdot \gamma' \cdot K_p \cdot b \cdot t_0^2$$

$$K_p = \frac{1 + \sin(\phi')}{1 - \sin(\phi')}$$

$$\phi' = 30\ °$$

These formulas have been entered in python, so that it is possible perform these calculation iteratively. The output is a unity check which is gathered by dividing the driving moment with the resisting moment. The resisting moment for an embedded length of 9.7 meters and a pile diameter of 1.2 equals:

$$M_{resisting} = R_1 \cdot \frac{1}{4} \cdot t_0 + R_2 \cdot \frac{1}{3} \cdot t_0 = 3817 \text{ kNm}$$

The acting moment is gather via Matrixframe. The supports are placed at 0.65t, but for the stability the moment is required at t_0.

$$t_0 = \frac{t}{1.2}$$

This means that the support reaction needs to be moved with a distance of:

$$\frac{t}{1.2} - 0.65 \cdot t$$

The acting moment at the bottom is:

$$M_{Ed} = M_{support} + H_{support} \cdot \left(\frac{t}{1.2} - 0.65 \cdot t\right)$$

In which $M_{support}$ and $H_{support}$ are the support moment and the horizontal support reaction respectively from the Matrixframe model.

$$u.c_{resisting\ moment} = \frac{M_{Ed}}{M_{resisting}}$$

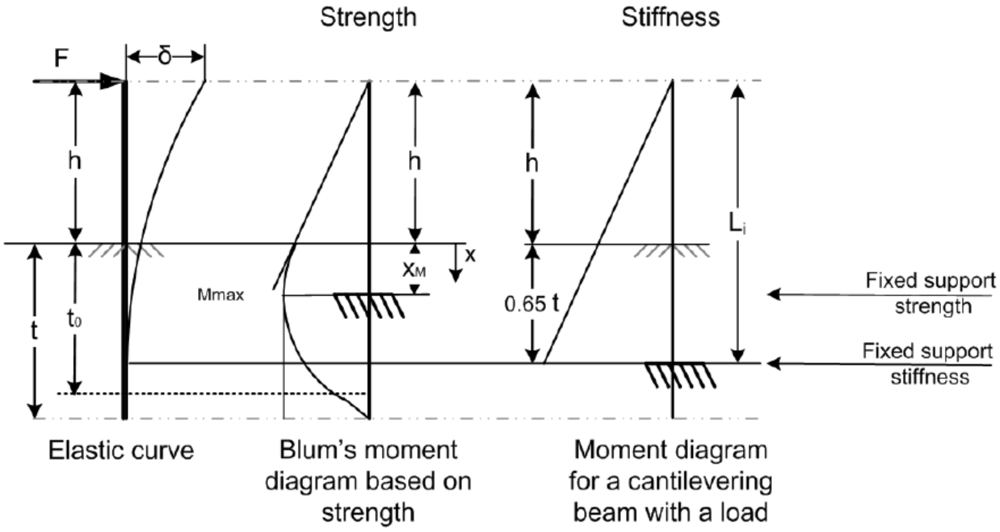


Figure 169: Schematisations for the deflection and the moment diagram (Molenaar & Voorendt, 2020).

Allowed deflection in the quick reference (Soons, van Raaij, & Wagemans, 2014)is given as $\frac{1}{500} \cdot h$. The total pile length is at approximately 25m. The maximum allowable deflection is 49.4 mm

$$u.c_{deflection} = \frac{w_{top}}{49.4}$$

N.2 Bearing capacity of the soil

The soil has a maximum bearing capacity. The axial force, which the pile exerts on the soil at that level cannot exceed this bearing capacity because this leads to structural failure. This mechanism has to be tested in the situation where the total downwards axial force is at a maximum. This is the usage situation, because this leads to high loads from the deck on the piles. This is displayed in Figure 170.

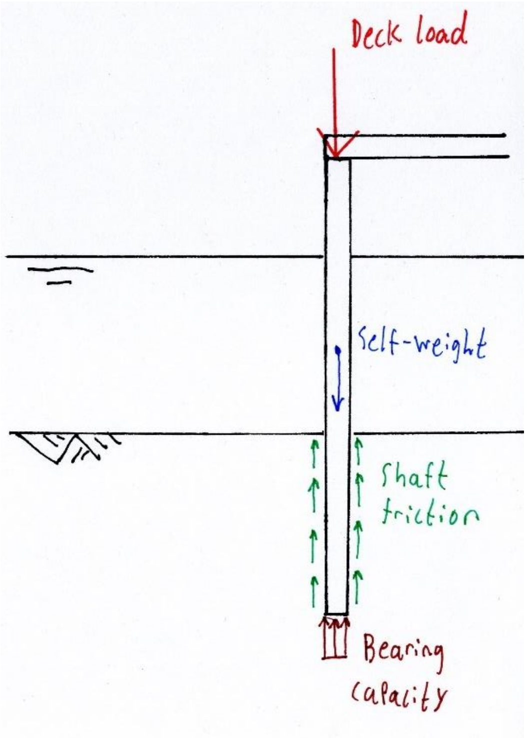


Figure 170: Critical load situation for the bearing capacity

Bearing capacity consists of the tip resistance and the shaft resistance. In the method of Koppejan, a negative shaft friction is also taken into account for the SLS check.

$$F_{r;max}\ [kN] = F_{r;max;tip} + F_{r;max;shaft} - F_{f;max;nk}$$

$$F_{r;max;tip} = A_{tip} \cdot p_{r;max;tip}$$

$$A_{tip} = area\ of\ the\ tip\ of\ the\ pile$$

$$F_{r;max;shaft} = O_{p;avg} \int_0^{\Delta L} p_{r;max;shaft}\, dz$$

$$O_{p;avg} = average\ circumreference\ of\ the\ pile$$

The unity check for the bearing resistance is given by:

$$u.c_{bearing} = \frac{F_{Ed,downwards}}{F_{r;\max}}$$

Two different methods have been performed to calculate the maximum bearing capacity. These are the method of Prandtl and the method of Koppejan. Both methods were performed to find the most governing method, which is the one with has the lowest bearing capacity as a result. In this thesis, the method of Prandtl is less complex than the method of Koppejan. For Prandtl, the soil is assumed to be uniform, while the method of Koppejan uses the results of a nearby Cone Penetration Tests (CPT). The different bearing capacities for both methods can be seen in Table 37.

Table 37: Bearing capacity of the soil at an embedded depth of 9.7m and a pile diameter of 1.2m.

Prandtl	Koppejan
4,964 kN	11,663 kN

Prandtl

The method of Prandtl uses the weight from the soil surrounding the pile, to calculate the soil resistance. This is multiplied with two factors which depend on the friction angle of the soil and the shape of the pile. The soil consists mainly out of sand with $\phi \approx 30$ °. Since the bed level is below the water level, the soil is completely saturated. The effective volumetric weight of wet sand is 10, $\gamma'_{soil} = 10\ \mathrm{kN/m^3}$. The section of the piles which are used are symmetrical, therefore $\frac{B}{L} = 1$. It is assumed that the three previous mentioned are consistent throughout the depth of the soil. Therefore, the tip resistance is only dependent of the depth of the piles, t.

The formulas from Prandtl and Brinch Hansen were used for these calculations.

$$p_{r;max;tip} = \sigma'_{v;z;o;d} \cdot N_q \cdot s_q$$

$$\sigma'_{v;z;o;d} = \text{surcharge} = \gamma'_{soil} \cdot t = 10 \cdot t$$

$$N_q = \frac{1 + \sin(\phi')}{1 - \sin(\phi')} \cdot e^{\pi \cdot \tan(\phi')} = 18.40$$

$$s_q = 1 + \frac{B}{L}\sin(\phi') = 1.5$$

The shaft bearing capacity can be calculated by determining the shear stress of the ground layer around the pile.

$$p_{r;max;tip} = \sigma'_v \cdot K \cdot \tan(\delta) + c'$$

$$\sigma'_v = \gamma'_{soil} \cdot z = 10 \cdot z$$

$$K = K_p = \frac{1 + \sin(\phi')}{1 - \sin(\phi')} = 3$$

$$\delta = \frac{2}{3} \cdot \phi' = 20\ °$$

$$c' = 0$$

It is assumed that $K_p, \delta, \gamma'_{soil}$ and c' are constant throughout the soil. The horizontal soil stress then become a triangle load, therefore the shaft force can be written as:

$$F_{r;max;shaft} = O_{p;avg} \cdot \frac{1}{2} \cdot \sigma'_{v;bottom} \cdot t \cdot K_p \cdot \tan(\delta)$$

With a pile diameter of 1.2m and a embedded depth of 9.7m, this results in an bearing resistance of 4964 kN.

Koppejan

The method of Koppejan makes use of Cone Penetration Tests (CPTs). The test which us located most close to the pier was taken near the harbour of Scheveningen. The CPT can be found in Figure 174. The location can be seen in Figure 175. To make calculations, the result of the CPT has been remodelled in Python. For every meter depth, the cone resistance was entered in an array. This led to Figure 171.

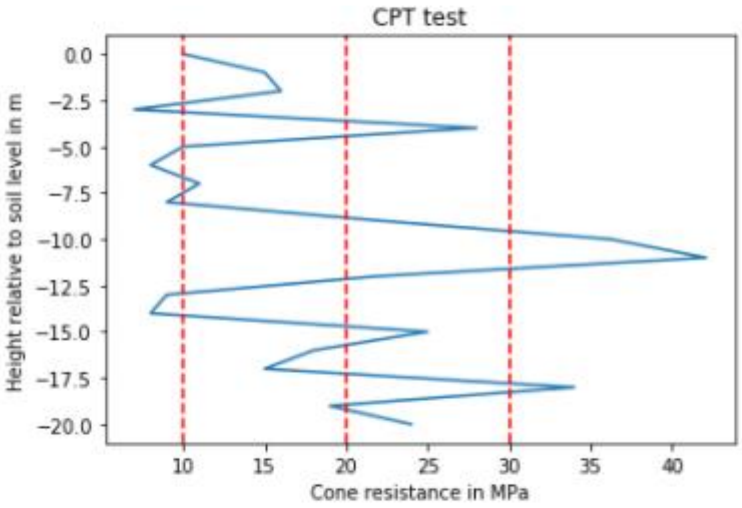


Figure 171: Remodelled CPT test, made with python.

The pile is cylindrical, therefore $D_{eq} = D$.

As can be seen in figure Figure 172, the depth of zone I and II can differ between $0.7\ D_{eq}$ to $4\ D_{eq}$ below the tip of the pile. This depth should be the one where the tip resistance is at a minimum. Python calculates the tip resistance for different lengths of zone I and II varying from $0.7\ D_{eq}$ to $4\ D_{eq}$ below the tip, with steps of 0.1m. After that, it finds for which depth the minimum value of the tip resistance occurs.

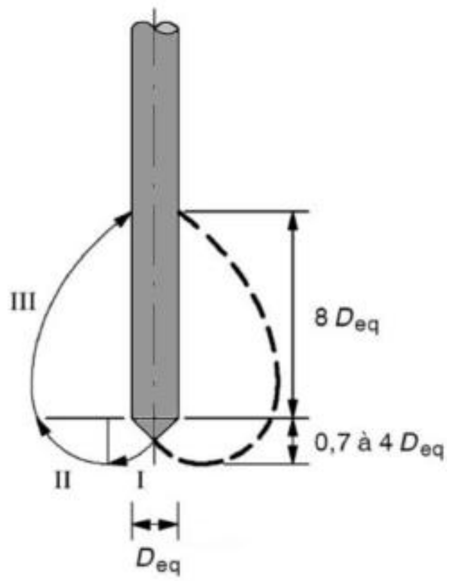


Figure 172: Schematisation of zone I, II and III, used in the Koppejan calculations (Molenaar & Voorendt, 2020)

The tip resistance, according to Koppejan, can be calculated with the following formula:

$$p_{r;max;tip} = \frac{1}{2} \cdot \alpha_p \beta s \left(\frac{q_{c;I;avg} + q_{c;II;avg}}{2} + q_{c;III;avg} \right)$$

$$\alpha_p = 0.7\ (for\ driving\ piles)\ or\ 0.63\ (for\ fundex\ piles)$$

$$\beta = 1 \text{ (assumed because no special footing)}$$

$$s = 1 \text{ (Symmetrical shape)}$$

$q_{c;I;avg}$, is the average cone resistance for zone I.

$q_{c;II;avg}$ is the average cone resistance for zone II, but the cone resistance of a layer cannot exceed the cone resistance of the layer underneath it.

$q_{c;III;avg}$ is the average cone resistance for zone III, but similar to zone II, the resistance of a layer cannot exceed the resistance of the layer underneath it.

The values which have to be used for the cone resistance in zone II and zone III are illustrated in Figure 173.

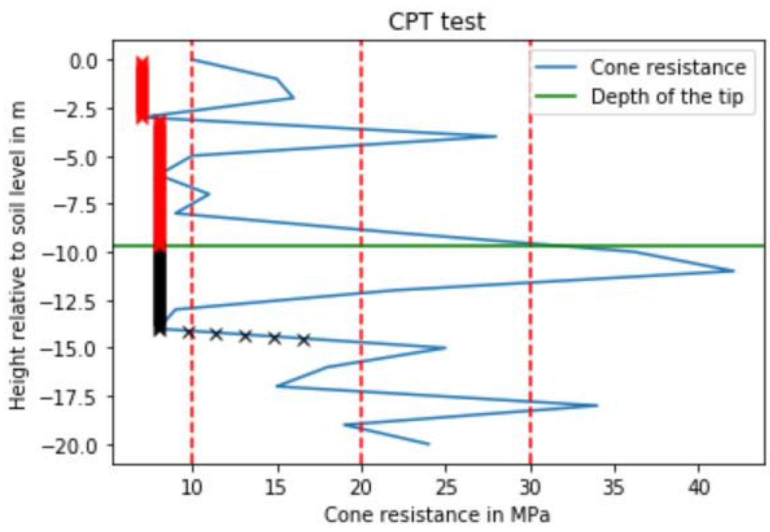


Figure 173: CPT test. The values taken for the calculations of zone II are given in black. The values taken for the calculations of zone III are given in red.

The shaft resistance, according to Koppejan, can be calculated as follows.

$$p_{r;max;shaft;z} = \alpha_s \cdot q_{c;z;a}$$

$$q_{c;z;a} = cone\ resistance \leq 15\ MPa$$

$$\alpha_s = 0.010 \text{ (for driven piles) } or\ 0.0014 \text{ (for fundex piles)}$$

With the method of Koppejan, there is also negative shaft friction, which works disadvantageous for the resistance. It should be noted that this negative shift fraction should only be considered for the serviceability limit state (SLS).

$$F_{s,nk} = O_S \cdot \sum h \cdot \sigma'_{v_{avg}} \cdot K_0 \cdot \tan(\delta)$$

If uniform soil is assumed, the formula can be rewritten to:

$$F_{s,nk} = O_s \cdot t \cdot \frac{1}{2} \cdot \sigma'_{v,tip} \cdot K_0 \cdot \tan(\delta)$$

$$K_0 = 1 - \sin(\phi')$$

The total bearing capacity according to Koppejan equals 11663 kN.

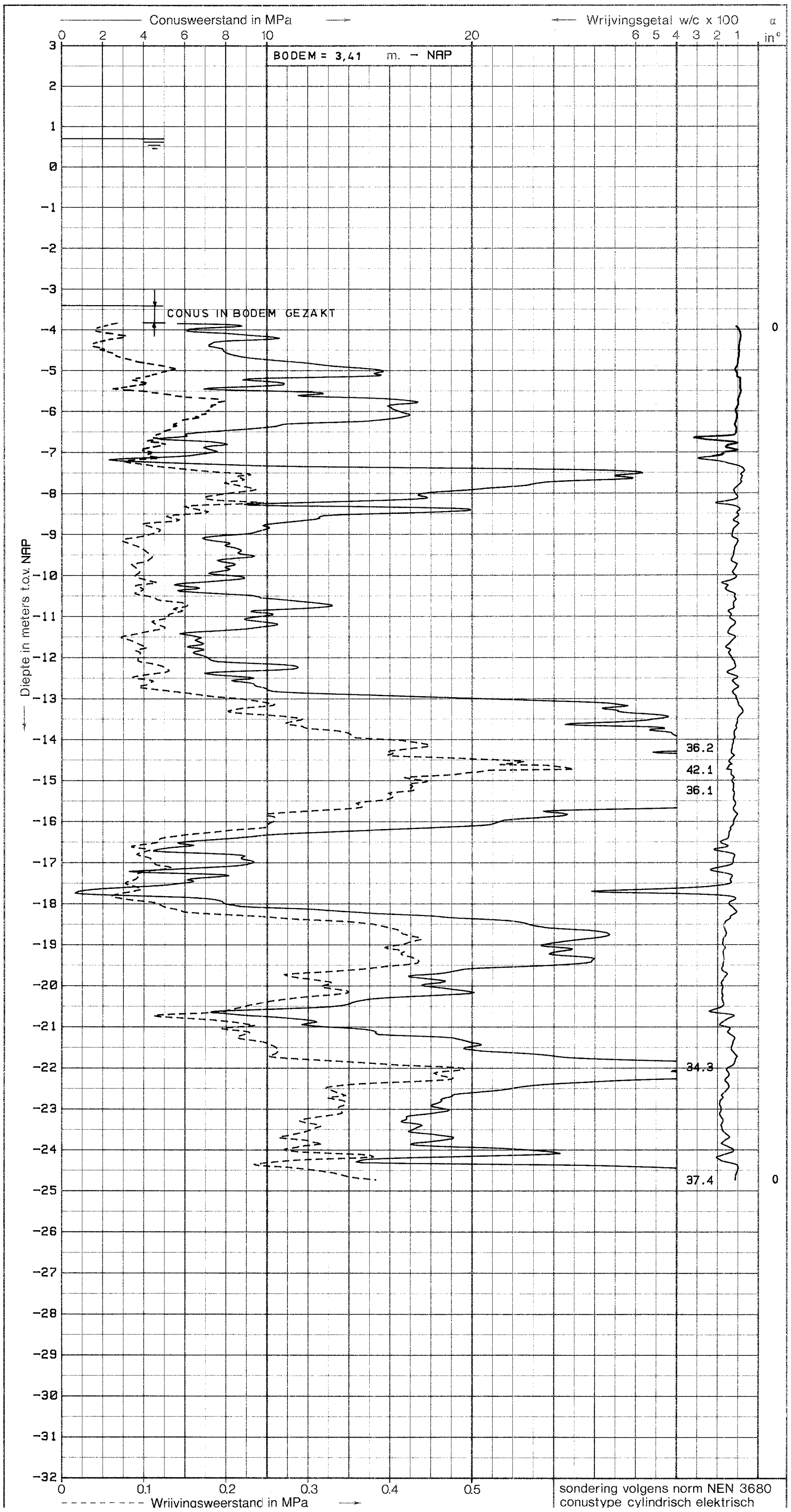


Figure 174: Cone penetration test S30D00193, (DINOloket, n.d.).

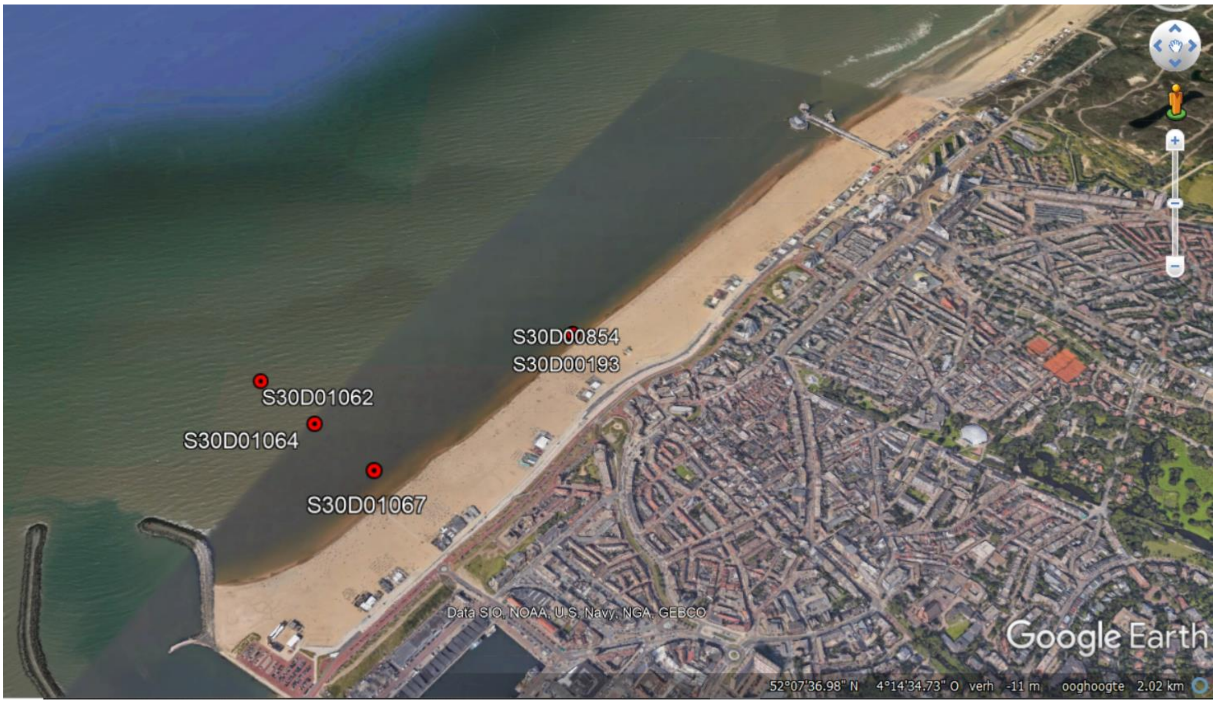


Figure 175: Location of the CPT (DINOloket, n.d.)

N.3 Compression stresses:

Compression stresses in the pile are caused by the axial force in the pile and the bending moment caused by the lateral forces.

$$\sigma_{comp} = \frac{N_{Ed}}{A} + \frac{M_{Ed} \cdot \frac{1}{2} \cdot D}{I_{zz}}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{50}{1.5} = 33 \text{ Mpa}$$

$$u.c_{compression} = \frac{\sigma_{comp}}{f_{cd}}$$

N.4 Reinforcement

As calculated before, the piles are going to be laterally loaded from loads from the sea and from the wind. This results in shear forces and bending moments inside the pile and this leads to shear, compression and tensile stresses. The concrete piles have a large resistance against compression stresses. But to be able to resist the shear and tensile stresses, reinforcement could be needed.

Figure 176 shows how a possible cross-section could look like. The diameter of the pile D, and the dimensions of the reinforcements are still to be determined. The figure shows a circular cross-section for the pile, and a square shape for the shear reinforcement with dimensions z by z. The tensile reinforcement is located on the inner side of the shear reinforcement.

Because the piles are placed in salt water, it is important that the concrete cover for the reinforcement is sufficient, or else concrete rot occurs. The minimal cover is located between the corner of the shear reinforcement square and the outer diameter. This is also displayed in Figure 176, where c is the cover. The distance for the cover can be calculated with the length shear reinforcement and the radius.

$$c = r - \sqrt{\frac{1}{4}z^2 + \frac{1}{4}z^2} = r - \frac{1}{2} \cdot \sqrt{2} \cdot z$$

The concrete cover however is a requirement, for hydraulic structures this often 50 mm. The dimensions of the stirrups are a result of the pile diameter and the concrete cover.

$$z = (r - c) \cdot \sqrt{2}$$

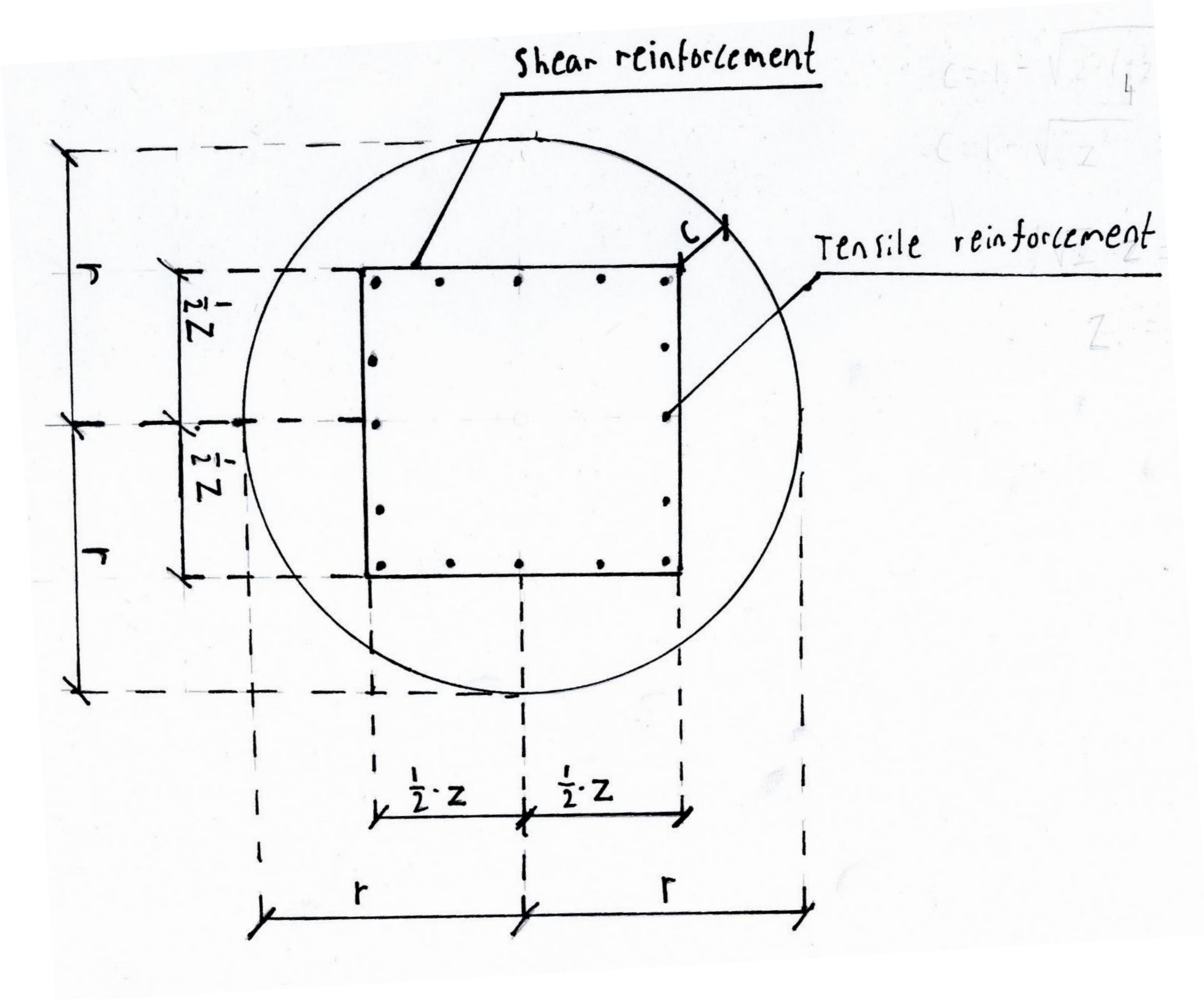


Figure 176: Cross section of a concrete pile including reinforcement.

Tensile Reinforcement

The tensile force, taken by the reinforcement should compensate for the bending moment. The arm between the tensile reinforcement and the centre of the compression stress is denoted as z_s

$$M_{Rd} = \frac{f_s}{\gamma_s} \cdot A_s \cdot z_s = 2227 \text{ kNm}$$

$$f_s = 500 \text{ Mpa}$$

$$\gamma_s = 1.3$$

$$z_s = 0.9 \cdot d$$

The unity check for the tensile reinforcement is given as follows:

$$u.c_{tensile\ reinforcement} = \frac{M_{Ed}}{M_{Rd}}$$

Shear without reinforcement

The resistance for shear-force of concrete is given in the following formula (Molenaar & Voorendt, 2020).

$$V_{Rd,c} = \frac{\left[C_{Rd,c} \cdot k \cdot \left(100 \cdot \rho_1 \cdot \frac{f_{ck}}{\gamma_c}\right)^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] A_{section}}{1000} \text{ [kN]}$$

$$C_{Rd,c} = 0.12$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad (d \text{ in mm})$$

$$\rho_1 = \frac{A_{tensile\ reinforcement}}{A_{concrete}} \leq 0.02$$

$$f_{ck} = \text{compressive cylinder strength}$$

$$\gamma_c = 1.5$$

$$k_1 = 0.15$$

$$\sigma_{cp} = \text{compressive stress} \leq 0.2 \cdot f_{cd}$$

This leads to a unity check of:

$$u.c_{shear} = \frac{V_{Ed}}{V_{Rd,c}}$$

If these requirement are not met, shear reinforcement must be added in the form of stirrups

Shear with reinforcement

When the concrete itself is not strong enough to resist the maximum shear force, reinforcement is required. Shear reinforcement consists of concrete stirrups. The resistance is calculated based on the truss model (Molenaar & Voorendt, 2020).

$$V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot \frac{f_{ywd}}{\gamma_s} \cdot \cot(\theta)$$

$$A_{sw} = \text{cross-sectional area stirrups} = \frac{1}{2} d_{stirrup}^2$$

$$s = \text{distance between stirrups}$$

$$z = \text{arm of internal leverage}$$

$$\theta = \text{angle concrete compression strutt}$$

$$21.8\,° \leq \theta \leq 45\,°$$

With a maximum resistance of:

$$V_{Rd,max} = \frac{\alpha_{cw} \cdot D \cdot z \cdot v_1 \cdot \frac{f_{ck}}{\gamma_c}}{\cot(\theta) + \tan(\theta)}$$

$$\alpha_{cw} = 1 \text{ (non pre-stressed)}$$

$$v_1 = v = 0.6\left(1 - \frac{f_{ck}}{250}\right)$$

$$\gamma_c = 1.5$$

The total shear force resistance is the minimum of these values.

$$V_{Rd} = \min(V_{Rd,s}, V_{Rd,max}) = 371.78\ kN$$

This gives the following unity check for shear resistance with reinforcement

$$u.c_{shear\ reinforcement} = \frac{V_{Ed}}{\min\ (V_{Rd,s}, V_{Rd,\max})} = \frac{V_{Ed}}{371.78}$$

N.5 Uplift

Uplift occurs when the water pressure underneath the pile exceeds the total downward force of the pile. During the usage situation of the pier, the downwards force on the pile is very large because the deck is exerting a large downward force on the piles.

The first check is calculating the weight of the pile. If this is larger than the water pressure underneath, the is no danger of uplift.

$$p_{water} = \gamma_{water} \cdot (h_{water} + t)$$

$$Q_{concrete} = \gamma_{concrete} \cdot (h + t)$$

$$\gamma_{water/concrete} = \text{specific weight}$$

$$h_{water} = \text{water depth above bed level}$$

$$h = u\text{nsupported height of the pile}$$

$$t = \text{embedded depth of the pile}$$

The piles are massive concrete cylinders and they are designed in such a way that the height exceeds the maximum water level. Since the specific weight of concrete is higher than the specific weight of water, the concrete weight always exceeds the upwards water pressure. Therefore uplift is not realistic when massive concrete piles are used.

However, the piles that are used are Fundex piles. In Section 6.2 it is explained how these piles are placed and this is displayed in Figure 37. The first phase of the placement of these piles is drilling a hollow steel tube into the soil to the required depth. This hollow tube has the most risk of uplift, since it is empty on the inside, but the upwards water pressure acts on the entire area of the pile. High water occurs during this critical situation, since this leads to the highest upwards water pressure below the pile. This load situation is displayed in Figure 177.

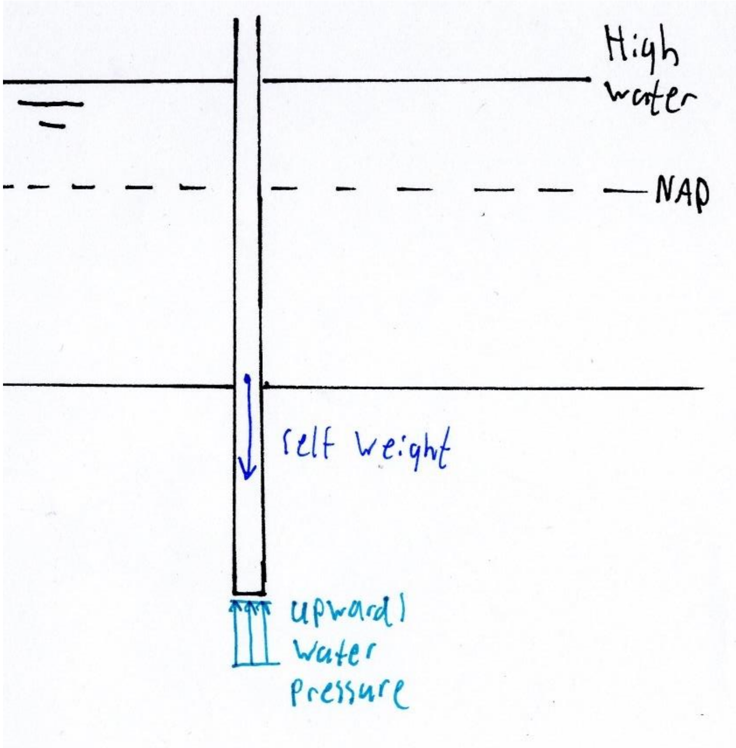


Figure 177: Schematisation of the critical load situation for uplift.

The relevant dimensions and conditions are given in Table 38.

Table 38: Relevant parameters for the uplift calculation.

Water level	NAP + 5.66 m
t_{shaft}	50 mm
$t_{embedded}$	9.7 m
D	1.2 m
γ_{steel} (Volumetric weight)	80 $\mathrm{kN/m^3}$
γ_s (Safety factor for steel resistance)	1.3

It is assumed that the hollow steel tube has a circular profile, and that it is closed at the downwards end. Both the shaft and the tip of the tube have a thickness of t_{tube}. The mass of the drill head is not taken into account.

The total gravity force from tube can be calculated as follows:

$$A_{shaft} = \frac{1}{4} \cdot \pi \left(\left(D + 2 \cdot t_{shaft} \right)^2 - D^2 \right)$$

$$V_{shaft} = A_{shaft} \cdot (h + t)$$

$$A_{tip} = \frac{1}{4} \cdot \pi \cdot D^2$$

$$V_{tip} = A_{tip} \cdot t_{shaft}$$

$$G_{tube} = (V_{shaft} + V_{tip}) \cdot \frac{\gamma_{steel}}{\gamma_s} = 268 \text{ kN}$$

$$\gamma_{steel} = 80 \text{ kN/m}^2$$

$$\gamma_s = 1.3$$

The uplift water force is calculated by multiplying the upwards water pressure with the area of the tip of the shaft.

$$F_{uplift} = p_{water} \cdot (\frac{1}{4} \cdot \pi \left(D + 2 \cdot t_{shaft}\right)^2 = 257 \text{ kN}$$

This uplift force has to be lower than the total weight of the steel tube. This leads to the following unity check.

$$u.c_{uplift} = \frac{F_{uplift}}{G_{tube}} = 0.96$$

N.6 Buckling

For the preliminary design, the buckling is tested with the buckling force of Euler. This is the resistance of the pile against buckling.

$$F_{Euler} = \frac{\pi^2 \cdot EI}{L_{cr}^2}$$

$$L_{cr} = buckling\ length$$

The beam is schematised as a vertical cantilever beam, with a length of $h + 0.65 \cdot t$, which is the same schematisation which has been used in calculating the deflection of the beam due to lateral loads with Blum's method. This is displayed in Figure 169. For a cantilever beam which is only supported on one side, the buckling length is twice the length.

$$L_{cr} = 2 \cdot L = 2 \cdot (h + 0.65 \cdot t)$$

$$h = \text{height of the pile above the bed}$$

$$t = \text{embedded depth of the pile}$$

The most governing situation for buckling is when the largest axial force possible occurs on the pile. This is during the usage situation of the pier. The deck then exerts a large load on the piles.

The unity check for buckling is given as follows:

$$u.c_{buckling} = \frac{N_{Ed}}{F_{Euler}}$$

N.7 Determining the dimensions of the piles

A python script has been made to perform the calculations of the pile iteratively. The calculations from this appendix have all been entered in a file. Eventually a function was made in Python, which takes multiple parameters as input. It then calculates the loads and unity checks for the previously mentioned failure mechanisms. These are given in Table 39 and Table 40. Figure 178 displays the sections of the piles.

Table 39: Parameters for the pile dimensioning.

Heights	
Embedded depth (t)	9.7 m
Length of the pile above bed level (h)	15 m
Water depth (d)	9.66m
Soil properties	
Effective soil volumetric weight (γ')	$10\ \mathrm{kN/m^2}$
Internal friction angle (ϕ')	30 °
Pile properties	
Diameter (D)	1.2m
Concrete strength class	C50/60
Thickness tube shaft t_{shaft}	50 mm
Reinforcement	
Concrete cover (c)	50 mm
Yield stress reinforcement (f_{ywd})	500 MPa
Diameter tensile reinforcement	50 mm
Number of bars	11
Diameter shear reinforcement	20 mm
Spacing shear reinforcement	400 mm
Theta (θ)	21.8 °
Koppejan parameters	
α_p	0.63
α_s	0.0014

Table 40: Unity check for the pile dimensioning.

Soil Resisting moment	0.55
Deflection at the top	0.64
Bearing resistance, Prandtl	0.79
Bearing resistance, Koppejan	0.34
Compressive strength	0.90
Buckling	0.19
Tensile reinforcement	0.93
Shear reinforcement	0.87
Uplift steel tube	0.96

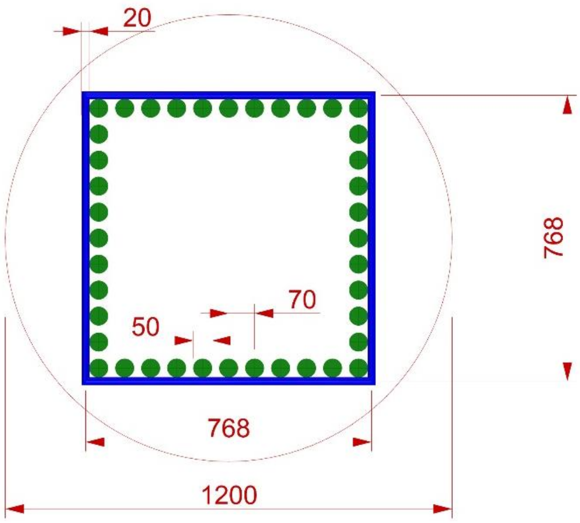


Figure 178a: Cross-section of the pile with reinforcement. Units in mm.

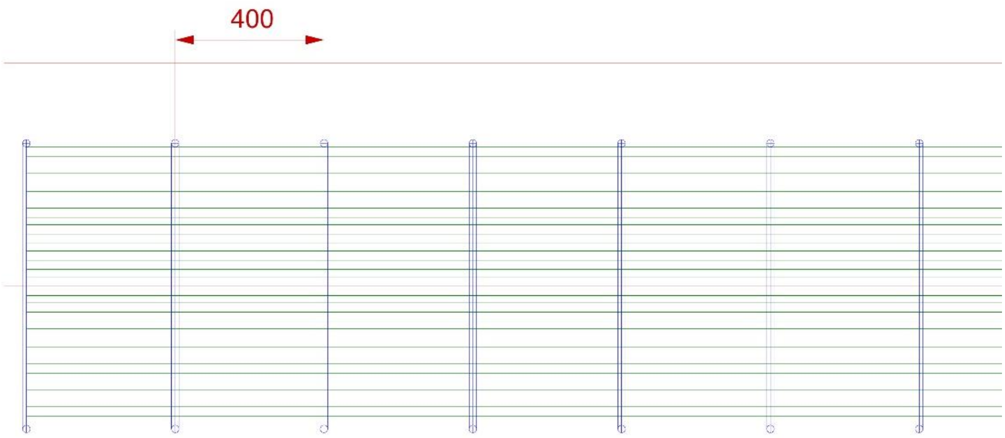


Figure 178b: Part of a longitudinal section of the pile, displaying the stirrup spacing. Units in mm.

Appendix O Calculation of the diagonal struts

Diagonal struts have been added to the structure of the pier. These were added to get a higher resistance from the lateral loads. This appendix will determine which orientation of the diagonal struts is most effective in reducing the effects of the lateral loads. These struts were added to the structural design of Chapter 6.

These go from the top of the column to the bed level of an adjacent column. Matrix frame was used to determine which orientation of the struts would be more effective. A simplified model was made in Matrixframe, this can be seen in Figure 179.

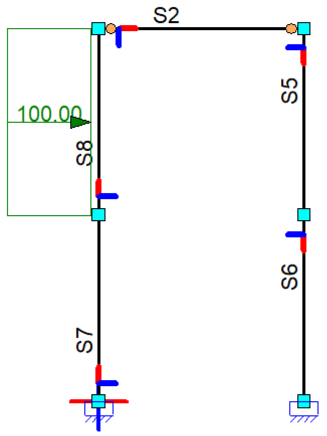


Figure 179a: No struts

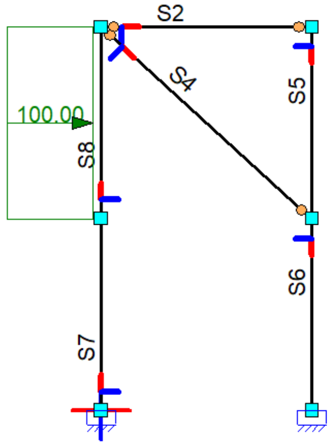


Figure 179b: Struts from the top left to the bottom right

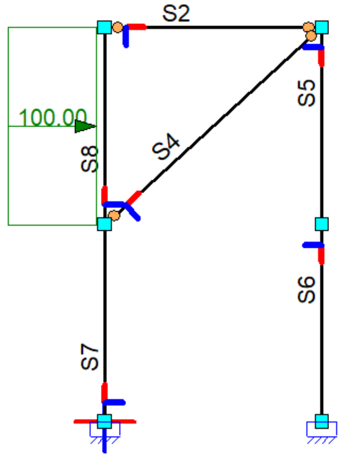


Figure 179c: Struts from the bottom left to the top right

Figure 179: Schematisation of two columns, a beam and struts. The left side is the offshore side, where the most lateral loads are coming from. S6 and S7 are below ground level.

The main goal of the struts is to reduce the internal forces and the deflection of the column, because the columns were dimensioned too large without the struts. A comparison of the deflection and the internal forces can be seen in Table 35.

Table 41: Results of the Matrixframe analysis for different orientations of the struts. Only the internal forces and deflections of the columns are analysed.

	Deflection (m)	***M*** **(kNm)**	***V*** **(kN)**	***N*** **(kN)**
No struts	0.4521	8595	678	0
Top left to bottom right	0.0682	3922	594	+723
Top right to bottom left	0.1092	6323	930	+662

From the results of Table 35 it can be concluded that struts from the top left to the bottom right are the most effective in reducing the deflection and the internal forces in the column. The only disadvantageous about these struts is that an additional axial tensile force occurs in the column. However, the reduction in the deflection and the bending moment still makes the struts a good addition. The struts themselves are mainly loaded in compression because they are hinged on both sides. Only self-weight of the struts causes a bending moment and a shear force. The profile of the struts are HP220x57. The maximum stress that occurs in the struts according to the Matrixframe model is 66.67 MPa. So it would be sufficient enough to make these struts out of S235 steel.

Appendix P Evaluation of different construction methods for the foundation

In chapter 6, the construction method of the structure of the pier is explained. The decision for this method has been made after comparing different construction methods with each other. This appendix shows the different construction methods and their advantages and disadvantages.

Dry work place:

Advantages

- Suitable for a shallow foundation or a pile foundation.
- The pier can be built in a dry situation.

Disadvantages

- The sheet pile structure is difficult to construct in the water due to the waves, a jack-up barge would be needed for this.
- The wave climate results in heavy loads on the sheet pile walls.
- The bottom of the workplace needs to be impermeable and resistant to prevent up burst.
- A large amount of water needs to be pumped out.

Submerging elements at the location

Advantages

- The elements are easy to transport.
- Elements are easy to construct in a dry situation.

Disadvantages

- The waves makes it difficult to put the elements at the correct location.

Using a jack-up barge with spud piles

Advantages

- The jack-up barge can stand steadily on the soil, therefore the platform does not move due to the waves.
- The jack-up barge can elevate itself among the spud piles and can therefore be used for the foundation and the pier deck.
- The jack-up barge can move to different locations.

Disadvantages

- The jack-up barge has limited space on the platform.

Appendix Q Sea level rise due to climate change

It is expected that the sea level will rise due to climate change. Because the new Scheveningen Pier is located at sea, it is important to take climate change into account when determining the hydraulic boundary conditions. This appendix will calculate the sea level rise for the design life time of the new Pier. The result is used in Section 3.2 to determine the design water level.

The KNMI (Koninklijk Nederlands Meteorologisch instituut) describes four different climate scenarios. These scenarios are a combination of a temperature change, which can be moderate or warm, and a change in the air flow patterns, which can be low or high. This is illustrated in Figure 180. The scenario with a warm temperatures and a high change of the air flow pattern (W_H in Figure 180) must be assumed when designing new structures according to the WOWK (Rijkswaterstaat, 2018).

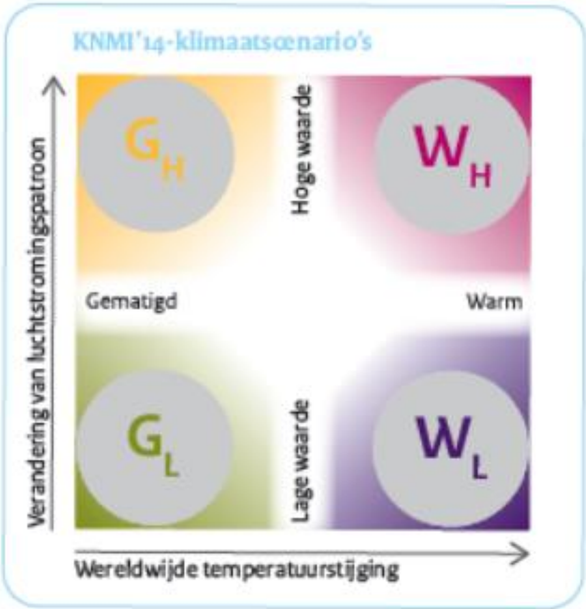


Figure 180: Description of the four different climate scenarios (KNMI, 2014)

The design life time of the new Scheveningen Pier is 100 years, so that would be until 2121. With the data of Table 42, an assumption can be made for the water level change in this year. It is assumed that in 2085 the water level has risen with 80 cm and that it is still growing with 10.5 mm/year.

$$\Delta h_{2121} = 80 + (2121 - 2085) \cdot 1.05 = 117.8 \text{ cm}$$

So this means that the average water level will be 117.8 cm above NAP according to this assumption in 2121.

Table 42: Water level change in the climate around 2085 (2071 – 2100) for the north sea coast (KNMI, 2014).

Indicator	Climate 1951-1980	Climate 1981-2010	G_L	G_H	W_L	W_H
Absolute water level	4 cm below NAP	3 cm above NAP	+25 tot +60 cm	+25 tot +60 cm	+45 tot +80 cm	+45 tot +80 cm
Rate of change	1,2 mm/year	2,0 mm/year	+1 tot +7,5 mm/year	+1 tot +7,5 mm/year	+4 tot +10,5 mm/year	+4 tot +10,5 mm/year

Appendix R Derivation of the safety factors and design values

This appendix gives the derivation of the safety factors and design values which are used for the semi-probabilistic calculations. The design values are used to for the boundary conditions in Section 3.2. The safety factors are used to determine the governing situations in the structural calculations for the construction in Section 6.4.

Partial factor for permanent load

It is assumed that the permanent load is normally distributed and the characteristic value is the mean of this distribution. The coefficient of variance is 0.10 (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017)

$$G_k = \mu_G$$

$$V_G = 0.10$$

$$g^* = \mu_G \cdot (-\alpha_s \cdot \beta \cdot V_G)$$

$$\gamma_g = \frac{\mu_G \cdot (1 - \alpha_s \cdot \beta \cdot V_G)}{\mu_G} = 1 - 0.10 \cdot \alpha_s \cdot \beta$$

Partial factor for variable loads (wind, floor and roof loads)

For variable loads it is assumed that they are Gumbel distributed and that the coefficient of variance is 0.08 (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017).

$$q^* = \mu + \frac{-0.5772 - \ln[-\ln \Phi(-\alpha_s \cdot \beta)]}{1.282} \cdot \sigma$$

$$Q_k = \mu - 0.4584 \cdot \sigma$$

$$V = 0.08$$

$$\gamma_q = \frac{1 + \frac{-0.5772 - \ln[-\ln \Phi(-\alpha_s \cdot \beta)]}{1.282} \cdot V}{1 - 0.4584 \cdot V} = \frac{1 + \frac{-0.5772 - \ln[-\ln \Phi(-\alpha_s \cdot \beta)]}{16.025}}{0.9633}$$

For variable loads, an additional factor, γ_{sd} is applied to account for model uncertainties. (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017).

$$\gamma_Q = 1.1 \cdot \gamma_q$$

Partial factors for concrete

The strength of concrete is assumed to be normally distributed. In the strength calculations, concrete is the dominant strength parameter. Similar to the variable loads, concrete also takes modelling errors into account with factor γ_{sd} which is equal to 1.1 (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017).

$$R_k = \mu_c \cdot (1 - 1.645 \cdot V_c)$$

$$r^* = \mu_c \cdot (1 - \alpha_r \cdot \beta \cdot V_c)$$

For C50/60, the following values are given in the Quick Reference (Soons, van Raaij, & Wagemans, 2014).

$$R_k = 50\ MPa, \qquad \mu = 58\ MPa$$

These values van be used to determine the coefficient of covariance of concrete.

$$50 = 58 - 95.41 \cdot V_c$$

$$V_c \approx 0.08$$

For concrete, an additional factor, γ_{sd}, for model uncertainties is added.

$$\gamma_{sd} = 1.1$$

$$\gamma_c = \gamma_{sd} \frac{R_k}{r^*} = \frac{1 - 1.645 \cdot V_c}{1 - \alpha_r \cdot \beta \cdot V_c} = 1.1 \cdot \frac{0.8684}{1 - 0.08 \cdot \alpha_r \cdot \beta}$$

Partial factors reinforcement steel

Similar to concrete, the reinforcement steel is assumed to be normally distributed. In the failure mechanisms where the shear stress or tensile stresses are exceeded, the steel strength is the dominant strength parameter. The safety factor for steel also takes modelling errors into account with the factor γ_{sd}.

$$R_k = \mu_s \cdot (1 - 1.645 \cdot V_s)$$

$$r^* = \mu_s \cdot (1 - \alpha_r \cdot \beta \cdot V_s)$$

$$\alpha_r = 0.80$$

$$V_s \approx 0.05$$

For steel, an additional factor, γ_{sd}, for model uncertainties is added.

$$\gamma_{sd} = 1.1$$

$$\gamma_s = \gamma_{sd} \frac{R_k}{r^*} = \frac{1 - 1.645 \cdot V_s}{1 - \alpha_r \cdot \beta \cdot V_s} = 1.1 \cdot \frac{0.91775}{1 - 0.05 \cdot \alpha_r \cdot \beta}$$

Design water level

To determine the design water level, partial factor for the water level and the characteristic value for the water level, the distribution of the water level needs to be known. It is assumed that the water level at Scheveningen has a lognormal distribution. The key figures (Rijkswaterstaat, 2013) for the water level show a similar distribution. The key figures are found in Table 43 and a logarithmic fit can be seen in Figure 181.

Table 43: Water levels and their exceedance frequency and the value of the CDF function. Water level and exceedance frequency are from the key figures (Rijkswaterstaat, 2013).

Water level	**Exceedance frequency**	**F(x)**
NAP + 2.85 m	1 / 5 years	0.8
NAP + 3.05 m	1 / 10 years	0.9
NAP + 3.25 m	1 / 20 years	0.95
NAP + 3.50 m	1 / 50 years	0.98
NAP + 3.70 m	1 / 100 years	0.99

NAP + 3.90 m	1 / 200 years	0.995
NAP + 4.20 m	1 / 500 years	0.998
NAP + 4.40 m	1 / 1,000 years	0.999
NAP + 4.60 m	1 / 2,000 years	0.9995
NAP + 4.80 m	1 / 4,000 years	0.99975
NAP + 4.90 m	1 / 5,000 years	0.9998
NAP + 5.20 m	1 / 10,000 years	0.9999

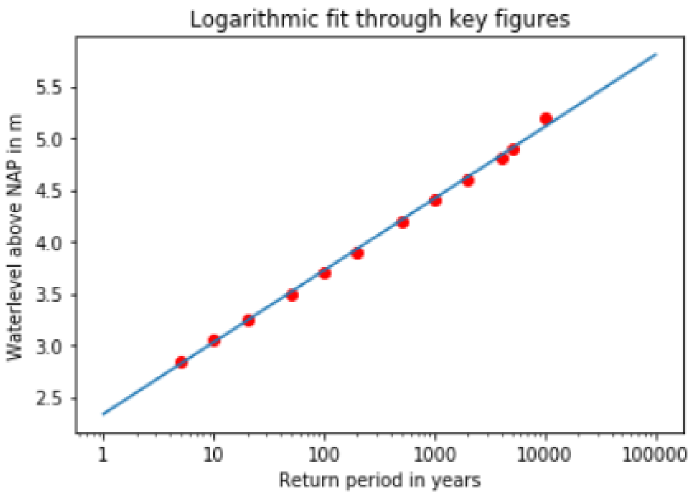


Figure 181: Logarithmic fit through data for the water level. Data from the key figures (Rijkswaterstaat, 2013)

The key figures (Rijkswaterstaat, 2013) are used to determine the distribution. Exceedance probability and the water levels are given. The exceedance frequency is equal to the exceedance probability, and the value of the cumulative distribution function can then be determined as follows.

$$F(x) = 1 - p_{exceedance}$$

For a water level of NAP + 2.85m this results in:

$$F(2.85) = 1 - \frac{1}{5} = 0.8$$

The values of the water levels with their exceedance frequency and CDF are found in Table 43. With python a lognormal distribution has been fitted through the data points. The fit can be seen in Figure 182 and Figure 183. The distribution has the following values.

$$\mu = NAP + 2.586 \text{ m}$$

$$\sigma = 0.357 \text{ m}$$

$$V = \frac{\sigma}{\mu} = 0.138$$

For the safety factor of the water level, an additional factor, γ_{sd}, for model uncertainties is added.

$$\gamma_{sd} = 1.1$$

The water level is a dominant load in governing load situations. For a governing load, $\alpha = -0.70$.

The characteristic value and the design point of a load with a lognormal distribution can be determined as follows (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017).

$$E_k\,[\mathrm{m}] = \mu \cdot \exp(1.645 \cdot V) = NAP + 3.244$$

$$e^*[\mathrm{m}] = \mu \cdot \exp(-\alpha_s \cdot \beta \cdot V) = NAP + 2.586 \cdot \exp(-0.138 \cdot \alpha_s \cdot \beta)$$

$$\gamma_{level} = \gamma_{sd}\,\frac{e^*}{E_k} = 1.1 \cdot \frac{2.586 \cdot \exp\left(-0.138 \cdot \alpha_s \cdot \beta\right)}{3.244}$$

Figure 184 shows the dependancy of the design water level, dependant of the reliability index β.

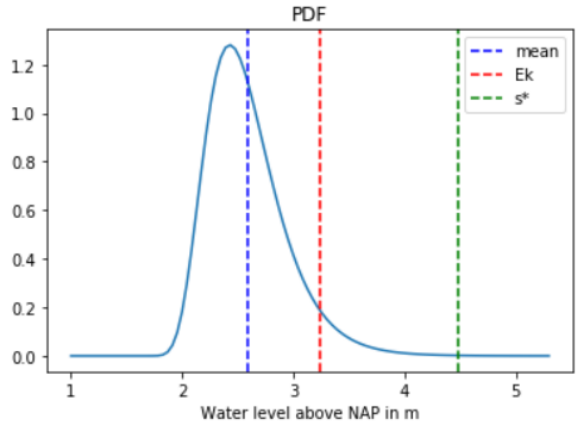


Figure 182: Fitted probability density function for the water levels at Scheveningen.$\alpha_s = -0.70$, $\beta = 5.7$

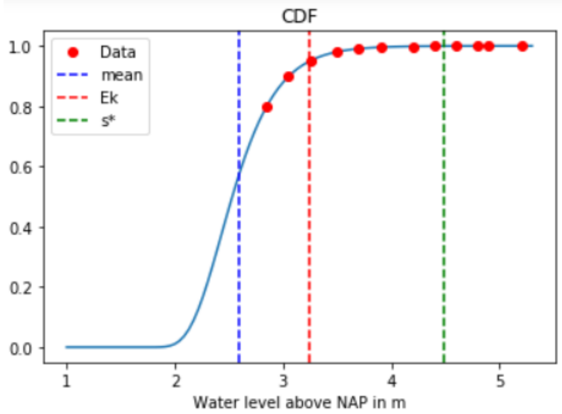


Figure 183a : Cumulative density function for the water level at Scheveningen.α_s *= -0.70,* β*= is 5.7*

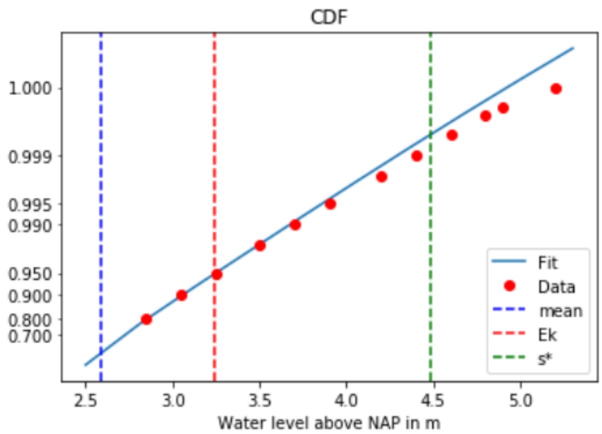


Figure 183b: Cumulative density function for the water level at Scheveningen.α_s *= -0.70,* β*= is 5.7, zoomed in at the end on logarithmic scale.*

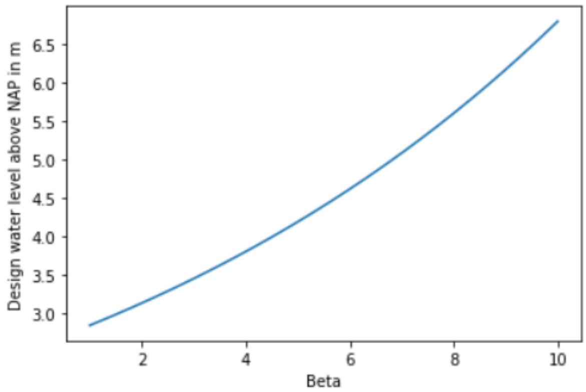


Figure 184: Design water level dependent of β for $\alpha_s = 0.70$

From the scenarios for which the water level is used in the calculations, the highest $\beta = 5.7$ and for a dominant load $\alpha_s = -0.70$. This results in a water level of NAP + 4.48m. An additional 1.178 m is added to this water level to account for climate change. A substantiation for this number to account for climate change is given in Appendix Q. This results in a water level of NAP + 5.66 m.

Only for the failure mechanisms where the deck is hit by the water, which are overtopping and waves hitting the deck from below, the reliability factor is higher, but no calculations are performed for this failure mechanism, this failure mechanism only determines the height of the pier deck.

Design wave height

To determine the significant wave height, a time series has been analysed with python. This data was gathered via Rijkswaterstaat (Rijkswaterstaat, 2020).The data has been taken at the Europlatform, of which the location can be found in Figure 185.

Figure 185: Location of Europlatform and Eurogeul E13 (Rijkswaterstaat, 2020)

The timespan is from 01-01-2000 to 31-12-2018. To reduce the computing time, the daily maxima have been used for this analysis instead of every measurement. The daily maxima have been plotted in Figure 186.

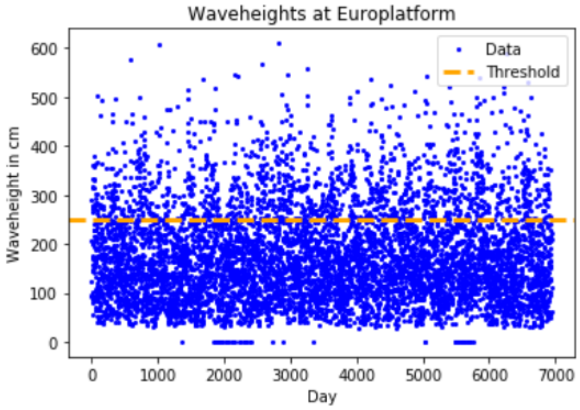


Figure 186: Daily maxima of the wave heights H_{m0} at Europlatform. Data from (Rijkswaterstaat, 2020).

Only the storm waves are taken into account. For this, a threshold has been set-up at a wave height 250 cm. This means that waves below this threshold are be taken into account. Figure 187 shows which waves are exceeding this threshold.

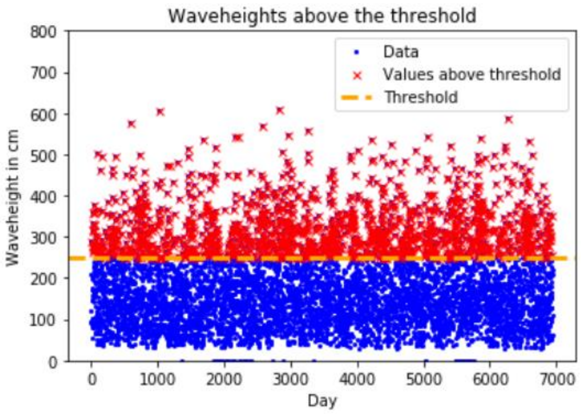


Figure 187: Wave heights with values indicated which are above the threshold.

The significant wave height is average of the highest third of the storm waves. This results in:

$$H_s = 3.97 \text{ m}$$

The design wave height is the wave height with a 10% exceedance probability (Rijkswaterstaat, 2018). For this, a Rayleigh distribution is assumed for the significant wave height. In Dutch conditions the following can be assumed.

$$H_d = 2.2 \cdot H_s = 8.74 \text{ m}$$

Figure 188 shows how the significant wave height and the design wave height relate to the data.

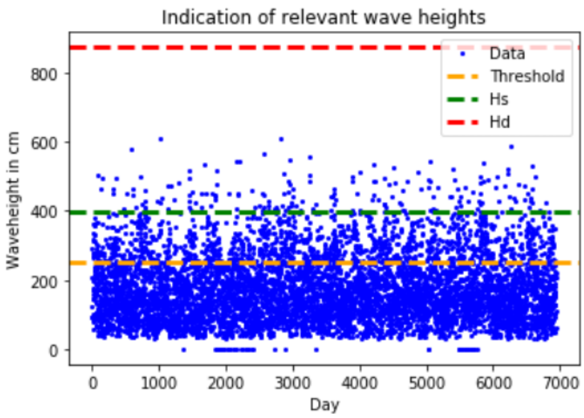


Figure 188: Indication of the relevant wave heights.

The design wave height is equal to 8.74 m, however it is possible that the wave height becomes smaller due to breaking. The wave height is then dependent of the breaker index,γ, and the water depth, d.

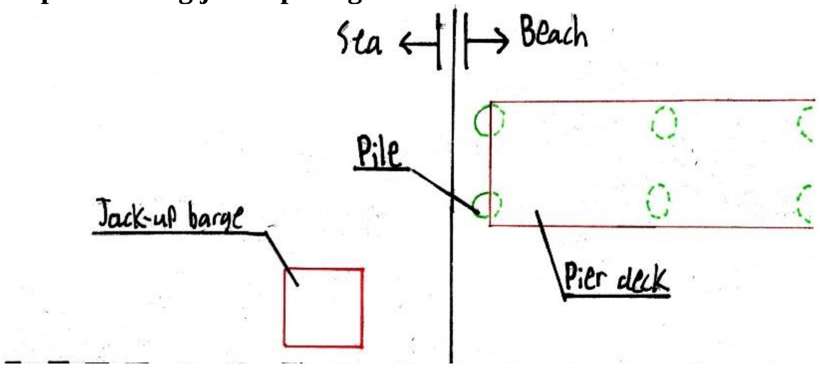
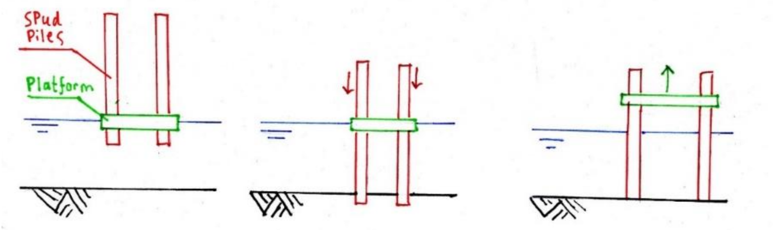
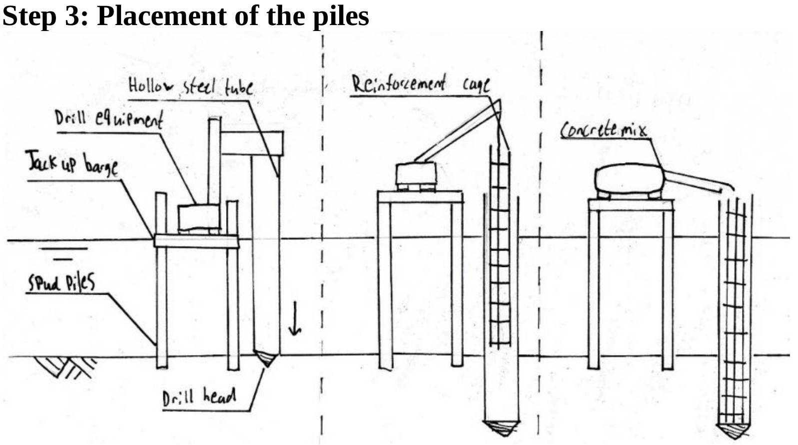
$$\gamma = 0.8$$

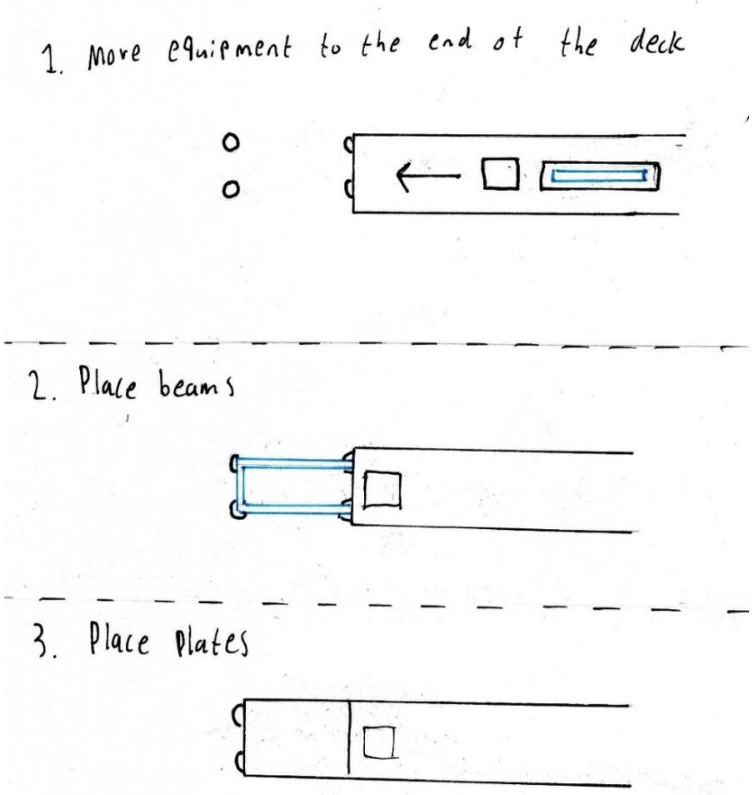
$$H_b = \gamma \cdot d = 0.8 \cdot d$$

$$H_d = \min \{8.74\, ; 0.8 \cdot d\}$$

Appendix S Construction sequence

This appendix shows a compact construction sequence. This construction has been explained in Section 6.2.

Step 1: Moving jack-up barge to location  *Figure 189:Top view, step 1*	The section of the pier which is located on the beach is be constructed first. For the part which is located at sea, a jack-up barge with spud piles is be used. This is a floating platform which can be placed on the seabed. This platform needs to move to the location first.
Step 2: Installing je jack-up barge  *Figure 190: Side view, step 2*	The spud piles at the corners of the platform of the jack-up barge moves downwards into the seabed, so that the platform stays stable and does not drift away. The height of the platform can be adjusted if necessary.
Step 3: Placement of the piles  *Figure 191: Side view, step 3*	When the jack-up barge is into location, the equipment which is on top of it is used to place the piles. These are Fundex piles. A hollow steel tube is drilled in the bed first to the required embedded depth. The inside of this tube is dry. After that, the reinforcement cage is placed and then the concrete mix is being poured in.
Step 4: Placements of the struts	When two adjacent piles are placed, the struts in between can be piles can be placed. Since the piles are placed in situ, the connections have to be made in on site as well. Equipment to get the struts onto locations can be placed on the jack-up barge

	as well. The connections would have to be placed with diver personnel equipped with underwater welding or drilling equipment.
Step 5: Place the deck beams and deck plates  *Figure 192: Top view step 5*	The already built pier deck and the piles placed in step 3 need to connected. This is done from the current platform itself. The beams and the plates are brought to the location with trucks, over the already existing pier. The beams are placed between the current piles, and after that, the slabs can be placed on these beams.
Step 6: Repeat step 1-5	Step 1-5 needs to be repeated until all the piles and the entire pier deck is built.
Step 7: Construct the super structure	After the deck is constructed, the buildings and the infrastructure can be constructed on top of the pier.

Appendix T Soil analysis

This appendix shows the analysis of a cone penetration test which has been taken on the beach of Scheveningen. The results are used to determine the soil type of the project area in Section 3.2.

Figure 194 shows the results of a CPT at the beach of Scheveningen. No CPTs were taken close to the Scheveningen Pier. The CPT from Figure 194 was taken between the harbour and the Pier. The exact location can be found in Figure 193.

The type of soil can be determined with the friction number of the CPT (Wrijvingsgetal in Figure 194). This soil classification is given in Table 44. The CPT in Figure 194 mostly shows friction numbers between 1 and 2. This means that the soil mainly consists out of sand. At a depth of NAP – 7m, a clay layer can be observed and at NAP – 18m a peat layer is observed. However, these are relatively thin layers. For further calculations, the soil is assumed to consist of sand only.

Table 44: Soil classification, dependent of the friction number (Van Tol, 2006).

Soil type	**Friction number in %**
Sand	1
Sand/clay mixtures	2
Clay	3-5
Peat	8-10

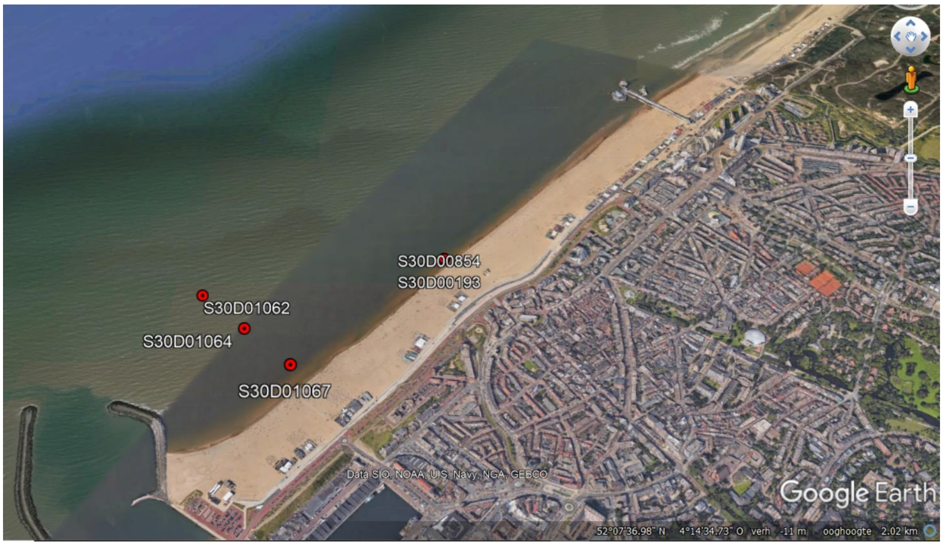


Figure 193: Location of the CPT (DINOloket, n.d.)

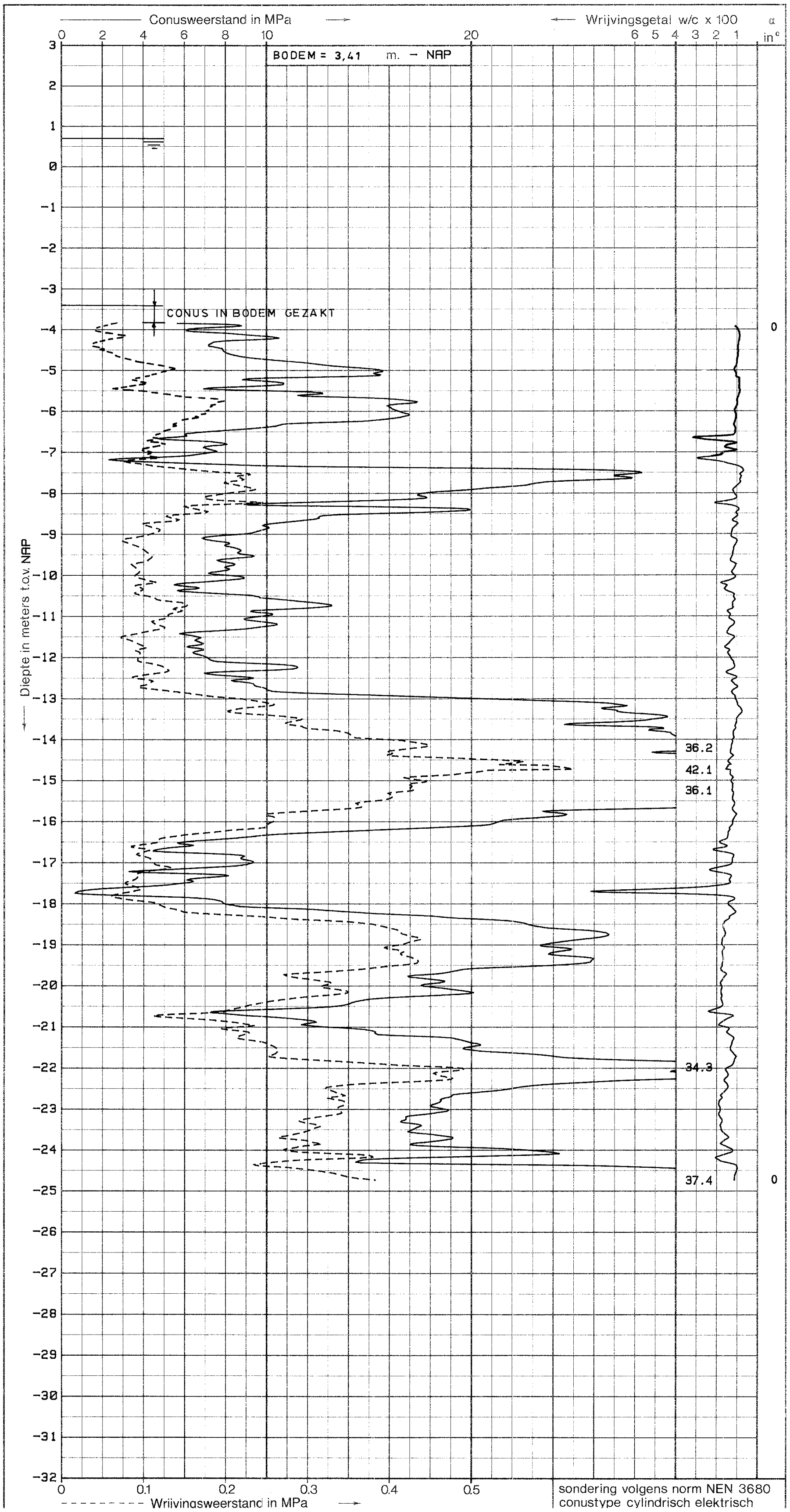


Figure 194: Cone penetration test S30D00193 (DINOloket, n.d.)

Appendix U Recommended design method

This appendix shows the new design method. This method was created after the design for a new pier was created in this thesis. After the design has been made, changes were made based on the experience gathered with the previous design method. This new method was mentioned in Chapter 8. The flowchart of this method is displayed in Figure 195.

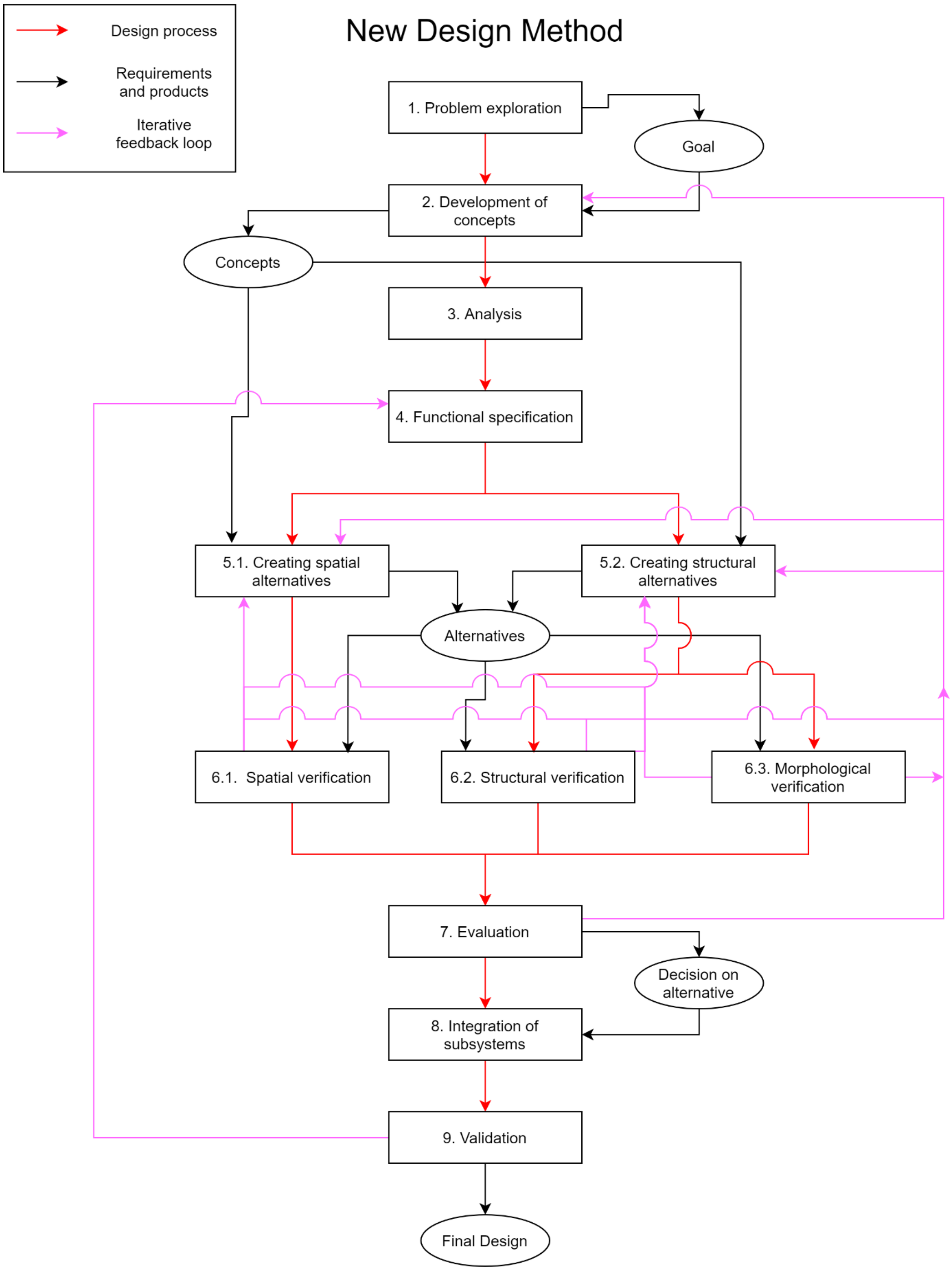


Figure 195: Flowchart of the new recommended design method.

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