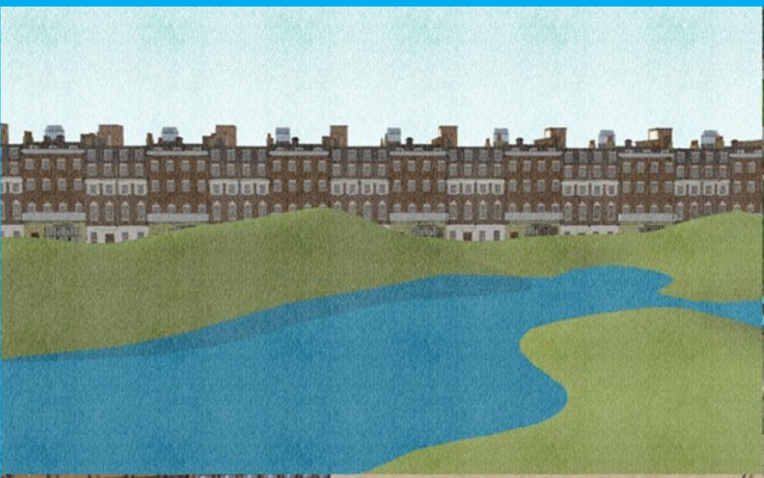
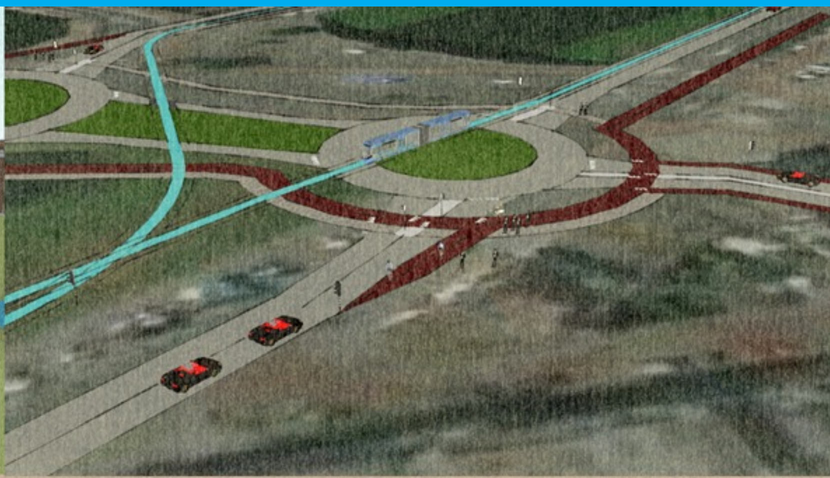
4th Harbour Scheveningen

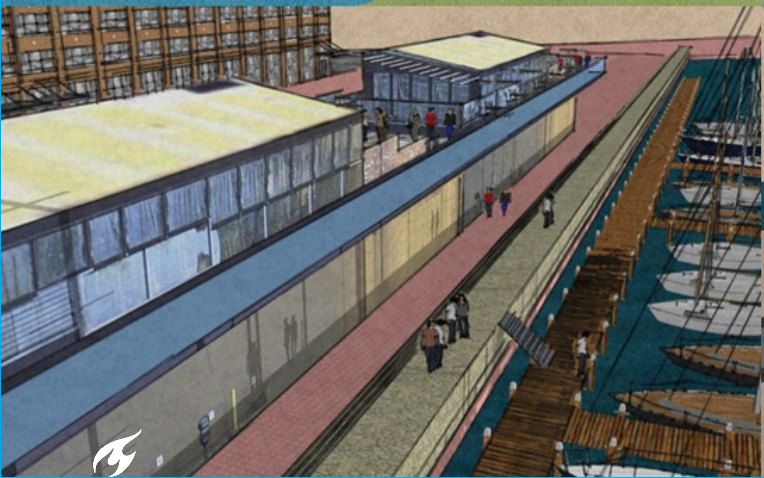
Masterplan

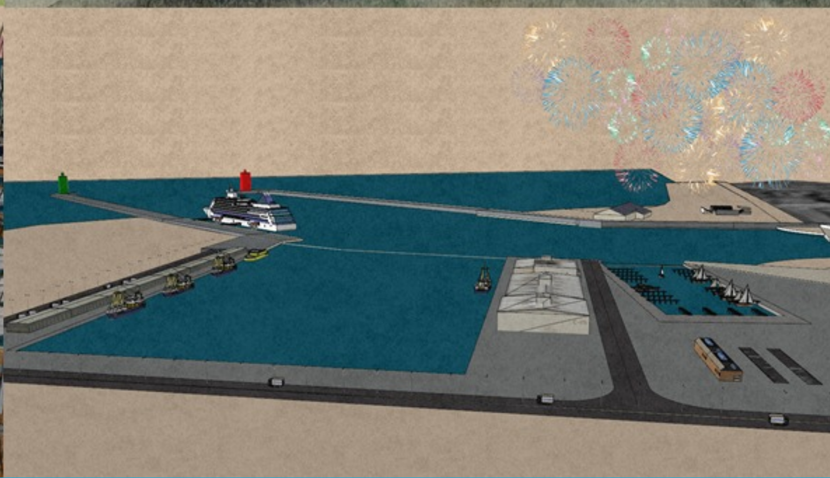
CIE4061-09
Multidisciplinary Project

S.A. Chotkan, M. Esmeijer, R.B.S. Goerdat, O. Hendriks, T. Stolp









TU Delft

4th Harbour Scheveningen

Masterplan

June 29, 2020

Students:	Shaniel Chotkan	4399862
	Marc Esmeijer	4375696
	Brahmanand Goerdat	4414012
	Onno Hendriks	4351118
	Thomas Stolp	4377907

Project duration: April - July, 2020

Supervisors: Dr. ir. A. van Binsbergen
Dr. ir. M.M. Rutten
Dr. ing. M.Z. Voorendt

TU Delft

Preface

In front of you lies the masterplan for the course CIE4061-09, which is the Civil Engineering Consultancy project. The purpose of this course is to perform a civil engineering related project with a group of multiple disciplines within the sector. Initially our group was supposed to follow up on the Pantai Project which concerns the plastic pollution problem on Bali. However, due to the Covid-19 situation a worthy alternative project had to be found.

Dr. ing. Mark Voorendt would have supervised us for the Pantai Project and we are delighted that he came up with the idea of elaborating on Dr. ir. Watermans project concerning the port of Scheveningen. This gave us the opportunity to still work on the multi-disciplinary project as a team. The composition of our team originates from the civil engineering Bachelor program, after which our paths diverged into different directions. In the beginning of our study career at the Faculty of Civil Engineering, the focus of our courses was distributed evenly over technical and design courses. As we progressed further into our curriculum, this shifted more towards the technical side. The execution of this project has been a learning opportunity for us all, as such did our appreciation for the other disciplines grow significantly.

We would like to express our gratitude to *Dr. ing. Mark Voorendt* and *Dr. ir. Waterman* for offering us the opportunity to do this project. Furthermore, we would like to thank our supervisors *Dr. Ir. Martine Rutten*, *Dr. ing. Mark Voorendt* and *Dr. Ir. Arjan Van Binsbergen* for their support and feedback during the project.

B. Goerdat, M. Esmeijer, O. Hendriks, T. Stolp & S. Chotkan

Summary

The Scheveningen harbour is a multi-functional harbour located near The Hague along the Dutch coast. Dr. ir. Waterman has proposed a plan for expansion of the harbour and an accompanying land reclamation program, based upon his Building with Nature principle. Seawards of the existing harbour area, a fourth harbour basin will be created that provides among others space for mini cruise ships. The reason for his ideas were based upon certain shortcomings of the harbour area. The various ports of Scheveningen are full most of the times, so extra capacity is preferred. A second reason for the upgrade of the harbour area, is the fact that the Doctor Lelykade adjacent to the second port flooded several times in the past years. Sea level rise will also result in higher flood risk, which is also part of the motivation for this project.

This report consists of two parts. In the first part, the plan of Waterman is developed further and various concepts are made on an urban scale. These concepts aim to provide a solutions which consists of five main elements: flood protection, harbour expansion, infrastructure, aquapuncture and salt intrusion. A total of 10 concepts were developed which were then verified to satisfy the functional requirements. The concepts were based upon several themes and different manners in which they could contribute to the surrounding area. The starting points for the concepts varied from minimizing changes and respecting the culture to the construction of tunnels and connections between the north side and south side of Scheveningen Haven. After designing the concepts they were compared with the requirements in order to specify which of them are adequate alternatives. The residuary alternatives were scored against one another in an MCA were the selection criteria were defined beforehand. The mutual importance of the selection criteria were expressed numerically using weight factors. The alternative which scored the best in the MCA is the one called Less is More, where the amount of changes to the area are minimized and the historical culture is respected. A last feedback resulted in the compensation for shortcomings in the final alternative, which explicitly was Building with Nature. The second phase of the project contains the detailed designs of the subsystems.

The flood safety of the Doctor Lelykade is safeguarded by increasing the elevation height of the Doctor Lelykade. Different concepts vary from increasing the height of the full street to solely placing concrete blocks on top of the quay wall. Due to the financial and aesthetic aspects of the different alternatives, the simplest one which comprehends placing blocks aligned best with the selection criteria. To optimize accessibility to and from the quay wall, the structure was built in the form of stairs.

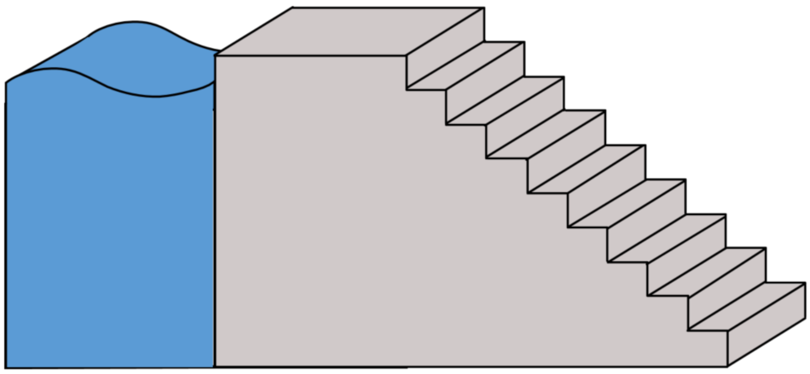


Figure 1: Breakwater design sketch with dimensions.

The breakwaters of the Scheveningen harbour have to be extended to the 10 meter depth contour. Because of the larger water depth and thus larger waves, a new design for a cross section of the Southern breakwater is made. The type of breakwater was chosen to be a caisson breakwater on top of a rubble mound foundation. The reasoning behind this was that the larger water depth would make this type of breakwater more economical than the conventional rubble mound breakwater. Different failure mechanisms were identified and each was assigned a certain design storm condition. The design method was used where all the uncertainty is assigned to the loading part of the limit state function. For this loading, extreme value distributions were fitted to find significant wave heights and storm surges corresponding to large return periods. To reduce over topping, a bullnose is present on the seaside of the caisson. A sketch of the design with dimensions and materials is presented in Figure 2.

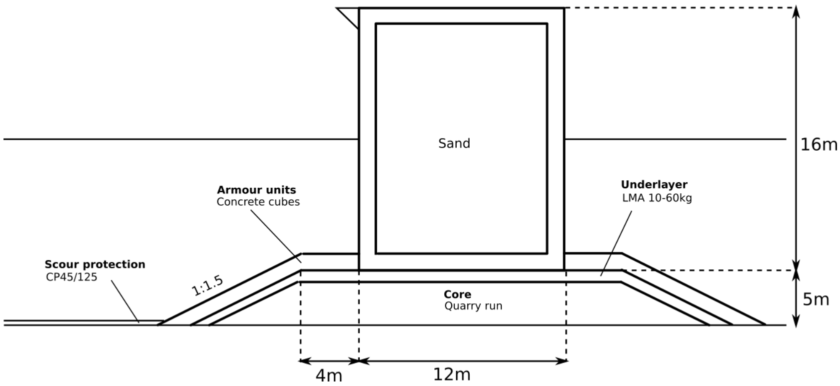


Figure 2: Breakwater design sketch with dimensions.

Layout of the harbour was made as a final design as this was necessary due to the implementation a new fourth harbour. This harbour ensures sufficient capacity for the marina and docking possibilities for larger fishing ships and a mini cruise. In order to achieve this, various elements were chosen for which dimensions had to be calculated. These elements are the access channel, the basins for the yachts and fishing ships and the docking of the mini cruise. The dimensions of the access channel have been determined such that design vessels can navigate safely through to the harbour. The basins for the yachts and fishing ships now has dimensions that provide sufficient capacity and the possibility for larger fishing ships to dock respectively. For the docking of the mini cruise, breasting and mooring dolphins are used. Dimensions for the rigid body of a breasting dolphin have been determined on the basis of strength calculations.The types of on-shore facilities have been determined along with their size and dimensions. This was necessary for the eventual layout of the harbour as that is where all the elements were added together. This was done to create a 3D sketch of the new harbour which can be seen in Figure 3.

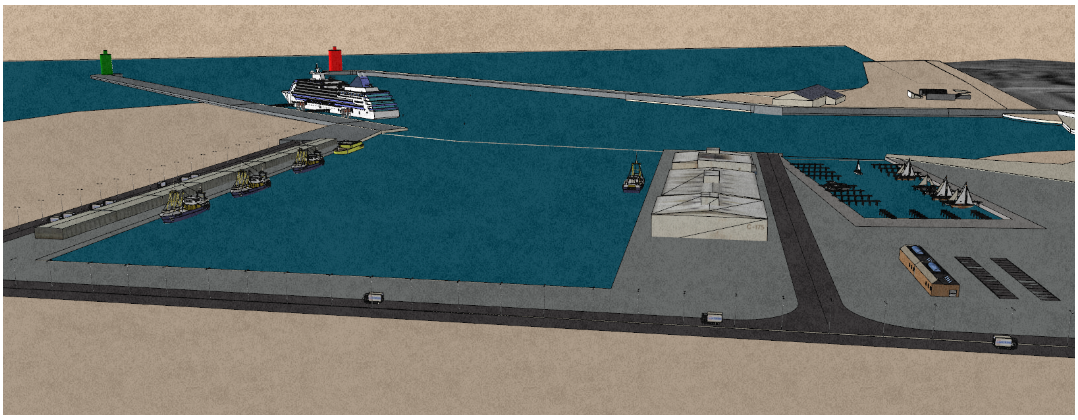


Figure 3: 3D Sketch of the entire port of Scheveningen including the fourth harbour.

The freshwater lens which starts under the dunes of Scheveningen is shrinking due to sea-level rise and more frequent/intense periods of droughts. Currently Water treatment plant Houtrust (WWTP Houtrust) discharges its effluent far into the North-Sea due to quality parameters not being sufficient. Due to incorporation of a new treatment scheme in WWTP Houtrust in the near future, fresh water will have the same quality parameters as surface water. This created an opportunity to use this fresh water for to recharge an infiltration pond that will provide an additional supply of water to the water lens. A model was created based on the water balance of the infiltration pond, which adjusts the discharge of the WWTP to the water levels found in the pond. Furthermore, a pipeline route and adequate pump were designed.

Figure 4: Overview of design including all components (OpenStreetMap contributors, 2017)

The infrastructure connecting Scheveningen to the hinterland is not sufficient for the future harbour expansion. For improvement, it was chosen to implement a tram track that will extend the current tracks (line 11), towards the harbour area. This tram track would extend across the cross-section of the Duindorpdam, which is identified as an unsafe combination of crossroads right now. Therefore, the crossroads at the Duindorpdam are redesigned too. The design of this tram track is catered towards cruise ship tourism, with an additional functionality to bring travelers from The Hague to Scheveningen beach/harbour. From a passenger analysis about the cruise ship passengers, the required capacity of the tram was determined. The placement of the tram tracks was determined per road segment, with the use of cross-sections, and more complex points were elaborated in further detail. The associated technical requirements were taken into consideration when creating these designs. After the physical aspect of the tram track was designed, the operational aspect was elaborated to ensure that the new tram would not clash with current tram lines 11 and 16.

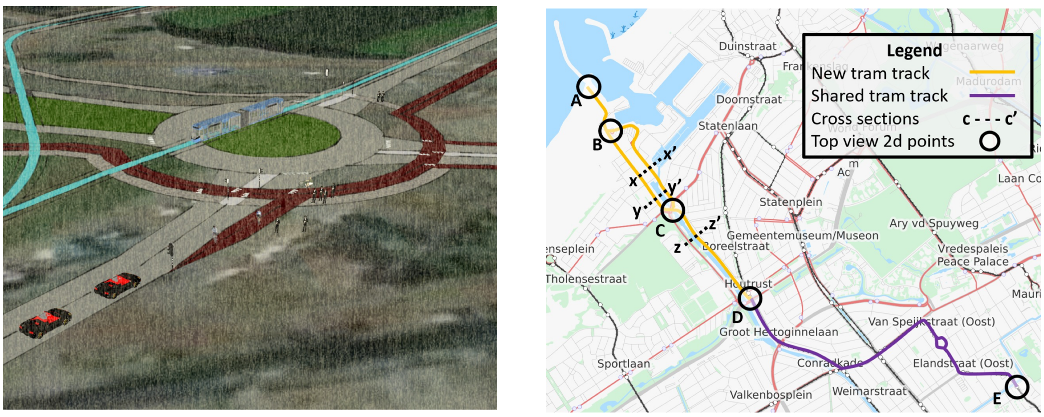


Figure 5: Overview of Duindorpdam and new tram track top view

Contents

1

Introduction

1.1. Motivation

The harbour of Scheveningen is a multi-functional harbour located in the Hague. Part of the harbour area, in particular the Dr. Lelykade, lies outside the primary flood defence. In the past years, multiple floods have occurred in the second harbour of Scheveningen under the influence of high-tide and strong wind surges (see Figure 1.1).

Figure 1.1: Flooding of the second harbour (Source: Omroepwest 2019)

Global warming leads to sea level rise and to an increased probability of flooding in coastal areas such as Scheveningen. To be prepared for the future, flood protections have to be improved. Besides sea level rise, climate change is likely to cause precipitation extremes to intensify i.e. more high intensity rainfall and longer droughts. Firstly, this can lead to fluvial flooding due to rain intensification. Secondly, problems related to the freshwater lens under the dunes of Scheveningen can occur due to intensification of precipitation extremes. The freshwater lens is a natural phenomenon under the dunes that stops salt water intrusion to the hinterland. Due to the sea level rise and drought intensification, the size of the freshwater lens could decrease which can cause problems in the hinterland.

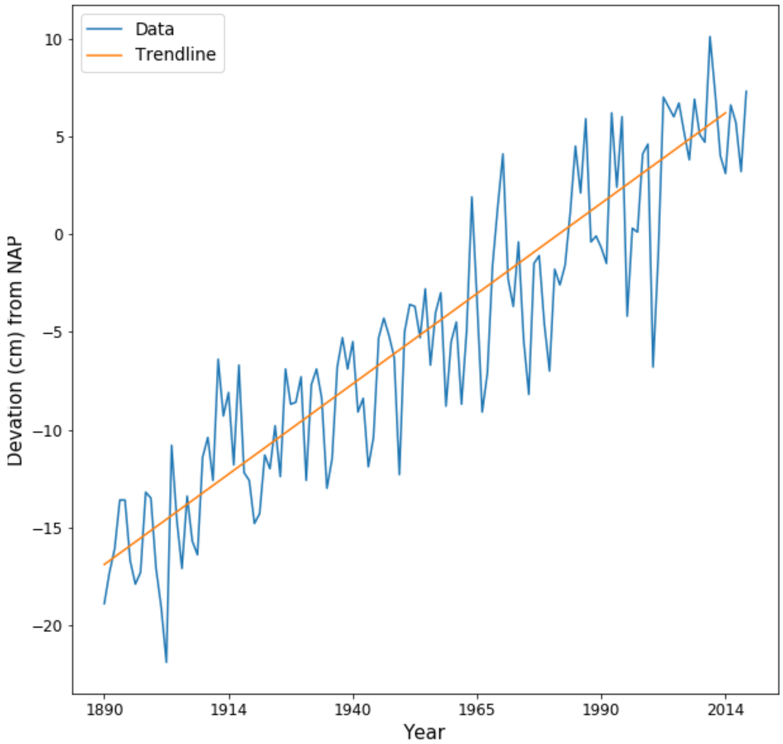


Figure 1.2: Sea level rise of the Dutch Coast (CBS, 2016).

Besides the complications induced by climate change, the second harbour of Scheveningen has been looking for expansion options due to a lack of capacity. There is a demand for berthing space for larger fishing ships and ships related to offshore industry. An expansion of the Scheveningen harbour and the population growth of Scheveningen and surrounding also asks for the improvement of the infrastructure and traffic disclosure of the area.

In line with the plans of the municipality of 'The Hague', an expansion of the current harbour is planned. In the past decade, an increase of unemployment was found in The Hague of which 10% coheres with the neighbourhoods of Duindorp and Scheveningen (Gemeente Den Haag, 2019). The expansion will create more job opportunities, thus improving the economic situation. Due to an expected increase of traffic flow throughout the harbour, a solution has to be implemented to not overflow the current network that links The Hague to the hinterland.

Expansion of the harbour of Scheveningen is one the plans for land reclamation along the Dutch coast proposed by dr. ir. Waterman. His ideas for land reclamation plan to improve the flood protection and his proposed solutions for the problems related to the Scheveningen harbour are the starting point for this project.

In his work, Waterman (2010) states that a major challenge of this century is to optimize space in coastal areas, guarantee safety and improve the economy and living environment. For the problem of space scarcity in coastal zones, Waterman identifies three main solutions. One of these solutions is building seawards by land reclamation which gives a possibility of multi-functional use according to Waterman. While improving flood protection of the hinterland, at the same time various watermanagement issues can be addressed.

Two principles that are central in the seaward option for spatial optimization are *Integrated Coastal Policy* and *Building with Nature*. The first principle is using an integrated approach for multi-functional use of coastal regions. Improving the overall economy as well as the environment. The second principle is a new approach in hydraulic engineering introduced by Waterman. This approach is based on incorporating material forces and interactions found in nature in engineering solutions.

1.2. Methodology

In this report, an integral solution for the issues stated in Section 1.1 is presented. A preliminary goal is defined to help stating the methodology. This goal is as follows:

Developing a realistic design for the expansion of the Scheveningen Harbour area

This report is based on a set of methods: Literature review, a modified hydraulic engineering design method, assessment of alternatives and eventually discipline specific methods. These methods are elaborated below.

1.2.1. Literature review

The literature review is mostly applied to to execute an in-depth problem analysis. This first step of the process concerns gathering of information on the case. This is started by gathering historical information on the project area, for which aspects considering historical value will be taken into account. And followed up by mapping the current situation and the future plans of the area. The mapping of the current situation will be taking into consideration attributes related to the surrounding areas, communities, port layout and infrastructure. The description of the current situation is followed up by a stakeholder analysis.

1.2.2. Modified hydraulic engineering design method

To stimulate the freedom in creativity during the creation of different concepts, the project group is split into 5 pairs that all create 2 conceptual designs. In contrary to the actual hydraulic engineering design method, where requirements and boundary conditions are stated first, the conceptual designs in the proposed modified method will be based solely on the problem analysis, again, to stimulate the freedom in creativity (See Figure 1.3). This line of thought is motivated by outcome of the research performed by Voorendt (2017). The problem analysis shows that there are multiple sub-problems that cannot be tackled by one single solution. Therefore, every conceptual design created then consists out of a set of solutions per sub-problem. The multidisciplinary composition of this project group is expected to create solutions from different visions.

1.2.3. Assessment of alternatives

To assess the conceptual designs, first all requirements and boundary conditions are composed. The requirements and boundary conditions will cancel out a few concepts that are not realistic enough, and some feedback will take place if the concepts could be corrected. The remaining concepts are now the input alternatives for the chosen assessment method, which will score these concepts.

The assessment method chosen in this project is the Multi-Criteria Analysis (MCA), because the MCA can take into account a broad set of criteria, without having to gather too much detail about the costs and benefits of the alternatives. The goal of the MCA can be to find solutions specific for different political visions, but in this project, the MCA will select 1 winning alternative based on the combined weight factors composed by the multidisciplinary project group. These weight factors will be constructed from discussion within the group and taking into account the known preferences of different stakeholders. The scoring of the criteria per alternative will be done individually, after which group discussion will lead to a consensus score presented in this report.

1.2.4. Discipline specific methods

After an alternative is chosen from the MCA, the multidisciplinary group works on problems coherent to their expertise to produce detailed designs. Additionally, not all sub-problem solutions will be elaborated due to time constrictions.

The challenge in this process is to maintain the multidisciplinary aspect in this phase, whilst working separately. Therefore, sub-groups are created where one person takes the lead in the corresponding expertise, whilst another helps to provide new thoughts and creativity from another perspective. Daily updates will keep the group informed of their peers progress and problems. The latter can than be discussed together for finding a multidisciplinary solution.

1.3. Report outline

This document started with an introduction, giving the relevancy, methodology (including objective). This section serves for clarification of the rest of the document. After this chapter, the problem analysis is presented in Chapter 2.

The problem analysis consists out a background analysis, stakeholder analysis, function analysis, the problem statement and the design objective. A total of ten concepts are developed in the third chapter, which are subject to requirements defined in the next chapter.

In Chapter 5, the concepts will be verified against those requirements. The remaining concepts are assessed by a MCA in Chapter 6. The criteria for the MCA are derived from Chapter 2. The best scoring concept will be further fine-tuned, by utilizing components from other concepts that scored better in the MCA.

The chosen conceptual design is then improved and elaborated in Chapter 7, as a base for the proceeding detailed designs. Chapters 8-12 starts with a selection of the components of the improved design. These components are then further dimensioned per component in these chapters. The content of those chapters is represented extensively and could be interpreted as a sequential report to the previous chapters, with corresponding conclusion and discussion per dimensioned component. The end of this report contains an overall conclusion and discussion of the full report. This process is schematized in Figure 1.3.

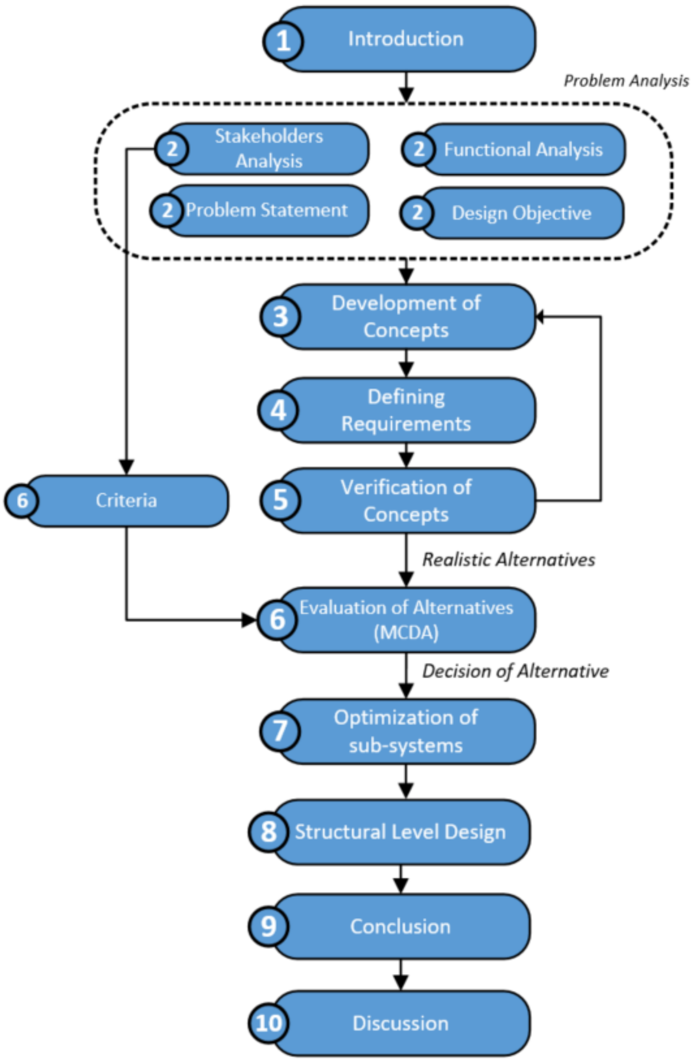


Figure 1.3: Flowchart of the design process

2 Problem analysis

This chapter describes the current situation and the problems associated with the plans regarding the future of the area. First, the current situation and nearby future are described, also addressing the cultural heritage of the area. Afterwards, the problem definition follows. In the next section the stakeholder analysis is presented, identifying main stakeholders and shareholders and their influences. From these sections the function analysis, problem statement and finally the objective follow.

2.1. Background

2.1.1. Plan area

The plan area is bounded according to decisions which have been made earlier by the municipality of The Hague. These boundaries are visualized in Figure 2.1 and correspond with the plans made for the area (Gemeente Den Haag, 2013). The defined boundaries are not fixed in time, as the possibility of the area developing thus diverging exists. At the landward edge of the boundary an urban area exists. No space is available for construction of new elements which implies that that specific boundary is fixed. The seaward side boundary may however translate more seaward, a phenomenon also in accordance with the land reclamation ideas of ir. Waterman.

Figure 2.1: Delimitation of the plan area of Scheveningen Haven (OpenStreetMap contributors, 2017).

2.2. Analysis of subareas

The harbour plan area of Scheveningen contains many subareas and objects which should be taken into consideration when making an integral design. In Figure 2.2, a brief overview is provided of the area.

Figure 2.2: Overview of subareas and structures in Harbour area (OpenStreetMap contributors, 2017).

- Living area 1 concerns Duindorp a neighbourhood that lies south-west of the harbour district.
- Living area 2 is the neighbourhood that lies behind the second harbour, it consists out of living and working space.
- Living area 3 is part of the development plans of the municipality of Scheveningen, new residential buildings are constructed here and an expansion of the Sand Theatre.
- The first and harbour are surrounded by hospitality areas, which are located nearby the water.
- Business area 2 functions as storage facility and transport hub for fishing companies
- Business area 2 is used by fishing companies and other businesses.
- Business area 3 contains a sail school and other storage facilities.
- A Natura 2000 area can be found on the map, further elaboration is found under Section A.3.
- the Port layout is explained under Section 2.2.1
- The pipeline on the map denotes the pipeline that discharges from the Watertreatment plant (WWTP) Houtrust into the North-Sea.

- In the current situation, the Verversingskanaal which lies along the Houtrustweg is subject to a few boundary conditions as presented in Figure 2.2. The function of the canal is the discharge of excess surface water into the sea. This channel is used to drain excess fresh water to the sea. Currently a pumping station is located at the Verversingskanaal with a capacity of $1170\ m^3/s$ that can elevate the water by $2.5\ m$. When the water level in the canal system rises, the pump discharges the excess water into the basins downstream (See Figure 2.2). The basins have buffer function, have bricked slopes coated with green and are connected through a culvert under the Duindorpbrug. The water is eventually drained from the basin through the scour sluice into the second harbour, disclosing it to the sea.

- The waterlens is a hydrological barrier which is formed under the dunes, it forms a natural protection against salinity intrusion towards the hinterland. The sea-wards boundary starts at the dune sections displayed on Figure 2.2.

2.2.1. Port layout

Currently the Scheveningen harbour consist of three harbour basins, the First Harbour, Second Harbour and Third Harbour. Just seaward of these basins the Outer Harbour is located, which is relatively unprotected from waves. Just north of this outer Harbour, a beach stadium is located with a parking area. The Scheveningen harbour is located outside the main flood defence. This primary flood defence is located at the Westduinweg. Scheveningen has a lighthouse that is located north of the 1st harbour. The First Harbour is the oldest part of the Scheveningen harbour dating from 1904. It was originally designed as a fishing harbour and is still used by fishing companies today. Large fishing vessels can berth along the quay of this harbour basin and facilities for handling catch are located on this side of the harbour.

In 1923, construction of the second harbour, ”2e Haven”, started. This harbour would be connected to the existing part by a small passage called ”De Pijp”. The second harbour first accommodated fishery, later around 1965, it mostly accomodated tourist activities and recreational. In 1972, a marina opened in the second harbour. Recently, the second harbour experienced flooding along the quay where some restaurants are located.

Due to the upcoming transportships, in 1973 a third harbour ”3e Haven” was constructed in behalf of transport company Norfolk Line. Between 1968 and 1972, the breakwaters got expanded. In 1972-1973 a terminal was constructed in the third harbour. In 2005, Norfolkline decided to leave Scheveningen. The terrain (See Figure 2.2, living area 3) will be redeveloped to become a attracting urban living and recreational are. The latter is further worked out in ”Masterplan Scheveningen Kust” from Januari 2010.

2.2.2. Infrastructure

On micro scale as presented Figure 2.3, there are two main entrance roads that lead towards the breakwaters. Namely the Houtrustkade and the Strandweg, these roads are both connected to a larger network of roads for disclosure. Important to note is that the Strandweg is currently a one-way traffic road. The Southern part of the harbour is mainly connected by the Houtrustweg to the S200 which towards the south flows into the Lozerlaan (N211) near Kijkduin and evnetually into the A4. When following the S200 towards the North it flows into the S101 and eventually into the A12 and A4. Another important connection consists of the N440 and N14 located northeast of Scheveningen that connect to the A4 near Leidschendam.

The Northern part of the harbour is connected to the Gevers Deynootweg which flows respectively north and south into the Zwolestraat and the Scheveningseweg. These roads are disclosing towards the A12 and A4 around Prins Clausplein by making use of the S-roads as can be seen on Figure 2.4.

Furthermore, the Rotterdamsebaan is a tunnel, currently under construction, that will connect the A13 with the centre of The Hague.
At the end of the 2e Haven, there is a drainage lock located that is used to drain excess precipitation to the sea. This lock can possibly allow to pass small ships who can sail further up the Afvoerkanaal and Verversingskanaal. In the later canal, a pumping station is located.

The most convenient way to reach Scheveningen by public transport right now is to use the tram. Trams 1, 9, 11, 16 and 17 all come from The Hague and pass through Scheveningen. Line 11 and 17 in particular have a stop in the harbour, however line 11 has a connection that not adequately linked to the centre.

The only buses operating in the municipality are bus 22 and 28. All public transport in Scheveningen is operated by *HTM personenvervoer*.

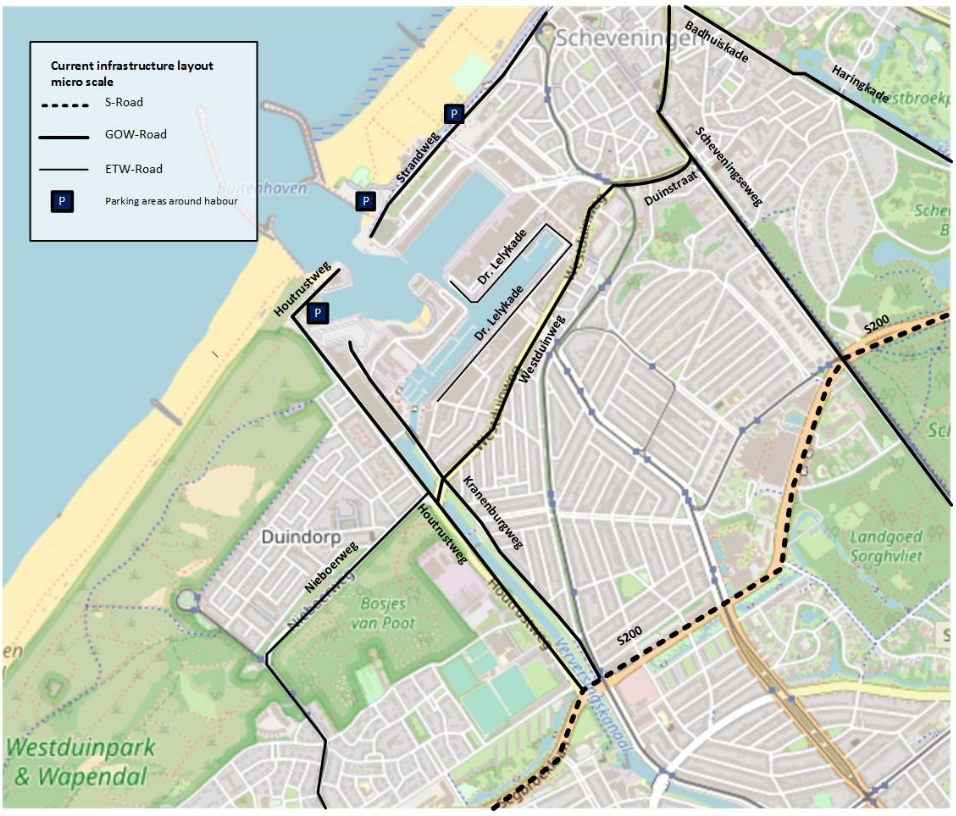


Figure 2.3: Micro scale: roads around the harbour (OpenStreetMap contributors, 2017).

Figure 2.4: Macro scale: Roads around Harbour to Hinterland (OpenStreetMap contributors, 2017).

	Stakeholder	Value
Public sector	Government	Project abide laws
	Municipality The Hague	Initiator of project
	Waterboard Delfland and Dunea	Reduce salt intrusion and Water control
	Rijkswaterstaat	Increase flood safety
	Province Zuid-Holland	Increase attractiveness of area
	WWTP Houtrust	Pumping treated water towards Northsea
	Metropoolregio Rotterdam Den Haag	Economic growth & accessibility
Community	Inhabitants of the plan area	No hinder, maintainance cultural history
	Agency for nature conservation	Preserve nature protection area
	Watersport community	More possibilities for watersport
	Media	Public opinion
Internal	Havenbedrijf, port employees	Increase capacity
Corporate bodies	Contractor	Maximize profit
	Fishing companies	Increase harbour capacity
	Transport Companies	Increase harbour capacity
	Cruise ship industry	Mooring of small cruise ships
	Hospitality industry	Increase attractiveness of area
	Tourism Sector	Increase tourism

Table 2.1: Overview of the main stakeholder and their values.

2.3. Stakeholder analysis

In this section, the main stakeholders of the project are identified. A distinction will be made between actively involved and other stakeholders. The needs, values and objectives of the actively involved stakeholders are also clarified in this chapter.
The decision context has been elaborated by distinguishing the stakeholders and shareholders. Their values and needs are important to consider and are used to develop criteria. A distinction will be made between the importance of the criteria by means of weight factors. The aim is to score the alternatives per criterion based on how well they implemented the values and needs by means of MCDA. This process will be elaborated and an MCDA will be performed in Chapter 6.

2.3.1. Stakeholder groups

First, the stakeholders are split into four different groups: public sector, community, internal and corporate. These groups can then also be split into a group of *stakeholder*, those who look for social benefits, and/or *shareholders*, those who look for financial benefits. Roughly, the public sector, community and internal groups are identified as stakeholders, whilst the corporate groups are also identified as shareholders.

Public sector

The stakeholders from the public sector are governmental instances that set up regulations, they need to be considered in order to comply to their rules and regulations. The local instances such as the municipality, are more involved as stakeholders with more interest as they are more affected by the project than the government for example. The higher instances do have a lot of power as well as the local instances and perhaps even more, but they are not likely to use it unless the project catches their attention.

Community
The community stakeholders are of low impact in the decision making process itself, but they do need to be taken into account during the decision making process. It is known that the community in this area has a strong opinion, and will stand together to achieve their goals.
Internal
The internal stakeholders are the stakeholders that operate the functional elements of the harbour. These stakeholders are directly impacted by the project. These are stakeholders, because their most important task is to facilitate the harbour, it is not essential to directly gain profit from that. The municipality of The Hague owns the Havenbedrijf and wants to see its company prosper for reasons other than stock performance.
Corporate bodies
The corporate bodies shareholders have little power in the actual decision making for the project. Although, together they can influence the outcome of the project. Some influence of the corporate bodies will also be influenced via the public sector, as it generally intends to stimulate the economic development as one of their important objectives.

2.3.2. Power-interest grid

A power-interest grid is created, with each actor located within this grid. The grid is made for the decision making process and indicates how to approach the actors. The axis does not indicate an absolute value of power or interest, it is rather relative. All mentioned actors have some interest and power, or they would have been left out of this analysis. The top right quarter contains all actors with relatively high interest and power. These actors need to be actively involved into the decision making process. Due to their values, most criteria in the multi-criteria analysis (MCA) are determined by this quarter. The actors in the top left quarter of the power interest grid have high interest in this project, but relatively low power. The role of these actors is passive, these stakeholders will be informed during the process. A risk is that these actors will team up to have power in case they feel left out. To avoid this, these actors need to be able to give their opinion about the project which should be taken into account. The bottom right part of the quarter are the sleeping giants. They have no direct interest in the project, and as long as the rules and regulations are followed, these actors will not play a direct role in this project.

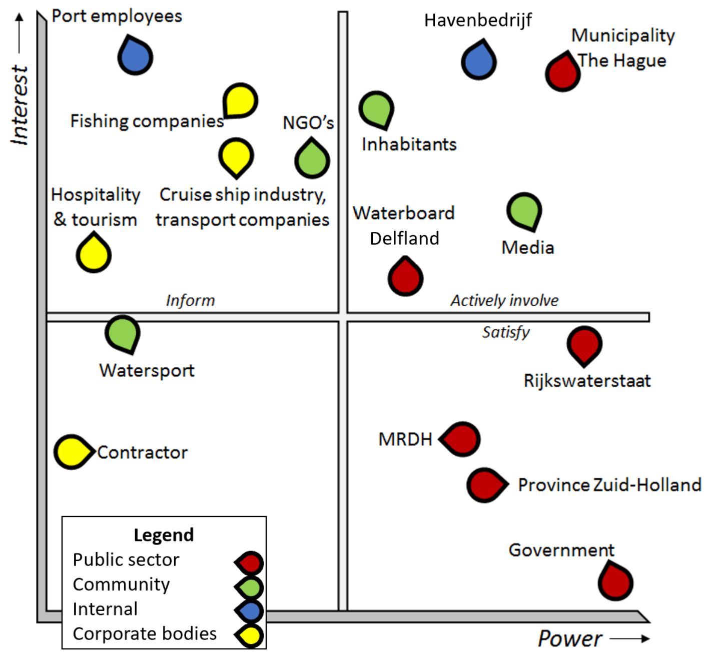


Figure 2.5: Power-Interest grid of stakeholders

2.3.3. Actively involved stakeholders

As mentioned before, the stakeholders with high power and interest are actively involved in the decision making process. In order to achieve this, the values and needs of all these stakeholders are written below that forms the basis from which the criteria in the MCA are formed. At first the values and needs of the municipality of The Hague are written down in terms of followed by that of the media, Havenbedrijf and the inhabitants.

Municipality The Hague

In 2013, the municipality of The Hague along with its corresponding stakeholders, developed the 'Bestemmingsplan Scheveningen Haven'. The document defines the area to be aimed at recreation and residence for the inhabitants and visitors trough out the whole year. The plan consists of the development plan for the area as indicated in Figure 2.1 (Gemeente Den Haag, 2013). The main objective of the document can be summarized in the following points:

- The area should become more attractive to water sports instances. This must be applied both on small scale and large scale, the latter implying that the area for example can be used as a station for the Ocean Race.
- The south part of the area and the north part can be modelled as two axes parallel to the sea. Stronger connection between the two axes is also one of the main points for the nearby future.

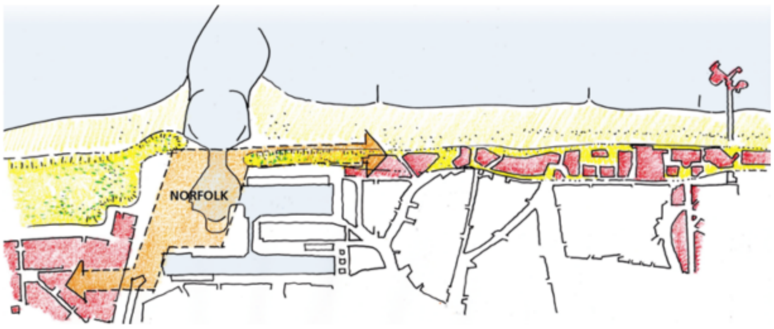


Figure 2.6: Connection between north and south Scheveningen Haven as depicted in Bestemmingsplan Scheveningen Haven (Gemeente Den Haag, 2016).

- Strengthen the public transport network from and towards the coast, as this is also beneficial for the hinterland of The Hague.
- Improve and extend walking paths and cycling paths along the coast.
- Reduce wandering traffic and enhance traffic structure and parking facilities. It is preferred that the traffic is transferred better to the access roads.
- Enhance routing of heavy (trucks) traffic as it is preferred that these do not pass through the city due to air pollution and road capacity reasons.
- Improve the road layout and reduce the bottlenecks as a way to increase the road capacity example given by the use of a green wave.
- The possibility to apply extra measures to guarantee a proper traffic flow during crowded days (Gemeente Den Haag, 2020).
- Preservation of the urban design of a certain area.
- The municipality of The Hague visions itself as the "Capital of the Northsea"
- Stimulation of innovation in the areas of blue economy (fishing, water sports, energy production and corresponding innovations) and flood defences.
- Further development and increasing attractiveness of the area to stimulate more tourism.
- Rise awareness about flood protection among wide audience such as inhabitants.

Media A special stakeholder in the project are the media. The media can create the public opinion and possibly activate the actors in the not-actively involved quarters to step up. The media can be your best friend in a project, but they can also be your worst enemy. Therefore, it is important to invite the media over, instead of obstructing them to do their job. Giving the media insight and openness about the decision making process is needed for a fruitful cooperation.

Havenbedrijf The Havenbedrijf desires to operate the harbour as efficient as possible. Their desire is to provide as much facilities as possible to attract new partnerships.

Inhabitants The people of Scheveningen have a great interest and power in the economic development of the area. Their power is significant because they are united through the means of resident's organizations such as the 'Bewonersorganisatie of Statenkwartier'. This organization aims to preserve as much cultural objects as possible and to keep the harbour's history present within the harbour.
In this project, tourism can therefore be positive for the inhabitants. On the other hand, too much tourism can decrease the livability which is bad for the inhabitants. For this masterplan, this assumption is sufficient, although, further research into the public opinion of the elements in this masterplan should be executed when the project will be carried out.

2.4. Function analysis

The current situation does not fulfill the expectations and wishes of the stakeholders. A new system must therefore be designed in which the subsystems meet the expectations of the relevant stakeholders. The desired system itself, and thus the subsystems, need to be able to perform according to defined functions. Defining these functions well results in a proper expression of the objectives which can later be considered when the quantity of success of the project is determined. The functions of the system are as follows:

- Guarantee sufficient flood protection for the harbour area of Scheveningen.
- Provide docking stations for bigger fishing boats, mini cruise ships and yachts.
- Create space and possibilities for the community The Hague to allow for the opportunity to become a global player in water sport activities.
- Prevent acceptable salt water intrusion from sea into the hinterland.
- Allow tolerable passage from harbor system to hinterland by nautical and vehicular traffic.

Figure 2.7: Schematic overview of the different functions of the system. Numbered are the current harbors already existing (OpenStreetMap contributors, 2017)

2.5. Problem statement

At the end of the problem analysis the problems themselves are stated in the so called problem statements. To formulate the problem statements one needs to consider the current situation and the desired one (formulated in the function analysis). The difference between these two defines the problem statement, and accounting for these discrepancies results in the objectives. The problem statements are therefore defined as follows:

- The dikes and quay walls do not provide a sufficient flood protection against the water levels.
- The harbor does not provide access for mini cruise ships and does not meet the capacity to dock yachts.
- The harbor lacks the possibility for water sport events in a way to become globally relevant.
- The present water-lens does not sufficiently hinder the salt intrusion to the hinterland.
- The current infrastructure situation does not provide enough capacity by car (persons and goods) and infrastructure for disclosing traffic from The Hague.
- The current waterway system is not sufficient for shipping.

2.6. Design objective

The difference between reality and the wishes of the client is essentially the response to the problem statements. The bridge between the current situation and the desired functions are summarised in the (sub)objective(s) of the project. Development of the harbour area will naturally require development in nearby and connected areas such as nearby infrastructure as well. Hence, the main objective can be identified as follows:

"Expanding the harbour while increasing the flood defence, capacity, nautical functionality and improving its connection to the main (aquatic) infrastructure of the nearby environment while safeguarding its historical culture."

3

Development of concepts

In this chapter the concepts for the Scheveningen harbour plan area are developed. A total of ten concepts will be proposed with coherent solutions to the subsystems. Elaborations on the plan will be incorporated in the form of preliminary maps per concept, which display overviews of the integral solution. The concepts are not necessarily feasible, but the line of reasoning is with the design objective in mind. All concepts will have broadening of the beach and an extension of the breakwaters to the 10m depth line. Furthermore, they will incorporate solutions for the expansion of the harbour, flood protection, aquapuncture, infrastructure, canal and pumping station capacity and salt water intrusion.

3.1. Concept 1: Less is More

The first concept is based on the idea of respecting the current situation and culture of the Scheveningen harbour area by making minimal changes, hence the name 'Less is More'. The main harbour expansion will be located seawards north-west of the current harbours. Another important aspect is the implementation of waterways and improving the current public transport network. The Scheveningen harbour will be connected to canals of The Hague and finally to the Binckhorst harbour in the hinterland, providing a new distribution network for fish. Important elements are the heightening of the quay walls around the second harbour and the harbour expansions on both sides of the breakwaters.

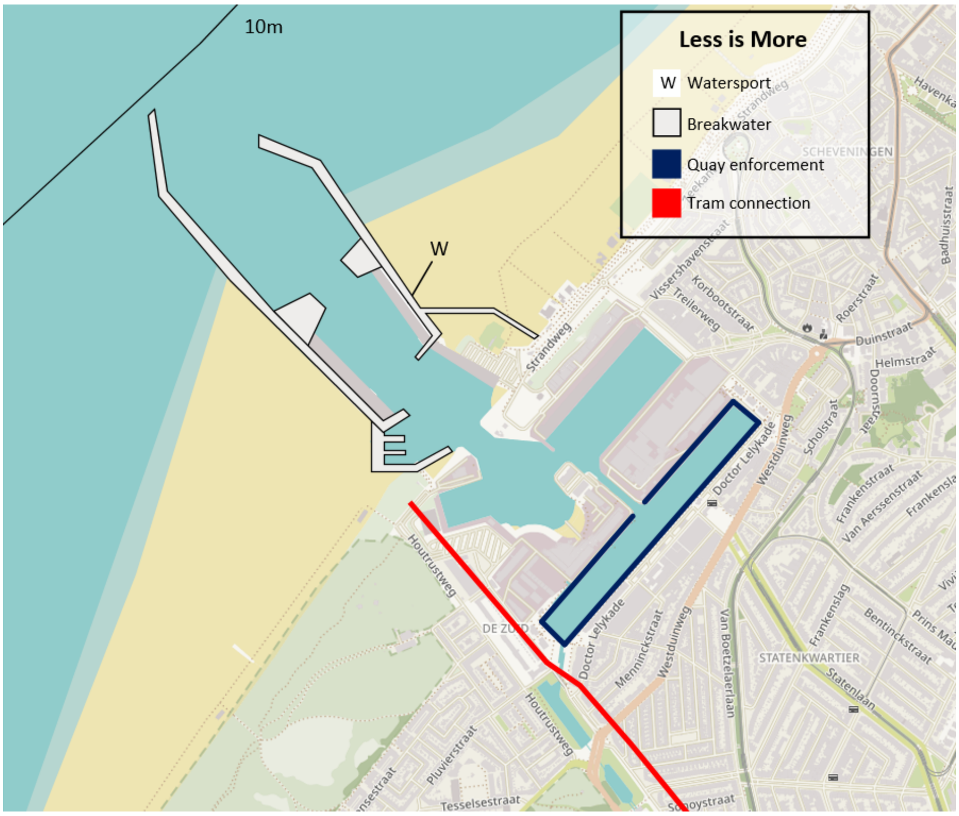


Figure 3.1: Overview of concept Less is More (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Improving quay walls of second harbour

- Flood protection of the harbour area will be improved by elevation of quay walls around the second harbour.
- Flood protection of the beach stadium by a hybrid of retaining wall and dune. This protection of the beach stadium also forms a shortcut to the watersports center which is located seawards of the stadium in line with the northern breakwater.

Water sports

- The watersports center will have training facilities and a quay for berthing of watersports related vessels.

Harbour: Quay along the access channel

- Harbour expansion is located in line with the southern and northern breakwater.

  - Northern quay will serve purpose for the watersports center, with berthing space for water sports related vessels and dry docks. More in the direction of the third harbour, there will be place for larger fishing ships.
  - Southern part will function as berthing place for small cruise ships and vessels for offshore industry.
  - At the landward end of the southern part, extra capacity for the marina will be available. The usage of the old-ship lock for aquapuncture reasons comes at the cost of capacity for the marina in the second harbour.
  - Both harbour expansions will be protected from oceanic currents by small extensions.

Infrastructure: Tram connection

- The tram track will be extended from the Van Boetzelaerlaan towards the new harbour. This will provide a connection towards the center of The Hague.
- The improvement of aquapuncture partially reliefs the road's usage as well as the usage of tramline. Therefore goods can be transported via waterways to the harbour in Binckhorst. From there it can be easily transported via the road as it is located next to the highway. Therefore no extra roads are added near the Scheveningen harbour.

Aquapuncture: Small boats through the Verversingskanaal

- For aquapuncture, the control sluice has to be adjusted which is located at the end of the second harbour. The following adjustments will be made to connect the second harbour with the canals of The Hague.
  - The door of this sluice will be removed such that the Verversingskanaal is connected with the second harbour.
  - The Duindorpdam has to be altered for allowing passage of small boats.
  - A ship lock will be constructed next to the pumping station to allow for passage of small boats.
- Removing the door of the sluice at the Verversingskanaal will have the following consequences:
  - Basins will no longer act as storage of fresh water
  - Salt water will intrude into the current basin areas
- To protect the area around the Verversingskanaal, the height of the quays have to be increased. Also an impermeable layer has to be added to halt salinity intrusion.

Salt intrusion: Storage facility in the dunes

- An infiltration pond will be designed in the area of Scheveningen. A transportline from Wastewater treatment plant Houtrust will discharge into the infiltration pond. Due to already having a line that discharges far into the North-Seam a bifurcation point should be made such that when the storage is full, the discharge will be lead into the North-Sea.
- The buffer capacity of the basins at the Verversingskanaal will be lost. The water in those basins is supplied by pumping-station Schouten, the buffer capacity will have to be re-located (which could be towards the infiltration pond).
- The functioning of the current spilling system will not be altered, however it will require more frequent water control due to the introduction of a the ship-lock.

3.2. Concept 2: Cruise Terminal De Zuid

This concept is based upon the notion of the municipalities plans, where most of the development happens at the base of the southern breakwater. Therefore, the expansion has been concentrated in this area. Important elements are the sluice between the breakwaters for protection of the full harbour, and the tunnel towards Kijkduin. The tunnel is situated between Duindorp and the Nature 2000 area to minimize impact to both. The natural view of the beach will not be hindered by the harbour expansion due to the dune rule being moved up (see Figure 3.2).

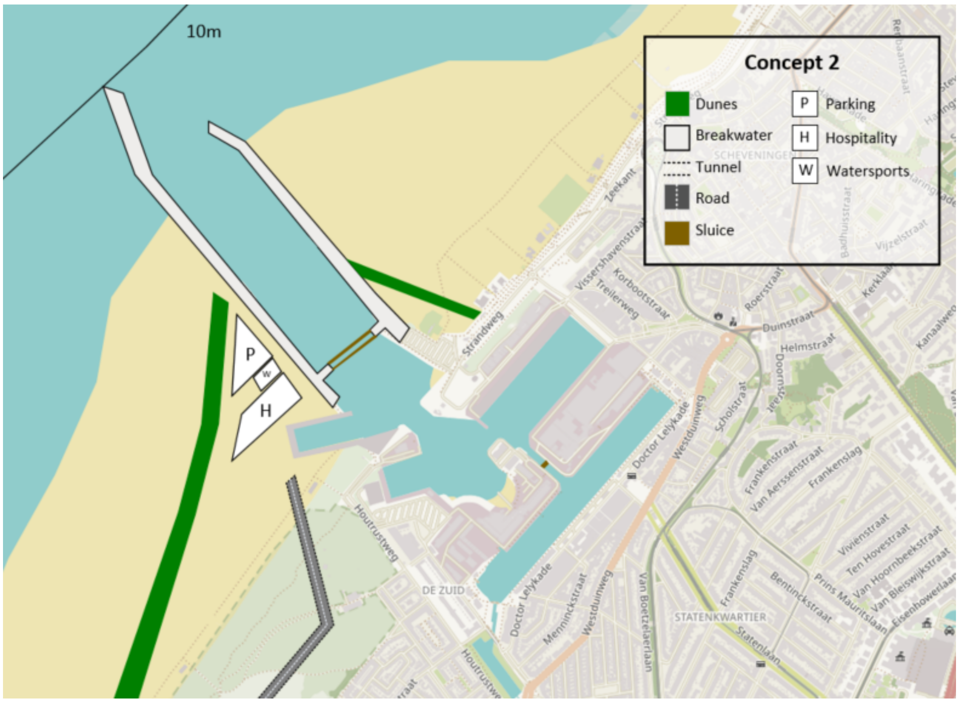


Figure 3.2: Overview of the Cruise Terminal De Zuid concept. (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Large sluice

- To improve the flood protection of the whole Scheveningen harbour area, a large sluice between the breakwaters will be constructed. This will be operated only when a large storm surge is present.
- A second sluice is placed between the third and second harbour, will be operated more frequently.

Harbour: Mini-cruise harbour at urban area De Zuid

- The new harbour is located near the new living area called De Zuid. This harbour will facilitate berthing place for mini cruise-ships.
- Around the new harbour there will be room for local businesses, hospitality and a parking spot. There will also be a cruise terminal that severs as departure and arrival point for the mini cruise ship. The area will be a hot spot for tourists.
- Training facilities for water sports will also be located in this area.

Infrastructure: Tunnel connection with Kijkduin

- To improve the accessibility of this area, a tunnel will be constructed that connects the fourth harbour with Kijkduin where a connection will be made to the S211 for disclosure (see Figure 3.2 & Figure 3.11).

Aquapuncture: No extra measures

- In this concept no extra measures are taken for improving aquapuncture of the area.

Salt intrusion: Storage facility in the dunes

- For minimizing salt intrusion sheet piles will be implemented up to the impermeable layer.

3.3. Concept 3: Reunion of north and south

In this design, the main focus is the improvement of the connection between beaches to the north and south of the Scheveningen harbour. To realize this, a tunnel and pedestrian bridge are constructed. A new harbour is located in the south with space for the watersports center and mini cruise ships.

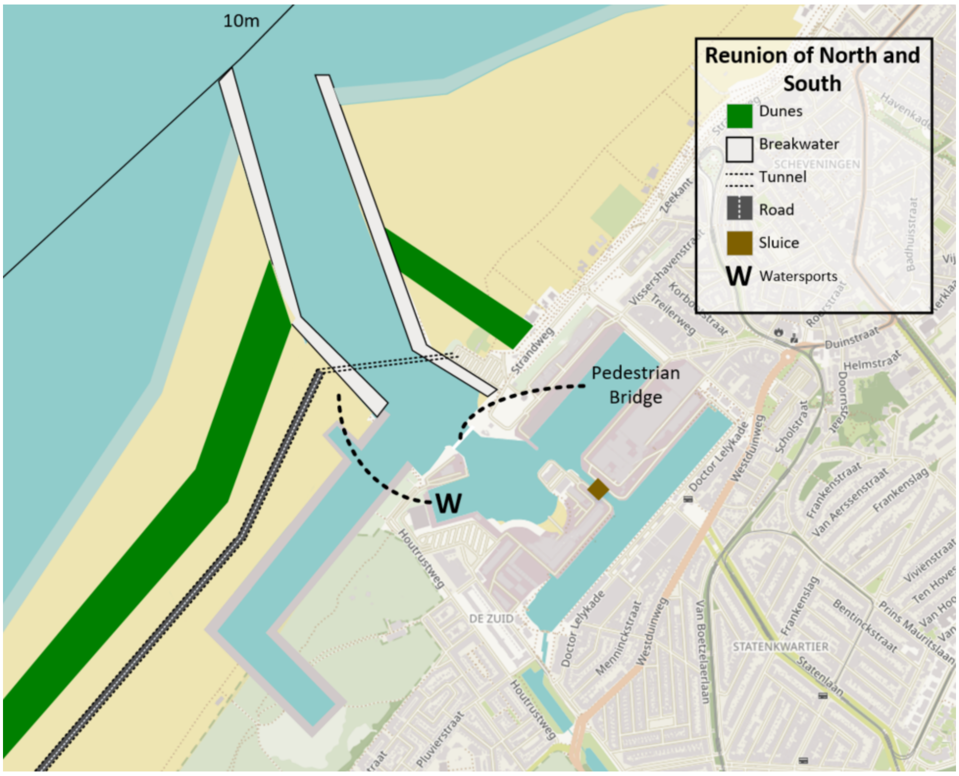


Figure 3.3: Overview of the third concept: Reunion of orth and south. (OpenStreetMap, 2017)

Design characteristics

Flood protection: Lock at small passage De Pijp

- To improve the flood protection of the second harbour a lock is constructed at the small passage called De Pijp.
- A dune rule will be created to protect the beach stadium against flooding.
- The current dune rule at the southern part will be (partially) shifted in seaward direction.

Harbour: Development at the south

- The fourth harbour is located at a landward incision at the southern breakwater.
  - The harbour will provide space for a second marina and will provide space for watersport related vessels.
  - There is space for mini cruise ships to berth.
  - At the end of the basin, there is room for vessels related to the offshore industry as well as for some larger fishing ships.

Infrastructure: Tunnel connection with Kijkduin

- A tunnel is constructed underwater between the breakwaters and between the fourth harbour to Kijkduin (see Figure 3.3 & Figure 3.11)

- The tunnel will connect with the current infrastructure at Kijkduin, such that traffic can go towards the S200 (The capacity at the Deltaplein has to be enlarged for this to occur).
- A pedestrian bridge is constructed just in front of the third harbour. This pedestrian bridge will be high enough to make sure that regular sized ships can still enter the existing harbour. Larger ships that are to high to pass under the bridge can berth in the new harbour.

infrastructure: Broadening Houtrustweg and Kranenburgweg

- The Houtrustweg and Kranenburgweg are broadened, they are connected to the S211 towards Kijkduin, which leads to disclosure form the city (see Figure 3.11)

Aquapuncture: Small boats through the Verversingskanaal

- For aquapuncture, the control sluice has to be adjusted which is located at the end of the second harbour. The following adjustments will be made to connect the second harbour with the canals of The Hague.
  - The door of this sluice will be removed such that the Verversingskanaal is connected with the second harbour.
  - The Duindorpdam has to be altered for allowing passage of small boats.
  - A ship lock will be constructed next to the pumping station to allow for passage of small boats.
- Removing the door of the sluice at the Verversingskanaal will have the following consequences:
  - Basins will no longer act as storage of fresh water
  - Salt water will intrude into the current basin areas
- To protect the area around the Verversingskanaal, the height of the quays have to be increased. Also an impermeable layer has to be added to halt salinity intrusion.

Salt intrusion: Storage in dunes

- Fresh water will be pumped to a storage facility around the dune area.
  - The storage facility will be created in the dunes for allowing recharge of the water lens in dry and wet conditions.
  - The pipeline will have a bifurcation which of which one will lead to the storage facility and the other one empties in the North-Sea. This depends on the size of the storage
- In addition to the reservoir for enlarging the waterlens recharge the principle of the Slok-op from a sustainable urban watermanagement concept, the WADI, will be used.
  - Infiltration pipes (Slok-op) will be introduced from the surface towards deeper layers for increasing the waterlens recharge. It will be used when the natural infiltration capacity of the toplayers is saturated.

3.4. Concept 4: Tidal park Scheveningen

This concept is based on the plans for a tidal park in the Rijnhaven in Rotterdam. The main functions of this park are leisure, nature and cultural appreciation. Floating pedestrian walkways will be constructed and there is room for a museum. In this design it was chosen to construct a dam for protection of the second harbour, thus creating a basin. Tidal energy will be generated by means of a turbine that will be installed in the dam. The loss of harbour capacity will be compensated by construction of a new harbour expansion south west of the current area. This will provide enough space for marine, mini-cruise ships and also for offshore and fishing industry.

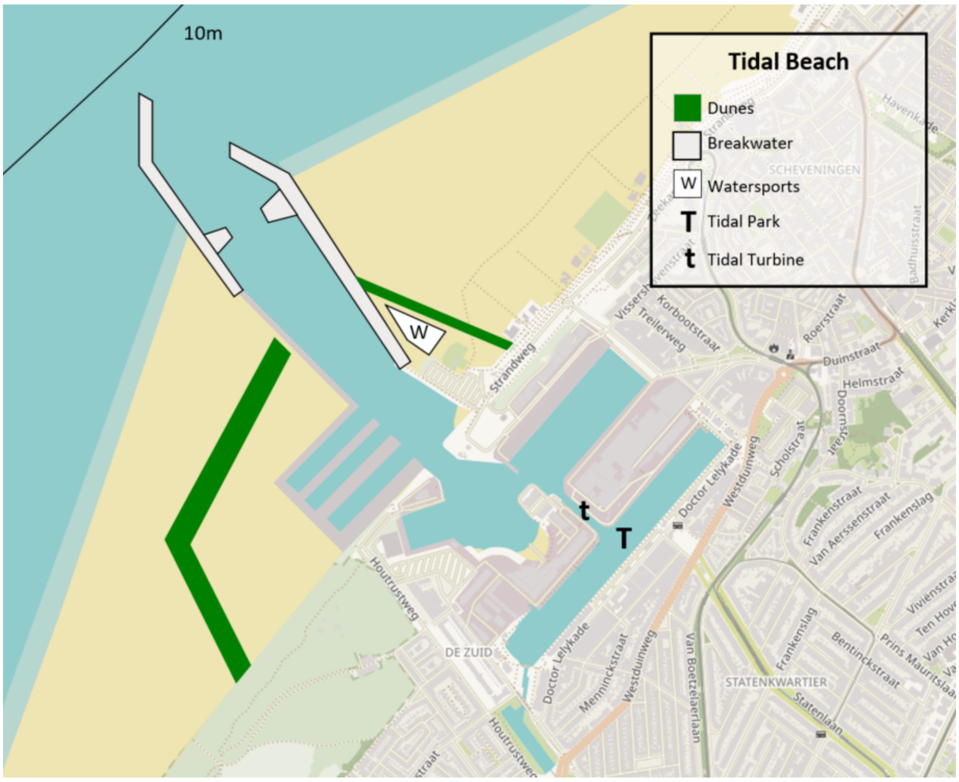


Figure 3.4: Overview of the tidal park concept. (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Closure dam at the second harbour

- Closing the second harbour with a closure dam will protect this part of the harbour against flooding. A turbine will be constructed in this dam to generate energy from the tidal water motion.
- Due to closure of the second harbor, it will not be accessible for ships any longer and the basin will be used as tidal park. Also a (floating) museum for cultural heritage will be located in the tidal park as well as floating green bodies and pedestrian walkways.
- The beach stadium will be protected against flooding by a dune stroke with a retaining wall. This retaining wall can provide the placement of stairs from the beach to the dune. On this dune a biking lane will be installed with ZOAB asphalt to minimize the water catchment capacity of the dune.

Harbour: Southern harbour expansion

- Since the second harbour will be closed in this concept, the marina will be moved seawards to the new harbour located close to the urban area De Zuid.
- Floating docking stations are installed inside the breakwater for the mini cruises. Passengers can then walk off the boat onto the breakwater to the harbour

- A watersports center is constructed next to the northern breakwater and close to the beach stadium. The watersport center will have training facilities for athletes, berthing space for watersports related vessels and dry docks.

infrastructure: Tunnel to the Hubertuspark

- A tunnel will be constructed from the first harbour to the Hubertuspark (See Figure 3.11).
- Taxis on the water will provide a connection between the harbour area and the Pier of Scheveningen.
- The closure dam will also function as a walkway and improve the internal connection of the harbour area.

Aquapuncture: Peaky Blinders tunnel

- It is impossible to reach the hinterland through the Verversingskanaal in this concept. Therefore, a canal extension is made that leads towards the sea at the Northern
  - Along the Haringkade (See Figure 2.4) towards sea a canal will be made following the structure of the streets.
  - A tunnel will be constructed from the Geverdeynootweg towards the beach which will allow ships to pass through the elevated residential area.
  - A sluice is incorporated on this system to stop the sea from intruding to the hinterland

Salt intrusion: Storage falicity in dunes

- The basins and pumping station will keep functioning as in the current situation
- WWTP Houtrusts transportline will discharge a storage facility around the dunes. It will have an operational bifurcation, when the storage is full it will discharge into the Northsea.
- The water area landinwards from the pumping station will have dockingstations for boats which will only be navigating on the canal system.

3.5. Concept 5: Aesthetic preservation

The Aesthetic Preservation concept focuses on keeping the visual appearance of the Scheveningen harbour intact. The new harbour is constructed at the northern breakwater and will have similar dimensions and orientation as the first harbour. In this way it respects the history of the Scheveningen harbour. The beach stadium will be moved to the south and at this location also a watersports center will be created.

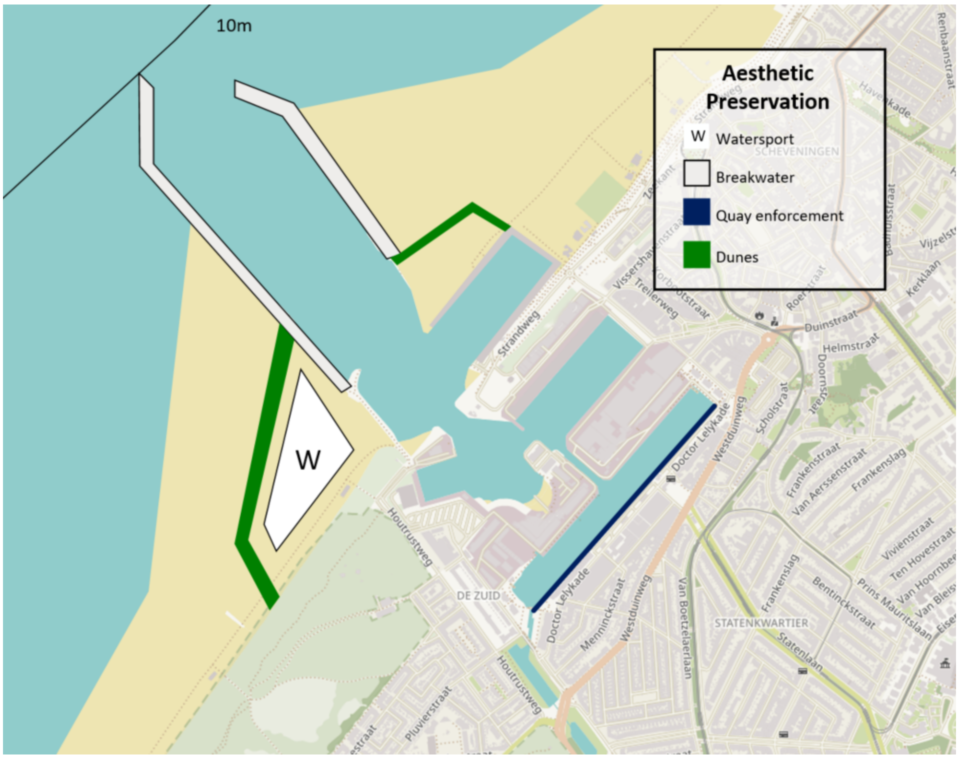


Figure 3.5: Overview of Aesthetic preservation concept (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Improving quay walls of second harbour

- To improve the flood protection of the second harbour the quay walls will be improved.
- The beach statium and watersports center at the south will be protected by a dune rule, see Figure 3.5.

Harbour: Northern harbour

- A new harbour will be build to the northwest of the current harbour. It will have the same dimensions and orientation as the historic first harbour. In this way it respects the cultural history of the Scheveningen harbour area.
- The new harbour will have berthing facilities for mini cruise ships and for larger fishing ships. Also some capacity for the offshore industry will be available.
- At the south, near the watersports center, a quay wall will be created for berthing of watersports related vessels. It will include special docking stations for sailing boats.

Infrastructure and salt intrusion: No extra measures

Aquapuncture: Small boats through the Verversingskanaal

- Same as concept 1: Less is More.

3.6. Concept 6: Sea Farm

In this concept a sea farm will be created offshore the Scheveningen harbour. This algae farm will contribute to the production of hydrogen. In addition, a beach pool will be constructed at the southern beach. This pool will provide the possibility to enjoy the sea water without the presence of high waves and strong currents.

Figure 3.6: Overview of the Sea Farm concept. (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Improving quay walls of second harbour

- Heightening of the quay walls around the second harbour is implemented for flood protection of the most vulnerable area
- A new dune rule is established from the Strandweg towards the boundary of the northern harbour for protection of the beach stadium.
- For the southern side the dune rule is moved partially north where it touches the breakwater.

Harbour: Seaward expansion

- The fourth harbour will be located on the southern breakwater. A small landinward incision will be made towards the south west, however will stick mostly to the current breakwater. It will be divided into an area for recreational boats and below this will be an area for the watersports center.
- On the northern side along the existing breakwater an area for small cruise ships to moor
- Across the boulevards at both parts of fourth harbour an area for hospitality will be created.
- A pool will be introduced to the beach, for recreational purposes. On the side of the dunes rocks will probably have to be placed for practical and aesthetic purposes.
- A space for a green algae farm production has to be implemented.

Infrastructure: Two-way traffic

- The passage under the Duindorpdam has to be enlarged (broadened and heightened) for allowing small ships to pass.
- The Strandweg will be transformed into two-way traffic road (allowing traffic in northern direction). Consequently the dike has to be broadened (see Figure 3.11)
- At the Scheveningseslag the parking spots will be removed for allowing more lanes at the intersection to the Gevers Deynootweg. From the Gevers Deynootweg, traffic flows into the van Alkemadelaan which leads to the N440 for disclosure (see Figure 3.11)
  - The Gevers Deynootweg has to be broadened for increasing the traffic capacity.
- Traffic flows towards the southern harbour require usage of the current infrastructure. A tunnel will be dug under the harbour inlet between the breakwaters to improve its reachability.

Aquapuncture: Small boats through the Verversingskanaal

- Same as concept 3.
- Across the pumping station on the northern side a small canal will be built, which has a lock. This will ensure that the pumping station will remain useful. Due to this canal the Kranenburgweg will probably have to be altered slightly as can be seen on the map.

Salt intrusion: Storage in dunes

- Same as concept 3.

3.7. Concept 7: Island harbour

This concept creates an island in the middle of the fourth harbour, with the Zuiderstrandtheater on it. Due to the shape of the fourth harbour, it is chosen to have one-way traffic in the harbour for easy access and exit operations for bigger ships. Characteristic to this design are the big sluice and the shape of the fourth harbour.

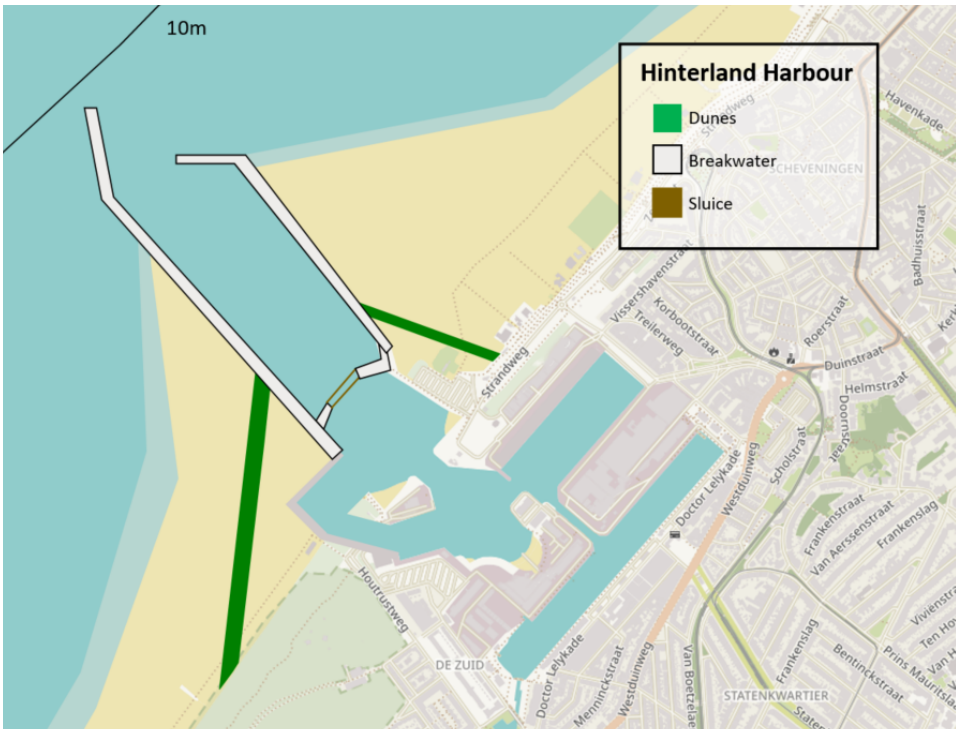


Figure 3.7: Overview of the Island Harbour concept. (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Large sluice

- To improve flood protection of the whole Scheveningen harbour area, a large sluice between the breakwaters will be constructed that will be operated when a storm surge is present.
- The beach stadium will be protected by an additional dune rule.

Harbour: Expansion around southern island

- The new harbour is situated near the southern harbour and stretches around the Zuiderstrandtheater as can be seen in Figure 3.7. It will contain docking station for mini cruise ships and larger fishing boats. The old harbours will provide space for docking of yachts and smaller private vessels.
- Construction of a sluice in between the breakwaters gives of control the water level in the harbour.
- The new harbour will have quay walls higher than that of the older harbours and an additional dune rule to ensure flood protection in the future.

infrastructure: Broadening Houtrustweg and Kranenburgweg

- The Houtrustweg and Kranenburgweg are broadened, they are connected to the S211 towards Kijkduin, which leads to disclosure form the city (see Figure 3.11)

Aquapuncture: No extra measures

- In this concept no extra measures are taken for improving aquapuncture of the area.

Salt intrusion: No extra measures

- No extra measures for were taken for salt intrusion.

3.8. Concept 8: Haagvlakte

For this design instead of building land inwards, it was decided to use the Maasvlakte as an inspiration for this concept. The Birmingham Canal Navigations in particular formed the motivation for improving the aquapuncture.

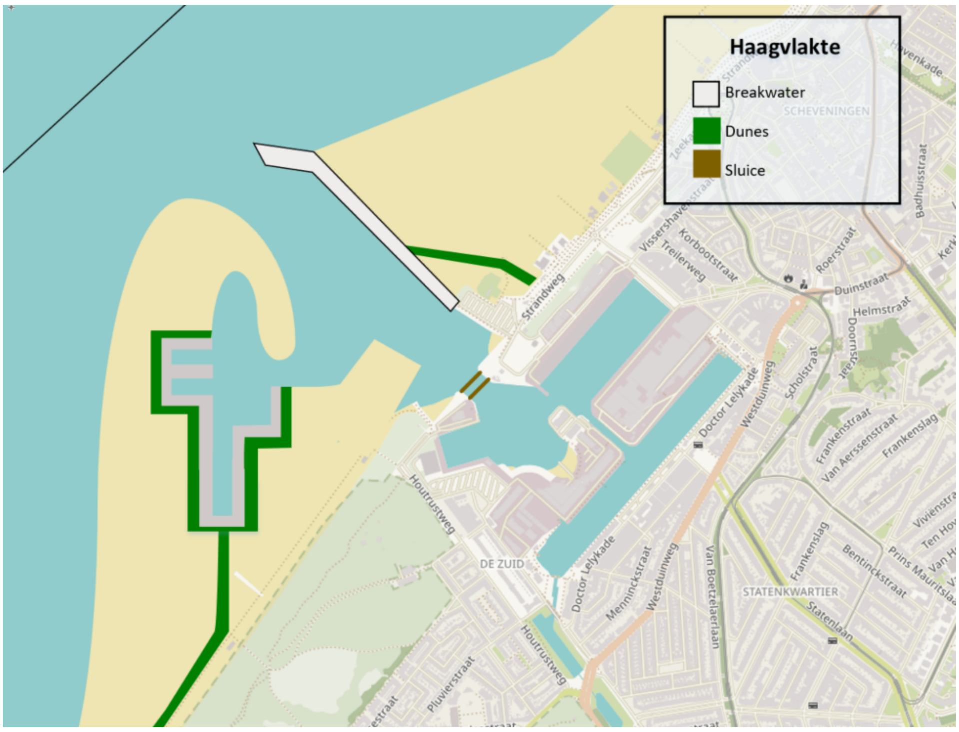


Figure 3.8: Overview of the Haagvlakte concept. (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Large sluice

- To improve flood protection of the whole Scheveningen harbour area, a large sluice between the breakwaters will be constructed that will be operated when a storm surge is present.
- For the protection of the sea theatre an additional dune rule will be implemented from the Strandweg towards the southern breakwater.

Harbour: Large westward expansion

- The harbour will be located at the southern breakwater and functions as breakwater extension towards the 10m depth line to break off north-eastern currents from the sea
- The third harbour will function as hub for the watersports center
- The traffic of the third harbour will be shifted towards the fourth harbour
- The fourth harbour will be oversized, to enable growth of the harbour in the future (similar to the Maasvlakte).
- The fourth harbour will function as a conduct hub for goods to the hinterland and as an extension for the lack of capacity of the second harbour
- The fourth harbour will be surrounded by a dune section and dike that will be multi-functional as a road.

infrastructure: Tram connection

- The channel that runs from the city center until the playground at Haringkade, will be extended in seaward direction. The extension follows along Badhuiskade and will run parallel to the Seinpostduin where it will end seawards.
  - To prevent the sea from intruding to the hinterland a sluice will be situated between the sea and the canal system.
- To disclose the traffic from the Fourth harbour it was chosen to construct a tunnel from the southern part of the harbour towards Kijkduin across the beach area. Due to the widening of the beach space would be created for this tunnel (see Figure 3.11).

Aquapuncture: No extra measures

- In this concept no extra measures are taken for improving aquapuncture of the area.

Salt intrusion: Deep quay walls

- In addition to the tunnel, deep quay walls can be introduced along the lateral side which counteracts seawater from seeping into the hinterland south of the breakwaters.

3.9. Concept 9: Oh Deer!

Concept 9 introduces a new harbour to the south of the current access channel that is disconnected from the excising harbour. The new harbour will be used by the larger fishing ships, mini cruise ships and ships used by the offshore industry. The existing harbour will be used by the marina, water sports vessels and the small fishing ships.

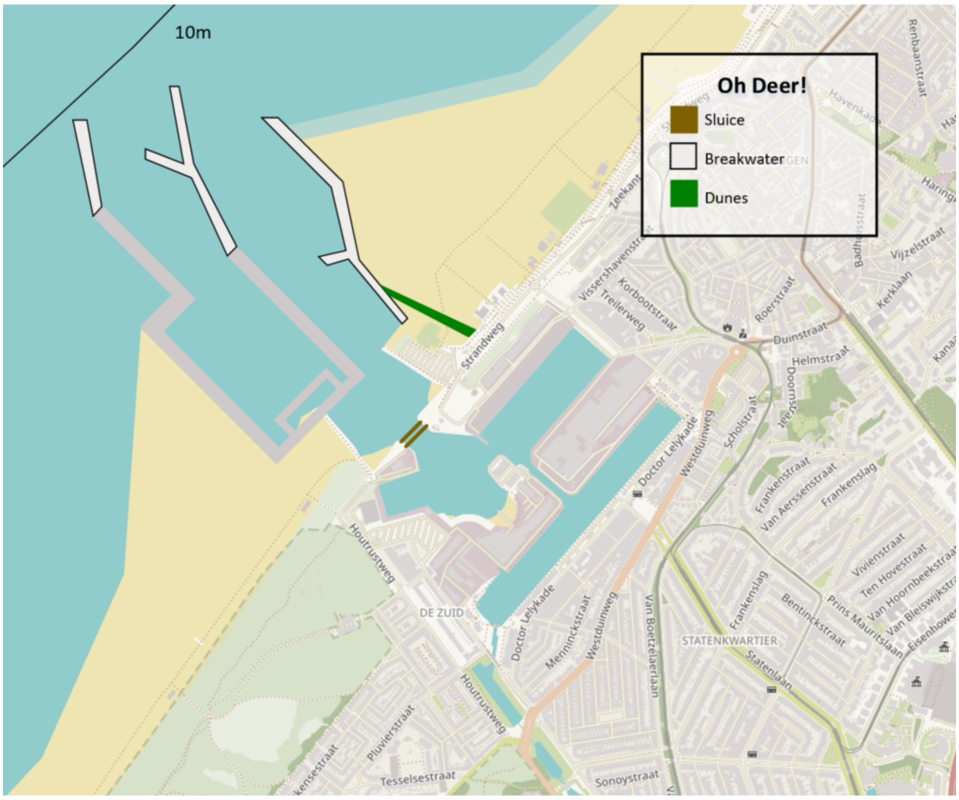


Figure 3.9: Overview of the Oh Deer! concept. (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Large sluice

- To improve flood protection of the whole Scheveningen harbour area, a large sluice between the breakwaters will be constructed that will be operated when a storm surge is present.
- A second dune stroke is added to ensure flood protection for the beach stadium.
- The new harbor will have quay walls with a height that provides flood protection for the upcoming years.

Harbour: Distinguish between the large and small ships

- The fourth harbour is located west of the current harbour area, in between the middle and southern breakwaters. This new harbour will be used by the larger ships. The mini cruise ships, offshore industry vessels and larger fishing ships will be able to berth in this area. The smaller fishing ships and yachts will still be using the existing harbour.
- The new harbour is located more seaward and will thus reduce the noise and visual nuisance introduced by the larger ships.

Infrastructure: Tunnel Scheveningseweg

- A tunnel of approximately 1 km will be constructed along the Scheveningseweg, starting from the Duinstraat, ending at the professor B.M. Teldersweg. The main traffic flow will then go east through the forest towards the N440, N44 and A12. The Duinstraat and the Westduinweg will be broadened, because otherwise this will be a new bottleneck of the route towards the highways

Aquapuncture:

- Waterways will be made usable for small vessels by opening the sluice at the second harbour. The function of the old harbour is entirely to stimulate the hospitality, as it now only consists of yachts and small vessels.

Salt intrusion: no extra measures

- In this concept no extra measures are taken against salt intrusion in the area.

3.10. Concept 10: Inside out

This concept is based upon the Port of Mackay, where the inner space of the breakwaters is used for mooring of small ships. The idea is to create enough space between the new and current breakwaters, in order to have a first part of the fourth harbour between the northern breakwaters (see Figure 3.10).

Figure 3.10: Overview of the Inside Out concept. (OpenStreetMap contributors, 2017)

Design characteristics

Flood protection: Lock at the port entrance

- The flood defence will consist out of a gate at the newly created entrance.
- The beach stadium will be protected against flooding by a dune rule.

Harbour: Building around the existing breakwaters

- Constructing two new breakwaters around the existing ones and redesigned such that they can function as quay. In this way a new harbour is created.
- Entrance of the current harbour is moved towards a new path created between the southern breakwaters. To create this entrance between the southern breakwaters some of the land north form the Zuiderstrand theater has to be excavated.
- A watersports center will be constructed in the new harbour with facilities for training athletes and berthing of watersports related vessels.
- The new harbour will have space for yachts, when there is no more space in the marina in the second harbour.

Infrastructure: Roads to northern breakwater

- Roads are added on the northern breakwaters to make the fourth harbour accessible for (un)loading of fishing boats
- The southern breakwaters will not be accessible by vehicles, therefore this area is allocated to recreational ships.
- The gate at the new entrance will function as a bridge to cross the breakwaters when closed, thus creating a shortcut between the northern and southern breakwaters.
- A parking space is added north from the Scheveningen Beach stadium. Thus making the entire harbour more accessible by car.
- For dealing with extra traffic the Strandweg will be broadened. Space for this broadening is created, because the beach is extended towards the seaside, and the beach activities can move with it.
- The Strandweg is extended towards Zwolsestraat. The Zwolsestraat also need broadening towards the Van Alkemadelaan, in order to have 2 lanes in each direction for the whole route. The choice is made to broaden this road at the expense of the cycling path and parking spaces along the road (see Figure 3.11).
- The parking spaces will be replaced by a (small) underground parking lot and the cyclists will be redirected to cycle through more through the center.

Aquapuncture: Small boats through the Verversingskanaal

- The same aquapuncture measures as in concept 3.

Salt intrusion: Storage in dunes

- Same measures as in concept 3.

3.11. Infrastructure Overview

In order to keep matters visually structured, the infrastructures ideas are all shown in a different figure. Some concept show equal solutions to the infrastructure problem. Concepts having matching solutions are shown using the same colour in the Figure 3.11.

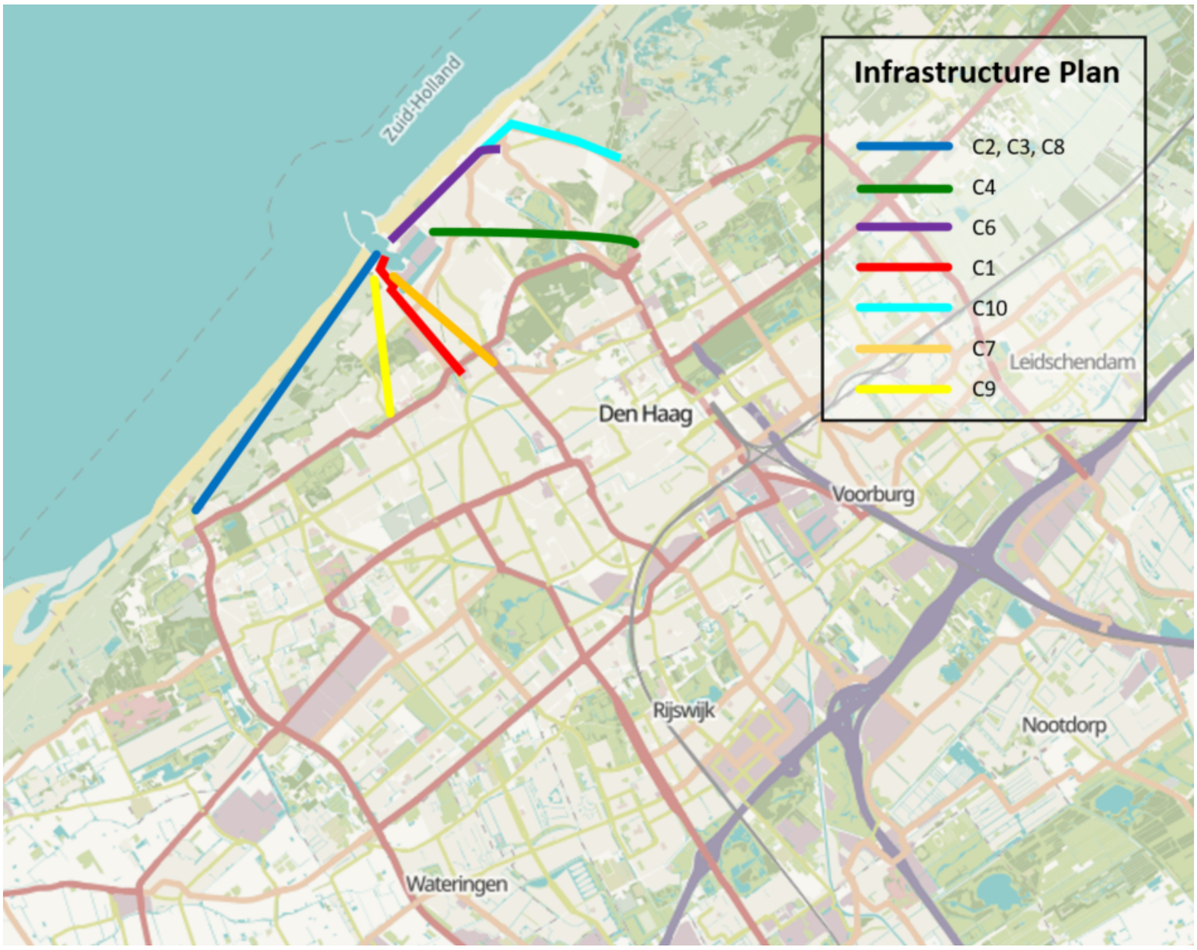


Figure 3.11: Overview infrastructure of all concepts. (OpenStreetMap contributors, 2017)

The different concepts have now been designed on a quite global scale. It may be possible that several concepts will be changed afterwards, or that some of them aren't even feasible to begin with. This results from the to be determined requirements. Another possibility is the creation of a new concept containing the 'best' sub aspects from the different concepts. This will be determined using evaluation criteria, which are defined after the requirements.

4

Program of requirements

The functional requirements can be derived from the system functions which is specified in the design objective. The program of requirements will cover the entire scope of the design. These requirements will be used for the verification of concepts in Chapter 5. Do note that more detailed requirements will be stated for the designs made in Chapters 8-12.

4.1. Flood protection

- The Scheveningen harbour area, located outside the primary flood defence, must be better protected against flooding to avoid the frequent flooding of for example the Doctor Lelykade. Sea level rise must be taken into account such that the protection is also sufficient in the near future.

- Flood protection measures for the Scheveningen harbour area must not conflict with the functions of the port in *normal* weather conditions. The entrance from the sea must have a good accessibility and passage must be safe.

- North and south of the harbour, the beach must be expanded to be better resistant against sea level rise. To realise this, the breakwaters must be expanded towards the 10 m depth contour.

- The beach stadium located north of the harbour must be protected against flooding taking into account sea level rise.

4.2. Harbour

- The capacity of the Scheveningen harbour must be increased to make the Scheveningen harbour attractive for coast hopping.

- The Scheveningen harbour must have mooring places for mini cruise ships to connect the Scheveningen harbour with the Baltic and Scandinavian countries.

- Larger fishing ships must be able to berth in the harbour for among others shipping lines Jaczon and Van Der Zwan.

- Space for facilities for offshore-industry (shipping line Groen) must be made available in the harbour. These facilities must be fit for maintenance of windfarms, cultivation of macro-algae and generating electricity by conversion of hydrogen.

- The harbour must have an access channel which must be long enough for a design ship to slow down and wide enough for save passage.

- For good manoeuvrability in the access channel of the harbour must have no bends, must be orientated in line with the dominant wave direction and must not form a narrow sleeve but provide space behind the opening.

Basin dimensions

- The basin must provide sufficient shelter for vessels from high sea waves.
- Basin must provide seasonal storage ashore of small ships in open yards or in sheds. Administrative or private services (harbour master's office, weather forecast, customs, clubs, medical needs, etc.)
- Basin width must be sufficient for the required amount of berthing stations to be installed.
- Basin must provide space for maintenance and repair operations in the marina (yards, dry-docking facilities).

4.3. Verversings kanaal and Salinity intrusion

4.3.1. Canal water and pumping station

- The canal (Verversingskanaal) has to be robust for future climate change influence.
- The pumping station has to have a sufficient capacity for discharging a flow that matches a higher rainfall intensity.

4.3.2. Enlarging the freshwater lens

- The water lens dimensions must be broad and deep enough to stop saline water from intruding to the hinterland. Therefore, the storage capacity of the freshwater lens has to be enlarged.
- The hydraulic pressure under nearby located houses may not rise to the point where it leads to flooding in the basements due to enlargement of the freshwater lens.
- In periods of drought the waterlens volume needs to be sustained to halt the salt intrusion.

4.4. Water sports

- A sufficient amount of space on the hinterland must be present to construct all facilities required to train Olympic athletes.
- The availability of mooring and dry docks for sailing vessels and other water sports vessels.
- Sufficient space for the Ocean Race, its entourage and possible spectators.
- Space must be available for the beach stadium.

4.5. Infrastructure

- Extra traffic demand created by the fourth harbour may not increase the road traffic density during peak hours in Scheveningen and The Hague.
- A plan is for traffic management is made to deal with traffic jams due to recreational traffic on the access roads.
- Enough space for parking lots must be available to supply the extra demand created by the fourth harbour.

4.6. Aquapuncture

- The harbour has to be connected to the hinterland (i.e. the canal system of The Hague) through waterways for small boats.

5 Verification of concepts

In this chapter, the concepts from Chapter 3 are verified to satisfy the program of requirements given in Chapter 4. A short explanation is given for concepts that do not meet with all requirements. The concepts were summarized adjacent to the requirements after which inadequate concept were eliminated.

5.1. Dropped concepts

Concept 2: Cruise terminal DeZuid

In this concept the requirements of manoeuvrability is not met because the mini-cruise ships are obligated to turn sharply after the sluice while there is insufficient space. Moreover, this concept does not provide an increase in capacity for the marina and offshore industry. Finally, in this concept a tunnel is planned to be built from the fourth harbour towards Kijkduin. However, this tunnel will partly be submerged through the dunes and the Natura 2000 area which is not allowed.

Concept 5: Aesthetic preservation

This concept does not meet the requirements. This concept does not provide enough space for extra capacity for the marina. In addition, there are no measures included for robustness of the canal and pumping station. Finally, this concept has no solution for the increase in road traffic density created by the fourth harbour.

Concept 7: Island harbour

This concept does not meet all requirements. Firstly, the harbour capacity is not sufficient. Only new space is available for the mini cruise ships and the larger fishing ships, but there is no additional space available in this concept for expansion of the the marina and offshore industry related vessels. In addition, the canal and pumping capacity is not sufficient and ready for climate change. Finally, there is no space reserved for watersports related vessel.

Concept 9: Oh Deer!

This concept does not meet all requirements. In the new situation, the access channel has a slight bend. This does not meet the requirements for a straight access channel that provide in safe entrance of the harbour. In addition, the canal and pumping capacity are not improved in this concept thus not meeting the requirement to make them robust for the future. Finally, this concept does not meet the requirements for water sports, since no space is reserved for training facilities, berthing for vessels and dry docks.

5.2. Alternatives

The following concepts were verified to agree with the program of requirements and are named alternatives. These alternatives will be evaluated in a Multi Criteria Analysis in the next chapter.

- Alternative 1: Less is More
- Alternative 3: Reunion of North and South
- Alternative 4: Tidal Park Scheveningen
- Alternative 6: Sea Farm
- Alternative 8: Haagvlakte
- Alternative 10: Inside Out

6 Multi Criteria Analysis

In the previous chapter, the alternatives were identified that match the requirements. In this chapter, Multi Criteria Analysis (MCA) will assess the alternatives based on the important criteria for stakeholders. These criteria are therefore partially adapted from the stakeholder analysis, but also from Department for Communities and Local Government: London 2009 and input from the authors of this document. When the criteria are established, weight factors are assigned to each criterion. This is done by creating sub-weight factors for the sub-criteria, and then creating weight factors for the main criteria. After these essentials are established, the final MCA results are shown.

6.1. Criteria

1. **Infrastructure**
   The infrastructure is an important element of the masterplan. Therefore, the different alternatives need to be valued in the various ways infrastructure has been implemented in those alternatives. Most sub-criteria for the infrastructure are derived from the municipality's vision on the accessibility of Scheveningen, as stated in the Stakeholder analysis. These sub-criteria are *public transport*, *traffic flow improvement*, *accessibility of the harbours* and the *internal reachability between the harbours*. The vision for the future of the municipality of The Hague is to improve the *public transport* towards, and along the beach. This criterion encloses the expansion of and addition of new bus and tramlines. The *improvement of the traffic flow* means that there will be less traffic jams within the city of Scheveningen. This criterion can be reached in multiple ways: road layout improvement, network adaptations or traffic management. The *accessibility of the harbours* is a criterion in addition to the criteria adapted from the municipality because a new harbour is designed. For this scenario, the degree of accessibility to the new harbour and increase of accessibility of the current harbours by new infrastructure should be accessed. Last but not least, the *internal reachability between the harbours* will be assessed based on the ability to move between the southern and northern parts of the harbour. This criterion also originates from documentation of the municipality, mentioned in the Stakeholder analysis.

2. **Cultural Heritage**
   The cultural heritage is a criterion that originates from the inhabitants, which are considered to be active stakeholders. As mentioned in the stakeholder analysis they have a lot of interest and quite some power as they are united through resident's organizations. The vision of these organizations is that the cultural heritage is to remain present as much as possible within the harbour and therefore the cultural heritage is an important criteria. It is divided into two sub-criteria respectively *Integration of urban design* and *Visibility of cultural objects*.

   Integration of urban design is an important criterion because of two reasons. Firstly, the inhabitants of Scheveningen and surroundings think it is important that the harbour is a part of their neighbourhood. Secondly, the urban design of the neighbourhoods surrounding the harbour are protected by either the government or the municipality. Therefore it is important that the harbour's aesthetics fit in with that of the surrounding neighbourhoods. *Visibility of cultural objects* is a criterion based upon the values of the inhabitants and the residents' organization. They find it important that the history of Scheveningen harbour remains present and visible within the harbour itself.

3. **Water sports**
The municipality of The Hague aims to become a global leader in the field of water sports. This criterion's value is based on the amount of area available for the practise of water sports and construction of training facilities. This holds under the assumption that more available area results in the construction of more facilities and more practise of water sports.
Good accessibility from the water sports center to the open sea is desirable, to further stimulate the practice of water sports. It is undesirable that water sports practitioners with jet-skis or sailing boats have to sail a long way from the docking station towards the open sea. Regarding the safety and the of these practitioners it is undesirable that they sail next or close to large fishing ships and mini cruises.

4. **Economic development**
The economic development of the harbour is a rather important criterion because this is one of the main drivers for the realisation of a fourth harbour.The measurement of well a alternative scores in terms of economic development is performed with three sub-criteria which are *Tourism*, *Capacity of the fourth harbour* and *Hospitality branch*.
Tourism (Attractiveness) is a rather broad definition and is expressed in many forms. This term encloses the recreational use of the area. For example the addition of nature can result in new walking and cycling roads that enhances the attractiveness of that specific area. Also the location of the fourth harbour affects the tourism. If the harbour is located in the north of the breakwaters it is easier for tourists to visit Scheveningen village or the peer, whereas tourists are dis-encouraged to do this if the harbour is located at the south. Namely, it is rather far and somewhat time consuming to walk from the south of the harbour to the north. *Capacity of the fourth port* is a large factor in the economic development of the area. This term includes the capacity of the marina and for that of larger fishing ships and mini cruises. The value for the criterion of the capacity of the fourth port depends on the amount of reserved area for the fourth port. It is assumed that more reserved area for the fourth port results in more capacity and therefore in more economic development. This is due to the fact that a larger port capacity results in the arrival of more mini cruises and fishing ships. This leads to more harbour activity which stimulates the economic development. *Hospitality branch (space and location)*

5. **Flood Defences**
The criterion flood defences is divided up into three sub-criteria, which are aesthetics and operational obstruction. As mentioned before in the Stakeholder Analysis, the community of Scheveningen Area holds great respect for the culture heritage and the way it is emitted visually from the current architecture. Flood defences often tend to be structures on a great scale, which relatively fast causes it to seem like a separate construction in the whole system. The score for the aesthetics part will therefore be based upon the size of the primary flood defence of the new system. Operational obstruction appeals to the way in which the flood defence might hinder the nautical accessibility of the port. Increasing the height of the quay walls will pose no obstruction for example while a sluice between the breakwaters hinders in- and outflow for the whole port. *Aesthetics* is in the current field of Hydraulic Engineering an important aspect. It is desirable that a hydraulic structure fulfills an aesthetic function besides its main operational function. A hydraulic structure can become a landmark in the scenery of the city if this criterion has been taken into account properly during the design phase. Examples, such as the central station of Rotterdam and the Maeslantkering which have become iconic for respectively Rotterdam and the port of Rotterdam. *Operational obstruction* is defined by the amount of time that a sub system cannot fulfill its operational function. In this project the function of a harbour is to provide docking possibilities for vessels. No new vessels can arrive and dock in the harbour if the sluice is closed and therefore an operational obstruction is present.This obstruction can occur for docking possibilities in the marina, larger fishing ships, mini cruises or for all three at the same time. The more obstructions present the higher the lower the score for the operational obstruction criterion. *Adaptability* is defined by how well the flood defences mechanisms can be altered to match the hydraulic conditions. This is a criterion because flood defences are designed on hydraulic conditions such as

water height, wave height and their respective return periods. However, values for these variables are mostly calculated and are therefore not exact. There is a probability that later in time it is discovered that these calculated values are wrong. Therefore to continue the guarantee with respect to flood protection, it is desirable that flood defences can be altered.

6. **Livability**
   Livability is a key criterion for the inhabitants of Scheveningen. Livability as a criterion contains the attractiveness to live or work in an area. this can be defined by the absence of nuisances and the presence of nice features like for instance nature. The presence of nice features is already included in other criteria and therefore left out in the livability criterion. This criterion therefore focuses on the minimisation of nuisances. A first nuisance that is treated within this criterion is the *visual nuisance*. *Visual nuisance* in this project mainly means obstruction of the sea view. *Noise nuisance* is the next form of nuisance that is taken into account for the livability. This sub-criterion can be caused not only by the proceedings in the fourth harbour, but also by an increase of traffic on the roads and waterways. The last sub-criterion taken into account to assess the livability is the *duration of construction*. This is important, because construction comes with many forms of nuisance (visual, noise, road blocks) for the direct environment.

7. **Investment costs** An estimation for the total investment costs is an important criterion to relate the benefits of the project to. Because the costs are not precisely estimated, the costs are assessed by scores, based on the different systems of the masterplan. Different weights can be given to the scores on the costs of the sub-systems. Operational costs are neglected in this MCA, because the masterplan's investment costs are presumed to be much higher.

8. **Nature preservation**
   The area of Scheveningen is situated next to the area which is defined by the Natura 2000. When the port gets situated at the south side of the current system, it will lay adjacent to the nature area. Some of the alternatives consist of building a tunnel underneath this area. The manner in which this area gets disturbed will indicate the score for this criterion.

9. **Technical feasibility**
   The aspect of technical feasibility is divided up into the following categories; flood protection, aquapuncture and infrastructure. The scores will be based on upon the difficulty of the engineering principles. If a alternative requires a specific tunnel for the sea to be connected to the canals in the hinterland, this alternative scores poorly in this aspect. The construction of tunnels in urban areas is often seen as quite an intense technical phenomenon.

10. **Building with nature**
    Building with nature is a relatively new alternative in hydraulic engineering which focuses on constructing systems using materials from nature itself. Ir. Ronald Waterman initiated the alternative. The manner in which the different alternatives utilise natural (in)organic material to (for example) build flood defences.

6.2. Scoring the criteria

The scoring of the criteria is done with a compensatory approach, which means good scores can eventually compensate for the lesser scores. In our approach, a value for each criterion is obtained for every concept and is determined by sub-criteria. The criteria are equal to the sum of the sub-criteria multiplied with the sub-criteria weight factors. This value is then multiplied by the criteria weight factor, these (sub)criteria weight factors are elaborated in the next section below. Values for sub-criteria are given by mathematically weighing up against each concept and lie between the 1 and 5 which is respectively the worst and best score. These values follow a ratio scale meaning that the intervals represent a ratio scale, such that a score of 4 equals a score of two times 2.

6.3. Weight factors

There are 2 types of weight factors used in this MCA: sub-criteria weight factors and main criteria weight factors. This distinction is made, because there is also a distinction between the main criteria and the sub-criteria which they are built up from. In order to create a meaningful score, the sub-criteria within one main criterion are weighted relative to each other, these weighted sub-criteria create the score for the main criterion. The main criteria are then also weighted relative to each other.

Sub-criteria weight factors are assigned so that they add up to 1.0, this is called the weighted sum approach. This approach is chosen because the values of sub-criteria for each main criterion are quite small, namely 0, to a maximum of 5. Thus, dividing the weight factors up to numbers that add up to one is structured.

Main criteria weight factors are weighted by taking one criterion as a base with weight factor 1 and relate the other criteria to this. This method is chosen because there is a total of 11 main criteria. Application of the same method here as for the sub-criteria weight factors would not be structured due to the low values this would result in.

Values that are used in the MCA for all weight factors are elaborated in Appendix C: Weight factors. Note that the values for livability might seem quite low for some designs, but all values between 1-5 are considered livable, they only score different for the degree of livability.

6.4. Scores

At last, the MCA criteria are scored. This is done individually by every member of this team. These individual scores were then put next to each other. Discussion about every score was then key to reach a consensus about the final score. This discussion lead to the MCA as presented in Figure 6.1. More detail about the scores that were given is elaborated in Appendix C: Scores.

Criteria / Design	1. Less is more	3. Reunion of North and South	4. Tidal park Scheveningen	6. Sea Farm	8. Haagvlakte	10. Inside out	Sub-weight Factors	Weight factor
Infrastructure	**2.3**	**4.4**	**3.15**	**2.9**	**3.4**	**2.7**		**1**
Public transport	5	1	3	1	1	1	0.15	
Traffic flow improvement	3	5	2	4	3	3	0.35	
Accessibility of the harbours (macro-scale)	1	5	4	3	5	3	0.35	
Internal reachability (micro-scale)	1	5	4	2	3	3	0.15	
Cultural Heritage	**2.125**	**1.75**	**1.375**	**0.875**	**1.375**	**1.25**		**0.5**
Integration of urban design	4	3	2	2	2	3	0.75	
Visibility cultural objects	5	5	5	1	5	1	0.25	
Water sports	**2.85**	**3**	**3**	**2.5**	**3**	**3.5**		**0.75**
Amount of space [m2] for the watersportscentre	3	3	3	3	4	4	0.6	
Reachability of open sea	5	3	3	2	2	3	0.4	
Economic development	**3.25**	**3.25**	**3.25**	**3.25**	**4.25**	**2.5**		**1**
Tourism (Attractiveness)	5	4	5	2	3	2	0.25	
Capacity of the fourth harbour	3	3	2	4	5	3	0.5	
Hospitality branche (space and location)	2	3	4	3	4	2	0.25	
Flood defences	**4.3**	**2**	**3.8**	**4.3**	**3.1**	**2.5**		**1**
Aesthetics	4	2	4	4	5	2	0.2	
Operational obstruction	5	2	5	5	2	2	0.3	
Adaptability	4	2	3	4	3	3	0.5	
Livability	**4.5**	**3.9**	**2.7**	**2**	**4.3**	**1.8**		**1**
Visual nuisance	4	2	3	2	4	2	0.3	
Noise nuisance	5	5	2	2	5	2	0.5	
Duration/hinder of construction	4	4	4	2	3	1	0.2	
Nature preservation	**5**	**1.275**	**2.325**	**5**	**3.225**	**5**		**0.75**
Preserved	5	2	4	5	4	5	0.7	
Compensated	0	1	1	0	5	0	0.3	
Technical feasibility	**4.25**	**3.75**	**2.5**	**4.5**	**3.5**	**3.5**		**1**
Flood protection	5	5	3	5	4	3	0.25	
Aquapuncture	4	5	2	5	5	5	0.25	
Infrastructure	4	1	2	3	1	5	0.25	
Accessibility of construction site	4	4	3	5	4	1	0.25	
Building with nature (4th harbour)	**2**	**1**	**3**	**3**	**5**	**1**		**1**
Investment costs	**3.75**	**3.5**	**2.75**	**4**	**2.25**	**3.5**		**1**
Aquapuncture	2	4	1	4	4	4	0.15	
Salinity intrusion	3	5	4	4	3	4	0.15	
Flood protection	5	4	3	4	1	2	0.3	
Infrastructure	5	1	3	4	1	4	0.4	
Totaal:	**34.325**	**27.825**	**27.85**	**32.325**	**33.4**	**27.25**		

Figure 6.1: MCA

6.5. Sensitivity analysis

In this section we describe the results of the sensitivity analysis on the weight factors. The goal of this analysis is to check if the result of the MCA is robust. The MCA is considered robust if by changing the weight factors for each criteria slightly, the result (the concept with the highest total score) does not change. Here, for each criteria the weight factor was varied with a range of 0.5 and steps of 0.05. For each change in weight factor, the total score of all concepts were plotted. Because the changes in scores by altering the weight factors are linear, a larger range can be extrapolated from these graphs.

The results of the sensitivity analysis are given in Appendix C.3. It is shown that the MCA is quite robust, since the *Less is More* alternative remains the highest scoring alternative when adjusting each of the weight factors separately. In the criterion *Building with Nature*, it can be seen that the *Haagvlakte* alternative approaches the score of the Less is More alternative when the corresponding weight factor is increased. This means that when *Building with Nature* is considered more important, and the weight factor is increased to more than 1.25, it can happen that Haagvlakte gets a higher total score. Moreover, if in addition to this the weight factor for *Economic development* is increased, the positive effect of *Haagvlakte* will strengthen, since Haagvlakte is also scores higher for this criterion, thus has a steeper slope of sensitivity.

6.6. Conclusion

One design alternative is chosen based on the MCA to continue with for a detailed design in the final report. To give more oversight, Figure 6.2 shows only the main criteria and the final scores of the alternatives. This overview shows that alternative 1: Less is more, robustly scores best overall, but it is closely followed up by alternatives 8: Haagvlakte and 6: Sea Farm. The final decision now has to be made on how to continue with for the final report.

Criteria / Design	1. Less is more	. Reunion of North and Sout	4. Tidal park Scheveningen	6. Sea Farm	8. Haagvlakte	10. Inside out
Infrastructure	2.3	4.4	3.15	2.9	3.4	2.7
Cultural Heritage	2.125	1.75	1.375	0.875	1.375	1.25
Water sports	2.85	3	3	2.5	3	3.5
Flood defences	4.3	2	3.8	4.3	3.1	2.5
Livability	4.5	3.9	2.7	2	4.3	1.8
Nature preservation	5	1.275	2.325	5	3.225	5
Technical feasibility	4.25	3.75	2.5	4.5	3.5	3.5
Building with nature (4th harbour)	2	1	3	3	5	1
Investment costs	3.75	3.5	2.75	4	2.25	3.5
Totaal:	34.325	27.825	27.85	32.325	33.4	27.25

Figure 6.2: MCA results

6.7. Discussion

The MCA is a suited method to compare design alternatives, as it gives global, qualitative insight. Because the exact values for conceptual designs are not known, a score from 1 to 5 is a suitable way of assessment. Although, MCA's in general are known to be sensitive for biased outcomes. A different scale for scoring, or a different way of implementing the (sub) weight factors (i.e. weighted product, (dis)concordance, regime analysis or pairwise comparison) could have led to a different conceptual design scoring better than others. Yet, the decisions are supported by the fact that a more precise scoring system would mislead the reader about the precision of this method. For weight factors, a different method could lead to polarisation of the scores. The chosen method is intended to be as transparent and unbiased as possible.
Missed criteria could also be a point of weakness in the MCA. Because the project includes a lot of different elements, the impact is also quite extensive. Department for Communities and Local Government:London (2009) was used to reduce the chance of missing important criteria.
Last but not least, the MCA scores are filled in by a homogeneous group of students with the same (civil engineering) bachelor background. Discussion about our individual scores showed that we tend to appraise big structural projects, whilst another person might not like this at all.

7 Final design

In this chapter, an improved alternative is presented that combines some of the design solutions of the best scoring alternatives on the MCDA. From the MCDA it was found that the alternatives Less is More, Sea Farm and Haagvlakte scored highest. The less is more concept is used as base since it was the highest scoring alternative on most criteria. This alternative will be optimized by adding elements from the other two alternatives.

7.1. Improved version of Less is More

Less is More is based on the idea of respecting the current situation and culture of the Scheveningen harbour area by making minimal changes. The main harbour expansion will be located seawards north-west of the current harbours. Another important aspect is the implementation of waterways and improving the current public transport network. The Scheveningen harbour will be connected to canals of The Hague and finally to the Binckhorst harbour in the hinterland, providing a new distribution network for fish.
From the MCDA results presented in Figure 6.2 from Chapter 6, the alternative Less is More scores relatively low on the criteria infrastructure and Building with Nature. The aim is to improve Less is more by the implementation of other alternative's elements.

First of all, the design solution for infrastructure of the Less is More concept is changed. The choise is made to use the infrastructure solution of the alternative Reunion of North and South. This alternative was best scoring on Infrastructure according to the MCDA. Note that one aspect is not being implemented into Less is More, namely the construction of a tunnel to connect the infrastructure at Kijkduin with the S200. This does indeed have a positive effect on the infrastructure, however it has been deemed unrealistic because the tunnel would have been constructed under the Westduinpark (Natura 2000 area) and under residential areas.

Building with nature can improved by implementing alternative's with high scores such as the Haagvlakte (score of 5). The Haagvlakte scored high because its breakwater is made of dunes and dikes and has as benefit that the 4th harbour could therefore be located within. This however cannot be implemented in Less is More as its fourth harbour is located closely to the 3rd harbour. Creating the breakwater by dunes and dikes would require a lot of investment costs with no extra purpose.

Finally, the quay walls around the second harbour and the harbour expansions on both sides of the breakwaters will be heightened. Also the aquapuncture of Reunion of North and South was incorporated due to the investment costs being severely lower than in the Less is More alternative. Figure 7.1 gives an overview of the improved less is more alternative.

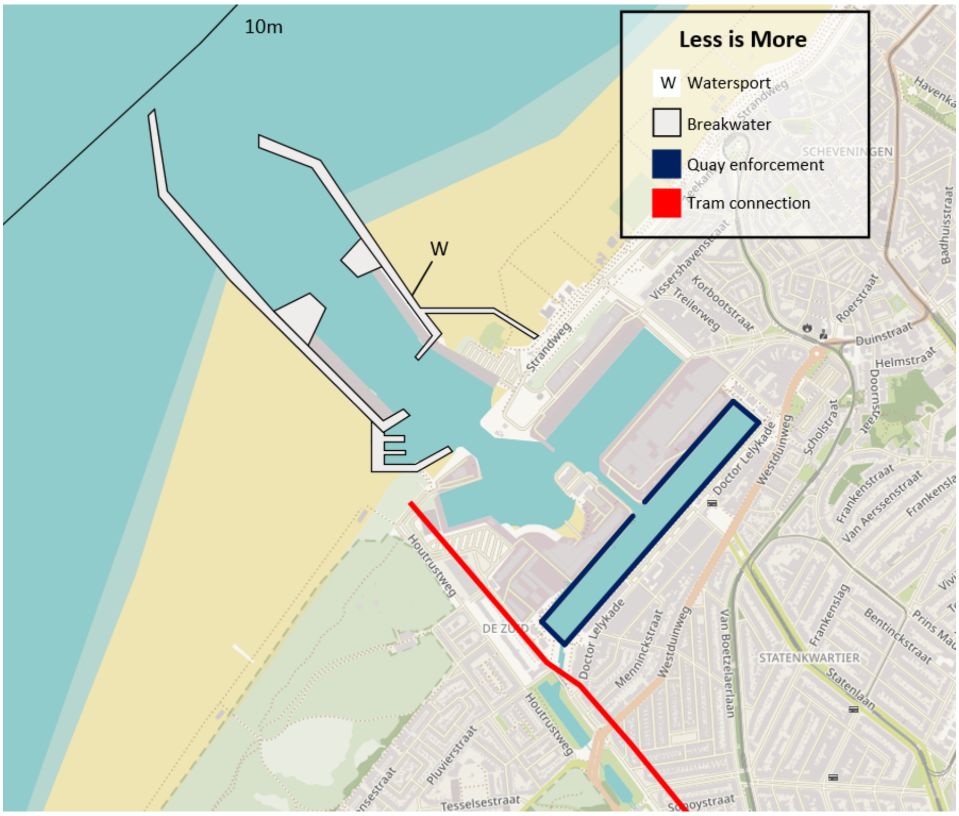


Figure 7.1: Overview of alternative Less is More (Openstreetmaps, 2017).

Design characteristics

Flood protection: Improving quay walls of second harbour

- Flood protection of the harbour area will be improved by elevation of quay walls around the second harbour.

- Flood protection of the beach stadium by a hybrid of retaining wall and dune. This protection of the beach stadium also forms a shortcut to the watersports centre which is located seawards of the stadium in line with the northern breakwater.

Harbour: Quay along the access channel

- Harbour expansion is located in line with the southern and northern breakwater.
  - Northern quay will serve purpose for the watersports centre, with berthing space for water sports related vessels and dry docks. More in the direction of the third harbour, there will be place for larger fishing ships.
  - Southern part will function as berthing place for small cruise ships and vessels for offshore industry.
  - At the landward end of the southern part, extra capacity for the marina will be available. The usage of the old-ship lock for aquapuncture reasons comes at the cost of capacity for the marina in the second harbour.
  - Both harbour expansions will be protected from oceanic currents by small extensions.
  - The watersports centre will have training facilities and a quay for berthing of watersports related vessels.

Infrastructure

Tram connection

- The tram will be extended from the Van Boetzelaerlaan towards the new harbour. Tram transfers will be provided towards the centre of The Hague.
- The improvement of aquapuncture partially reliefs the road's usage as well as the usage of tram. Therefore goods can be transported via waterways to the harbour in Binckhorst. From there it can be easily transported via the road as it is located next to the highway. Therefore no extra roads are added near the Scheveningen harbour.

Broadening Houtrustweg and Kranenburgweg

- The Houtrustweg and Kranenburgweg are broadened, they are connected to the S211 towards Kijkduin, which leads to disclosure form the city (see Figure 3.11)

Reunion of north and south

- A pedestrian bridge will be constructed to connect the north with the south.

Aquapuncture: Small boats through the Verversingskanaal

- For aquapuncture, the control sluice has to be adjusted which is located at the end of the second harbour. The following adjustments will be made to connect the second harbour with the canals of The Hague.
  - The door of this sluice will be removed such that the Verversingskanaal is connected with the second harbour.
  - The Duindorpdam has to be altered for allowing passage of small boats.
  - A ship lock will be constructed next to the pumping station to allow for passage of small boats.
- Removing the door of the sluice at the Verversingskanaal will have the following consequences:
  - Basins will no longer act as storage of fresh water
  - Salt water will intrude into the current basin areas
- To protect the area around the Verversingskanaal, the height of the quays have to be increased. Also an impermeable layer has to be added to halt salinity intrusion.

Salt intrusion: Storage in dunes

- An infiltration pond will be designed in the area of Scheveningen. A transportline from Wastewater treatment plant Houtrust will discharge into the infiltration pond. Due to already having a line that discharges far into the North-Seam a bifurcation point should be made such that when the storage is full, the discharge will be lead into the North-Sea.
  - The infiltration pond will be created in the dunes for allowing recharge of the water lens in dry and wet conditions.

7.2. Detailed design elaborations

In the next phase of the project, some design solutions of Less is More will be designed on a more detailed level. Due to time constraints not all design characteristics could be elaborated. The choice is made to elaborate a few characteristics with more quality rather than elaborating everything broadly. The chosen characteristics match the respective disciplines of the group working on the case. The following characteristics were chosen to work out in further detail in this document:

- Quay wall design
  - Elevation of the quay walls around second harbour
- Breakwater design
  - Extension of breakwaters to the 10 m line
- Port layout
  - Dimensions of the access channel
  - Design of berthing facilities
- Watermanagement
  - Recharge of the fresh water lens
- Infrastructure
  - Tram connection between The Hague and the fourth harbour

The following chapters contain the topics in the same order as the list mentioned above. Per chapter, the dimensioning steps will be supported with calculations and visualisations. To retain order in the document, more complicated calculations are placed in the Appendix.

8

Quay wall design

8.1. Introduction

The Doctor Lelykade is probably the weakest subsystem of the port of Scheveningen. It flooded twice in the last ten years ,which is not in accordance with the Dutch flood safety philosophy. This can be explained due to the fact that the quay wall lies relatively low. According to the Actueel Hoogtekaart Nederland, the quay wall at the Tweede Haven has the lowest elevation of all the ports. In an earlier phase of this project, the decision was made to increase the height of the quay wall in order to decrease the probability of flooding. The quay wall needs to perform the following functions:

- Enabling berthing and mooring of ships.
- Retaining of soil and water.

The first defined function is already satisfied by the system. The problem lies in the second function.Both of these are directly correlated with structural functions as well. In this phase of the design, the quay wall will also be design regarding the structural mechanics. The problem concerning the current subsystem can be defined as follows:

The height of the current quay wall of the second port of Scheveningen does not meet the requirements defined in the Dutch national flood safety assessment.

8.2. Basis of the design

Whereas the design on urban level followed the method proposed by Voorendt, the structural level design will on the contrary ’simply’ follow the usual hydraulic engineering method. Therefore the requirements, boundary conditions and the starting points will be defined prior to designing the concepts. Because the design merely concerns an increase in height of the quay wall, only one requirement will be defined. This requirement will indicate the frequency in which the quay wall may fail. The frequency itself may be based upon the regulations given by the Dutch safety assessment.

8.2.1. Boundary Conditions

The current boundary conditions are responsible for the current frequency of flooding of the Doctor Lelykade. The boundary conditions are generally based upon probabilistic calculations done in Python, of which the details can be found in the Appendices.

- The first effect of the wind is a storm surge increase in the port. The storm surges require an elevation of +2.8 meters with respect to NAP.
- The second effect of the wind is the cause of wind waves which may over top the quay wall. The design wave height equals 0.20 meters.
- Climate changes causes a permanent sea level rise, which is directly ’felt’ in the port. The sea level rise is based upon a research done by Deltares, which states that the sea level rise the ongoing 100 years will be 1 meter.

- The quay wall is located directly at the Doctor Lelykade. This street houses several restaurant, which act as physical boundaries. Google Maps was used to compute the shortest distance between the port and facilities. This distance is 4 meters.

Combination of the assumption leads to the following statement; the water level in the second port of Scheveningen is governed solely by the tide and by wind waves.

$$h_{wall} = h_{storm} + \Delta d_{waves} + \Delta d_{sea} \tag{8.1}$$

8.2.2. Requirements

The requirements regarding national Dutch flood safety are not met with the current quay wall as it is. Therefore a program of requirements will be set up in order to develop different concepts, which will later on be compared to one another. The requirements mainly follow from the functions, but are elaborated more in detail such that concrete numerical values can be substituted in the concepts.

1. The new quay wall requires an elevation height of +3.65 meter with respect to NAP, according to Equation 8.1. Waves forming in the port will then over top the quay wall once in 200 days on average and flooding once in a hundred years. This implies a design water level of +3.55 meter NAP because the waves are not taken along.
2. The quay wall needs to be accessible from both sides. This implies that people need to be able to access it after docking their ships, and to access their ships from the Doctor Lelykade.
3. The quay wall needs to fulfill the structural requirements in terms of strength, stability and stiffness.

Figure 8.1 shows an indication of the current quay wall and elevation heights of relevant elements. The derivation/sources can be found in the relevant appendices.

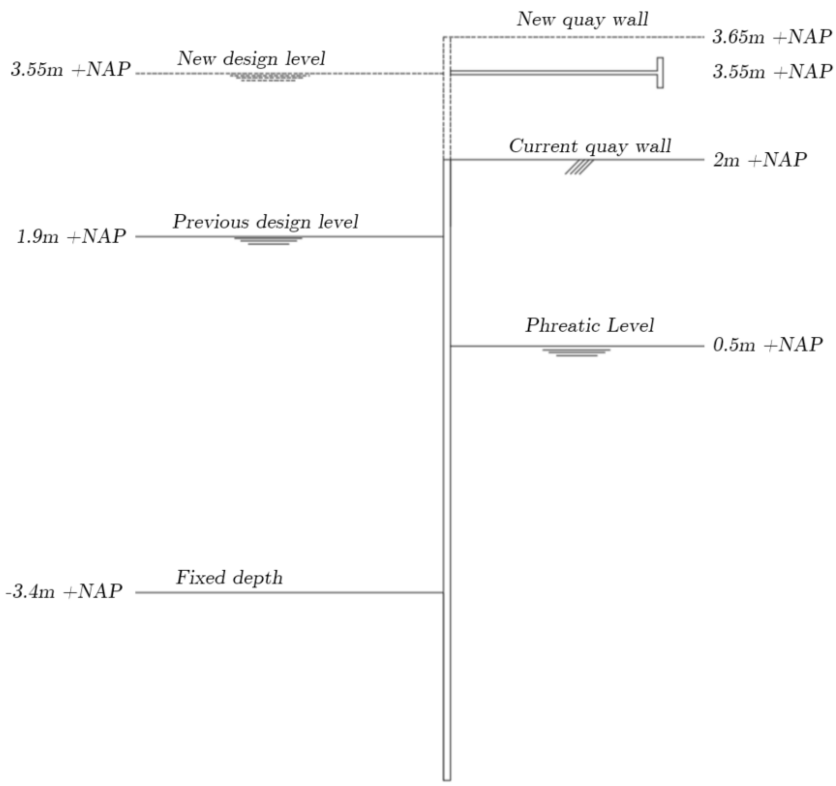


Figure 8.1: The assumed configuration of the quay wall in its current state along with the required heights. The figure is only an indication of elevation heights and no aesthetic impression of the to be designed system.

8.3. Development of concepts

After the definition of the requirements and the boundary conditions, concepts were developed. The concepts all naturally fulfill the requirements, but do this in different manners. Certain aspects which were taken along in the development, are the available space on the Doctor Lelykade, and the space (thus capacity) in the port. One can choose to reduce one to increase (or safeguard) the other. Bearing this in mind, the following ideas were created as foundation for the concepts.

1. The first concept aims at increasing the height of the full Doctor Lelykade, including the part used by traffic (cars and bicycles).
2. The second concept aims at creating more mooring and docking space at the expense of port capacity (area) by placing a gravity wall in front of the current wall.
3. The purpose of the third quay wall is placing a plateau in front of the current quay wall, supported on a concrete compression pile.
4. The fourth option tries to fulfill the new requirements by placing a new concrete structure on top of the current quay wall, thus at the expense of space on the current Doctor Lelykade.

All the concepts will be checked upon their ability to fulfill the requirements, this means that til a certain degree, their strength will be calculated (amount of details differs per concept). In the more global phase of the project, an MCA was used in order to evaluate the urban level concepts. Because this was already done, this design will solely incorporate a qualitative analysis in order to choose one concept which will be worked out more extensively.

8.3.1. Concept 1: New quay wall and street elevation

Figure 8.2 shows a global overview of the first concept. As specified before, in this concept the elevation of the whole street is increased. This implies that the traffic on the street of the Doctor Lelykade is elevated as well. A new quay wall will be built in front of the current one, which does require a dry construction situation. The quay wall itself needs to be tested upon strength and stability. Regarding the available time for the project, the stability was not accounted for. The situation in which the quay wall is loaded the most intense, is the one in which it operates under Lowest Astronomical Tide (Appendix E.2.). In this configuration the quay wall is exerted with the greatest bending moment.

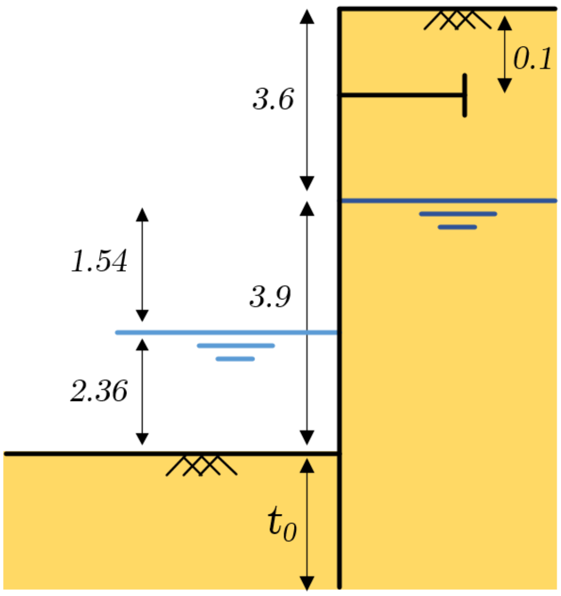


Figure 8.2: A basic overview of concept 1, in which the full Doctor Lelykade is elevated upwards.

The calculations regarding strength and stability were done using the Hydraulic Structures Manual (M.Z. Voorendt, 2019). Multiple methods are available as to model the sheet pile, the most occurring ones are Blum's method and the American method. The manner in which they differ from one another is the way in which the bottom section of the sheet pile wall is "fixed" in the earth. Blum's method assumes no rotation at the bottom, and a horizontal support (the anchor) causing the structure to be statically indeterminate. The American method assumed no horizontal support in the lower section because the active and passive soil pressures balance one another. The latter method was used, as it is an easier method in general. The calculations were all done by hand, so that the embedded depth t_0 could be left arbitrary (causing the balance between active and passive soil pressure). The calculations can be found in the Appendix. The required section modulus however is stated in Figure 8.3.

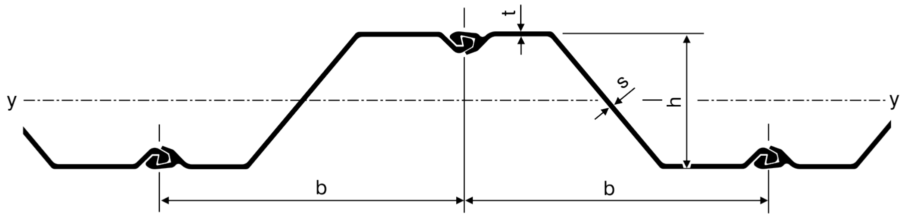


Figure 8.3: A schematic overview of the profile section which will be used as the wall.

Characteristic	**Value**
Section Modulus	1600 cm3/m
Weight	118 kg/m2
Second moment of inertia	28000 cm4/m
Section width	575 mm
Wall height	350 mm
Back thickness	9.2 mm
Web thickness	8.1 mm

The section modulus of the chosen profile is larger than the required one, which implies that in terms of strength the design of the quay wall should be sufficient. For stability however the anchors need to be checked upon their strength, as the anchors provide the forces for horizontal force equilibrium. The design of the anchor will be done in the case when this concept is chosen for the final design.

8.3.2. Concept 2: Gravity wall

This concept will aim at minimizing the amount of changes to the Doctor Lelykade while still fulfilling the retaining role with respect to the water level rise. The concept which aims at installing a new quay wall in front of the current one, also aims at increasing the elevation of the whole Doctor Lelykade. Enheightement of the street as a whole can be quite an operation, therefore this concept tries to build more seaward instead of landward. Figure 8.4 shows an overview of the new situation.

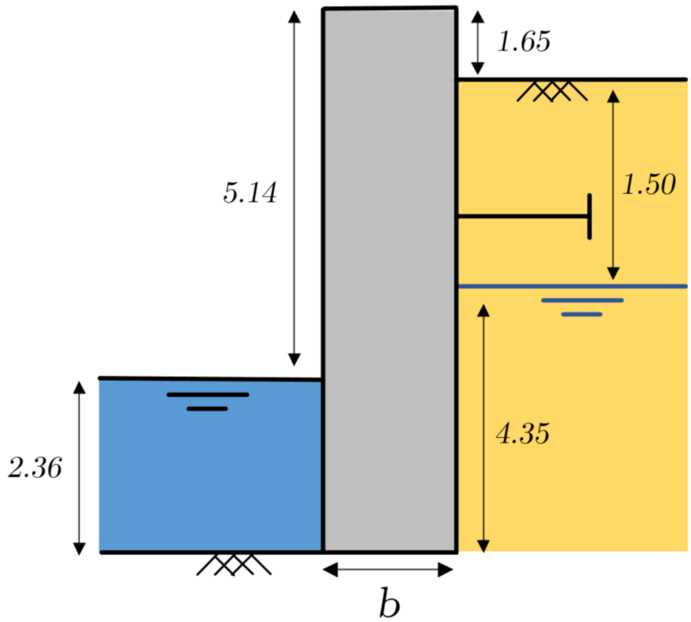


Figure 8.4: A schematic overview of Concept 2 in which the quay wall is heightened and broadened.

The main idea behind this concept is that a new wall gets constructed some meters in front of the current one, after which the gap is filled with concrete. The advantage of this concept is that no changes are made to the Doctor Lelykade and more functional space becomes available for docking and mooring. A disadvantage is that the basin size of the port gets reduced dependent upon the (on paper) width of the concrete slab. Two obvious choices come to mind with a such a structure:

- The first option is the construction of a gravity wall. Gravity walls derive their stability from a combination of their shape and mass, according to the Hydraulic Structures Manual.
- The second option is by placing a caisson next to the current quay wall.

Due to easier calculations, the first option was chosen. The calculations regarding the strength and stability can be found in Appendix D.

8.3.3. Concept 3: Compression piles

The third concept will also try to aim at expanding the quay wall functional area by extending it 'seawards'. It will however reduce the amount of (concrete) material required by constructing a compression pile for the quay wall to rest on. Different failure mechanisms may now occur, for which the structure needs to be checked. In this configuration the compression pile also needs to checked upon the strength of the pile itself as well as the bearing capacity of the pile-soil system. Buckling of the compression pile may also occur if the compressive force becomes too great. Figure 8.5 shows an overview of this situation.

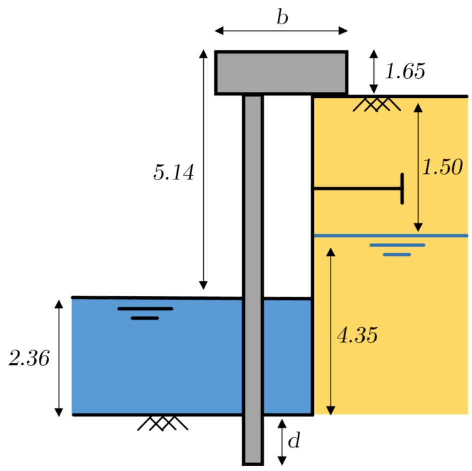


Figure 8.5: Schematic overview of Concept 3 in which the new quay wall rests on top of a compression pile.

A challenge in this approach is thus the design of the compression piles. The calculations will be done observing the cross section, such that forces are expressed per unit depth. The difficulties in the design will then lie in the dimensions of the piles and the amount of them (these may of course compensate each other). For the detailed calculation one can refer to Appendix D.

8.3.4. Concept 4: L-shaped wall

The fourth concept will try to solve the problem by minimizing the amount of material necessary to retain the new design water level. No new quay wall will be made available which was the case for Concept 2 and 3 however. Therefore it has been chosen to reinforce the flood safety in this concept by constructing an L-shaped wall on top of the quay wall. The vertical part will then retain the water from the port and the horizontal part will function as a stability element. One disadvantage of an L-shaped wall is that the horizontal part needs to rest upon the Doctor Lelykade, hindering traffic on top of it. Concrete blocks or concrete caissons could also be chosen but these need to be supported rotationally fixed which might pose a problem. Figure 8.6 displays the 2 mentioned possibilities.

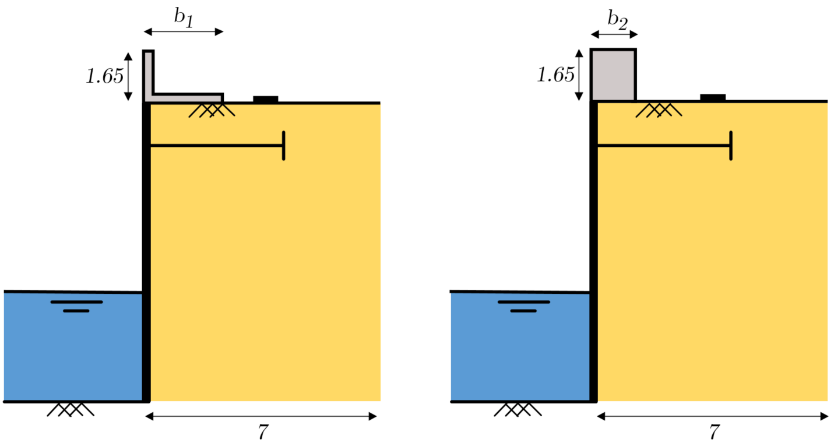


Figure 8.6: Schematic overview of Concept 4 in which the new quay wall is constructed using either an L-wall, a concrete block or small caisson.

An assumption which will be made in this concept is that the construction within the current quay wall is able to withstand the increase in water level. This would imply that only the new element has to be tested upon strength, stability and stiffness. The construction of the concrete blocks would unfortunately contradict with a certain part of the Doctor Lelykade, as is shown in Figure 8.7.

Figure 8.7: The current configuration of a part of the Doctor Lelykade. One can obviously see how a construction on top of the current quay wall would impose problems for stakeholders making use of the quay wall. The small black boxes indicate the/a physical boundary between the driving lanes. This is however a small part of the area. (Google Street View)

The case with putting the blocks on top of the quay wall is that the blocks need a certain width b in order to withstand the moment from the water, and to attain stability. This however counteracts the operative functionality on top of the wall. All these considerations will be taken into account however when a concept is chosen.

A difficulty in this concept is the fact that no information is available about the current configuration of the quay wall. Therefore it is not possible to say anything about whether the current quay wall construction will endure the new design water level. The concrete blocks on top of the quay wall would also put an extra weight on the quay wall, forcing a compressive pressure on it. Considering the space on top of the quay wall as mentioned on the previous page, Figure 8.8 gives an idea of the main configuration of the Doctor Lelykade (Figure 8.7 solely shows the setting of a small part).

Figure 8.8: The current configuration of the other part of the Doctor Lelykade. (Google Street View)

One can see that part of the street (at the waterside) might be turned into higher quay wall by putting the blocks on top of it. A stairs construction might also make 'traffic' possible. This would however imply that lateral traffic (parallel to the long side of the port) is not possible anymore. This will be worked out in smaller detail when the concept gets chosen for the final design, as these aspects were not part of the program of requirements. The stability of the construction however is, so this will be worked out in further detail. Figure 8.9 shows the internal forces in the concrete member.

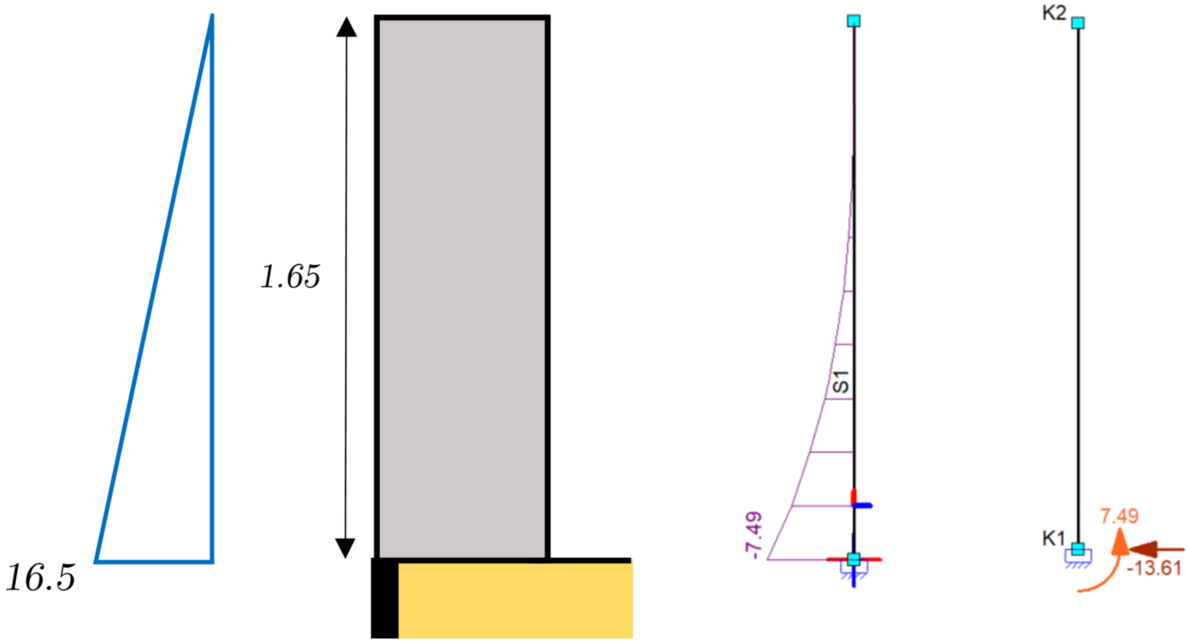


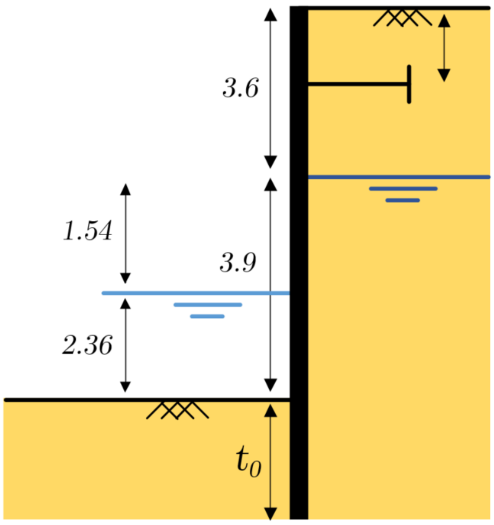
Figure 8.9: The current configuration of the other part of the Doctor Lelykade. (Google Street View)

The forces in the concrete member are not significantly high, so it will be assumed that they can be withstood. More detailed elaboration on the design will happen when the concept gets chosen.

8.4. Evaluation of the concepts

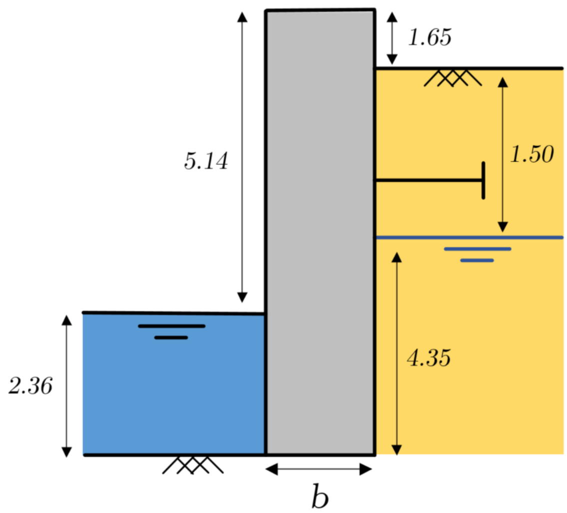
After designing the concepts on a global scale, one of them has to be chosen as the best alternative. In an earlier stage of this project, this was done using a quantitative Multi Criteria Decision Analysis. This is quite an intensive method, so in this case a qualitative method was chosen. The concept will be summarised once more along with the advantages and disadvantages, after which one will proceed to the final design.

Concept 1



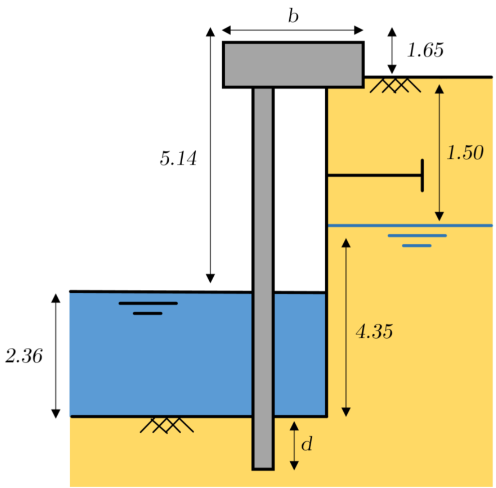
- The first concept focuses on increasing the elevation height of the full Doctor Lelykade. A new sheet pile will be designed which will create force balance between active and passive soil pressure, along with an anchor creating horizontal force equilibrium.
- An advantage is that no space of the port is used. On the other side the construction will take a long time and will be very expensive. The Doctor Lelykade itself cannot operate during the construction. Some stakeholders will suddenly encounter an enheightened street in front of their door. (Part of) the port needs to be pumped dry for the construction.

Concept 2



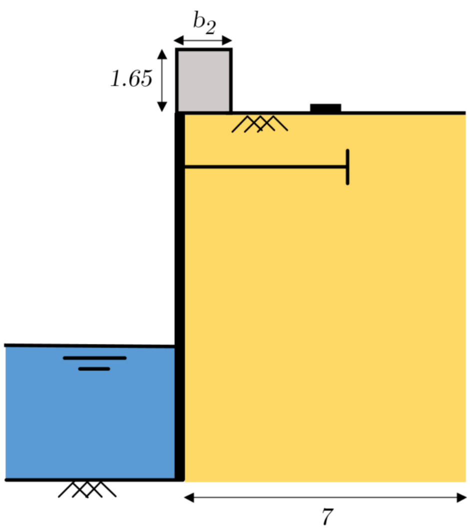
- The second concept aims at retaining the increasing water height by placing a gravity structure between the port and the current quay wall. The top of the structure can this way also be used for docking of ships.
- An advantage is that no changes are made to the Doctor Lelykade. A disadvantage however is a decrease in basin size and costs of structures with these dimensions usually tend to be quite high. Again (part of) the port needs to be pumped dry for the construction.

Concept 3



- The third concept in essence does the same as the second one, only without using as much material. The quay wall is again enheightened using concrete, while it rests upon a concrete compression pile.
- The advantage from Concept 2 applies here as well. This concept is actually an improvement on the latter one as it tries to minimize the amount of material necessary for the new quay wall to rest one. However the construction costs will be fairly high and the wall cannot be used when under construction.

Concept 4



- The fourth concept aims at retaining the new water height by placing the simplest structure possible on the current quay wall, while fulfilling the program of requirements. No connection between current quay wall and new structure has been defined.
- An advantage is that no space of the port is used. The Doctor Lelykade is affected dependent upon the width of the structure (in plane). The construction costs will however be quite low and part of the quay can be used when under construction. A disadvantage may be that part of the wall cannot be used for car (and bike) traffic anymore.

Comparison of all of the different concepts leads to a fairly straightforward decision. When the area of interest flooded, the damage was not that high. As this is the case, one could say that no extreme measures need to be taken in order to fulfill the requirements. Therefore the decision was made to proceed with the fourth concept, the simplest one. One could however argue that the structure hinders traffic on that part of the road, but this can easily be refuted by considering the increased flood safety. Hinder of the view is true, but this is the case for all the concepts, as there will surely need to be a structure with the predefined elevation height.

8.5. Final Design

Now that the fourth concept has been chosen, several details need to be worked out, which consist of the following:

1. The structure itself needs to be checked upon strength, stability and stiffness thoroughly.
2. The connection between the structure and the soil/ground upon which the structure rests needs to be modelled well. Because of the manner in which the water pressure acts on the structure, it will tend to rotate. This can be counteracted by the weight of the structure or because of soil-structure interaction.
3. The enheightened quay wall must be accessible from the part with the lower elevation. This can be realised using stairs or such. Some parts of the Doctor Lelykade do not allow for this however.

At first, the horizontal stability will thoroughly be checked. The horizontal forces working upon the structure are the hydro static pressure from the (highest possible) water level and the incoming wave force. Again, the design water level will be taken into account, along with a wave height with return period of 200 days. A safety factor has been applied to the water level, in the case that ships cause the water level to rise even more. Figure 8.10 shows the situation of the final design. Keep in mind that the left part of the quay wall (now used for walking and biking) has a width of 7 meters (Google Maps), which is quite big as it is the solution space).

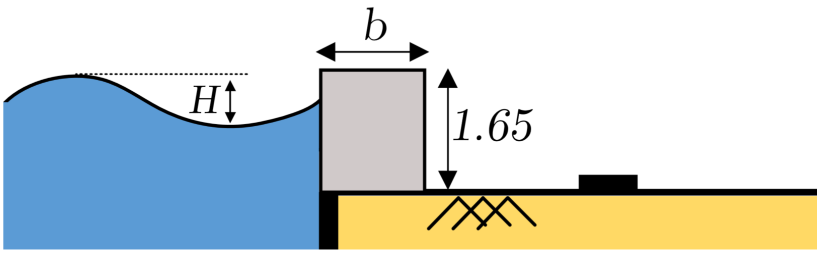


Figure 8.10: The mechanisms which cause horizontal forces on the structure.

$$F_{wave} + F_{hydrostatic} < f \cdot \gamma_c V$$

For the horizontal wave force, the most conservative expression from the Hydraulic Structures Manual was used.

$$F_{wave} = \frac{1}{2}\rho g H^2 + d\rho g H \tag{8.2}$$

Because the weight (or the volume) is dependent upon the width of the structure, the width can be left arbitrary in order to calculate the minimum value. For the hydrostatic pressure, a safety factor of 1.5 was taken along. This was not done for the wave force as H was computed using a fully probabilistic method. Equation D.6 is immediately used here to obtain the required width (and thus cross section in plane).

$$\frac{1}{2}\rho \cdot 9.81 \cdot 0.^2 + 1.65 \cdot \rho \cdot 9.81 \cdot 0.20 + 1.5 \cdot 13.61 < \gamma_c \cdot 1.65 \cdot b$$

When the water density $\gamma = 10\mathrm{kN/m3}$ and the concrete density $\gamma_c = 25$ kN/m3 are substituted one obtains the following which allows the requirement of horizontal stability to be satisfied.

$$b_{min} = 0.61 \text{ meter or } A_{min} = 1 \text{ m2}$$

In the earlier concept design, the wave force was not taken along, and the pressure was taken over the full height of the structure. This is not right, because the pressure works from the design water level (hence 0.10 meters down the top of the structure) to the bottom of the structure. Therefore, Matrixframe was used once more in order to calculate the internal forces in the concrete structure (still neglecting the cross section). Figure 8.11 shows the results obtained in Matrixframe.
Now one can prove that the rotational stability requirement is not met when one uses the predefined required in plane cross section. Equation D.7 is hence again used (the weight of the structure is equal to the specific concrete density multiplied by A.

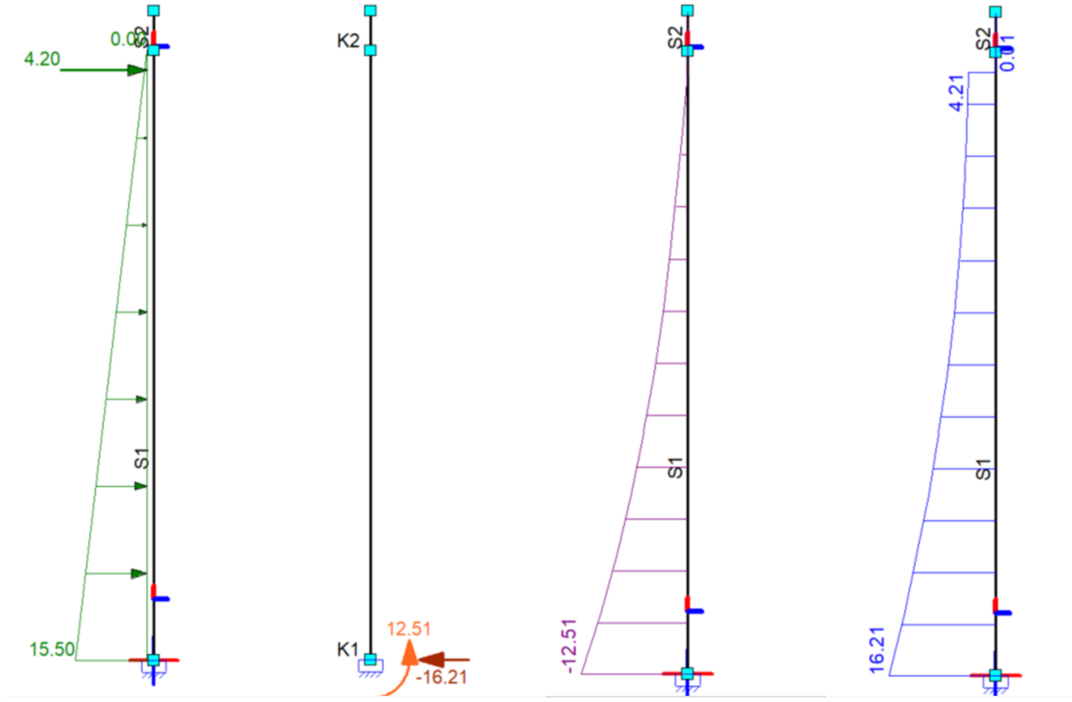


Figure 8.11: The internal forces in the concrete element when it is assumed to be rotationally fixed at the bottom.

$$e_R = \frac{\sum M}{\sum V} = \frac{12.51}{25 \cdot 1} = 0.5 \text{ meter} > \frac{b_{min}}{6}$$

The rotational stability requirement is therefore not met which means either the gravitational force must become greater or the width must increase. Both of these phenomena happen when the dimensions of the structure are increased. Therefore the structure dimensions are increased, the shape as well. As mentioned earlier, an idea rose to construct the quay wall in the form of stairs, so that the third detail is on the previous page is taken along as well. This will immediately cause the structure to be significantly wider, but as stated before (and to be seen in Figure 8.8), 7 meters of loss can be justified quite well.

From "Bouwbesluit", the regulations regarding stairs were sought up. It turns out that the maximum height of one step should be 188 millimeters and that the width should be between 210 and 270 millimeters. The ships will need some space for docking, so 2 meters are chosen for this purpose. this means that 2 meters landwards of the structure the stairs structure will begin. Nine steps were chosen, which means that the height of each step should be 180 millimeters. The width is chosen to be 220 millimeters, which means $9-1$ horizontal small plateaus.
Figure 8.12 gives an indication of the quay wall stairs design. The corresponding width turns out to be $2+8 \cdot 0.18 = 3.76$ meter. The corresponding area can be calculated by computing a Riemann integral. The computed area turns out to be bigger than 1 square meter, which implies the horizontal stability criterion is satisfied.

$$A = 2 \cdot 1.65 + \sum_{i=1}^{8} 0.22(1.65 - 0,18i) = 4.76 \text{ m2}$$

One can now calculate whether the structure fulfills the rotational stability criterion. For this exercise, the horizontal position of the normal force center was calculated, as the structure does not have a homogeneous shape. The expression according to (M.Z. Voorendt, 2019) states to calculate the length

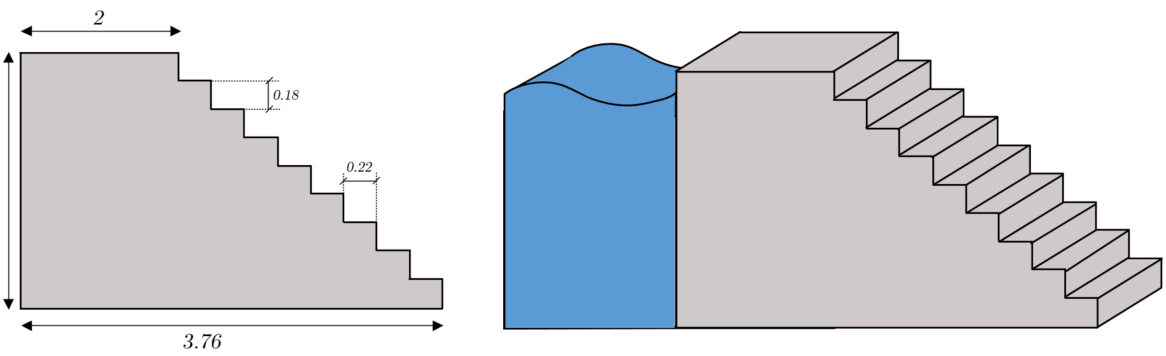


Figure 8.12: The stairs structure width the predefined (inter)dimensions shown.

of the core, assumes a rectangular shape. Therefore an equivalent width will be used here, as if the stairs structure were a rectangle. Figure 8.13 shows these calculations visually.

$$x_{NC} = \frac{\sum_{i=0}^{0} 0.18(2+0.22i)\cdot(0.5(2+0.22i))}{A} = 2.33 \text{ m}$$

$$b_{eq} = \frac{4.76}{1.65} = 2.975 \rightarrow \frac{1}{6}b_{eq} = 0.495 \text{ m}$$

Given the new, especially bigger form, the gravity acts with a greater force upon the structure, which causes the eccentricity to become smaller with respect to the previous rectangular shape.

$$e_R = \frac{\sum M}{\sum V} = \frac{12.51}{25\cdot 4.76} = 0.1 \text{ meter} < \frac{b_{eq}}{6}$$

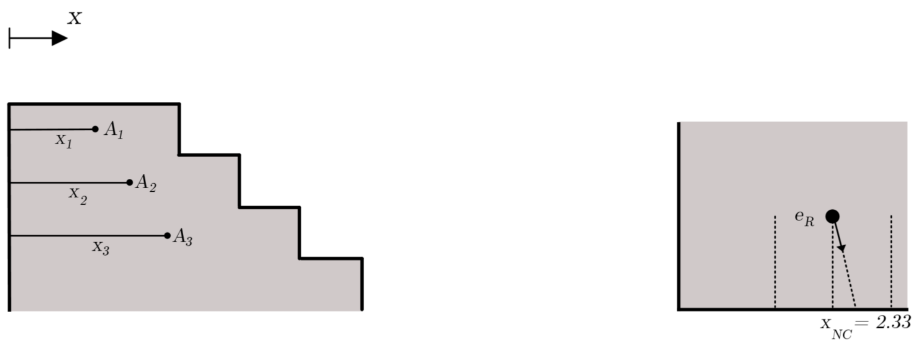


Figure 8.13: The left picture displays the manner in which the position of the normal force center was determined (by using static equilibrium). The right image shows why the structure will not rotate.

In order to satisfy the vertical stability requirements, Equations D.8 and D.9 can be used in order to compute the maximum load and minimum load on the soil respectively. Both of the criteria (M.Z. Voorendt, 2019) are met which means that vertical stability is satisfied.

$$\sigma_{k,max} = \frac{|25\cdot 4.76|}{3.76} + \frac{|12.51|}{\frac{1}{6}\cdot 3.76^2} = 37kPa < 400kPa$$

$$\sigma_{k,min} = \frac{|25\cdot 4.76|}{3.76} - \frac{|-12.51|}{\frac{1}{6}\cdot 3.76^2} = 26kPa > 0kPa$$

The tension forces in the structural member can be calculated using the general formulas of structural mechanics (Hartsuijker, 2010). In the formula the second moment of inertia is transformed to the section modulus to simplify computations. The tensile stresses in the members turn out to be really small, which is not unnatural given the dimensions of the structure. Hence no steel reinforcement is necessary in order to withstand the defined loading.

$$\sigma_{max} = \frac{M_z e_b}{\frac{1}{12}bh^3} = \frac{12.51 \cdot 10^6}{\frac{1}{6} \cdot 1000 \cdot 2760^2} << 1\text{kPa}$$

The design of the quay wall thus suffices in terms of strength. The next element which needs to be investigated is the current quay wall itself. Compared to the current situation, the wall will need to be able to support more loading, as extra elements are added to the system. For starters, the water level will increase causing the water pressure to be greater (at every vertical coordinate). Second, the new concrete quay wall system will put a vertical force on the quay wall, via the horizontal soil pressure (which will be the result of an increased surcharge).

Current quay wall

As specified before, the new concrete structure on top of the quay wall will exert extra loading on the current quay wall. The current configuration and dimensions are unfortunately not known, so the required profile will be determined after which a conclusion will be drawn. In the first concept, the manner in which horizontal stresses are to be computed was elaborated in great detail. Therefore this will not be done in this final design. The only aspect which is specific in this design, is the fact that the concrete wall acts as an extra surcharge load on top of the quay wall. Figure 8.14 shows an overview of the working forces on the quay wall in the new configuration.

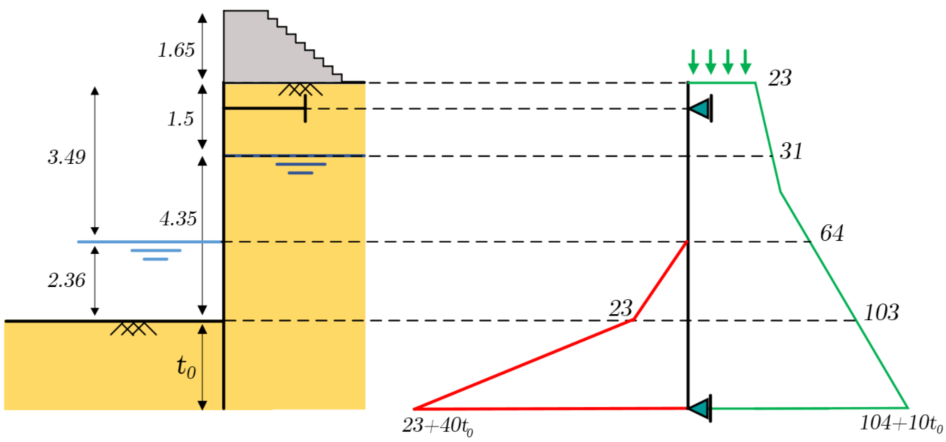


Figure 8.14: An overview of the forces working on the quay wall. The most heavy loading situation is again the one in which the water levels equals Lowest Astronomical Tide. The numeric values indicate the total horizontal pressures, which were derived using the method as specified beforehand in the chapter.

As one can clearly see, the embedded depth of the quay wall is left open, because it needs to result in a balance between active and passive forces. Matrixframe was used in a manner in which the depth was determined iteratively until the lowest part of the wall did not contain any shear forces (hence no reaction force). The final embedding depth which results in soil force equilibrium turned out to be 6 meters. The mechanic diagrams are shown in Figure 8.15.

As specified before, regarding forces the bending moment will be assumed as the failure mechanism. In most of the quay wall cases this is the case so this is a safe assumption. According to Matrixframe the maximum moment in the sheet pile is 620 kNm, which is almost twice the value as calculated for the first concept in which the whole street was elevated. The sheet pile anchor must able to resist a force of 273 kN. Regarding the bending moment, the following section profile is necessary to withstand it:

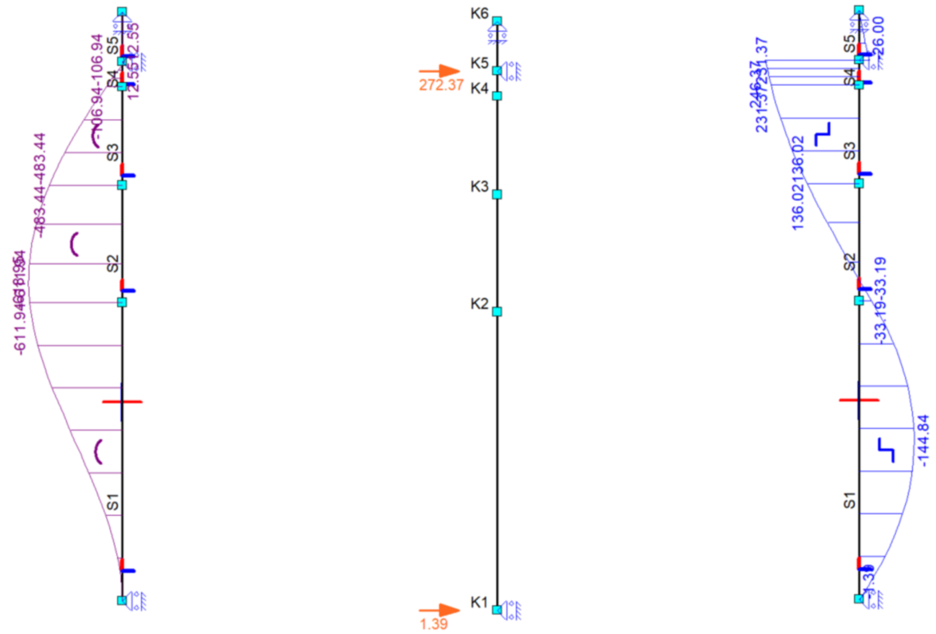


Figure 8.15: The moment diagram, reaction forces and shear force diagram shown from left to right.

$$W_{z,req} = \frac{M_{z,max}}{\sigma_{max}} = \frac{620 \cdot 10^6}{235} = 2638297mm^3 = 2638cm^3/m$$

Because no details regarding the current configuration are given, one can not say whether the system will be able to fulfill these conditions. The discussion of the project will elaborate this aspect in further detail. For now the required dimensions or materials of the elements will be calculated.
To maintain stability, an anchor needs to be constructed in order to maintain horizontal force equilibrium and force equilibrium. This tie rod needs to be checked upon strength, stiffness and stability. Regarding the strength, the diameter of the rod should be sufficiently big to withstand the force of 273 kN. Assuming steel is used having an elastic yield strength of 235 N/mm2, the following calculation can be made:

$$A_{req} = \frac{F_{anchor}}{f_y} = \frac{273 \cdot 10^3}{235} = 1162mm^2$$

$$d_{req} = \sqrt{\frac{4A_{req}}{\pi}} = 40mm$$

The sheet pile anchor must also be resisted by the soil pressures. Figure 8.17 shows the manner in which this happens physically.
To be positioned stable in the soil, the soil pressures must be able to form an equilibrium regarding the sheet pile. Unfortunately, due to lack of time, this could not be done in great detail. An indication will however be given which this can be calculated. The resisting soil forces are the sum of the the friction between anchor and soil and the soil forces acting upon the plate.

1. The friction between the anchor and the soil can be computed using the Cone Penetration Test. The value at the elevation of the anchor determines the value.
2. The active and passive soil forces can be computed by considering the which has been used before.
3. The pressures need to be transformed to forces by multiplying them with the corresponding areas.

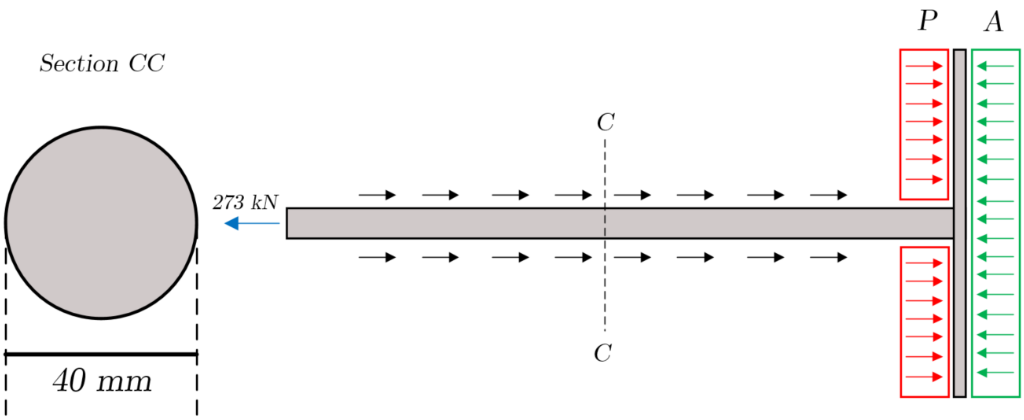


Figure 8.16: The cross section of the sheet pile anchor and the configuration of it. The passive soil forces are shown in red and the active forces in green. The black arrows indicate the friction between the anchor and the soil.

One can mathematically denote this criterion in the following manner: L stands for the length of the anchor, O for the circumference and A_{base} for the area of the base wall. Further, σ_{active} and $\sigma_{passive}$ denote the active and passive soil pressures respectively.

$$F_{resist} = \sigma_a L O_{anchor} + \sigma_{passive} A_{base} - \sigma_{active} A_{base} \leq 273kN$$

After the definitions of the dimensions of the structure an impression can be made using visualizing software. Google Sketchup was used in order to deliver this impression. One of the initial selection criteria was the manner in which the structure fits into the environment. This aspect was taken along in the aesthetic design of the quay wall.

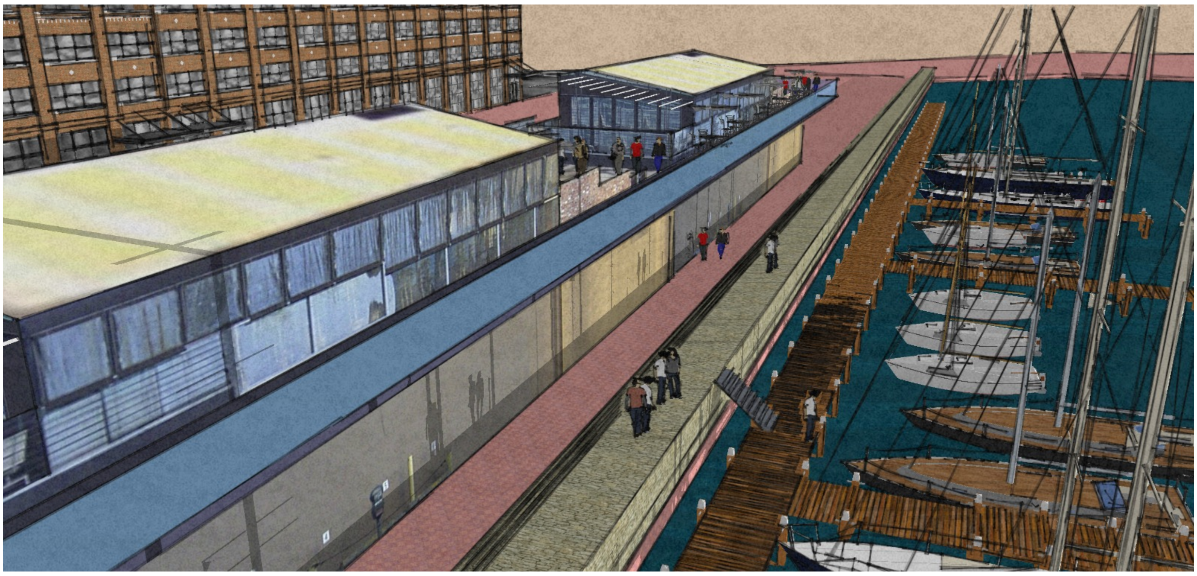


Figure 8.17: An aesthetic impression of the manner in which the quay wall is supposed to look within the environment. The image was made in Google Sketchup.

8.6. Conclusion

The quay wall at the Doctor Lelykade is primarily meant to protect the inhabitants and their assets in the hinterland from high water levels. In its current state the frequency of flooding is too high, indicating that the elevation of the quay wall is too low. Therefore different approaches were used to design constructions in which this problem was tackled. Four concepts were designed, which comprehend an increase in height of the full street and quay wall, construction of a gravity, construction of a plateau on compression piles and a new (smaller) structure on top of the current wall. The best alternative turned out to be a concrete structure on top of the current quay wall. Evaluation of the different concepts was not done using a quantitative method, as this was already used for the urban spatial design during the previous phase of the project. Aspects which were taken along were the aesthetic representation of the structure, the space limitation and the construction costs. The concept which aims at placing a new structure on top of the quay wall, is the only one which allows for port operation during the construction phase. It can also be designed to fit nicely in the environment while the construction costs are relatively low. Therefore the decision was made to choose that specific concept. The structure was built in the form of stairs, such that it provides access to both sides of the wall. Present-day, people also use the area for recreative purposes, for example by walking upon the Lelykade with a view on the Tweede Haven. This aspect was maintained by building the concrete structure in the form of stairs. This way the people are able to walk upon the new quay wall while still respecting the required flood safety. The reason that opposite variants were not chosen is mainly because of their financial and time-consuming (construction wise) character. An indication of the costs can also be given due to a research describing the relationship between construction costs and retaining height for a quay wall (Gijt, 2011). The costs can be estimated at 6 million euros.

8.7. Discussion

The fact that the structure on top of the current street was chosen is not very surprising. The Doctor Lelykade did flood several times in the past but compared to floods in other areas the damage remained relatively low. From a financial perspective one then tends to choose a solution which aligns with the relatively low damage costs. The concept which was chosen is assumed to be the cheapest while it does fulfill the requirements. To deduct whether this is actually the case one could also do a quantitative Cost Benefit Analysis. This also increases the accuracy of the evaluation of the concepts. Regarding the strength calculations of the concepts, several approaches could be modified to enhance to model accuracy. In the first concept, where a new quay wall was designed, the American method was used. The method assumes a perfect balance between active and passive soil forces to provide rotational stability. This also implies that the shear force in the bottom section of the sheet pile is zero. Whether this is really the case is not guaranteed to be right. In the future one could better analyze general soil structure interaction to characterize the mechanics of the bottom section. Regarding the final concept, elementary calculations were done to ensure the strength and stability of the concrete stairs structure. Because the forces on the structure are relatively low (especially compared to the dimensions) the stiffness calculations were not accounted for. The design could however be improved if the anchor was designed in more detail.

The overall quality of the design could improved if more was known about the state/configuration of the current quay wall. Certain assumptions were made and photos were taken during a field trip to the project area. Unfortunately nothing could be deduced from the material. Phone calls were however made with the municipality of The Hague, whose employee told that the construction drawings would be sent, which did however not happen. For a future research it is therefore recommended to ensure that these drawings are obtained. The assumptions which are now made are actually based upon pure logic because otherwise no starting point would be available to begin from.

9

Breakwater design

9.1. Introduction

In this chapter, the design of the breakwater will be discussed, which will be extended to the 10m depth contour. This extension will facilitate the land reclamation at the north and south beaches. In Figure 9.1, an overview of the new situation in presented. Here a technical design is presented for the individual cross-section A as indicated. From analysis of the wave- and wind climate at the site, this section was found to be subject to most critical loading conditions. Since the breakwaters are extended further seawards than in the current situation, the extensions will be located in deeper water. This means that the structure will be exposed to larger waves. For this reason, the existing breakwater design cannot be used for the extended part and a new design should be made for the breakwater extensions. Part of the existing breakwaters can remain and function as access road to the new breakwater section. The design process starts with a brief description of the current situation and a functional description of the breakwater after which a decision for the type of breakwater is made. The lifetime of the structure and target reliability for different failure mechanisms of the structure will be discussed after that. The hydraulic boundary conditions are discussed and finally a deterministic design for the breakwater cross section is presented.

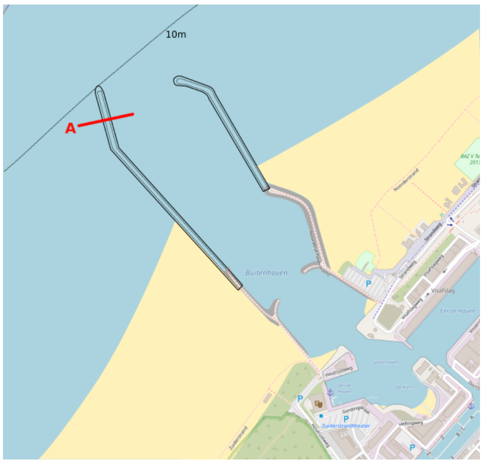


Figure 9.1: Planview of the breakwaters and cross-section A, which will be designed in this section.

9.1.1. Current Situation

The current breakwaters are conventional rubble mound breakwaters with concrete armour units. A pedestrian walkway is present at the crest of the breakwaters. It is also possible for cars to access the walkway in case of emergency.

It is assumed that the current orientation of the breakwaters is such that vessels can safely enter the Scheveningen harbour and these orientations will thus be adopted for the extended parts. Cross section A, as indicated in Figure 9.1 will thus have orientation $340°N$, which means that normally incident waves come from the direction $250°N$.

The breakwaters consist of two types of concrete armour units. Smaller concrete cubes are placed at the harbour end of the breakwaters. The diameter is estimated to be 1 m. More seawards, larger armour blocks are placed with holes in them. The diameter of these blocks was estimated to be around 1.2 m. Figure 9.2 shows the two types of armour units.

Figure 9.2: Currently the breakwaters are build with two types of concrete armour blocks.

Functional requirements
The main functions of the breakwater will remain remain unchanged for the new situation where they reach further seawards. An overview of the functions is given below.

- Provide shelter from waves and protection of vessels harboured in the Scheveningen port.
- Provide protection against currents.
- Prevent sedimentation of the access channel.
- provide space for access roads to the berthing facilities of the mini cruise ships.

The breakwater will remain accessible for the public and thus a pedestrian walkway should be present on the crest of the structure. This access will have consequences for the amount of overtopping since this could be hazardous to pedestrians. The design life of the structure will be 100 years. Figure 9.1 gives the layout of the breakwater as determined in the development of concepts.

9.1.2. Type of breakwater

Selection of the type of breakwater depends on environmental conditions at the site and conditions for construction. For small to medium water depths a rubble mount breakwater is most appropriate. Caisson breakwaters are generally more expensive, however the amount of material needed does not increase as much for larger water depths than for rubble mount breakwaters.

The new breakwaters should extend towards the 10m depth contour. Choosing a rubble mount breakwater like the existing ones in Scheveningen will result in a breakwater that is very wide. This is because the slope of the structure cannot be to steep since this leads to instability and damage of the armour units. Considering this, it will likely be more economical to choose for a caisson breakwater. After land reclamation, the equilibrium profile of the sea bed will not yet be reached and will have a higher slope. The water depth at the toe of the breakwater will thus still be 10 meters with respect to NAP. A disadvantage of caisson breakwaters is that they give more wave reflection compared to a rubble mount breakwater. This will result in standing waves in front of the structure with relatively large orbital velocities of water particles near the bed. This should be taken into consideration and a scour protection layer could be placed in front of the structure to prevent large scour holes.

For the reasons above, it is chosen to make a design for a caisson breakwater. This structure will be supported by a foundation to distribute the forces on the sea bottom. The foundation should have armour protection units to prevent damage during storm conditions. As discussed above, vertical wall structures result in standing waves in front of the structure and thus an appropriate scour protection layer should be designed. In the next section, the failure modes of the caisson breakwater will be identified.

9.2. List of failure modes to include in design.

In this section the failure modes of the caisson breakwater are identified. The design life of the was determined to be 100 years. In general, one can identify a limit state function for each failure mechanism with a resistance side and a loading side. Here, the conventional design method is used where all the uncertainty is assigned to the loading side of the limit state function. A certain design storm is identified which has a certain return period R. There are two different limit states: the Serviceability Limit State (SLS) and Ultimate Limit State (ULS). There is no real consensus on the right target failure probabilities for breakwaters, for determining acceptable target failure probabilities some guidelines are used. Also for each failure mechanism the consequences of exceeding of the limit state is discussed and then an attempt is made to choose an appropriate return period. The probability of failure for the lifetime of the structure for a certain return period can then be calculated using:

$$p_{f,TL} = 1 - e^{-T_L/R} \tag{9.1}$$

In total, five failure mechanisms are considered for the caisson breakwater. For each failure mechanism, the choice for the criterion is made and a design storm condition corresponding return period is stated. Here each failure mode is evaluated separately which is a simplification. In reality failure modes can occur at the same time and one failure mode can influence others.

Failure mechanism	**Limit state**	**Criterion**		**Return period**	**$P_{f,TL}$**
Wave overtopping	ULS	No damage to crest	$q < 50\ l/sm$	500	0.18
	SLS	No hazard for pedestrians	$q < 1\ l/sm$	1	1.00
Armour foundation stability	ULS	no damage	-	1000	$9.51 \cdot 10^{-2}$
Sliding	ULS	No damage allowed	$\gamma_f \geq 1.2$	800	0.12
Overturning	ULS	No damage allowed	$\gamma_f \geq 1.2$	800	0.12
Seabed scour	ULS	Scour at structure	N/A	800	0.12

Table 9.1: Return periods failure modes

Wave overtopping Overtopping is an important mechanism that determines the crest level of the breakwater. Caisson breakwaters can withstand heavy overtopping (Van der Meer, 2018), so for the ULS a relatively large mean overtopping discharge of $q = 50\ l/s$ per m is allowed for storm conditions with return period 500 years. This overtopping discharge will likely not lead to structural damage of the caisson. Some of the infrastructure on the crest can be damaged due to this loading. The breakwater will also function as pedestrian walkway, for which only limited overtopping is acceptable. When overtopping is too large, the breakwater will have to be closed. For the SLS, an overtopping rate of $q < 1\ l/s$ is allowed for storm conditions with return period 1 year.

Sliding

Sliding of the caissons normally also moves the foundation, reducing the overall stability of the structure. This also makes the structure more vulnerable for other failure mechanisms. In addition, the repair cost for sliding are relatively high. For these reasons, the criterion for sliding is that no damage is allowed for storm conditions with return period 800 years. This corresponds to a safety factor for sliding of $\gamma_{sliding} \geq 1.2$ according to Goda (Goda, 1992).

Overturning
Overturning of the caisson around the heel is another failure mechanism that should be taken into account in the design of the vertical structure. The same reasoning as for sliding holds, which is that overturning reduces the overall stability of the structure and is not easy to repair. Considering this, the criterion is that no damage is allowed for storm conditions with return period 800 years corresponding to a safety factor for overturning of $\gamma_{overturning} \geq 1.2$ according to Goda (Goda, 1992).

Armour foundation stability
The caisson breakwater will be supported by a rubble mound foundation, which will have armour protection and an underlayer. High wave reflection that is present for vertical wall breakwaters will result in amplification of near-bed water particle velocities. In addition, damage of the foundation of the structure can rapidly develop and lead to loss of stability of the overall structure. This makes that instability of armour foundation is an important failure mechanism. A criterion is chosen of no damage for storm conditions with the larger return period of 1000 years.

Sea bed scour
Seabed scour in front of the vertical breakwater can be expected due to the high wave reflection caused by the structure. Standing waves will occur in front of the structure which introduce flow circulation cells. Sediment on the seabed starts moving with the circulation when the flow velocities become large and the bed material is fine, the reshaping of soil is characterized by Xie (Xie, 1981). For soil mechanical stability, scour protection over a length of approximately 1/2 wavelength should be safe with a minimum length of 10-15 m (J.P. Bos and Verhagen, 2018). Scour can develop into soil mechanical instability and finally sliding of the structure. For this failure mechanism the ULS is that scour in front of the structure will not occur for storm conditions with return period 800 years.

9.3. Boundary conditions

In this section the hydraulic boundary conditions will be discussed. For the failure modes of the caisson breakwater different loading condition may be normative. First of all, water levels at the location of the breakwater should be known. These water levels consist of tidal levels, storm surge and sea level rise. The tide is deterministic and different tidal levels are calculated from measurements of Rijkswaterstaat in Appendix E. The highest tidal level is mean high water spring (MHWS) and it was found to be at 1.48 meters above NAP. The lowest atmospheric tide was found to be 1.04 meters below NAP, other tidal levels are presented in Table E.2.

The storm surge and significant wave heights are stochastic in contrary to the tidal levels. To get the extreme values corresponding to different return periods, data from Rijkswaterstaat is used. For the significant wave height the data is obtained from a measuring location EuroPlatforn. The storm surge was obtained from a measuring location inside the Scheveningen harbour. For both the wave heights and storm surge, a peak over threshold (PoT) analysis is performed. The extremes, values larger than the threshold, were selected from a dataset of 20 years of measurements. After that a Generalized Pareto distribution was fitted to obtain the values corresponding to different return periods. The calculations can be found in Appendix E.

A design water level consists of MHWS, storm surge and sea level rise (SLR). Normative condition is at the end of the design life, since the sea level is expected to rise during this period. Table E.3 an overview is given for the water levels where the design water level is found with:

$$h_0 = MHWS + SS + SLR$$

R [yr]	SS [m, NAP]	Tidal Level [m, NAP]	SLR [m, NAP]	**Total [m, NAP]**
1	1.62	1.48	1.00	**4.10**
500	2.77	1.48	1.00	**5.25**
800	2.82	1.48	1.00	**5.30**
1000	2.85	1.48	1.00	**5.33**

Table 9.2: Design water levels corresponding to different return periods

To get appropriate boundary conditions for the caisson breakwater design, the offshore wave conditions should be transformed to nearshore wave conditions. This is done using the SwanOne program, which is the 1D mode of the Swan wave model developed at Delft University of Technology. The program computes random, short-crested wind-generated waves in coastal regions and inland waters. The input for SwanOne are first of all the bottom profile given in Figure E.11. This is a simplified bottom profile that was estimated using depth information from Navionics. Besides the bathymetry, the wave conditions consisting of H_{m0}, T_P and wave direction. Also the wind conditions consisting of wind intensity and wind directions should be specified. The output of SwanOne are the spectrum and related parameters such as H_{m0}, T_p and $T_{m-1,0}$ at any given position x. For the caisson breakwater, the wave parameters at the toe of the structure should be used. An overview of the parameters of interest at the toe of the structure (nearshore) are presented in Table 9.3.

R [yr]	h_0 [m]	Offshore conditions			Nearshore conditions				
		H_{m0} [m]	T_p [s]	Dir [°N]	H_{m0} [m]	$H_{1/3}$ [m]	T_p [s]	T_{m01} [s]	Dir [°N]
1	NAP+4.10	4.88	6.96	295.33	3.98	4.10	8.96	7.66	287.37
500	NAP+5.25	6.95	8.29	295.33	5.10	5.29	11.16	8.28	292.96
800	NAP+5.30	7.08	8.36	295.33	5.14	5.33	11.16	8.31	294.38
1000	NAP+5.33	7.14	8.41	295.33	5.14	5.33	11.16	8.33	294.30

Table 9.3: Nearshore wave conditions for various return levels.

9.4. Deterministic design of cross-section

In this section the cross-sectional design of the caisson breakwater is made for the cross section as indicated in Figure 9.1. The design is deterministic, meaning that for the resistance of the structure uncertainty is not taken into account. The design is based on the boundary conditions that were determined in Section 9.3. Figure 9.3 gives a simple sketch of the cross section. The composite vertical wall breakwater has a foundation berm with berm width B_M and height h_{berm}. Here R_c is the freeboard, h is the waterdepth at the toe of the structure. The dimensions are given in Table 9.4

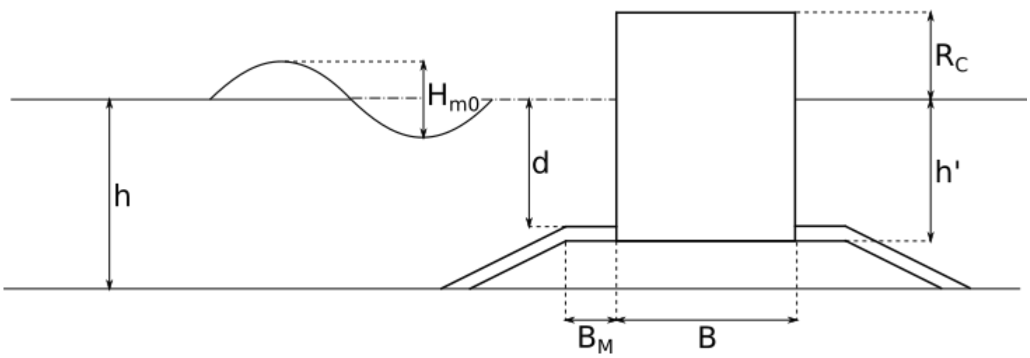


Figure 9.3: Composite caisson breakwater dimensions.

Caisson height, $h_{caisson}$	16 m
Caisson width, B	12 m
Wall thickness, t	0.5 m
Foundation height, h_{berm}	5 m
Foundation width, B_M	4 m
Foundation slope	1:1.5

Table 9.4: Dimensions of the composite vertical wall breakwater.

9.4.1. Wave overtopping

For determining the amount of wave overtopping over the caisson breakwater, the wave / structure regime has to be determined by evaluating the influence of the foreshore, the significance of the berm and the likelihood of impulsive breaking.

The first step is to determine whether we are dealing with a vertical structure with or without influencing foreshore. To determine if an influencing foreshore is present, we should check if the structure is found at the end of a sloping foreshore so that waves are depth limited. The ratio of the water depth at the toe of the structure and the deep water significant wave height gives an indication of whether we are dealing with deep water conditions or shallow water. For the return period of $R = 1$ years and $R = 500$ years, this ratio equals $h/H_{m0,\text{deep water}} = 2.89$ and $h/H_{m0,\text{deep water}} = 2.19$ respectively. Since $1 < h/H_{m0,\text{deep water}} < 4$, it is clear that the toe of the structure is in shallow water (Van der Meer, 2018), and thus an influencing foreshore is present.

The second step is to check if there is a significant mount in front of the breakwater. to check if this is the case for our situation, the ratio d/h is computed for both the $R = 1$ year and $R = 500$ year return period. For design storms, the ratio $d/h > 0.6$ was found which indicates that the mound has no significant influence.

Lastly, we check if we are dealing with non-impulsive waves or impulsive waves. Non-impulsive waves are waves with lower steepness where the significant wave height is relatively small compared tot the depth at the toe of the structure. For impulsive waves the significant wave height is large compared to the local water depth and the overtopping is violent. To check whether the waves are impulsive, the modified 'impulsiveness' parameter d_* is computed. The parameter is given by (Bruce et al., 2010):

$$d_* = 1.3 \frac{d}{H_{m0}} \frac{2\pi h}{T_{m-1,0}^2} \tag{9.2}$$

For return periods $R = 1$ and $R = 500$ years, we find $d_* = 0.46$ and $d_* = 0.37$. For both it holds that $d_* > 0.3$ indicating non-impulsive waves.

From the above steps, it is found that we are dealing with a composite vertical structure with influencing foreshore, no significant mound and non-impulsive conditions. For this wave / structure regime, it is suggested to use the following formula for calculating the mean overtopping discharge q for deterministic design (Van der Meer, 2018).

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.062 \cdot \exp\left(-\frac{2.61}{\gamma_\beta} \cdot \frac{R_c}{H_{m0}}\right) \tag{9.3}$$

Here γ_{beta} is a reduction factor for the angle of wave attack. For $\beta = 43\,\text{deg}$, $\gamma_\beta = 1 - 0.0022\beta = 0.9$ was found. The wave overtopping can be reduced further by constructing a bullnose at the top the structure. This is a seaward overhang which deflects the water that runs up the vertical wall. The bullnose has an effectiveness factor k_{bn} that determines how much the wave overtopping over the vertical wall breakwater is reduced.

$$q_{\text{with bullnose}} = k_{bn} \cdot q_{\text{without bullnose}} \tag{9.4}$$

The effectiveness factor k_{bn} can be determined using the decision chart presented in Figure 9.4 (Van der Meer, 2018). Here B_r is the horizontal extension of the bullnose, h_r is the height of the bullnose and α is the angle of the bullnose with the horizontal.

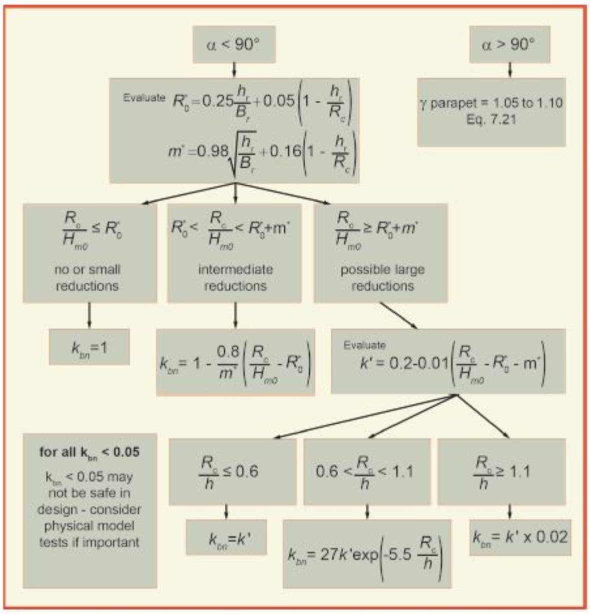


Figure 9.4: Decision chart for calculating the effectiveness factor k_{bn} of bullnose for reducing wave overtopping (Van der Meer, 2018).

Using the above equations, the overtopping discharge over the caisson was calculated. With the dimensions of the bullnose being $B_r = 0.4\ m$ and $h_r = 0.5\ m$. Here the angle of overhang $\alpha < 90$ such that wave overtopping is reduced. The results of the calculations are sumarized in Table 9.5.

Return period [years]	$R = 1$	$R = 500$
$H_{m0}\ [m]$	3.98	5.10
$R_c\ [m]$	6.90	5.75
$q_{\text{without bullnose}}\ [l/s\ /m]$	11.76	96.86
k_{bn}	0.05	0.50
$q_{\text{with bullnose}}\ [l/s\ /m]$	0.59	48.85

Table 9.5: Wave overtopping for design storm with return periods R=1 year and R = 500 years.

9.4.2. Stability of caisson

Wave pressures on the caisson

For the deterministic design of caisson breakwaters the practical formulas from the Goda method can be used (J.P. Bos and Verhagen, 2018). Goda analysed vertical wall breakwaters and developed practical formulas for the stability of these structures (Goda, 2010). The Goda method is based around the trapezoidal pressure distribution on the seaside of the caisson as sketched in Figure 9.5. Here the maximum elevation of wave pressure is $\eta^* = 0.75(1 + \cos\beta)H_{max}$ with $H_{max} = 1.8H_{m0}$ the design wave height.

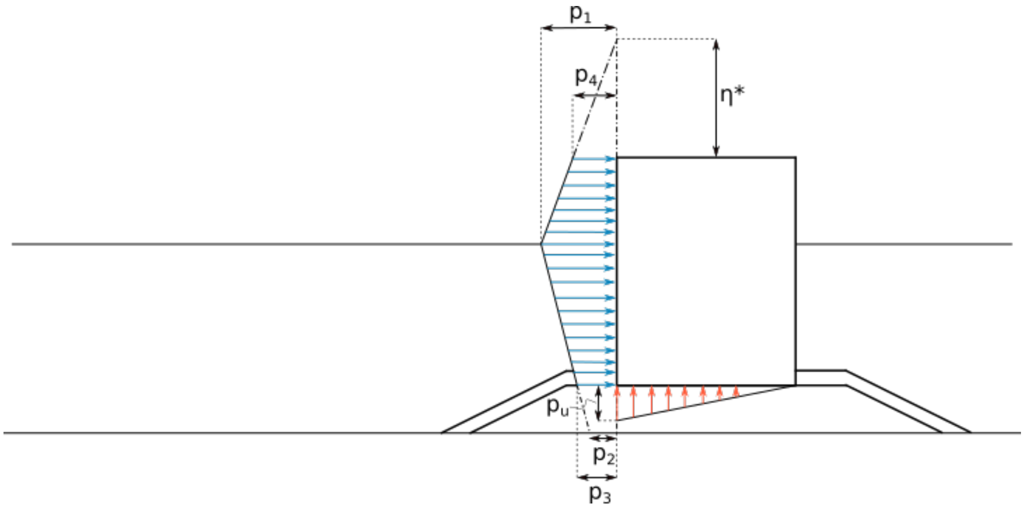


Figure 9.5: Wave pressure distribution by Goda's formula's (Goda, 1992)

The pressures p_1, p_3, p_4, p_u as shown in Figure 9.5 can be computed as follows.

$$p_1 = \frac{1}{2}(1 + cos(\beta))(\alpha_1 + \alpha_2 \cdot cos^2(\beta))\rho_w g H_{max} \tag{9.5}$$

$$p_3 = \alpha_3 p_1 \tag{9.6}$$

$$p_4 = \begin{cases} p_1(1 - \frac{R_c}{\eta^*}) & \eta^* \geq R_c \\ 0 & \eta^* \leq R_c \end{cases} \tag{9.7}$$

$$p_u = \frac{1}{2}(1 + cos(\beta))\alpha_1 \alpha_3 \rho_w g H_{max} \tag{9.8}$$

in which β is the angle between the direction of wave approach and a line normal to the breakwater. R_c is the freeboard, $rho_w = 1025\ kg/m^3$ is the density of seawater and $g = 9.81\ m^2/s$. α_1, α_2 and α_3 are given by:

$$\alpha_1 = 0.6 + \frac{1}{2} \cdot \left(\frac{2kh}{\sinh 2kh}\right)^2 \tag{9.9}$$

$$\alpha_2 = min\left(\frac{h_b - d}{3h_b}\left(\frac{H_{max}}{d}\right)^2; \frac{2d}{H_{max}}\right) \tag{9.10}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left(1 - \frac{1}{\cosh kh}\right) \tag{9.11}$$

Here $k = 2\pi/L$ and L is the local wave length based on $T_{1/3}$ at depth h which can be calculated using the dispersion relation for shallow water. h is the water depth at the toe of the structure, h_b is the water depth at the location at a distance $5 \cdot H_{1/3}$ from the breakwater. The water height above the foundation is denoted by h', which differs from d which is the water depth above the armour units foundation. Goda suggests to always adjust the angle of incident for the incoming waves 15° shore normal (Goda, 1992). In our case the breakwater cross section has orientation 250 °N and the direction of incomming waves is 294.30 °N. This gives $\beta = 294.30 - 250 - 15 = 29.30°$. Values used in the calculation are given in 9.6.

H_{m0} [m]	H_{max} [m]	β[rad]	η^*[m]	R_c[m]	h_b[m]	h[m]	h'[m]	d[m]	L[m]	$k[m^{-1}]$
5.14	9.24	0.51	12.98	5.70	15.32	15.30	10.30	9.76	86.64	0.07

Table 9.6: Boundary conditions for Goda formula's

The thickness of the armour layer was found to be 0.6 m. With this design the different parameters and resulting pressures in the Goda formula's are given in Table 9.7.

α_1[-]	α_2[-]	α_3[-]	p_1[kPa/m]	p_3[kPa/m]	p_4[kPa/m]	p_u[kPa/m]
0.72	0.11	0.73	61.73	50.71	39.10	45.49

Table 9.7: Parameters en pressures in Goda formula's

Sliding and overturning

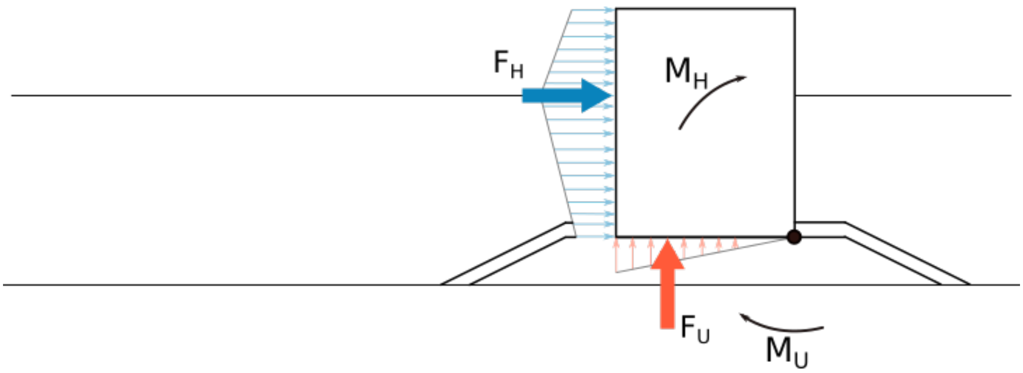


Figure 9.6: Resulting forces on the caisson and resulting overturning moments around the heel.

The wave pressure distribution in front of the caisson and the uplift pressure distribution under the caisson give resulting forces F_H and F_U and resulting moments M_H and M_U around the heel as shown in Figure 9.6. The resulting horizontal force F_H and resulting uplift force F_U can be calculated as:

$$F_H = \frac{1}{2}(p_1 + p_3)h' + \frac{1}{2}(p_1 + p_4)h_c^* \tag{9.12}$$

$$F_U = \frac{1}{2}p_u B \tag{9.13}$$

In the above equation, $h_c^* = min(\eta^*, R_c)$ and B is the width of the caisson. The overturning moments around the heel are given by:

$$M_H = \frac{1}{6}(2p_1 + p_3)h'^2 + \frac{1}{2}(p_1 + p_4)h' h_c^* + \frac{1}{6}(p_1 + 2p_4)h_c^{*2} \tag{9.14}$$

$$M_U = \frac{2}{3}F_U B \tag{9.15}$$

Using the values for the wave pressures as given in Table 9.7, the resulting forces and overturning moments around the heel can be computed and the results are presented in Table D.3.

F_H[kN/m]	F_U[kN/m]	M_H[kNm/m]	M_U[kNm/m]
930.43	272.95	7358.29	2183.58

Table 9.8: Resulting forces on the caisson and resulting moments around the heel.

With the computed forces and moments on the caisson breakwater, the stability of the structure can be evaluated. This is done using safety factors for sliding and overturning. The safety factor for sliding is defined as follows:

$$S.F. = \frac{\mu(Mg - F_U)}{F_H} \tag{9.16}$$

Here, M is the mass of the caisson per unit width, μ is the coefficient of friction between the caisson and the rubble mound foundation. The mass can be computed using wall thickness $t = 0.5\ m$, concrete density $\rho_c = 2400\ kg/m^3$ and assuming the caisson is filled with sand with density $\rho_s = 1900\ kg/m^3$. This gives a total mass of the cross section per unit with of $M = 251610\ kg/m$. A common value of $\mu = 0.6$ should be used as an estimate (J.P. Bos and Verhagen, 2018). With the values of Table D.3, the safety factor for overturning around the heel $S.F. = 1.42$ is found. Since the condition $S.F. \geq 1.72$ is met, the structure is considered safe against sliding for the considered design storm. Overturning of the caisson breakwater around the heel should be prevented. The safety factor against sliding is defined as follows:

$$S.F. = \frac{Mgt - M_U}{M_H} \tag{9.17}$$

Here t is the horizontal distance between the center of gravity and the heel of the caisson and equals $t = B/2$ for symmetrical caissons (M.Z. Voorendt, 2011). With the values of Table D.3, the safety factor for overturning around the heel $S.F. = 1.82$ is found. Since the condition $S.F. \geq 1.2$ is met, the structure is considered safe against overturning for the considered design storm.

9.4.3. Armour foundation stability

For determining the dimension of the armour units of the foundation, the formula proposed by Tanimoto is used (Tanimoto et al., 1982) where D_n is the nominal armour unit diameter.

$$\frac{H_{1/3}}{D_n} = N_s \tag{9.18}$$

The stability number N_s is calculated with:

$$N_s = max\left(1.8,\ 1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8\exp\left(-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_{1/3}}\right)\right) \tag{9.19}$$

For obliquely incident waves, the parameter κ can be calculated with:

$$\kappa = \frac{2k'h'}{\sinh(2k'h')}\sin^2(k'B_M) \tag{9.20}$$

Here $k = 2\pi/L'$ and L' is wavelength at depth h', which can be calculated with the dispersion relation:

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

The depth at the crest of the rubble mound foundation is $h' = 10.33\ m$. The wave height for the $R = 1000$ years return period equals $H_{1/3} = 5.33\ m$ and peak period $T_p = 8.33\ s$. The results are presented in table 9.9.

$H_{1/3}$	T_p	h'	L'	B_M	κ	N_s
5.33	8.33	10.33	72.50	4	0.08	5.97

Table 9.9: Stability number N_s found with the formula proposed by Tanimoto et al (Tanimoto et al., 1982).

With this stability number, the following nominal armour unit diameter is found.

$$D_n = \frac{H_{1/3}}{N_s} = 0.54\ m$$

It is chosen to use single layer concrete cubes for the protection of the caisson foundation with diameter $D_n = 0.6\ m$. The layer thickness is then $t_d = nk_t D_n = 0.6\ m$, using for $n = 1$ (single layer) and $k_t = 1$ for single layer cube (CIRIA, 2007).

9.4.4. Seabed scour

Scour in front of the vertical wall breakwater can lead to failure of the structure and thus scour protection should be placed on the seabed. For vertical wall breakwaters, standing waves will occur and will result in cells with oscillatory motion of water particles and nodes and antinodes. The scour depth in front of the structure can be found with the following equation (Xie, 1981):

$$h_{max} = 0.4 \cdot H_{m0} \left(\sinh \frac{2 \cdot \pi \cdot h_0}{L} \right)^{-1.35} \tag{9.21}$$

The used parameter and the resulting maximum scour depth are given in Table 9.10

H_{m0} [m]	h_0 [m]	T [s]	L [m]	h_{max} [m]
5.14	15.3	8.30	86.64	1.37

Table 9.10: Parameters and resulting maximum scour depth.

Here, H_{m0} is the significant wave height, h is the water depth in front of the structure and L is the local wave length that is found using the dispersion relation. For a given design storm, h_{max} is the depth of the scour hole that could develop in front of the structure. Based on model tests, it is found that the scour protection should have length equal to $\frac{1}{4}L$ to $\frac{3}{8}L$ (Xie, 1981). For the given parameters, this gives a length of 13.75 to 20.63 meters. Also the length should be larger than 10 to 15 meters according to (M.Z. Voorendt, 2011). Considering these ranges, a scour protection length of 15 meters is chosen.

$$D_{50} = \frac{\tau_w}{(\rho_R - \rho_w) \cdot g \cdot \psi_{cr}} \tag{9.22}$$

The grading of the scour protection can be calculated with the above equation (CIRIA, 2007). Here τ_w is the maximum shear stress for oscillatory flow and the rock density $\rho_R = 2650\ kg/m^3$ is used. The maximum shear stress τ_w is computed with:

$$\tau_w = \frac{1}{2} \rho_w \cdot f_w \cdot u_0^2 \tag{9.23}$$

Where the maximum horizontal orbital velocity near bed u_0 and the bed friction factor f_w are computed with equations below. Here the bed roughness parameter $k_s = 0.04\ m$ is used and amplitude of horizontal motion $a_0 = \frac{u_0 \cdot T}{2\pi}$. Results are presented in Table 9.11, a D_{50} of 38 mm is found. With a safety factor of 1.2, the diameter becomes $D_{50} = 46mm$. The standard grading CP45/125 is chosen for the scour protection which has range 45/125 mm.

$$u_0 = \frac{H}{2} \omega \frac{1}{\sinh kh} \tag{9.24}$$

$$f_w = 0.237 \cdot \left(\frac{a_0}{k_s}\right)^{-0.52} \tag{9.25}$$

u_0 [m/s]	a_0 [m]	k_s [m]	f_w [-]	τ_w [N/m2]	ψ_{cr} [-]	D_{50} [m]
1.44	1.90	0.04	0.032	33.68	0.055	0.038

Table 9.11: Results for computation of the D_{50} of the scour protection.

9.4.5. Under layer, toe and core

The foundation of the caisson breakwater has concrete armour protection units which require an underlayer to ensure proper transfer of loads. Also the layer should be large enough and have sufficient permeability and prevent losing fines from the core (CIRIA, 2007). The size of the underlayer material is based on the mass of the armour protection units. With the concrete density $\rho_c = 2400\ kg/m^3$ and the diameter that was determined to be $D_n = 0.6\ m$, the mass of the concrete armour protection blocks equals $M_a = \rho_c D_n^3 = 382\ kg$. The stones in the underlayer should fall between 1/10 and 1/25 of the weight of the armour protection units (CIRIA, 2007). Therefore, the standard grading LM_A 10-60kg is chosen for the underlayer. The nominal diameter of this grading is given by $d_{n50} = 0.23 - 0.28\ m$. The thickness for double layer standard irregular rock, with $k_t = 0.87$ and $n = 2$, will be $t_d = nk_t d_n = 0.45\ m$.

The foundation of the caisson breakwater has concrete armour protection blocks which should be protected by a toe. According to the Rock Manual, it is suggested for randomly placed concrete armour units to construct a double row of armour as toe of the foundation (CIRIA, 2007).

For the core material, the economical choise is to use quarry run. According to The Rock Manual [6], the core material should have weight of 1/10 - 1/25 of the weight of the underlayer.

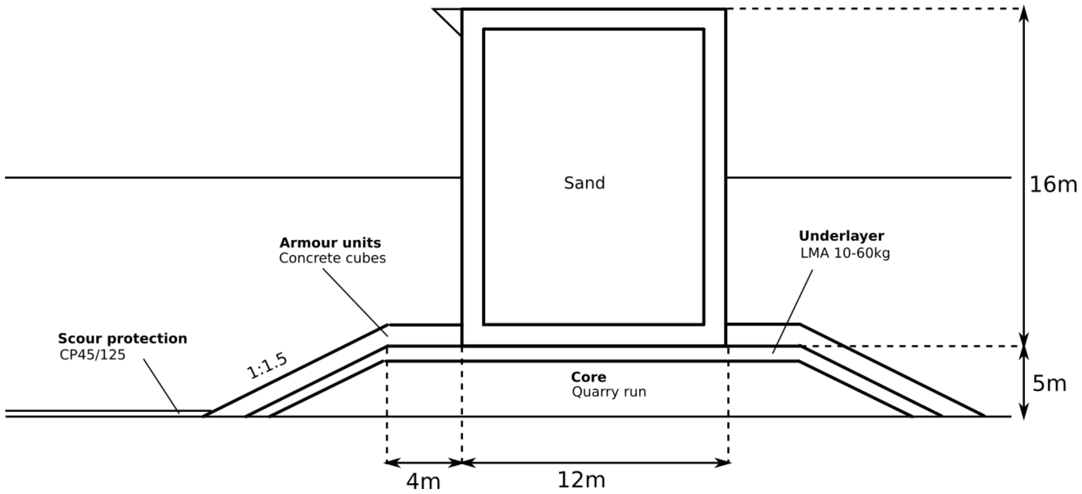


Figure 9.7: Breakwater design sketch with dimensions.

9.4.6. Summary on required dimensions of breakwater

Section	Parameter	Value
Caisson	Material	Reinforced concrete
	Wall thickness	0.5 m
	Fill material	Sand
	Dimension H · B	16 m · 12 m
Armour layer	Material	Single layer concrete cube
	Layer thickness [m]	0.6
	Diameter D_n [m]	0.6
Under layer	Material	Quarry rock
	Layer thickness [m]	0.45
	Diameter d_{n50} [m]	0.23-0.28
Core	Material	Quarry run
Toe	Material	Single layer concrete cube
	Layer thickness [m]	0.6
	Diameter D_n [m]	0.6
Scour protection	Material	CP45/125

Table 9.12: Summary dimensions

9.5. Conclusion

The current breakwaters of the Scheveningen harbour are conventional rubble mound breakwaters with concrete armour units as protection. In the new situation, the breakwaters should be extended to the 10 meter depth contour. For these extended breakwaters a cross-sectional design is made for the breakwater section with most critical loading. The type of breakwater was chosen to be a composite vertical wall breakwater using caissons. This type of breakwater was chosen because of the relatively large water depth at the location of the cross section. In these water depths caisson breakwaters are likely to be more economical since conventional rubble mount breakwaters will be very wide structures and thus require large amounts of material. The failure mechanisms of the caisson breakwater were identified and for each failure mode a choise was made for the ULS and or SLS and an accompanying return period. The local wave climate was analysed and other hydraulic boundary conditions were identified. Extreme value distributions were fitted to data of wave heights and storm surges to obtain values larger return periods. The offshore wave climate was transformed using SwanOne to get nearshore wave conditions at the toe of the breakwater. The design can be summarized as follows:

- Caisson breakwater with dimensions $H \cdot B = 16\ m \cdot 12\ m$ and wall thickness $t = 0.5\ m$.
- A rubble mound foundation with dimensions height $H = 5\ m$, berm width $B_m = 4\ m$ and a 1:1.5 slope. The foundation is protected with concrete cube armour units.
- To reduce overtopping the breakwater will have a bullnose on the sea side with dimensions $B_r = 0.4\ m$ and $h_r = 0.5\ m$.

9.6. Discussion

In this section, first the breakwater design choices and approaches are discussed after which recommendations will be given for further research.

- The choice for a caisson breakwater was made on the assumption that this type of breakwater would be most economical for water depth and loading conditions on site. This assumption was however not evaluated with a cost estimation.
- Only a cross sectional design was made for the section of breakwater in the deepest water resulting in a relatively large structure. More towards shore, another cross sectional design may suffice with smaller dimensions resulting in a more economical solution.
- The design considerations of the breakwater contains a lot of uncertainties. The design load of the breakwaters is one of the most important parameters for the breakwater. The magnitude of the storm surge and wave heights are stochastic and were estimated using extreme value distributions. Due to limited data and a lack of goodness of fit tests, the uncertainty in the used design values are high.

Recommendations for further research for the breakwater are listed below:

- In this cross-sectional design, the uncertainty was only considered in the loading side of the limit state function. The resistance of the breakwater was only determined in a deterministic way. In reality, the resistance of the structure is also uncertain. For further research, a probabilistic design method could be chosen such as a Monte Carlo simulation. In this way the uncertainty of all relevant parameters is taken into account.
- It is recommended to make an economic optimization for the dimensions of the breakwater and the amount of used material. In this way an optimum design can be made that satisfies all the criterion and is thus safe while the cost are minimized. In addition, an optimum for the allowed probability of failure of the structure during the lifetime can be found taking into account loss of life, ecological damage, damage and cost of repairing and loss of income during the time the harbour is not operational.
- In a continuation of this design, the construction phase can be described including a planning. An elementary project execution plan could be made that describes the most important construction phases of the caisson breakwater as well as the logistics of the materials used for the project.

10

Port design

10.1. Introduction

The harbour of Scheveningen will be extended by the construction of a 4th harbour to create the possibility for the berthing of mini cruises increase in capacity in the Marina. The capacity of the marina is not sufficient at the moment and its capacity limit is reached most of the time (See Figure 10.1).

Figure 10.1: Overview of the marina in Scheveningen.

The placement of the fourth harbour was determined during the development of concepts of which an overview can be seen in Figure 10.2. It can be seen that the fourth harbour will be located at the end of the southern breakwater and water sports facilities will be located at the northern breakwater.

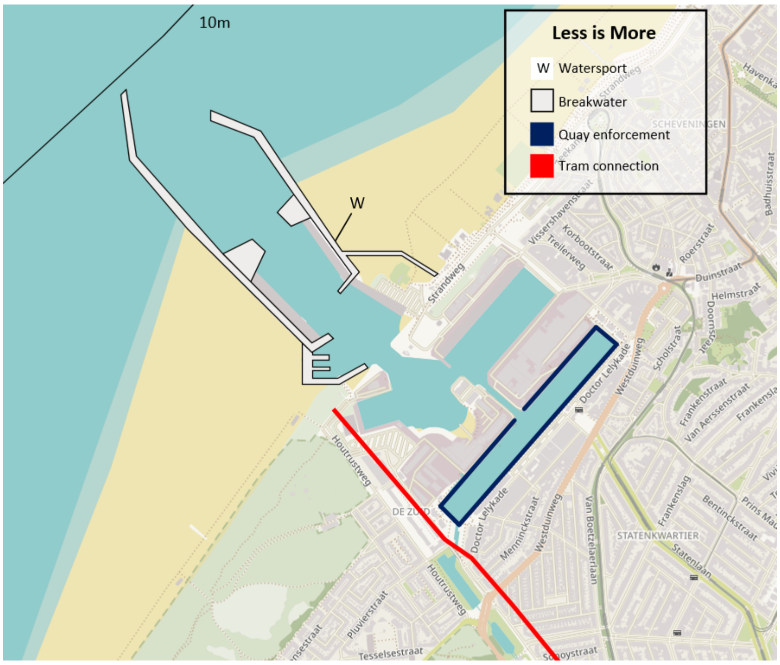


Figure 10.2: Less is more concept

However this is just a rough sketch of where the harbour extension will be located and its size is rather inaccurate. It is desired to know the actual required area and dimensions of the fourth harbour and for the installment of other on land facilities. In order to achieve this, the fourth harbour will be divided into various elements which are the following:

- Access channel
- Cruise docking
- Basin dimensions
- Zoning plan

Access channel
The dimensions of the access channel (length, width and depth) will be calculated. As well as the costs that come along in order to achieve the required water depth so that all vessels can navigate through.

Cruise docking
The Cruise docking will ensure the possibility for mini cruises to dock on the inner breakwater. This will be realised with breasting and mooring dolphins. For the breasting dolphins strength calculations will performed to obtain its required material, cross-section and length. It will also be shown where the piers and quay walls will be constructed, however the design and calculations for these quay walls will not be performed here.

Basin dimensions
The area and dimensions of the basins will be calculated which will be done for both the marina and that for the larger fishing ships. A sketch will also be provided of a possible layout for both basins.

Onshore facilities
This elements regards the required area and dimensions for the onshore facilities to ensure functional operations for fishing ships. In this section all previously mentioned elements will come together in a layout and a sketch will be provided. Within this zoning plan the possibility for water sports must also be present, however the determining of training's facilities will not be categorized and elaborated in this project.

10.2. Basis of design

10.2.1. Function analysis

The current situation does not fulfill the functions set for the harbour. The system must therefore be designed such that it meets the expectations of the relevant stakeholders in order to be able to perform accordingly. If all of these functions are acted on by the system then it means that the system acts accordingly. The functions of the system are defined as follows:

- The fourth harbour also offers space for mini cruises and larger fishing ships to moor.
- the light offshore industry must be facilitated during the construction and maintenance of windmills at sea.
- Providing the facilities for the storage of cargo, processing, auction and transport of cargo.
- Providing opportunities of transport from and towards hinterland for tourists from mini cruises and workers of shipping companies.
- Provide mooring opportunities for water sports vessels and training facilities.

10.2.2. Boundary conditions

The boundary conditions define system or elements that are present in the designated area that cannot be changed or altered. These are mostly structural boundaries as mentioned above which are in terms of physical boundaries.

- At the moment there is a physical boundary of the Zuiderstrandtheater. However it must be noted that this is a temporal building and perhaps can be removed at an earlier stage but requires consultation between the municipality of The Hague and of the owner and user of the building.
- The Westduinpark is located to the southwest of the third harbour and is a physical boundary as it is prohibited to alter it. This is prohibited according to the Natura 2000 and must therefore remain completely unchanged from its current situation.
- The third harbour must remain unchanged as well as the location of where breakwater is present. Its position will not change even though given the fact that it will be stretched towards the ten metre depths line.

10.2.3. Program of requirements

The requirements are derived from the functions provided in the previous chapter. In order for the system to fulfill all its functions, it must meet the following requirements which are obtained from (PIANC, PIANC121,2014), the lecture notes (J.P. Bos and Verhagen, 2018) and (H. Ligteringen, 2000):
Access channel The following design cruise ship, with characteristics as given in Table 10.1, must be able to navigate through the access channel.

Quantity	value
Length (LOA)	183.40 m
Length Between Perpendiculars	160.00 m
Beam (B)	25.00 m
Max. Draught (D)	6.50 m
Deadweight	4.202 t
Passenger capacity	720 persons
Maximum speed	21.97 knots

Table 10.1: Characteristics of the Design Ship (medium size cruise ship).

- The tugboats must be able to guide mini cruises to their designated location. In order to achieve this, a sufficient amount of tugboats must be present.

- Vessels must be able to turn when they have reached the end of the access channel. Therefore the possibility for vessels to turn with a turning circle diameter of 2 $\cdot L_{ship}$ must follow up the access channel.

- The access channel must have a sufficient guaranteed water depth in order to match the draught of the vessel including certain buffers so that the vessel can safely navigate through.

Cruise docking The mini cruise must be able to dock next to the inner of the breakwater.

- Berthing and mooring possibilities must be present for the mini cruise.
- The structures ensuring the berthing and mooring must have sufficient strength in order to remain in tact correctly transfer forces to the soil.

Basins This section is divided into the requirements for the marina and for the basin of the larger fishing ships.
Dimensions of larger fishing ships and its basin

- The larger fishing ships have a ship length of between 40 - 60 metres. This coincides with fishing ships used for the offshore fishery.
- These larger fishing ships can contain up to 100 tons of cargo.

Dimensions of marina and its basin

- The harbour must have sufficient dimensions to provide space for the placement of the docking stations.
- There must be a sufficient amount of harbour to provide enough space for the placement of the required amount of docking stations.
- The boats require no tug assistance or long access channel to slow down. They will turn in front of the berth.
- Basin width must be 5 $\cdot L_{max}$, where L_{max} is the total length of the largest vessel that will berth in the marina. This length also includes bowsprit, pulled up outboard, dinghy and other protrusions.
- The berthing arrangement options must be either one of the following two: parallel and finger piers.
  - Parallel: this means that the vessels berth parallel to the quay wall which does result in a high unloading rate, but a large quay length is required. For each vessel, a length of about 1.15 $\cdot L_{max}$ is reserved.
  - Finger piers: Here two vessels berth perpendicular on both sides of the pier. It is capable of berthing large vessels up to 15 metres.

Onshore facilities

- Berthing along quay or jetty
- The market or auction hall must have an area of 5 $\frac{m^2}{t}$.
- Processing facilities and administration building of 25 - 1000 m^2.
- Ice production facility and a storage general output of 50 tones per day:
  - Ice factory with an area of 1-6 m^2 per ton per day.

– Ice storage of 0.5 - 1.0 $\cdot \frac{m^2}{t}$

- Cold storage building buffer with an area of 1 $\frac{m^2}{t}$.
- Slipway 500 tonnes and repair shops must be present.
- Net repair facilities with an area of 50 - 1000 m^2.

10.3. Dimensioning of elements

The layout is divided it into sub elements in order to develop an alternative. The characteristics and dimensions for these sub-elements will be calculated in their respective section so that they can be implemented in the final design. The sub-elements are derived from the program of requirements and are listed below:

- Access channel
- Cruise docking
- Basin dimensions
- Zoning plan

10.3.1. Access channel

The dimensions of the access channel will be determined by calculating the required length, width and eventually the depth.
Length of access channel The total length of the access channel is equal to the required stopping distance of the design vessel and can be divided into the following three parts:

L_1 : Slowing down
L_2 : Making fast
L_3 : Final stop

$$L_1 = (V_{s,crosscurrent} - V_{s,min} \cdot \frac{3}{4} \cdot L_s \tag{10.1}$$

The $V_{s,crosscurrent}$ depends on the velocity of the cross current. It is equal to 4 knots for cross current velocities below 0.5 m/s (PIANC, PIANC121,2014). The $V_{s,min}$ is the minimum vessel speed for control which equals 4 knots. This results in a stopping distance L1 of 0 metres as the vessel speed due to cross currents and the minimum vessel speed are equal. This means that the vessel slows down to 4 knots outside of the breakwater resulting in a shorter required the breakwater length.
The second part of the stopping length is used for the tugs to make fast. The dominant wave conditions above a wave height of (H_s) of 2.5 metres.

$$L_2 = T_{fastening} \cdot V_{s,min} \tag{10.2}$$

The fastening time of the tugs commonly is approximately 10 minutes with a vessel speed of 4 knots (2 m/s). This results in the following stopping distance:

$$L_2 = 10 \cdot 60 \cdot 2 = 1200 metres. \tag{10.3}$$

The length for the final part depends on the maximum length of vessels entering which is 180 metres:

$$L_3 = 1.5 \cdot L_s = 1.5 \cdot 160 = 270 metres. \tag{10.4}$$

$$L_{total} = L_1 + L_2 + L_3 = 0 + 1200 + 270 = 1470 metres. \tag{10.5}$$

The turning circle is located at the end of the stopping distance and the rule of thumb to calculate for the turning circle is the following (PIANC, PIANC121,2014):

$$D_{turningcircle} = 2 \cdot L_s = 2 \cdot 160 = 320 metres. \tag{10.6}$$

Width of access channel The port's access channel width is determined with equations that depend on the type of ships navigating, the traffic intensity, the wind, current and waves. The traffic intensity through the desired access channel is not sufficiently large to opt for two lanes of the design vessel. It is possible that many ships arrive on the same day but it is assumed that the arrival of vessels nicely distributed over the year. Therefore the following equation will be used which is originated from Figure 10.3.

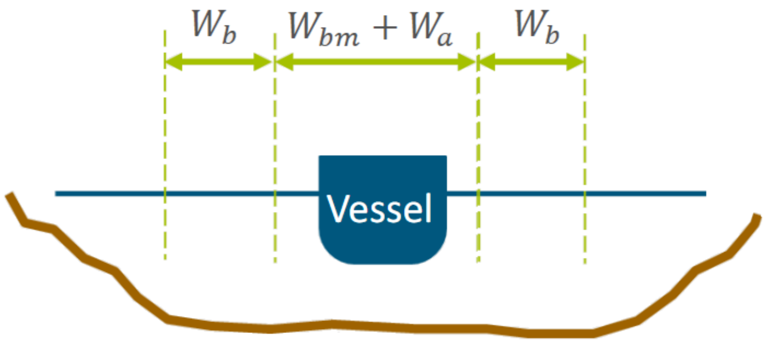


Figure 10.3: Width of the access channel . Figure obtained from the lecture notes (Lansen, 2019).

$$W = W_{bm} + \Sigma W_a + 2 \cdot W_b \tag{10.7}$$

The W_{bm} and W_a are determined by boundary conditions of variables such as wave and wind and is shown in the table below in its entirely. The significant wave height in Scheveningen is larger than 2.5 meter (Pilarczyk, 2000) and the other values have been obtained from (Lansen, 2019).

Component	**Condition**	**Width [m]**
Normative Vessel	Largest width	Bs = 25.00 metres
Cross current	<0.5 knots	0.3 Bs
During a storm	Hs > 2.5m	0.5 Bs
Soft soil	-	0.1 Bs
Channel depth	Dredged	0.2 Bs
Cross wind	30.5 knots	0.6 Bs
Bank clearance	Steep, hard embankment	0.5 Bs

This has resulted in the following width for the access channel:

$$W = 1.5 * B_s + 2.2 B_s + 2 \cdot 0.5 \cdot B_s = 115 metres. \tag{10.8}$$

Guaranteed water depth The access channel must provide the passage of vessels to enter the harbour which can only be achieved if a certain depth is present within the channel. The depth of the channel is not simply the draught of the design vessel as there are many factors that influence how deep the vessel lies in the water. An overview of these factors is shown in Figure F.1 in Appendix F. Combining these factors results in the following deterministic formula (Lansen, 2019):

$$d = D + h_T + s_{max} + z + h_{net} + T \tag{10.9}$$

The only variable in this formula that must be calculated is the squat, which is sinkage due to the flow around the vessel. The maximum sinkage due to the squat of the vessel can be described and calculated with the Barrass formula (Lansen, 2019):

$$s = \frac{C_b}{17.4} \cdot k^{0.76} \cdot V_s^2 \tag{10.10}$$

All variables within the equation have been specified in Appendix F and have the following values:

D	h_T	z	h_{net}	T
6.5 m	0.78 m	1 m	0.5 m	0.75 m

Iteration has resulted in a guaranteed water depth of 10.4 metres.

Dredging It is desirable that the port design is economically attractive and therefore it is desirable to have as low investment costs as possible. Dredging plays an important aspect in this matter as it is an expensive and time consuming task. As calculated in the previous section a guaranteed depth of 10.4 metres is required over the entire length of the access channel. The costs due to dredging are significant due to the long length and width of the access channel. In order to cut investment costs it is opted to use the tidal window. The extra depth that can be used due to the tidal window and the saved costs will be elaborated in the upcoming paragraph.

Tidal window The tidal window is a period of high water time during which the water level exceeds a certain value. However, high water is not always present which has effect on whether or not ships are able to navigate on the access channel. This depends on the tidal restriction which is the minimum water level at which channel passage is safe. The tidal window is often effective for large tidal variations, short channels, infrequent vessel visits and out of boundary vessels. Applying a tidal window to the access channel can reduce dredging costs and can therefore be economic beneficial.

The tide can be described with a cosine function as follows:

$$x(t) = 0.78 \cdot \cos \pi + \frac{t}{6} \cdot \pi + 1.03 \tag{10.11}$$

The water level given by the previous equation has been plotted with the characteristic values of MHW,MSL and MLW.

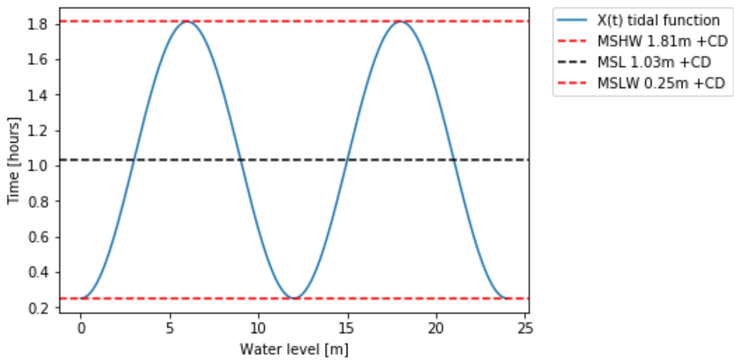


Figure 10.4: Water level caused by the tide as a function of the time.

Now it is desired to calculate when it is possible and when not for vessels to enter the access channel. This can be done by using the equation for the water level and set it equal to its own equation after a time step equal to the maximum waiting time: x(t) = x(t+maximum waiting time).

$$0.78 \cdot \cos \pi + \frac{t}{6} \cdot \pi + 1.03 = 0.78 \cdot \cos\,(\pi + \frac{t+10}{6} \cdot \pi) + 1.03 \tag{10.12}$$

Solving this equation gives a value for t of 7.0 hours and at this time the surface level can be calculated by solving equation F.4 for x(7.0) which gives a value of 1.80 meter. Looking at Figure F.3, if 7.0 hours have passed then it will be unable for mini cruises to navigate through the access channel for 10 hours (maximum waiting time). In other words, at t = 7.0 + 10.0 it will again be possible for mini cruises to navigate through the channel again. By this tidal window the depth of the channel can be reduced with the following value:

$$h_t = 1.80 - 0.25(MLWS) = 1.55m$$

The guaranteed depth without the use of a tidal window as calculated in the previous section was 10.40 metres. However this will be reduced with 1.55 meter resulting in a new guaranteed depth of 8.85 metres. This depth reduction applies to the entire length of the access channel and should be dredged if the tidal window was not opted for.

Cost estimate of dredging The costs to dredge the soil directly depends on two factors which are the volume of the soil and the dredge costs per cubic meter The price range that lies more to the price of 6 euro's per cubic metre will be used in this project namely between 10 and 15 euro's per cubic metre. The volume of soil that requires dredging depends on the dimensions of the access channel and have been visually represented in the table below and its water depth as a function of the length in Figure F.4 which can be found in Appendix F:

Variable	Channel length	Turning circle	Total length	Width
Value	1470 m	320 m	1790 m	115 m

Variable	L1	L2	L3	Guaranteed water depth	Wet earthwork suction costs
Value	620m	136 m	1034 m	8.85 m	10 - 15 euro per m^3

Now the total volume of soil that requires dredging can be calculated with Equation 10.13 and the costs with it can be calculated with Equation 10.14.

$$Volume = \text{Depth } \cdot Length \cdot \text{ Width} \tag{10.13}$$

$$Costs = \text{Volume } \cdot dredgingcostperm^3 \tag{10.14}$$

The total dredging costs that are saved due to the use of a tidal window is thus between 2.34 and 3.52 million euro's depending on the amount of volume that requires dredging. This needs more looking into so that this value can be predicted more precisely.

10.3.2. Basin dimensions

In this part the basin dimensions are quantified and consists of two parts as two basins will be created, one for the larger fishing ships and for the marina respectively. As defined in the requirements, the basin width must be at least $5 \cdot L_{max}$ where the L_{max} is different for the larger fishing ships and yachts.

	Marina	Fisher ship basin
Design length (L_{max})	15 m	60 m
Basin width	75 m	300 m

The length of the basin depends on the amount of vessels that must be able to moor which results in a certain amount of required berthing stations. This amount is different for the fishing ship's basin and for the marina: for the fishing basin it depends on the demand of fishing companies to make use of the harbour and for the marina it depends on the expected future demand. The dimensioning of the two basins is performed separately first that for the the larger fishing ships and then for the marina.

Fishing ship basin As mentioned earlier, the minimum width of the fishing ship basin is 300 metres so now it is only required to know the its length. The formula to calculate the total required quay length for the berthing of vessels (J.P. Bos and Verhagen, 2018) and is given below:

$$L = \frac{Q \cdot (1+s) \cdot f_1}{r \cdot h} \tag{10.15}$$

The is required quay length has been calculated with values from the table below:

Total daily peak discharge	Space in between vessels	Irregularity of vessels	Unloading rate	Unloading hours per day
4000 tonnes	5 m	1.1	10 t/hr	10 hr/day

Now with the variables known the required quay length can be calculated with the use of Equation F.6 (J.P. Bos and Verhagen, 2018).

$$Quaylength = 330metres.$$

This is the required length of the quay walls on both side of the larger fishing ship's basin. On one side the unloading of cargo into the storage areas will be performed and on the other side repair and preparing of ships for their upcoming fishing trip. This gives a restriction on the possible zoning plan of the onshore facilities which will be performed in section 'Zoning plan'.

Marina The basin width of the marina must be at least 75 metres following the requirement when a design vessel length of 15 metres was used. The required length of the basin depends on the amount of berthing places that must be present and on the layout of the pier. It is opted for the finger pier which can be seen in Figure F.5 in F and this is also similar to the layout in the present marina in Scheveningen.

In the layout for the marina, vessels are moored perpendicular to the quay walls and are lined next to each other. Each gang board is used double sided to enhance the efficiency and space limitation. The values from Table **??** from Appendix F have been used to provide rough estimations for the dimensions of the marina. This has resulted in the sketch of the possible layout for the marina and can be seen in the figure below:

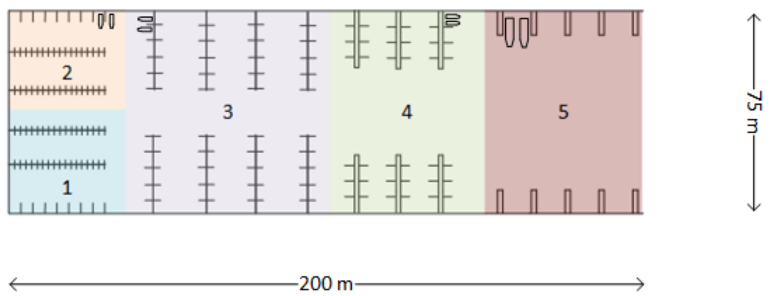


Figure 10.5: Layout of Marina

- Number 1 represents the area where 80 vessels of size category below 4 metres can berth.
- Number 2 represents the area where 160 vessels of size category between 4 and 5 metres can berth.
- Number 3 represents the area where 100 vessels of size category between 5 and 6 metres can berth.
- Number 4 represents the area where 40 vessels of size category between 6 and 8 metres can berth.
- Number 5 represents the area where 20 vessels of size category between 8 and 15 metres can berth.

10.3.3. Cruise docking

The location for the mini cruise to berth was chosen next to the inside of the breakwater during the development of concepts. However, in present situation it is impossible for a mini cruise to moor at that location as there is no quay wall or any berthing or mooring possibilities. It would be required to construct a quay wall along the inner breakwater and to dredge soil to match the required vessel's draught to make it possible for mini cruises to moor. This however requires large investment costs and construction time and therefore it is opted to use a pier construction for the mooring of mini cruises similar to that in Figure F.7 in the Appendix.

To create the possibility for the berthing of the mini cruise, it is opted to use a combination of breasting and mooring dolphins as these dolphins provide the transfer of forces induced by the vessel onto the soil.

These dolphins allow the mini cruise to berth by absorbing forces which must then be transferred to the soil. This is commonly realised by the use of pile foundations. The amount of forces that must be transferred and the thereby required dimensions of the breasting dolphin will be calculated in the next paragraphs.

Laterally loaded pile foundation

During the berthing of the vessel forces are imposed on the berthing dolphins. The value of the forces depends on variables such as the length of the vessel, its mass and what kind of vessel it is (inland going or sea-going). As the mini cruises are sea-going vessels it means that table 29-3 from the Manual Hydraulic Structures (M.Z. Voorendt, 2019) shown in FigureF.9 can be used. This has resulted in a design value for the force of 85000 kN perpendicular to the fender and 22776 kN parallel to the fender.

The force acting on the fender must be transferred to the soil. For large sea-going vessels a rigid breasting dolphin is commonly used according to the General lecture notes Hydraulic Structures (M.Z. Voorendt, 2020). A possibility is to transfer the forces due to the load on the dolphin to the ground with a vertical pile embedded in the floor bed. For such pile foundations, Blum's method is commonly used to compute the required embedded depth that provides enough soil resistance to absorb the lateral loads. Blum schematises the situation of which a figure including an elaboration on the method can be found in the Appendix.

This schematization by Blum is a reasonable approximation to reality as long as the soil can be approximated as one layer. This is the case for the chosen location for the placement of the breasting dolphins as can be elaborated in Appendix F. Although the results can be rather reasonable, Blum's method is still an approximation that includes assumptions. For this method the following assumptions have been made:

1. The embedded part of the foundation is regarded as an elastically supported beam.

2. The soil response is perfectly plastic.

3. The soil reaction on the deeper part of the pile is substituted by a concentrated force known as the Ersatzkraft indicated as R3 in Figure F.11.

4. The pile is thought to have a fixed support at the depth where the Ersatzkraft (R3) is acting on the pile.

In order to calculate the maximum load that the soil can resist in terms of the embedded length can be calculated by taking the sum of the moments. The point at which the moments are calculated must be chosen such that as many unknown variables as possible can be eliminated. Therefore the chosen rotation point is located in the extension of the Ersatzkraft (R3). A schematisation of the loads and their arms is given in Figure 10.6. Note that the loads induced by the water pressure have been drawn in Figure 10.6 however, they are insignificantly small in comparison with the other loads.

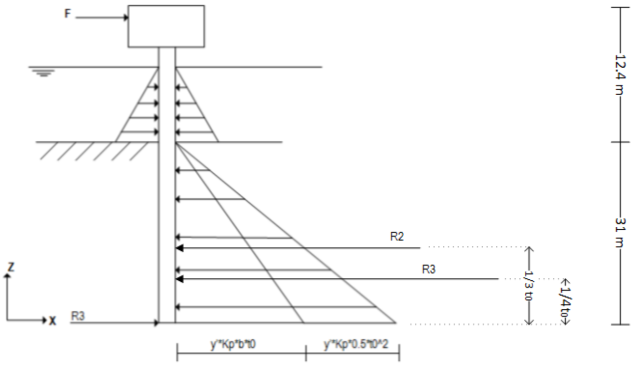


Figure 10.6: Schematisation of loads acting on the pile

Taking the momentum equation around a point where the Ersatzkraft (R3) engages which results that it is being cancelled out due in the momentum equation to an arm of 0 metre. Therefore only the external applied force on the structure and the two forces by the soil (R1 and R2) remain in the momentum equation. Solving that equation results in the following expression for the maximum load that the soil can resist:

$$M_0 = 0$$

$$\rightarrow F_{max} = \gamma \cdot K_p \cdot \frac{t_0^3}{24} \cdot \frac{t_0 + 4 \cdot b}{t_0 + h} \tag{10.16}$$

$$K_{p,h,\sigma} = \frac{cos^2(\phi - \alpha)}{cos^2(\alpha) \cdot \left[1 - \sqrt{\frac{\sin(\phi-\delta)\cdot\sin(\phi+\beta)}{\cos(\alpha-\delta)\cdot\cos(\alpha+\beta)}}\right]^2} \tag{10.17}$$

- $\alpha = \beta = 0$

The wall friction can be calculated with $f = \tan(\delta)$ which for a friction coefficient f results in δ = 23.33 degrees. This coincides with an internal friction angle of 35 degrees. These values including the given values for α and β are used to calculate the passive soil pressure coefficient and has resulted in $K_{p,h,\sigma}$ = 6.35.

The maximum force that the soil can withstand must be equal or larger than the maximum impact load caused by the mini cruise which had been defined previously at 85000 kN. As there are three variables that are unknown within equation F.21, this must be solved iteratively. This has been done and has resulted in the following values:

$$\rightarrow F_{max} = \gamma' \cdot K_p \cdot \frac{t_0^3}{24} \cdot \frac{t_0 + 4 \cdot b}{t_0 + h} = 18 \cdot K_p \cdot \frac{26}{24} \cdot \frac{26 + 4 \cdot b}{26 + 12.4} = 9.15 \cdot 10^5 kN$$

As the shear forces are known, the momentum equation can be used to calculate the location in which the maximum momentum occurs and subsequently calculating the maximum momentum. The momentum equation can be written in terms of 'x' instead of $'t_0'$. The term 'x' then stands for the distance below the bed level which can also be seen in Figure F.12 in Appendix F and results in the following equation:

$$M_x = F \cdot (h + x) - \gamma \cdot K_p \cdot (x + 4 \cdot b) \cdot \frac{x^3}{24} \tag{10.18}$$

The maximum momentum occurs for which the derivative of the momentum equation equals zero ($\frac{\delta M}{\delta x} = 0$) and this must be calculated for when the maximum allowable lateral force is applied (F_{max}). This bending moment curve can be seen in Figure F.15. This moment curve takes an x-value of 0 on the bed floor and is positive towards the soil. This results in the following equation for the depth at which the maximum bending moment occurs:

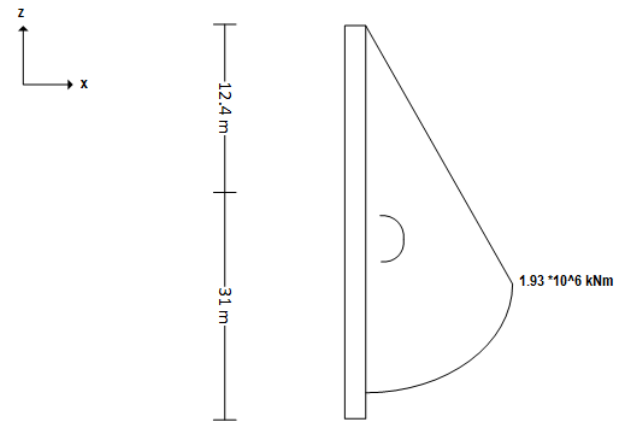


Figure 10.7: Momentum acting over the length of the pile

$$\rightarrow x_m = 15.07 metres \tag{10.19}$$

The momentum equation can now be filled in to obtain the maximum occurring moment:

$$\rightarrow M_x = F \cdot (12.4 + 15.07) - \gamma \cdot K_p \cdot (12.54 + 4 \cdot 4) \cdot \frac{12.54^3}{24} = 1.93 \cdot 10^6 kNm$$

The maximum shear force diagram can be calculated from the momentum equation as it holds that $V_{Ed} = \frac{\delta M}{\delta x}$. This has resulted in the following shear force cuve with a maximum shear force of 4.04 $\cdot 10^5$ kN.

Figure 10.8: Shear force acting over the length of the pile

So the length of the pile, embedded depth, width and the maximum moment have been calculated so that the soil can provide a large enough force to withstand the berthing forces. Values for these variables are shown in the table below. Now the cross-sectional area of the pile must be determined as well as the type of quality of steel to be used. The material used for the pile foundation is steel because of the large bending moments and torsion that will occur as this is a weakness of concrete. In the table below is an overview of the values have been calculated within this section of which some will be used for strength calculations further on in this document.

Embedded depth t_0	**t**	**h**	**Width b**	**Max moment**	**Max shear force**
26 m	31.2 m	12.4	4	1.93*10^6 kNm	4.04*10^5 kN

The previously mentioned pile of the breasting dolphin will be a steel wall cylindrical tube. This has been chosen to use because it must be able to withstand the present forces and significant bending moments as it does rather well according to Figure F.18. The calculation methods for the structural elements partially depends on the dimensions and characteristics of the chosen profile such as width over thickness ratio which is equal to 4 resulting in class 1 as can be seen from Appendix F. This means that the profile will deform in a plastic way and plastic theory can be applied for strength calculations.

Strength The strength calculations depend on the internal forces in a cross-section which in this case are the following:

- Bending moment
- Shear
- Torsion
- Combination of bending moment and shear

Strength calculations for these sections will be performed for the following cross-sectional dimensions:

Steel class	Yield strength	Width (b)	Wall thickness [m]	Diameter
S235	235*10^3 [kN/m^2]	4 [m]	0.8	4 [m]

Bending moment The design value of the present bending moment must be smaller than the design value of the maximum resistance for bending moment: $\frac{M_{ed}}{M_{c,Rd}} \leq 1.0$ which is also called the unity check. The design resistance for bending moment depends on the class of the cross-sections and is the following for class 1:

$$M_{c,Rd} = 1.97 \cdot 10^6 kNm$$

The maximum present moment had previously been calculated in the pile foundation and was equal to $1.93 \cdot 10^6 kNm$ and thus the unity check can be calculated:

$$UC = \frac{M_{ed}}{M_{c,Rd}} = \frac{1.93 \cdot 10^6}{1.97 \cdot 10^6} = 0.98$$

These profile dimensions suffice against the present bending moment because the unity check is lower than 1.

Shear The shear force that acts on the hollow cylindrical tube must be smaller than the shear force resistance thus $\frac{V_{Ed}}{V_{pl,Rd}} \leq 1.0$ must be satisfied. The shear force acting that is present is equal to 4.04 $\cdot 10^5$ kN whereas the design plastic shear resistance is given by the following equation:

$$A_v = 8.04 m^2$$

$$V_{pl,Rd} = 1.09 \cdot 10^6 kN$$

$$UC = \frac{V_{Ed}}{V_{pl,Rd}} = \frac{4.04 \cdot 10^5}{1.09 \cdot 10^6} = 0.05$$

These profile dimensions suffice against the present shear stress because the unity check is lower than 1.

Torsion It is possible for the vessel to hit the fenders in an angle and therefore imposing torsion on the pile. Calculations regarding the strength due to torsion will be performed using the yield criterion by Von Mises which must be smaller than the yield stress of steel. This is shown in the equation below:

$$\sigma_{vgl,Ed} = \sqrt{\sigma_{max}^2 + 3 \cdot \tau_{max}^2} < f_{yd} \tag{10.20}$$

$$\tau_{max} = 16091.03 kN/m^2$$

$$\sigma_{max} = 0 kN/m^2$$

$$\sigma_{vgl,Ed} = 1.11 \cdot 10^5 kN/m^2$$

Now the unity check can be calculated because the Von Mises yield stress is known and equal to $1.11 \cdot 10^5 kN.m^2$.:

$$UC = \frac{\sigma_{vgl,Ed}}{f_{yd}} = \frac{1.1 \cdot 10^5}{2.35 \cdot 10^5} = 0.47$$

These profile dimensions suffice against the present torsion because the unity check is lower than 1.

Combination of bending moment and shear The bending moment resistance of the cross-section is affected when shear force is also present. This should only be taken into account if the shear force is less than half the plastic shear resistance. The effect of the shear force on the bending moment resistance can be neglected because the shear force is smaller than half the plastic shear resistance.

All of the unity checks have been written in the table below and all have a value below 1.0. A sketch of the cross-sectional area and the lateral area are also shown in Figure 10.9 including its dimensions.

Unity Check	Bending moment	Shear stress	Torsion
Value [-]	0.98	0.05	0.47

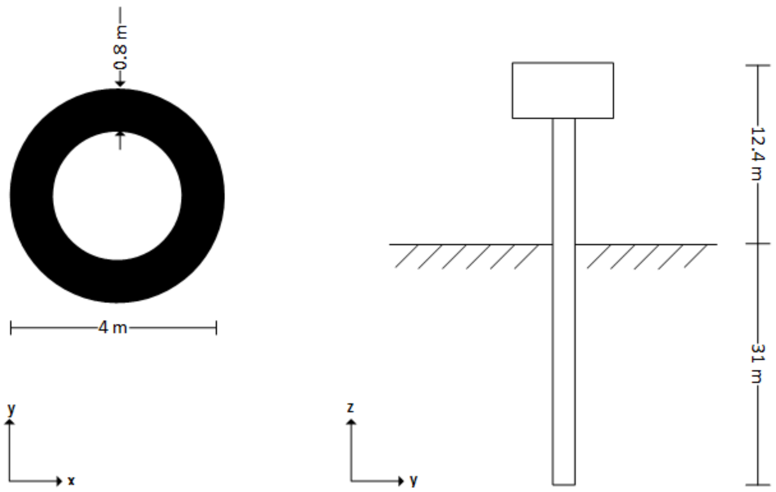


Figure 10.9: Cross-section and lateral view of the breasting dolphin.

Zoning plan The zoning plan concerns the layout of the harbour in which the land- and water-based facilities required for the harbour extension, access channel, basins and water sports facilities are graphically shown. The required on-shore facilities and their area and dimensions are determined whereas that of the access channel and basins have already been defined in earlier sections. These area size and dimensions and of what must be present for the water sports is defined in Appendix F. Combining all the previously mentioned elements has resulted 3D sketches of the port including the fourth harbour. The figure below shows an overview of the 3D sketch but more overviews can be found in Appendix F.

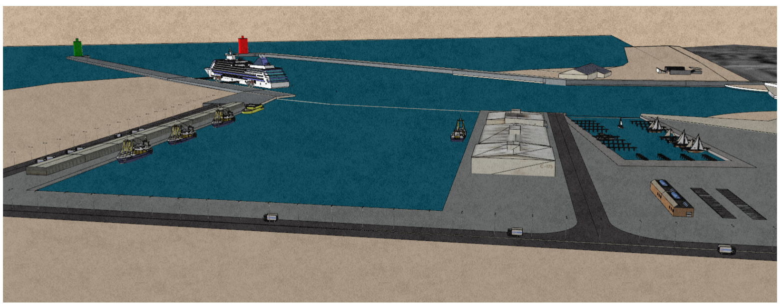


Figure 10.10: 3D Sketch of the entire port of Scheveningen including the fourth harbour.

10.4. Conclusion

The required length of the access channel including the required turning circle is reached when the breakwater is extended to the 10 metre depth line. This means that mini cruises can safely navigate through the access channel towards the harbour. To ensure the guaranteed water depth is realised with the use of a tidal window which also cuts on the investment costs. The marina now has a sufficient capacity to meet the new demand and the basin for fishing ships now also provides berthing places for larger fishing ships. The berthing of mini cruises will be ensured with a rigid body for a breasting dolphin for which dimensions have been calculated. In the final design, all elements have been combined in one 3D sketch to provide an overview of the proposed harbour layout including the new fourth harbour. This sketch can be seen in Figure 10.10 which is presented above the conclusion.

10.5. Discussion

In this section, certain decisions that have been made and uncertainties will be discussed.

- The calculation of the dredging costs that are saved by the use of a tidal window shows a rather large variation. It was mentioned earlier that this is mainly due to the price per cubic meter dredged soil. This uncertainty can mostly be taken away by getting advice from experts or a dredging company that can determine the price per soil depending on the size of this project. Note that also the access channel was assumed to be a rectangular channel with a constant width which is not the case in reality.
- The basin for the marina contains rough sketches and requires more research. Exact equations or rules of thumbs with regard to specific dimensions per pier. Think of rules such as spacing between piers and turning circles for yachts and sailing lanes between piers. Also, the value for the required capacity for the marina was assumed and must be specified by the Municipality of The Hague or other stakeholders but the municipality is the most important one.
- For the calculation with Blum's method the effective volumetric weight of the soil and soil resistance coefficient have been calculated. Both these variables depend highly on the type of soil that is present and must be known. However, this is not the case for this project as the soil profile was not taken inside the breakwater and only represented the soil until 7 meter depth. It is therefore desirable to perform new probings at the correct location.
- The current structure of the breasting dolphin is one vertical pile. This has resulted in a rather large embedded depth and significant wall thickness, whereas other layouts such as multiple and perhaps diagonal piles could resist the acting forces more efficiently. Calculations are therefore required for such design.
- The soil wedge resistance calculated with Blum's method was calculated for a rectangular pile, whereas the chosen pile was a wall cylindrical tube. The use of a different type of cross-section has effect on the soil wedge resistance.
- As mentioned within the report, Blum's method is an approximation and therefore can contain errors. Such errors were visible with the momentum equation and shear force equation when these equations had been plotted over the length of the pile. It was visible that the shear force was not constant over the part of the pile that was above the bed floor. The shear force was fluctuating around the value of the imposed force by the vessel whereas in reality this must be a constant line as there is no other force acting on the pile.

Recommendations

- The calculation of the guaranteed water depth was performed with a deterministic formula. However, a full probabilistic analysis can be performed to calculate the exact guaranteed water depth which could lower the volume soil that requires dredging.
- As not the rectangular cross-section was used for the design of the breasting dolphin it is necessary to investigate the effect this has on the soil wedge resistance.
- The water sports facilities must be determined with the stakeholders such as the Municipality of The Hague and the NOC*NSF. Especially the last party has knowledge on what facilities and their dimensions are required to fulfill the wish of the Municipality for The Hague to become relevant in field of water sports.

- It was stated that a slipway and repair facilities must be present. However, calculations regarding the slipway and structure that carries the vessel while it is being repair can be made. These calculations could regard strength and stability, dimensions of the structure and what materials will be used.
- Scour of the seabed

11

Watermanagement

In this chapter a solution is elaborated concerning enlargement of the current fresh water lens. The solution consists out of 3 components which will be elaborated in their own subsections: 1) Infiltration pond, 2) Pipeline from WWTP Houtrust to the infiltration pond and 3) Pumping station at WWTP Houtrust.
The following stake holders are described in addition to the stakeholder analysis provided in Section 2.3, due to more power and interest in the specific watermanagement design.

Hoogheemraadschap Delfland in general is in charge of the regional waters, in this case the Verversingskanaal and channel network in The Hague. They are liable to protect their region from floods, preservation of water quality in regional waters and providing agriculture with sufficient water. The water treatment within this scope translates to the Delfluent WWTP Houtrust that discharges into the North-Sea in the current situation. In addition, they carry responsibility for management of nature around waters (i.e. the Dunes).

Provincie Zuid-Holland is in charge of translating the national watermanagement plan to regional measurements. The main tasks concerning the case are the (possible) extraction of groundwater and the supervision over the Natura-2000 areas Wapendal and Westduinpark. The province also issues permits towards infiltration projects according to the groundwater law (e.g. Deep infiltration projects by Dunea in Meijendel) (Provincie Zuid-Holland, 2010). These permits are evaluated every 7 years to verify whether they are up to date or not.

11.1. Current situation

This section will describe the system in its current state which includes that of the geo-hydrological structure, the WWTP and the fresh water lens recharging method that is used in similar areas. This knowledge is required in order to create a realistic design.

11.1.1. Geo-hydrological structure of fresh water lens

The fresh water lens functions as a natural boundary that hinders the salt-water intrusion into the hinterland. The ground underneath the area of Scheveningen and surroundings consists mainly out of sandy layers, up to a depth of ±60 m. These sandy layers have the ability to store and infiltrate water. The infiltration capacity of these layers is estimated to be 0.1 - 0.3 m/d in the dunes (Van Dijk, 1984). At the bottom of the sandy layers (± 60 m), there is a semi-permeable clay layer. This layer reaches from ±60 - 65m. It is classified in the Formation of Waalre (Dinoloket, 2020). A confined aquifer is found next, as can be seen in Figure 11.1, which reaches up to 110m depth, where an impermeable layer is found. This impermeable layer is classified in the Formation of Maassluis, regionally it is seen as the base of the aquifer or separating layer (Ingenieurs Bureau Oranjewoud, 2013). Therefore, the actual aquifer reaches up to a depth of 110m where it is limited by this impermeable layer. A visualisation of the above described layers can be found in Figure 11.1.

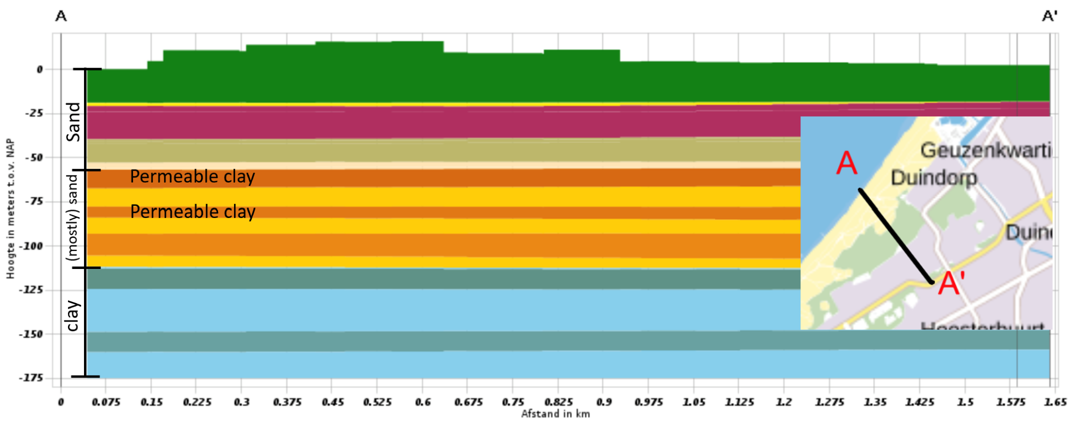


Figure 11.1: Overview of cross section geo-hydologyA - A' (Dinoloket, 2020)

In Figure 11.2 the water lens in Meijendel is displayed, which is a dune area located 4 kilometers north of Duindorp. As can be seen in the overview of the water lens at Meijendel, from the cone penetration tests a semi-permeable layer is identified at ±60 m depth. Due to this layer, two separate aquifers are defined shown in the figure. The upper aquifer ranges from 10-65 m below NAP and is unconfined, whereas the lower one is confined and ranges from ±60 - 120 m below NAP.
Furthermore, it can be seen that the groundwater is divided into fresh, brackish and saline water. The boundary of saline water lens decreases towards the impermeable clay layer when moving land inwards from the North-Sea, as it is blocked by the fresh water aquifers. In the design it is assumed that the fresh water lens has a similar structure to that of Meijendel, due to having a similar soil structure as Meijendel.

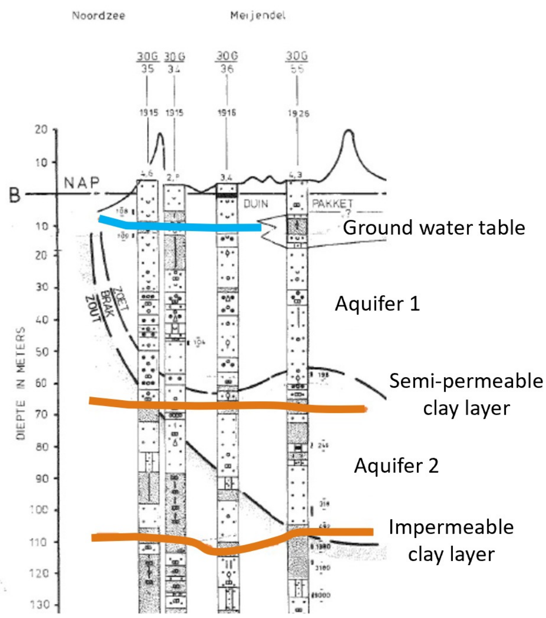


Figure 11.2: Overview waterlens in Meijendel (Ingenieurs Bureau Oranjewoud, 2013)

The groundwater table The groundwater tables in the areas of Vogelwijk and Duindorp are fluctuating around +1.5m NAP, whereas the ground-levels vary from 3 - 8m NAP (see Figures 11.3 and 11.4). The green line in Figure 11.4 displays the ground level. The largest storage capacity in the unsaturated zone is found in the area of Duindorp (A), which is about 6,0 m, as opposed to to 1,5 - 2,0 m at Vogelwijk (B, C). Therefore, it is favourable to design an infiltration pond nearby Duindorp.

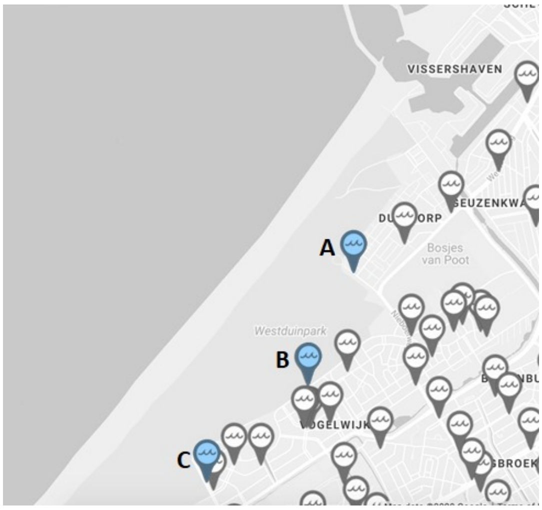


Figure 11.3: Locations of boreholes (Wareco Engineers, 2020)

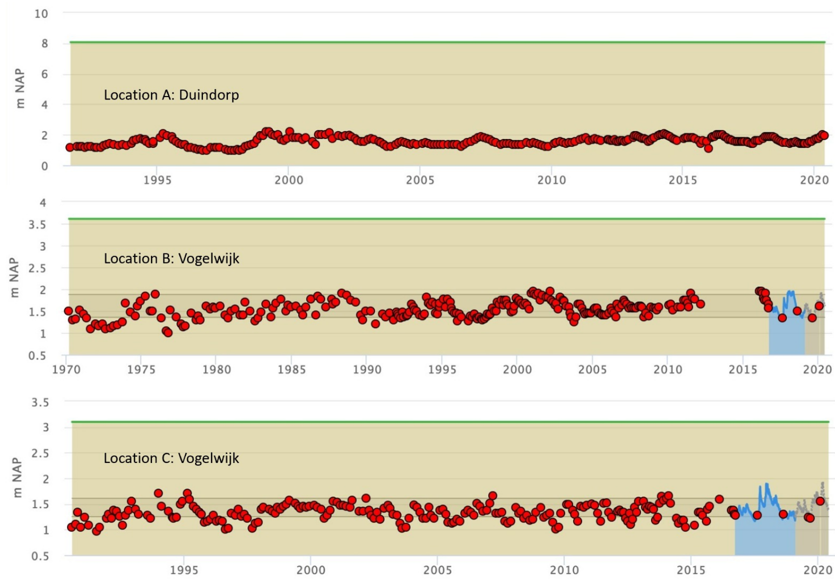


Figure 11.4: Recorded groundwater levels [m NAP] for respectively boreholes A,B and C (Wareco Engineers, 2020)

11.1.2. WWTP - Re-use of water

In the current situation WWTP Harnaschpolder treats water in its surrounding areas and transports its effluent with two transport lines towards the North-Sea. One of those lines transports the water towards WWTP Houtrust. Together with the effluent of WWTP Houtrust the combined effluent is released 2.5 km from the coast at Scheveningen on the North-Sea. The WWTP currently discharges a peak of 3,600 m^3/h during dry weather conditions, and a peak of 113,900 m^3/h for wet weather flow (Delfluent services, 2020).

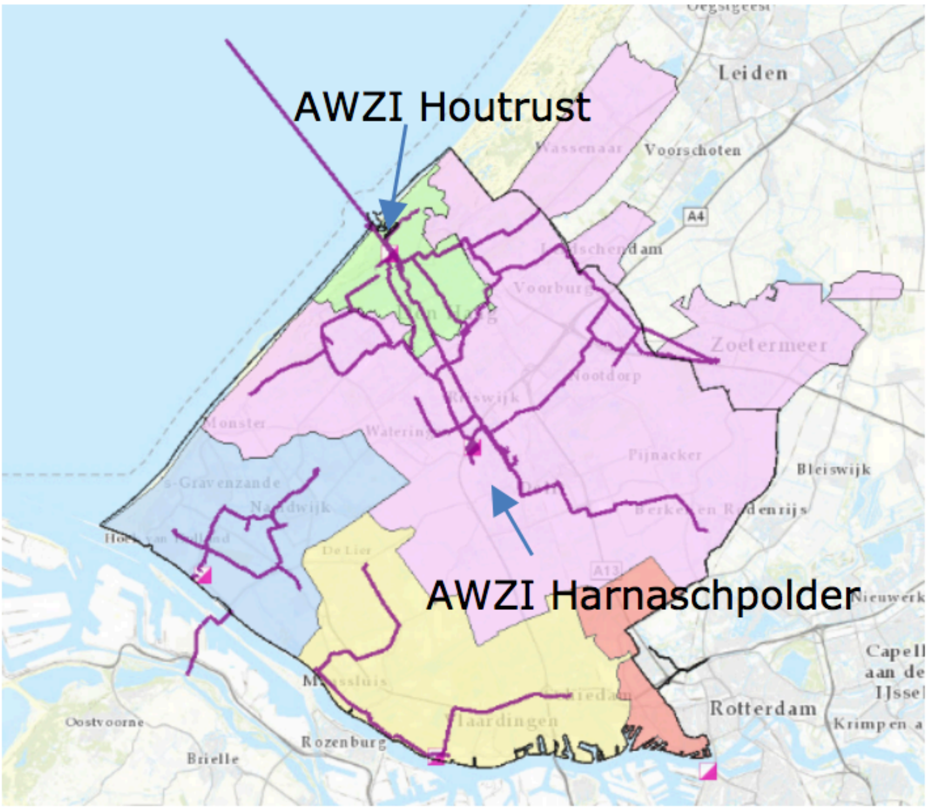


Figure 11.5: WWTP Transport lines towards the North-Sea (Koeman-Stein et al., 2014)

Delft Blue Water B.V. is a partnership between Delfland, Delfluent services and Evides to look into the re-use of effluent of the WWTP's Houtrust and Harnaschpolder (Koeman-Stein et al., 2014). This initiative stated fresh water quality parameters in 2015 which the effluent will meet by 2027 (Koeman-Stein et al., 2014). From research conducted by the partnership, the water produced from their pilot treatment plant is within the range of rain water quality as Table 11.1 displays. In addition to those parameters, the bacteria and virus concentration score very low according to (Besluit kwaliteitseisen en monitoring water, bijlage III, 2009). All values are within the range of the European parameters for surface water.

Table 11.1: Results of treated effluent versus national average of rainwater (Delft Bluewater, 2013)

Water quality parameter	Unit	Rainwater	Treated effluent
Cl-	mmol/L	0.05 - 0.26	0.03
Na+	mmol/L	0.05 - 0.25	0.12
EC	µS/cm	10 - 150	10
Organics	µg/cm	7	<detection limit

11.1.3. Fresh water lens recharge

Currently dune water treatment is executed by Dunea, who transports water from the Afgedamde Maas towards 2 dune locations around Scheveningen: Meijendel and Berkheide, as shown in figure 11.6. There is no water transport towards Westduinpark. One reason for this is that *Westduinpark is not used as a resource for drinking water production*, whilst the other dunes are used as a resource for drinking water production (Dunea, 2020). Another reason for not infiltrating Maaswater into Westduinpark is that this area is mostly constrained by the built environment of The Hague (See Figure 11.6).

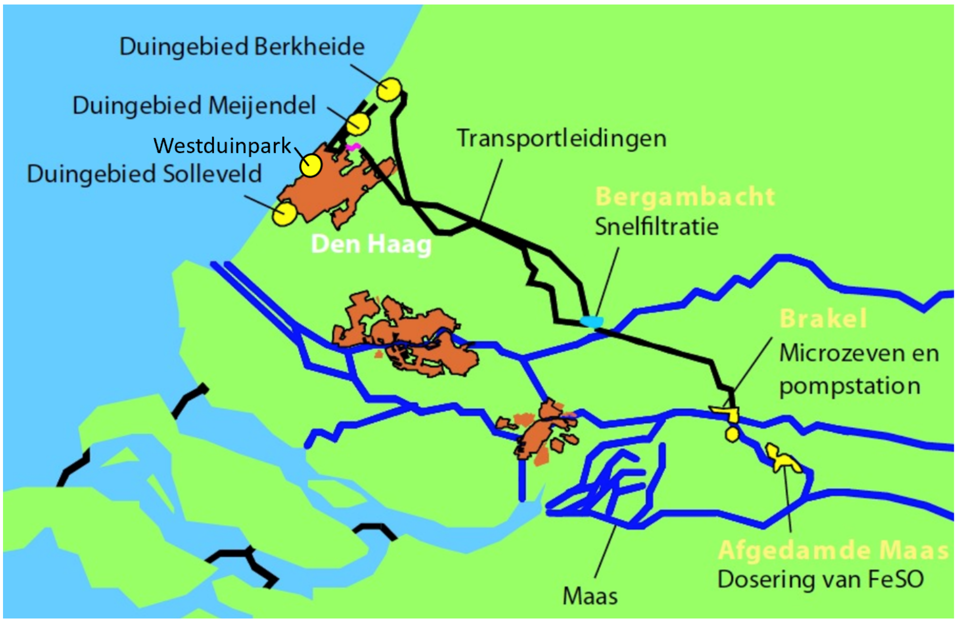


Figure 11.6: Transportlines Afgedamde Maas towards the dunes (Dunea, 2020)

11.2. Design

This section starts with a description of the concept and underlying reasons for decisions made in this concept. Then, specific boundary conditions and design goals that are needed to design this concept are listed shortly. After this, the different elements of the design are elaborated in detail.

11.2.1. The concept

The concept of this design is to use the treated wastewater from wastewater treatment plant Houtrust as a fresh water source to recharge the fresh water lens under Westduinpark. In order to achieve this, an *infiltration pond*, *pipeline system* and *pumping station* need to be designed with the main function of recharging the fresh water lens. Furthermore, the ecological variety can be expanded when fresh water is added to a dry area and the system should be the least of a nuisance to the surrounding inhabitants. The idea of using the wastewater as a fresh water source instead of discharging wastewater into the North-Sea, was discussed in an article by Koeman-Stein et al. (2014). From their research it was concluded that the treated water from WWTP Harnaschpolder was a promising addition of water as alternative fresh water resource.

11.2.2. Boundary conditions

The following boundary conditions are composed in order to create a design for the concept. In the permits according to the waterlaw, given by the Provincie Zuid-Holland, for incorporating changes into the Natura 2000 area, it is stated that these changes may not lead to disturbance of species or degradation of the area. Furthermore, with regards to groundwater it is stated that with deep infiltration no groundwater level rise is expected, hence the permit was issued.

- The design needs to be eligible for obtaining permits given by the Province of Zuid-Holland (Provincie Zuid-Holland, 2010) on the following bases:
  - Incorporating changes into a Natura 2000 area may not lead to the disturbance of species.
  - Incorporating changes into the Natura 2000 area may not lead to degradation of the area.
  - Infiltration projects may not lead to groundwater level rise in the neighbouring areas.
- Pipelines may not be constructed under buildings in order to have a minimal impact to nature. It follows than that pipeline segments will be constructed mainly under cross-section segments of roads, which are suitable to break open.
- The pipeline circumference is also limited by the maximum commercially available pipe radius of 1.5m (Kapelan, 2020).
- The pipeline diameter is limited by the width of the pavement, which is 2 meters.
- A solution should take into account the connection of the WWTP effluents pipeline in the future when it becomes of sufficient quality towards the new pipeline that will run towards the infiltration pond.
- The infiltration ponds may not lead to flooding in the cellars of nearby neighbourhoods.
- The minimum velocity of the water flow in the pipeline must be larger than 0.7 - 1.5 m/s, to have self-cleaning capacity and limit corrosion (Kapelan, 2020).

11.2.3. Design goals

The design goals are not strict, but these criteria have to be kept in mind when designing the system, or the system will be inefficient.

- Local head losses minimization in the pipelines.
- The infiltration pond has to be placed inside the dunes above the fresh water lens.
- The pump should be able to pump water towards the infiltration pond by overcoming the dynamic and static head losses associated with the pipeline.

- The infiltration pond capacity [m^3] should not overflow for rainfall events with a return period of 100 years.

- The pipeline radius and velocity in the pipe should meet the necessary discharges of the WWTP.

11.2.4. Location of Infiltration pond

For the location of the infiltration pond the groundwater tables of 3 locations around Westduinpark were retrieved (see Section 11.1.1.) It was found that the groundwater table was the lowest relative to the ground level at location A in Duindorp (See Figure 11.4), which makes the unsaturated zone ±6 m thick. From the cross-section of Figure 11.1 which is assumed homogeneous throughout Westduinpark, the unsaturated zone is predominantly sandy soil. With regards to the groundwater table at location A having relatively more room for fluctuation opposed to the other borehole points, the reservoir is chosen to be placed in this area. From the elevation map and base map (Figure 11.9), the area north of Duindorp had enough space for a reservoir and contained relatively low areas which are favourable for the static energy loss the pipeline and pumping station have to overcome.

To show the impact of the infiltration pond on the groundwater table, a short calculation is made based on the infiltration rate and the dispersal area for this water. The boundary of the aquifers towards the hinterland goes all the way to Leidschendam (See Figure 11.7). Hence, the dispersal of water will be over this full area. This area is hard to quantify due to the unknown dimensions of the dispersal surface parallel to the North-Sea. Therefore, a simplified area is chosen with Westduinpark as its boundaries for indicative purposes.
The boundary conditions of the fresh water lens are now assumed to surround at least the entire Westduinpark, which is an area of 2.35 km^2. A calculation will bring forward that the rise of the groundwater table caused only by the infiltration pond is then 6 - 18 mm/y, using the infiltration capacities elaborated under Section 11.2.7. In reality this will be lower, since the calculation is done over the surface area of the dunes instead of over the full aquifer.

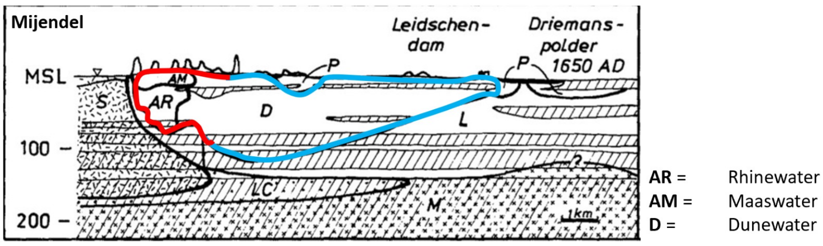


Figure 11.7: Side overview of full aquifer range (Stuyfzand, 1993)

Due to the size of the fresh water lens and its large boundary conditions, the assumption is made that the storage of the lens is infinite. This implies that the groundwater level cannot be increased significantly by the infiltration pond. For comparison, a typical rainfall event is in magnitudes of mm/d while the rise of the groundwater table due to infiltration is in mm/y.

In Figure 11.8, the location of the infiltration point is sketched on smaller scale to show how the infiltration pond interacts with the fresh water lens, and how it will not interact with Duindorp.

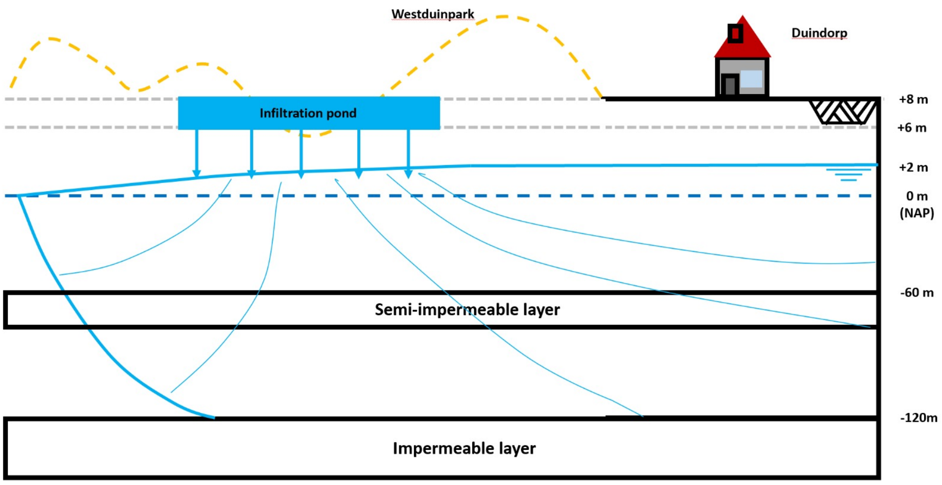
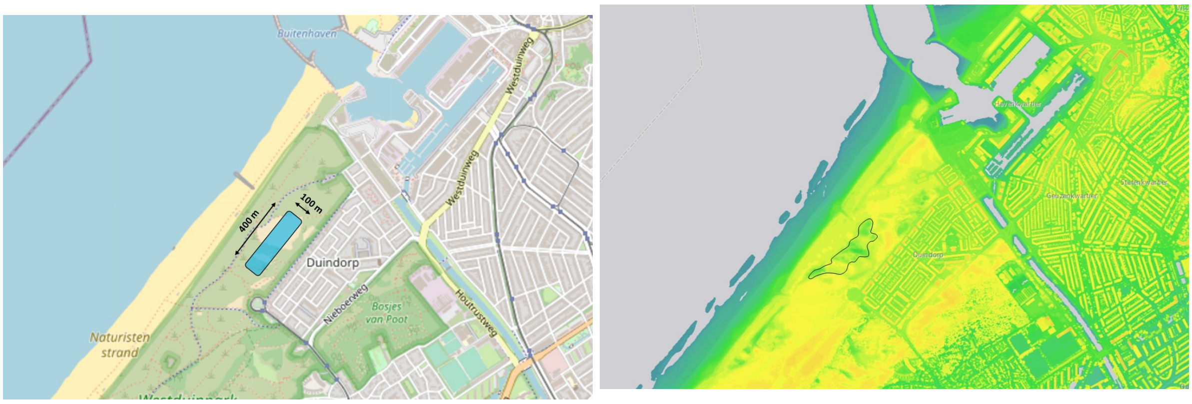


Figure 11.8: 2D geo-hydrological overview infiltration pond

Cellars around the infiltration pond will therefore not be flooded due to this infiltration. Once the water reaches the fresh water lens, the horizontal flow starts to play a significant role. The water starts to disperse from the initial infiltration area into all directions across the full surface area of the fresh water lens. The boundaries of the infiltration pond follow the natural elevation lines which are formed by the dunes surrounding this location. This can be identified as *building with nature*, which was a low scoring criteria on the MCA for the concept of Less is More. Implementing this ideology for the infiltration pond, would compensate for this. For the calculations this was assumed as a rectangle which is seen in Figure 11.9b.



(a) Base map (OpenStreetMap contributors, 2017)

(b) Elevation map (Algemeen Hoogtebestand Nederland (AHN), 2020)

Figure 11.9: Location infiltration pond

11.2.5. Dimensions of infiltration pond

It is chosen to design an infiltration pond with an area of 4 ha (Figure 11.9), because the location of the infiltration pond has natural boundaries formed by the dunes. The current hiking trail distance will remain similar after implementation of the pond in this manner.
The depth of the infiltration ponds in Meijendel are mentioned to be 1.5 - 3 m (Dunea, 2020). Mijendel's infiltration pond is proven functional in practise for both infiltration and ecological reasons, therefore it is decided to use a depth in this range as a rule of thumb. Since the infiltration flow is much smaller than the discharging capacity of WWTP Houtrust, the storage capacity of the infiltration pond does not have to be deeper than 2m. This is due to the main function of the infiltration pond being provision of infiltration rather than provision of storage.
A depth of 2 m is eventually chosen as this is a value on the lower side of the range between 1.5 - 3 m and will save excavation costs in the design. The average ground level is at approximately +8 m

NAP, hence the bottom of the pond will be located at +6 m NAP (See Figure 11.8). An impression of the infiltration pond is provided in the figure below.

Figure 11.10: Impression of the infiltration pond, Duindorp in the background

11.2.6. Water balance

The water balance is set up for the infiltration pond from which the water infiltrates into the fresh water lens. The water balance is a function of the precipitation, evaporation, infiltration and discharge from the WWTP. A schematic overview is presented in Figure 11.11.

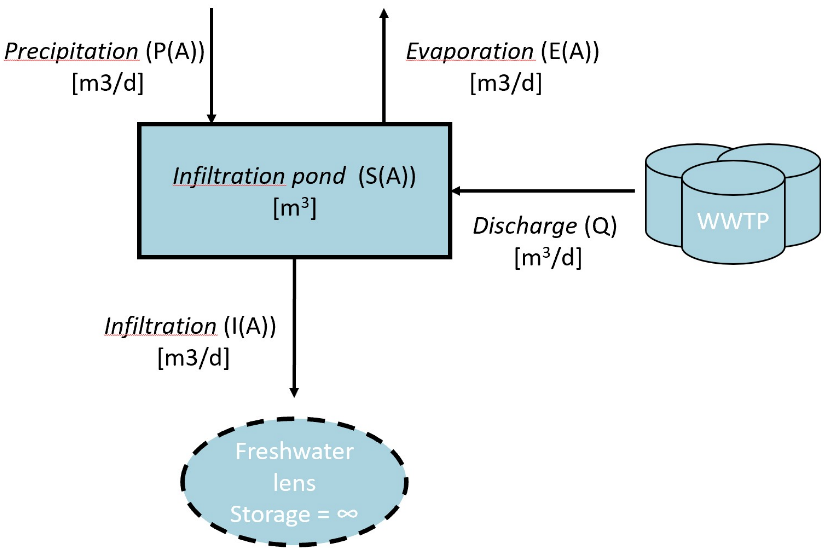


Figure 11.11: Water balance of infiltration pond

The equation describing the water balance is as follows:

$$S(t) = S(t-1) + Q(t) + P(t) - E(t) - I(t)$$

- S(t) = Storage per day [m^3/d]
- Q(t) = WWTP discharge [m^3/d]

- E(t) = Evaporation per day [m^3/d]
- P(t) = Precipitation per day [m^3/d]
- I(t) = Infiltration per day [m^3/d]
- t = time [d]

The water balance is utilised to determine the design discharge of the WWTP towards the pond of which the dimensions are determined in Section 11.2.4. The discharge that is pumped through the pipe is a variable in time, the aim is to define a water control plan for the discharge that sustains the water level at a height of 1.75 m, with a free board of 0.25 m as buffer for when a higher precipitation intensity occurs.

11.2.7. Infiltration capacity

As mentioned in subsection 11.1.1, the sand layers in the dunes can infiltrate about 0.1 - 0.3 m/d (Van Dijk, 1984). With an area of 40,000 m2, this means that the total water flow towards the fresh water lens will vary between 4,000 - 12,000 m^3/d.

Precipitation

The precipitation data set was retrieved through the KNMI website (KNMI, 2019) for area of Scheveningen. The data contained daily measurements for the period of 1951-2015. The mean daily rainfall and its standard deviation were determined per year. A Gumbel analysis is then conducted with the use of the mean and standard deviation (std) values per year. Using this analysis the mean and std were retrieved for return periods of 10 years and 100 years. When following this method, the rainfall values retrieved are expressed in mm/d.

The rainfall values are assumed normally distributed throughout the year with respect to seasons, opposed to the data set itself of values per year which is Gumbel distributed. In Figure 11.12 the plot is displayed of the analysis, where the values above are displayed.

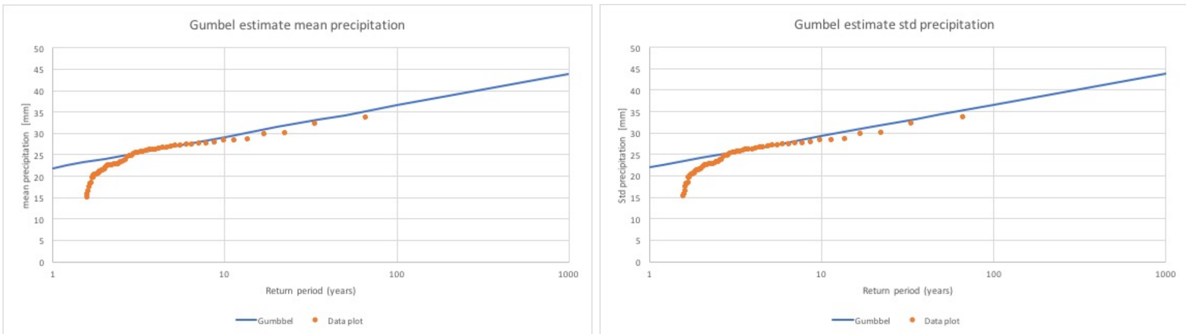


Figure 11.12: Gumbel plot for mean and standard deviation, T =10 years: N(29.2, 57.1) , T = 100: N(36.5, 70.9)

A normal distribution for the values generated by the gumbel analysis is then used to randomly generate realistic values for the rainfall in the model described under Section 11.2.10. When using extreme rainfall scenarios as presented in the method by (van Weeren et al., 2018), the pond would be over designed. Our method takes a mean and std, which by using the normal distribution enables extreme values, that confirmed possible in their findings, to be taken into consideration in the model. In addition, the findings of extreme rainfall for both return periods and their respective duration are covered in the values of the normal distribution presented above.

11.2.8. Evaporation

The data for the evaporation was also retrieved through the KNMI, however, for the area of Zoetermeer. The Evaporation is a very small flux compared to the other fluxes from the water balance, therefore it was decided to calculate the yearly cumulative evaporation. To create the model, a normal distribution with random picks from this distribution is used. This is done to create a variation of possible outcomes when running the model. From parameters (mean & std) of this distribution, the evaporation values were generated in the model (See Section 11.2.10). A mean evaporation of 2.32 mm/d was found, with a std of 1.21 mm/d were retrieved through KNMI data (KNMI, 2019).

11.2.9. Effluent Discharge

The Dry weather flow from the WWTP is characterized my a maximum of 3,600 m^3/h. In the figure the typical diurnal pattern in The Netherlands is shown. The peak factor used in the Netherlands is 2.4 (Faculteit Civiel Techniek en Geowetenschappen, 2013), hence the average flow per hour reads 1500 m^3/h.

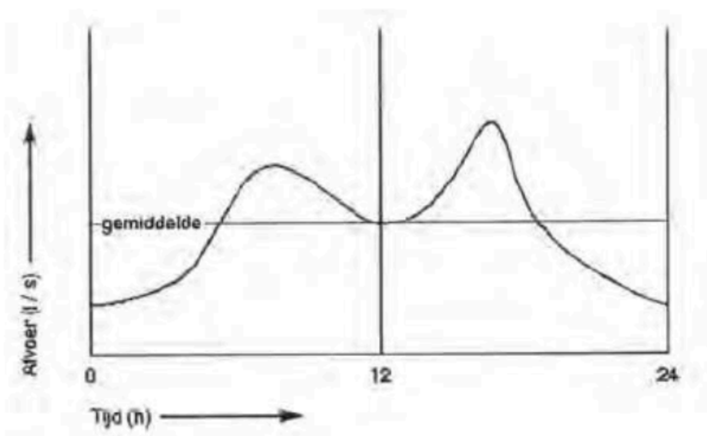
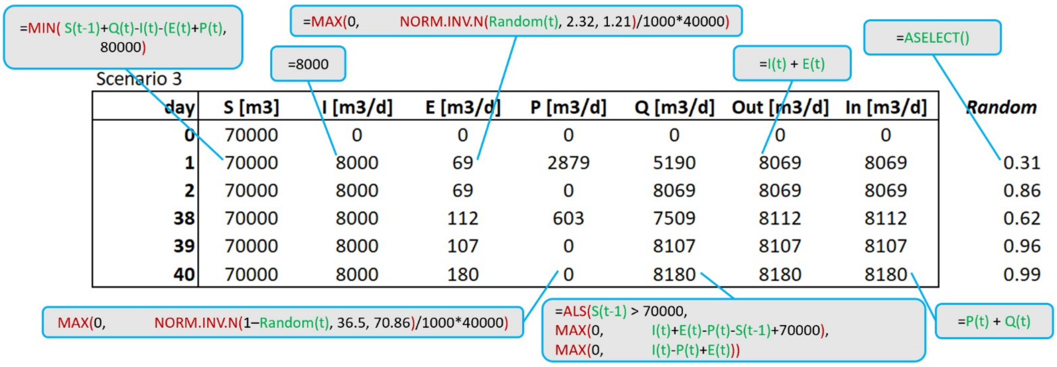


Figure 11.13: Diurnal pattern WWTP (Faculteit Civiel Techniek en Geowetenschappen, 2013)

It is important to note that the discharge from the WWTP to the infiltration pond will be varying based on the real time water height measurements in the infiltration pond. When a lower height than 1.75 m is measured, a higher discharge will be pumped from the WWTP to the infiltration pond, and when a higher height is measured, there will be no discharge. This can only be realised when there is sufficient discharge capacity on a daily basis. This means that in the WWTP a separate pumping installation is required to be installed that pumps towards the infiltration pond, instead of the North-Sea. Once the infiltration pond reaches the desired maximum water level, the pump will be shut down and full discharge to the North-Sea will occur. An average discharge of 36,000 m^3/d (1500 m^3/h) indicates the magnitude for a possible discharge that can be used for the water balance. It is expected that the maximum discharge of the WWTP will be too large for the infiltration pond, hence the pipeline will be fitted considering the average discharge.

11.2.10. Design scenarios

Three scenarios are described in this chapter, completing the water balance for multiple time steps, which is eventually used to calculate the necessary pipeline dimensions It is chosen to work with 3 scenarios because this can cover a minimum, maximum and likely situation for the water balance in terms of flows. The scenarios are sketched for a period of 40 days, with a day 0 to create the initial storage. Day 0 has no water flows in forms of evaporation, precipitation, discharge or infiltration. This gives a vertical line at the start of some graphs, which should not be considered as a fluctuating value.



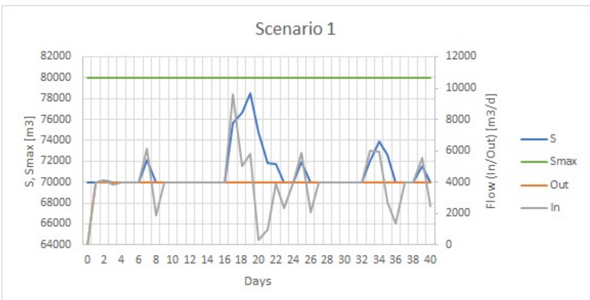
Scenario 3

day	S [m3]	I [m3/d]	E [m3/d]	P [m3/d]	Q [m3/d]	Out [m3/d]	In [m3/d]	*Random*
0	70000	0	0	0	0	0	0	
1	70000	8000	69	2879	5190	8069	8069	0.31
2	70000	8000	69	0	8069	8069	8069	0.86
38	70000	8000	112	603	7509	8112	8112	0.62
39	70000	8000	107	0	8107	8107	8107	0.96
40	70000	8000	180	0	8180	8180	8180	0.99

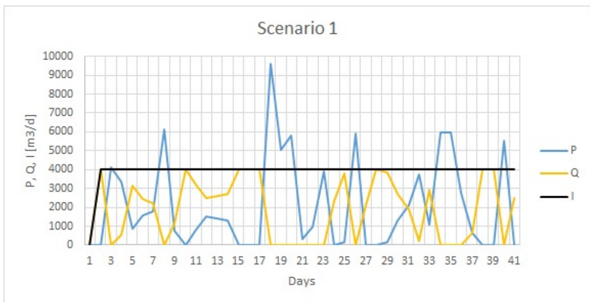
Figure 11.14: Utilised model for the different scenarios

Figure 11.14 explains the Excel model that is used to create the different scenarios. The red text shows the Excel functions, with language settings on Dutch. The green text shows cells references. For readability purposes, the cell references are written down as functions of time. In the real situation, the discharge will be adapted based on water level measurements in the infiltration pond. In this model, this measurement is a given and the daily discharge is then calculated by using the water balance of that day and the desired water level, which is set at 1.75 m (70,000 m^3). The MAX() functions are used to rule out negative numbers, because the discharge, evaporation and precipitation will never be negative. The MIN() function for the storage is used in the same way, to make sure the storage will never exceed a value of 80,000 m^3, because that is the storage limit of the infiltration pond. ASELECT() picks random values between 0 and 1, which is the input for the NORM.INV.N normal distribution function to pick random values for the evaporation, based on the mean and standard deviation from the Gumbel analysis. For the generation of precipitation normal distribution picks, an inversely proportional relationship of the random value is used.

Scenario 1 covers the minimum flow for the water balance. To create these conditions, the minimum infiltration rate of 0.1 m/d is used, in combination with precipitation generated according to the normal distribution (see Section 11.2.7 and neglected evaporation. This scenario is important, because the precipitation in a very wet year (with return period 100 years) can be higher than the infiltration now, causing storage fluctuations regardless of the discharge. In the other scenarios, the storage will not fluctuate as much.



(a) Storage (S), maximum Storage (Smax), total Outflow (I+E), total Inflow (Q+P)

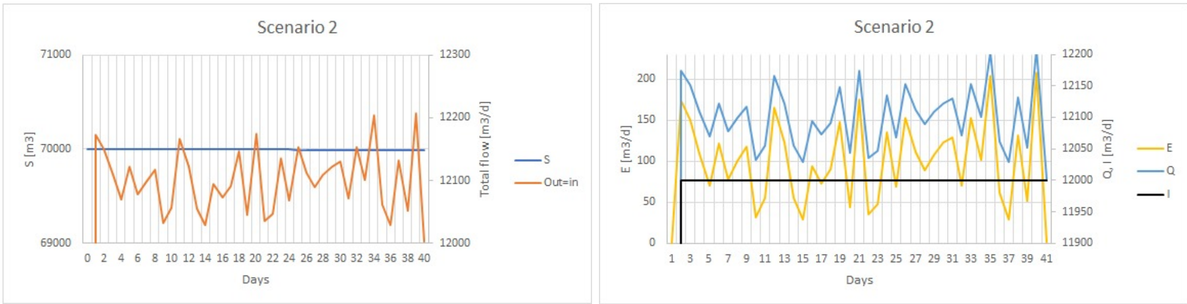


(b) Precipitation (P), Discharge (Q), Infiltration (I)

Figure 11.15: Graphical representation of the water balance of scenario 1

In Figure 11.15a, the maximum storage of 80,000 m^3 is plotted and the baseline storage of 70,000 m^3. The out going flux is constant throughout time, which just consists of the infiltration. Fluctuations in this scenario of the in going flow are largely related to the rainfall. When a larger precipitation event occurs, effluent of the WWTP adjusts to revert back to the base storage of 70,000 m^3 (see Figure 11.15b). Figure 11.15a shows that the discharge is designed to be inversely proportional to the precipitation, but high peak of the precipitation can only be dissolved by discharge values of 0 over a longer period of time.

Scenario 2 covers the maximum flow for the water balance. These conditions are created by using the maximum infiltration rate of 0.3 m/d, combined with neglected precipitation and randomly picked evaporation. This scenario is important because it will generate the maximum discharge Q that needs to be able to flow through the pipeline from WWTP Houtrust towards the infiltration pond. In the other scenarios, this discharge is considerably lower.



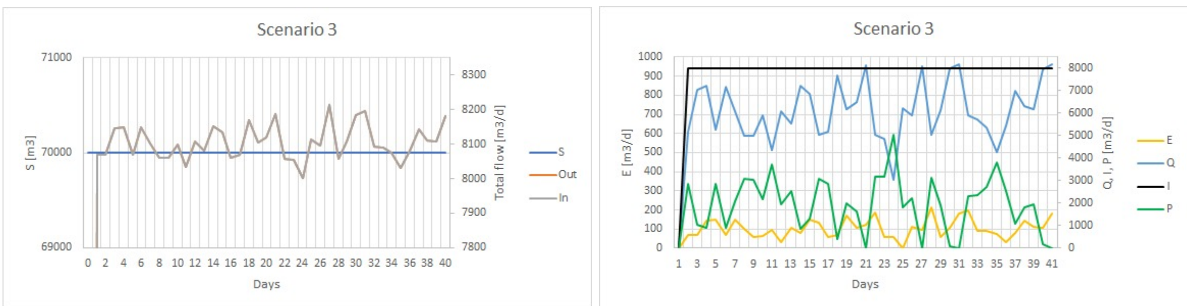
(a) Storage (S), total Outflow (I+E) = total Inflow (Q+P)

(b) Evaporation (E), Discharge (Q), Infiltration (I)

Figure 11.16: Graphical representation of the water balance of scenario 2

The total outflow in this scenario equals the total inflow, as shown in figure 11.16a. The underlying reason is that the precipitation is neglected, therefore the inflow (discharge) can always be adjusted to match the outflow (evaporation and infiltration). The total outflow is matched by the inflow of the WWTP discharge, therefore it is easy to maintain the base level of 70,000 m^3. Both graphs (Figure 11.16b) follow the same pattern because the infiltration is constant and the discharge is defined by infiltration and evaporation. The only difference is that the magnitude of the value is much higher for the discharge due to the infiltration.

Scenario 3 will be the most realistic scenario to occur, and takes into account all fluxes. This scenario therefore makes use of the infiltration rate in the middle of the range, namely 0.2 m/d, combined with a randomly generated evaporation which is linked to the generated precipitation with a return period of 10 years.



(a) Storage (S), total Outflow (I+E), total Inflow (Q+P)

(b) Evaporation (E), Discharge (Q), Infiltration (I), Precipitation (P)

Figure 11.17: Graphical representation of the water balance of scenario 3

In this scenario, the storage is constant most of the time. When a very high flux of precipitation comes in, this storage could temporarily increased, but this is not the case in this scenario. A mean and standard deviation of the daily rainfall in a year is set at 57.07 mm and 29.21 mm, which are lower values than given in scenario 1. With the infiltration being constant over time, the height of the water table in the infiltration pond will only increase when a precipitation larger than 8,000 m^3/d occurs, which equals 200 mm in a day. This precipitation on a daily basis rarely happens with the current mean and std values.

Furthermore, the discharge is inversely related to the precipitation again, just like in scenario 1. The discharge is also following the pattern of the evaporation, just as in scenario 2.

From this model it is found that the maximum necessary discharge of the WWTP towards the designed infiltration pond is 12,207 m^3/d (scenario 2). The infiltration pond will have a maximum volume of 80.000 m^3 (2.00 m), of which 10.000 m^3 (0.25 m) is used for compensating for extreme weather. This buffer for extreme weather is deemed sufficient, as shown in scenario 1. The total recharge of the fresh water lens due to the infiltration pond will range between 4,000 and 12,000 m^3/d, not accounting for precipitation.

11.3. Pipeline system

The pipeline system will be fitted to the discharge of the effluent which was retrieved in the previous section. Only a part of the discharge can be redirected towards the infiltration pond due to having only a limited amount of surface area. The rest of the WWTP effluent will still be discharged into the North Sea. A proposal is to connect the dune area's Meijendel and Solleveld, which are nearby. This idea would require further research and is not within the scope of this project. The design is made for scenario 3, for which a pipe diameter will be taken. This pipe diameter is then checked for the discharges found in the other scenarios. To calculate the area for a pipeline to fit for the discharge, the following formula is used:

$$Q(t) = \pi * r^2 * v(t)$$

- Q(t) = WWTP discharge [m^3/s]
- r = radius of the pipeline [m]
- v(t) = flow velocity [m/s]
- t = time [s]

The results of this formula are shown in the table below. Area* and Radius* implicate the values that are necessary to fit the discharge of the scenario. Then, a final radius is chosen by checking the commercially available standard pipe dimensions (Kapelan, 2020).

Table 11.2: Pipeline design values

Scenario	Design velocity (m/s)	Design discharge [m^3/d]	Design discharge [m^3/s]	Area* [m2]	Radius* [cm]	Final radius [cm]	Final area [m2]	Final velocity [m/s]
3	1	8183	9.47E-2	9.47E-2	17.4	17.8	9.95E-2	0.95
2	1.5	12207	1.41E-1	9.41E-2	17.3	17.8	9.95E-2	1.42

The pipeline will be placed under the paved area surrounding Duindorp, the shortest way towards the infiltration pond from the WWTP is along the Houtrustweg along the North-side of Duindorp (see Figure 11.19). In this setup, the nature 2000 area and surrounding neighbourhoods will be impacted the least and the dynamic head loss is minimized. Reasons to put the pipeline on 7 m NAP (1 meter below ground level) are minimization of excavation/maintenance costs in combination with a protecting layer of soil on top of the pipeline. The construction of the pipeline at the Houtrustweg (the first 560 m as depicted in Figure 11.18 can be combined with the construction of the tram track in the same street, as elaborated in the Infrastructure section. The static head loss is represented in Figure 11.18, the pump takes water in at a level of +8m NAP and transfers it to the same level, hence the total static head increase if zero.

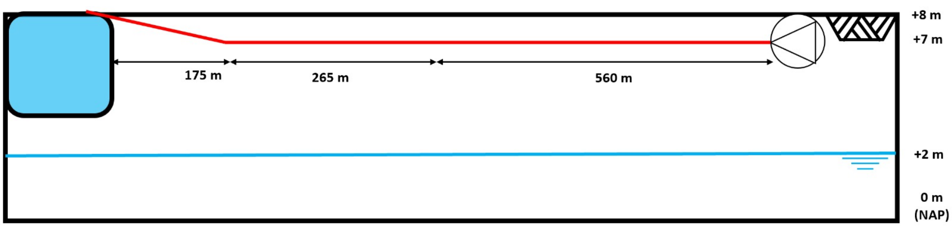


Figure 11.18: Schematic overview of pipeline situation side

The pipeline will be approximately 1000 m long, and includes one major bend. The dynamic head loss is described with the following formula.

$$hl = \sum \lambda \frac{L}{D} * \frac{v^2}{2g} + \sum \xi \frac{v^2}{2g}$$

- H_l= Total headloss [m]

- λ= Friction factor [-]
- v = Flow velocity [m/s]
- g = 9.81 [m^2/s]
- ξ= local loss factor [-]

Where ξ, equals the local loss coefficient. The lowest possible value for this coefficient is 0.16 adapted from (Elger et al., 2015). To achieve this lowest value, an r/D ratio of 4 is needed, which makes the inner radius of the bend 1.424m (See Figure 11.19). λ, is described by the simplified-formula of White-Colebrook:

$$\frac{1}{\lambda} = 2 * log(\frac{3.71d}{k})$$

- λ= Friction factor [-]
- d = pipe diameter [m]
- k = roughness [mm]

In practice, sewer systems are usually over designed causing their lifespans to be longer than anticipated when proper maintenance is executed. Systems are usually still utilised when their roughness has decreased over a long period of time, thus making use of a pro-active asset management strategy the roughness of old pipes was chosen. Hence the k-value (roughness) of the PVC pipe was assumed according to this criterion to be at 0.30 mm (Butler and Davies, 2000). The total dynamic head loss from the above is than calculated to be 38.13 m.

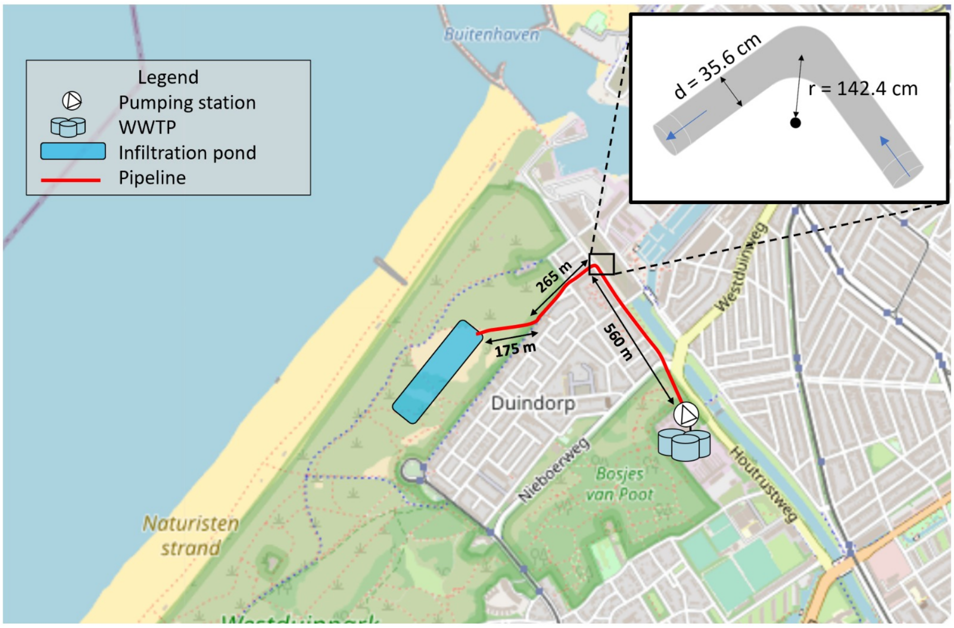


Figure 11.19: Schematic overview of pipeline situation (OpenStreetMap contributors, 2017)

11.4. Pumping station system

The pumping station will have to be able to overcome a total head loss of 38.13 m and requires a maximum discharge of 0.14 m^3/s, as depicted in Figure 11.20. The Q-H curve is displayed below, where the pumping characteristic is described by the total head loss (See section 11.3.
The pump characteristic belongs to pump type: Flygt N-Technology N 3301. In the Q-H diagram the pump curve is plotted versus the pipeline characteristic in the following figure:

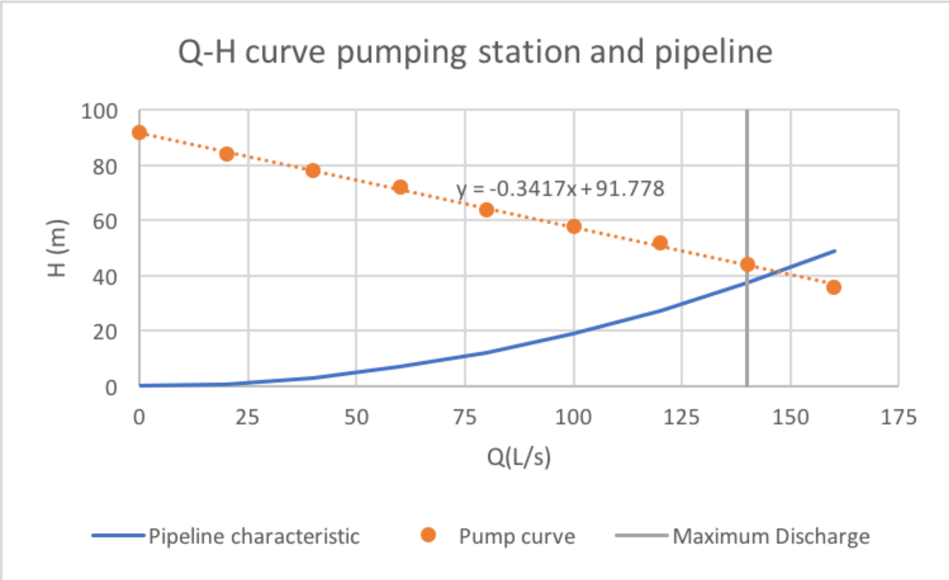


Figure 11.20: Schematic overview of pipeline situation

From the Figure above it is seen that the duty point (intersection of both curves) lies at 145 l/s which can overcome a head-loss of 40m, therefore the pump fits the pipeline. The efficiency will be between 65-70% , which is nearby the maximum efficiency of 73% of the pump.

11.5. Conclusion

This design proposes an infiltration pond in Westduinpark to recharge the fresh water lens. The infiltration pond is shaped in a manner that complements the current dune layout, for improving the score of the criterion of building with nature of the overall Less is More concept. The infiltration pond itself will be recharged by treated wastewater from WWTP Houtrust, pumped via a pipeline system. An overview of the designed components are mapped on Figure 11.21. The following physical design components were established in this report:

- The area of the infiltration pond will be 40,000 m^2.
- The total recharge capacity of this infiltration pond ranges between 4,000 and 12,000 m^3/d, which will not increase the groundwater table.
- The pipeline leading to the infiltration pond will have a final diameter of 17.8 cm and made of PVC material.
- The pump that was has been selected is type Flygt: N-technology N3301, which functions optimally at 145 l/s.

Figure 11.21: Overview of design including all components (OpenStreetMap contributors, 2017)

11.6. Discussion

Various points of discussion of this research are listed first, after which a list with recommendations for further research is listed.

- The actual dimensions of the fresh water lens at Westduinpark and how it stores infiltrated water could make it more evident in how much the salinity intrusion will be slowed down. An addition to this would than be taking into consideration the broadening of the beach.
- The Gumbel analysis used for the precipitation extreme values, however it fitted our data points very well. Also, when comparing it to the regular method by Van-Weeren et al. (2018) it accommodated for those extreme values listed for a duration of larger than 12 h.
- The normal distribution that was assumed for the evaporation, was done to generate random picks. This flux is relatively small compared to the others, hence it wouldn't matter match.

Recommendations for further research:

- Gemaal Schouten discharges into the ocean currently, this water could be discharged into the infiltration ponds around Meijendel or Solleveld instead, or new storage basins. This could include routing towards these new locations. When considering new storage basins, note that buffers in the city itself that collect the runoff can be given.
- Extra locations for infiltration ponds in Westduinpark could be found to accommodate for the discharge of Gemaal Schouten and the excess discharge of the WWTP.
- When the fresh water lens has sufficiently recharged, it could be interesting to look into the retrieval of drinking water from Westduinpark. This would include a design of a drinking water treatment plant and how it could be distributed through the (surrounding area) of Scheveningen.
- After the treated wastewater from WWTP Houtrust is used for the infiltration pond, the rest of the discharge could be directed towards the greenhouses in the Haaglanden. The suitability of the treated water for cultivation purposes is already proven by (Koeman-Stein et al., 2014). The next step is to design a way to move this water from the WWTP to the Haaglanden.
- More insight into the costs of the implementation of the infiltration pond and its necessary infrastructure could be given.
- The infiltration pond could be used for recreational purposes, the research to back this up would need to include a way to ensure the water quality within the infiltration pond. Because a slight reduction in the quality of the WWTP effluent could lead to a potential risk for visitors.
- In addition to the infiltration pond for enlarging the water lens, a slok-op can be implemented. These are infiltration pipes that are sunk into the deeper ground water layers. When the natural infiltration is clogged, or the top-layers are saturated the water can still infiltrate to the deeper layers. Further research could look into the increased infiltration rate of slok-ops.

12

Infrastructure

12.1. Design concept Infrastructure

The current infrastructure between the harbour of Scheveningen and the city center of The Hague is not designed to accommodate for cruise ship tourism, which is added at the fourth harbour in Less is More. To accommodate for this extra flux of tourism, a tram shuttle will be added from the fourth harbour to the city center of The Hague. On its way back, it can provide a connection towards the fourth harbour for beach visitors.
The tram is chosen over other modes of transport for this new flux of tourism, because this is in line with the transport vision for Scheveningen 2025 (Gemeente Den Haag, 2016). The implementation of a tram track towards the harbour is already in the plans of municipality, although these plans are in a very early stage.
The tram line will connect the fourth harbour to The Hague via the Kranenburgweg and Houtrustweg, which inevitably will cross the Duindorpdam. The Duindorpdam includes two crossroads and is known for not being safe in the current situation due to its road layout. Improvement of the road layout is another main theme mentioned in the transport vision for Scheveningen 2025 (Gemeente Den Haag, 2016). In order to create a design for the tram track, a more detailed re-design of the road layout of the Duindorpdam crossroads will be proposed.
The design deliverables will consist of a design of the tram line that will mainly connect tourists between the cruise ship terminal and the city center of The Hague and a more detailed re-design of the crossroads at the Duindorpdam.

12.2. Market analysis

Normall, a market analysis would be conducted for the future scenarios where cruise ships will be docking in the fourth harbour. This leads to an increase in movements will be caused mostly by tourism influx. Part of this analysis would be a comparison to see which transport method would be the most feasible for linking the harbour to the city center based on the modal-share. However, due to lack of time in this department and the vision of the municipality, it was chosen to opt for a tram sans this analysis.

The second part of a normal market analysis would focus on mapping the current and predicted fluxes. However, the added tram in this design will mainly function as a shuttle between the cruise terminal and the city center of The Hague, with a second function to create a stronger connection between The Hague and Scheveningen beach. Hence, it will only operate for the cruise ship tourists, rendering a regular market analysis redundant. The market analysis for our will focus on the magnitude of the extra demand created by the cruise ship tourism.

Passenger analysis The passenger analysis is conducted to quantify the passenger fluxes that will be using the tram. This analysis consists of:

1. Quantification of the cruise ship tourism.
2. Quantification of the exit rate at a cruise ship (maximum boarding rate of the tram).
3. Determining the share of travellers of the cruise ship that is heading to the city center of The Hague.

One cruise ship at a time will be allowed for Scheveningen (reference to harbour berths chapter). Those cruise ships are estimated to carry 720 tourists each (See Table 10.1). A cruise ship will arrive in the morning and leave in the afternoon/evening, therefore only 1 cruise ship can be serviced a day.

When the passengers leave the ship, they will have to check out at the exit point of a cruise ship. This is usually simply done by swiping a personal card passed the security computer combined with a facial check. It is assumed that this process takes 5 seconds per person, therefore the maximum outflow of passengers is set at 12 persons/minute. This does not imply that everyone will stand in a line to exit the ship, because there is a distribution of desired exit times. Not only desired exit times play a role here, but also the limitation of not having simultaneous breakfast on cruise ships will distribute the outflow of passengers. Although, for the tram shuttle design, the maximum outflow of passengers will be used.

The predicted distribution of tourists going to the the city center of The Hague versus Scheveningen harbour is estimated by the current annual tourist ratio. The tourists visiting the harbour and the center are respectively 13.5 million and 30 million, resulting in 70% of the 720 tourists visiting the city center by tram and 30% staying in Scheveningen (Gemeente Den Haag, 2018), (PSO, 2015). This results in a traveler rate for the tram with a maximum value of 8.4 travelers/minute.

An extra market for this tram is to bring a fraction of the inhabitants and visitors of The Hague to Scheveningen beach in the morning, and back to The Hague in the evening. Otherwise, the tram would be empty during the way back. This flux of passengers is not used as a design criteria, because it is out of the scope of this project. Although, this design will leave possibilities to account for these flows.
The capacities and frequencies that will supply for this demand will be elaborated in section 12.5.

12.3. Technical requirements

In order to design a realistic tram system, technical requirements are listed below. It is important to know technical requirements concerning standard dimensions of street cross-sections, the radii for the rails and clearance zones of the tram in order to integrate the tram track into the existing infrastructure. These technical requirements are mostly spatial, therefore spatial sacrifices could be necessary to implement a tram track.

- The clearance / safety zones next to the tram tracks are conform to document 'Kadernota straten, wegen en lanen', written by the municipality of The Hague (Gemeente Den Haag, 2015).
- If a specific clearance / safety zone is not listed in 'Kadernota straten, wegen en lanen', 'Guidance on Tramways' by Office of Rail Regulation (2006) is used (Office of Rail Regulation, 2006).
  - The edge of the tram needs a minimum distance of 300 mm from the sidewalk for safety purposes.
  - A traction pole, placed between 2 tram tracks is required to have a minimum distance of 100mm from each tram (200mm total, excluding it's own diameter).
- Width of parking lots, cycling lanes, pedestrian roads, roads are conform to the 'Kadernota straten, wegen en lanen', written by the municipality of The Hague (Gemeente Den Haag, 2015).
- The new tram track must fit to tram characteristics of the Regio Citadis tram. Which is a modern bi-directional tram type that is currently in use in The Hague (See Figure 12.1).

  - The minimum curve radius is 23 m (Hansen et al., 2011).
  - The distance between the rails is 1.435 mm (Hansen et al., 2011).
- The tram track requires a reversing/switching possibility at both system ends.

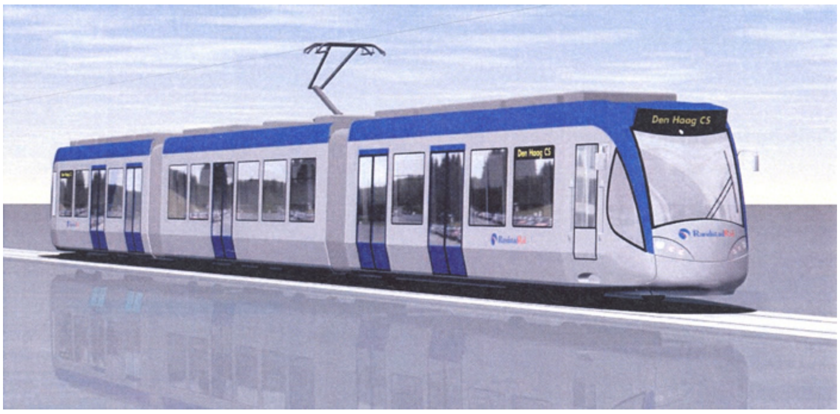


Figure 12.1: Regio Citadis tram (Randstadrail, 2006)

12.4. Tram track layout

A tram track between the fourth harbour in Scheveningen and the city center of The Hague needs to be designed on various levels. The tram track is split in a new built part, and a current existing part. The layout for a route will be developed in this chapter. First, multiple options for the tram track are identified, then cross sections and more detail will be elaborated.

12.4.1. Identified options for tram track

It is most evident for the new tram track to converge with the current infrastructure of Line 1. In Figure 12.3 the current track is displayed, a few possibilities for how the new tram track can be connected to it are identified. Table 12.1 connects the options from Figure 12.3 to the layout of the tram track that is considered when a certain route is chosen. When considering the blue route, there are options to implement a single track for 2 directions on one street, Gauntlet track for 2 directions in one street, 2 single directional tracks in one street, or 1 single directional track per street. An overview of the three track types is provided in the figure below.

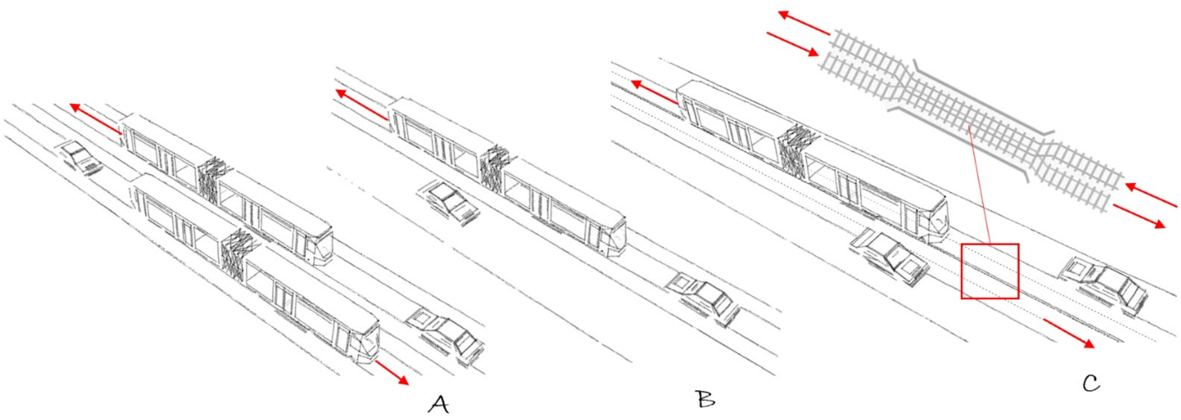


Figure 12.2: Different tram track styles (a) 2 way, (b) 1 way, (c) gauntlet track. The red arrow indicates the tram direction.

The 2 directional track and single tracks are quite standard forms for a tram track. The gauntlet track for 2 directions is basically 2 tracks sharing the middle clearance zone where their inner rails are placed (See Figure 12.2c), this creates a situation where only 1 tram can drive on this track at a time, but the tram will drive on the right side of the road at all times, instead of in the middle.

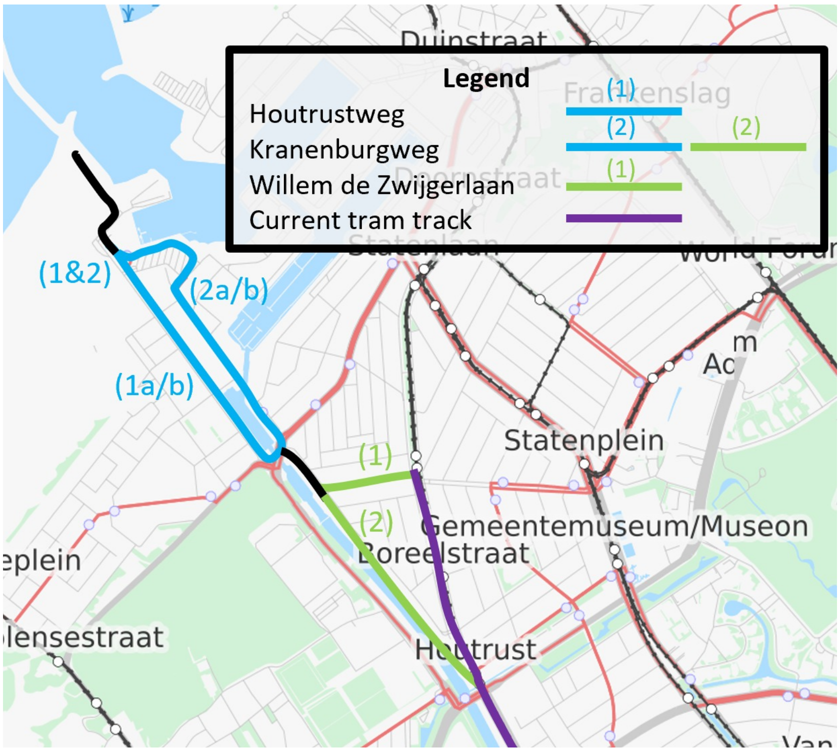


Figure 12.3: Overview of tram track options to reach current infrastructure (OpenStreetMap contributors, 2017)

Table 12.1: Options overview

	Blue			Green
Option	***a***	***b***	***-***	***-***
1	2-way track	Gauntlet track	-	2-way track
2	2-way track	Gauntlet track	-	2-way track
1 & 2	-	-	Single way track	-

Figure 12.4 displays the possible combination which are elaborated in Table 12.1. The black sections along the Kranenburgweg and at the harbour are set for any of the considered routes, they consist out of 2-way tracks. The blue and green routes lead to the possible combinations.

(1a) + (1) (1a) + (2)
(1b) + (1) (1b) + (2)
(2a) + (1) (2a) + (2)
(2b) + (1) (2b) + (2)
(1&2) + (1) **(1&2) + (2)**

Figure 12.4: Considered routing options, bold the chosen option

The blue part, as indicated in Figure 12.3, consists of 2 narrow streets with residences on the side(s): (1) Houtrustweg and (2) Kranenburgweg. To design a double track in one of these streets seemed off (options (1a) & (2a)), because the noise of a tram would could cause nuisance to the surrounding area, and the roads are designed for a living area with parking lots on both sides. A single track for 2 directions would have been a good solution, if there would be enough space for a tram track next to the road. A single tram track can not be placed on the existing roads because the tram would interfere with oncoming traffic. To solve this, the gauntlet track is considered (options (1b) & (2b)). last but not least, the option to have a single directional tram track in each street (option (1 & 2)) is considered. This options is favorable over the others, because the tram track can in this case be built as far as possible from the residences, and this decreases the frequency of a tram passing by these residences. Although this option increases the amount of residences affected, due to the minimal impact of the nuisance, this option is chosen.

The green part consists of just two options: (1) Along the Willem de Zwijgerlaan and (2) along the Kranenburgweg. For these sections it would be best to implement a 2-way track, as only 2 separate locations are available for merging towards the current 2-way track. In addition this is an evident option to minimise the waiting times for when a single- or gauntlet track is implemented towards the blue section and to match the current tracks layout.

Figure 12.5: Willem de Zwijgerlaan cross section in its current situation (Gemeente Den Haag, 2015)

The first option requires less distance to be covered by rails than option two. The current cross section consists out of multiple parking lanes including trees in the middle. Removing a large number of parking lots from a residential area is seen as undesired, therefore in a potential new situation the amount of parking opportunities should be retained. To implement a two-way traffic track, both driving lanes in Figure 12.5, would be switched into multi functional lanes utilised by car traffic and trams.

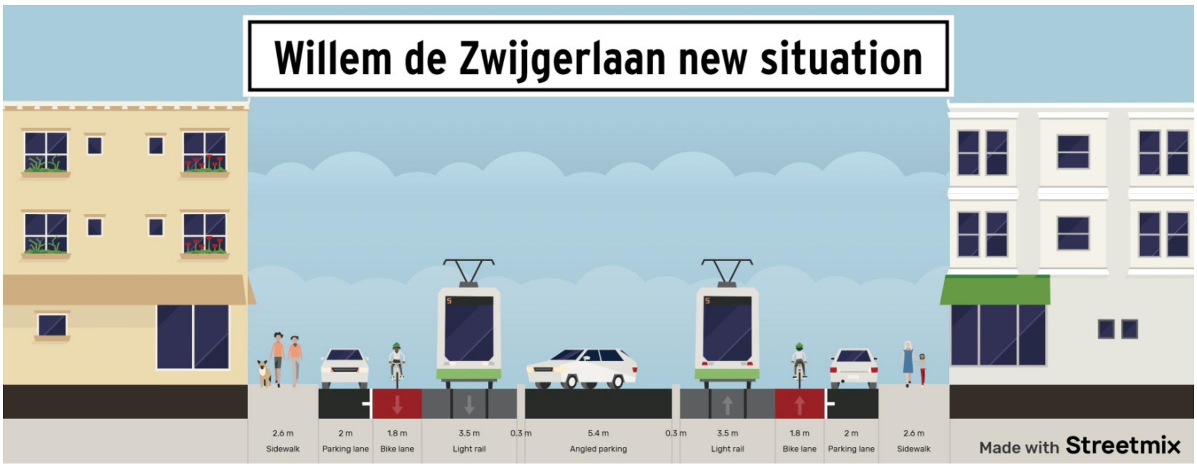


Figure 12.6: Willem de Zwijgerlaan cross section possible new situation (Gemeente Den Haag, 2015)

What hinders this option is that it is surrounded by residential buildings from both sides, which makes it less viable to be accepted by the community. The second option has a large amount of space along the Verversingskanaal which requires less change of the current situation, in addition there is more place for merging the tracks. Although option (1) requires less rail construction, option (2) is favorable due to nuisance reasons.

This concludes into a tram design where option (1 & 2) is chosen for the blue part and option (2) is chosen for the green part of the tram track. This design will now be further elaborated. In addition, after these options were elaborated, it was found that the solution presented above partly overlaps with a document produced by ZKA Consultants & Planners 2011 (See Appendix G.2). What sets our solution apart from theirs, is that for the blue line the only option considered in the referred document is our solution 2a. However, due to lack of space from our analysis it was found that this is a less appealing solution. Option 2 was more appealing for the green part for having less boundary conditions. This verifies the idea of using these streets for the tram track.

12.4.2. Developed tram track concept

The new tram track is constructed at the Houtrustweg and the Kranenburgweg. The tram route has been displayed in the figure 12.7 below, it will span from the new fourth harbour towards the stop at The Hague Noordwal. A choice is made to use a single direction tram track at the Houtrustweg and one parallel to this at the Kranenburgweg. This decision is made because these streets are small streets, with houses next to it. Having only a single lane through those streets creates a situation where the track can be placed with more distance from the houses along the road, and the amount of trams that will pass by will be halved, as it is shared between the 2 lanes. This should give minimum nuisance to the inhabitants. The overview of figure 12.7 shows multiple components that will be designed.

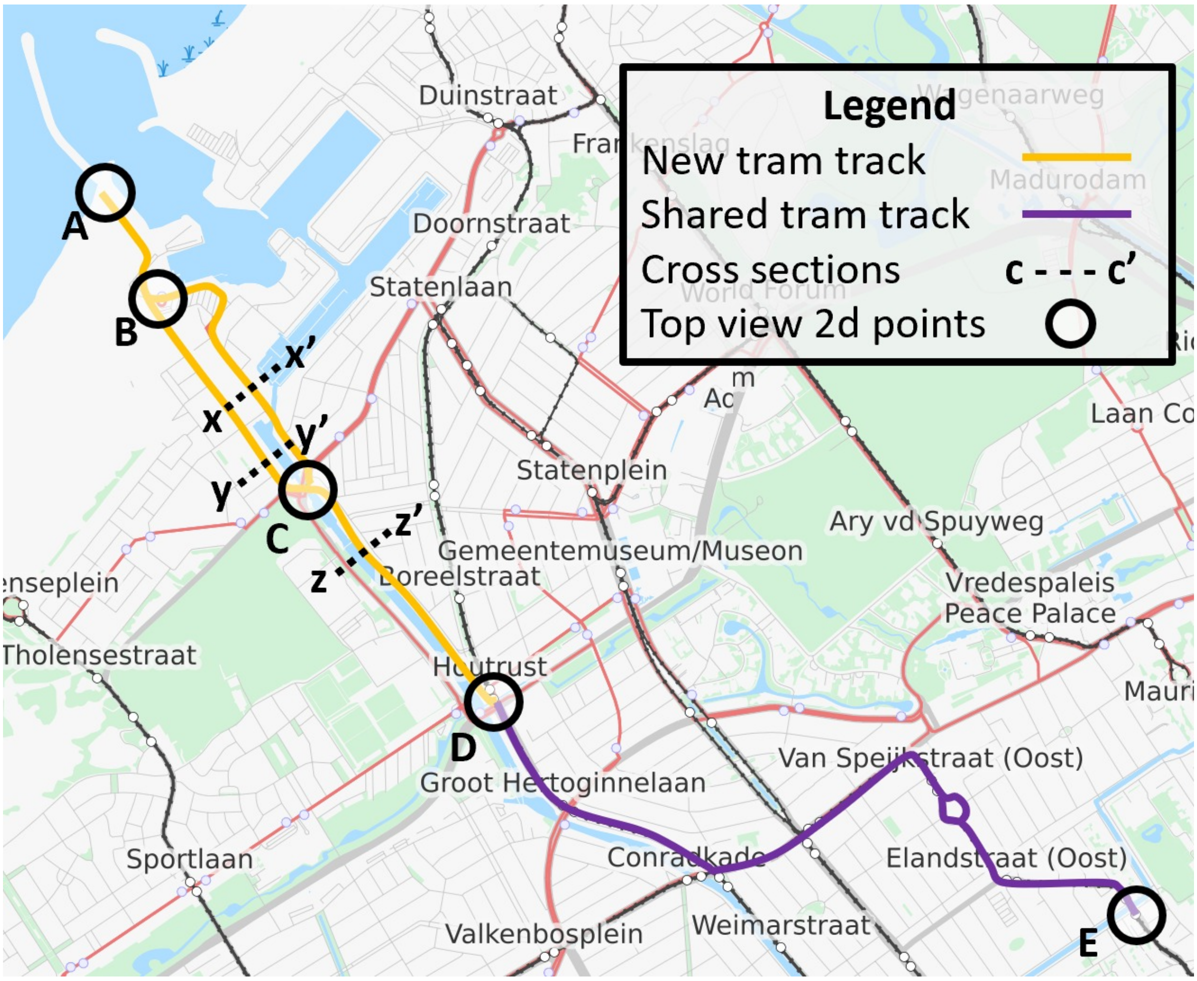


Figure 12.7: Overview of the tram track route and its elements (OpenStreetMap contributors, 2017)

- The new tracks from the harbor towards the Van Boetzerlaerlaan, (the old tram track), which is subdivided in the following segments:
  - **Section X-X'** - A representative section of the Kranenburgweg nearby the harbour. Due to space limitations, this is a single track. Its counterpart will be on the Houtrustweg.
  - **Section Y-Y'** - A leading cross-section of the Houtrustweg. The Houtrustweg spanning from the harbour towards the Nieboerweg can be subdivided into two components. The first component, close to point C has a water basin on one side and buildings on the other sides, whilst the second component, close to point B, has new buildings on its side. The leading cross-section for the design is the first part with the water basin, because the available space is more limited in this part of the street.
  - **Section Z-Z'** - A leading cross-section of the Kranenburgweg where 2 tram tracks will be placed. This cross-section forms a general representation of the Kranenburgweg, important to take away is the grass patch providing a large amount of space for the tram.
  - **Area A** - The point of departure from Scheveningen harbour.
  - **Area B** - This is where the tram shuttle of the Kranenburgweg and Houtrustweg merge together to get to the starting point of the tracks.
  - **Area C** - This point contains the crossroads at the Duindorpdam. Here the tram segment of the Kranenburgweg coming from the South-East is split up into two lines which merge again at point A. The northern line at the Kranenburgweg functions for transport from the harbour towards the center, whereas the Houtrustweg accommodates for the opposite direction.

  - **Area D** - The new tram track connects to the current infrastructure here.

- The current tram track that will be used by the new tram shuttle to reach its destination in the city center of The Hague.

  - **Area E** - This is the endpoint of the new tram shuttle, an important aspect is the switch, which is elaborated under section (12.4.6). Therefore the current stop will have to be re-designed.

Note that the cross sections are all made based on the orientated in North-Western direction. Therefore, x, y, z always indicate the left side of the figures, whilst the x', y', z' always indicate the right side of the figure.

12.4.3. Kranenburgweg (x-x')

The Kranenburgweg is a road which is surrounded by residential building blocks on both sides. The cross-section where this is the smallest has been taken as leading case. As can be seen on Figure 12.8, these buildings are accommodated with parking lanes on both sides and a relatively large part is is used for planting strips and side walks. A two-directional driving lane is found in the middle of the street.

Figure 12.8: Kranenburgweg section X-X' in its current situation (Gemeente Den Haag, 2015)

To retain the current two directional flow of traffic without impacting the possibilities of traffic flow, it was chosen to create a shared carriageway for trams and cars in this cross section. To create space between the tram line and residential units, parking lanes have been adjusted into an angled parking lane on the right side. This results in a distance of the tram of 8.0 and 8.5 meters from the edges of the residential units. Angled parking is usually only suitable for single directional traffic. However, in this design a 7 m wide road is available to maneuver into the parking lot. This enables parking even when you're in the opposite direction of the parking lot orientation. Additionally the angled parking has been given a margin zone of 0.5 m for the safety of parking next to the tram track. This gives a total distance from the parking lane to the tram of 0.5 + 0.425 m. Conform the guidelines found by Hansen et al. 2011, this suits to all parking angles of 60-70 degrees (See Appendix G.3).
The overhead wires are placed using the residential buildings on both sides, which doesn't require the placement of additional traction poles.

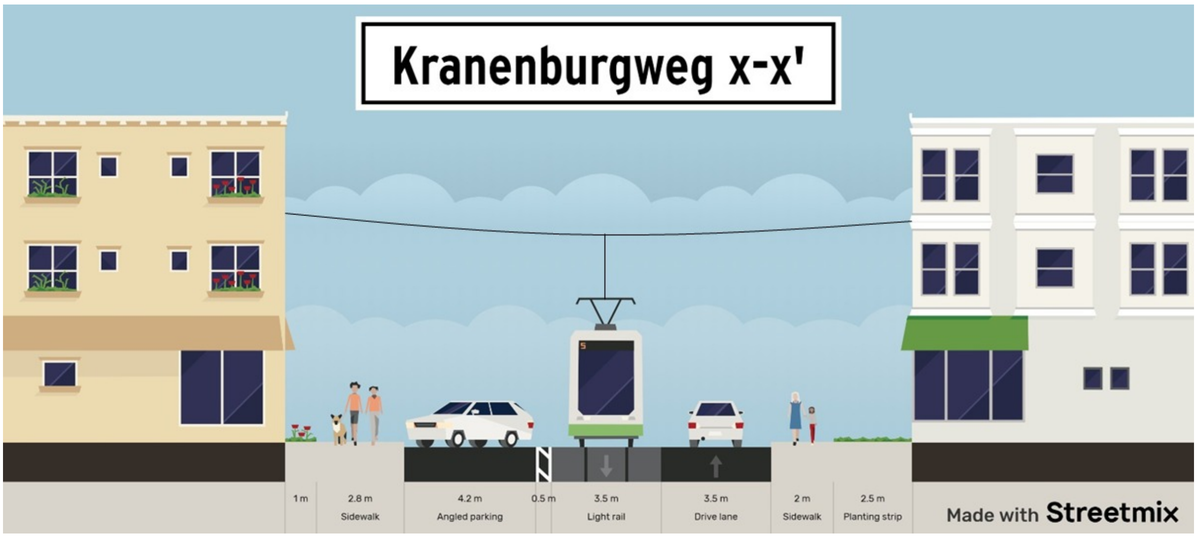


Figure 12.9: Kranenburgweg section X-X' in the new situation (Gemeente Den Haag, 2015)

12.4.4. Houtrustweg (y-y')

In the current situation in the Houtrustweg, there is a single broad road which functions for two-directional traffic. On both sides of this road there are parking lots and sidewalks. There are houses placed on the right side, and the Verversingskanaal on the right side. More North of this road, construction of new buildings is ongoing. There is a lot of space between these newly constructed buildings and the current road, which makes the Houtrustweg at the Verversingskanaal the normative cross section.

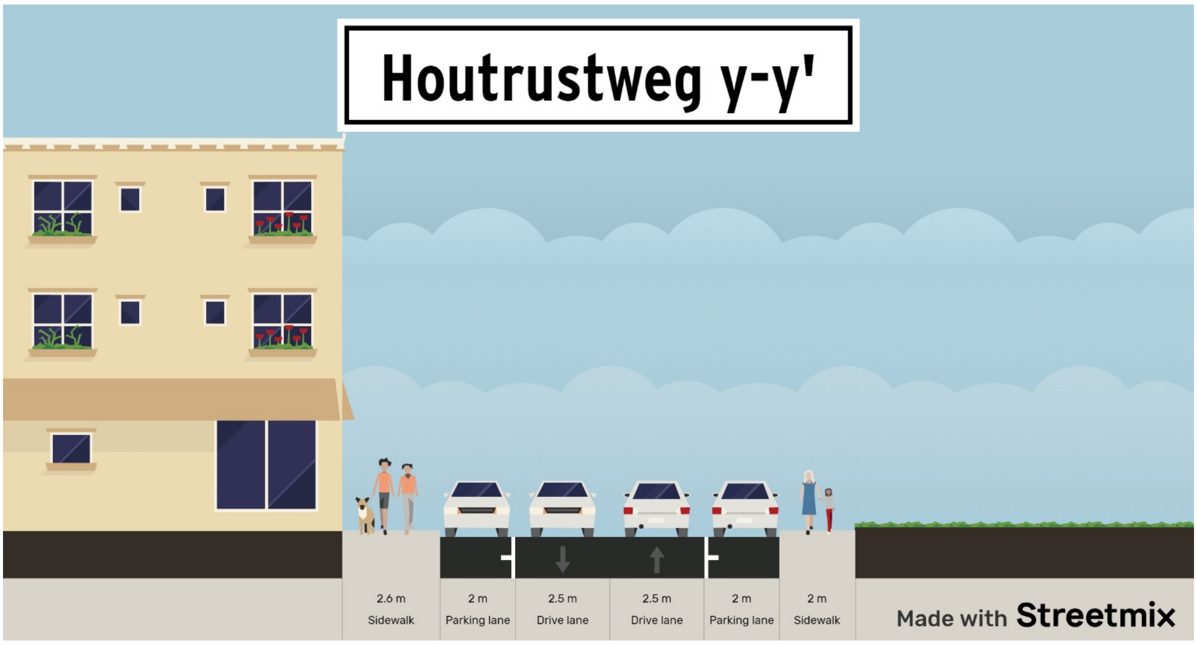


Figure 12.10: Houtrustweg section Y-Y' in its current situation (Gemeente Den Haag, 2015)

The new tram track at the Houtrustweg is orientated towards the right side of the cross section. This is done to have the tram track as far as possible from the houses along this road, whilst preserving the sidewalk along the Verversingskanaal. The parking lots on the right side of this cross section could not be spared, as an extra meter is needed for a shared carriageway for trams and cars. The other meter that has come available is also given to the road, on the right side, to create more space between the houses on the right side and the tram track.

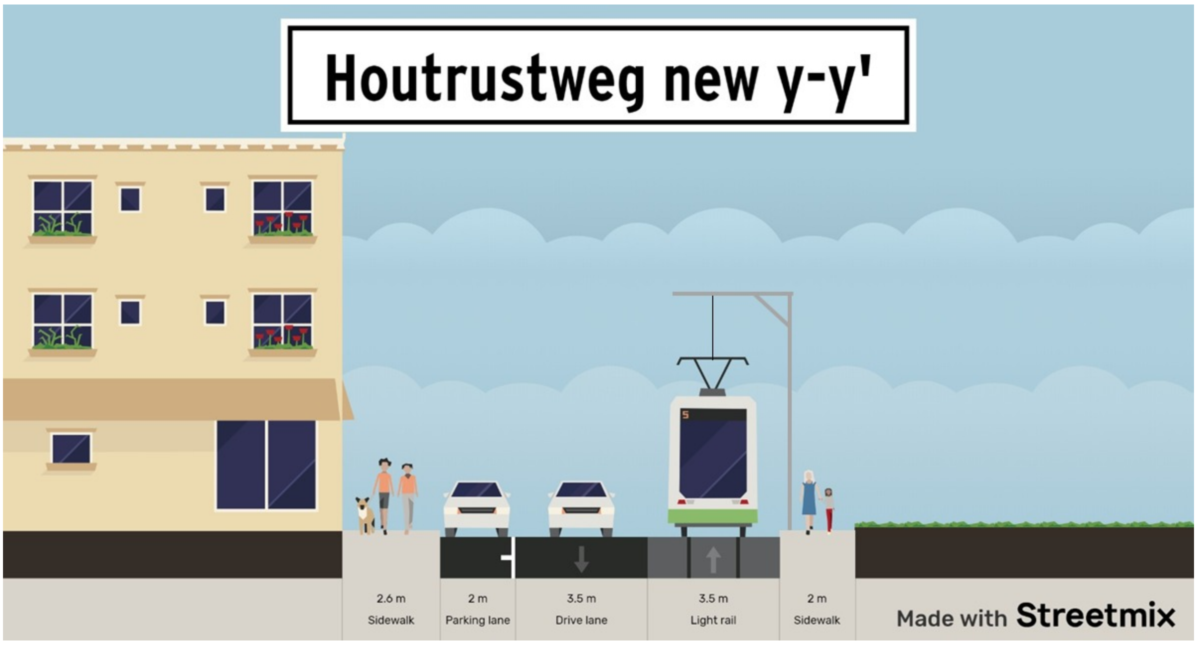


Figure 12.11: Houtrustweg section Y-Y' in the new situation (Gemeente Den Haag, 2015)

For the overhead wires a traction pole is placed on the right sidewalk of cross section y-y' in figure 12.11. The space between the edge of the sidewalk and the tram is 425 mm, using the 3.5 m allocated tram space according to Gemeente Den Haag 2015. The traction pole is placed 325 mm from the edge of the sidewalk. A schematic overview is presented in Figure 12.12.

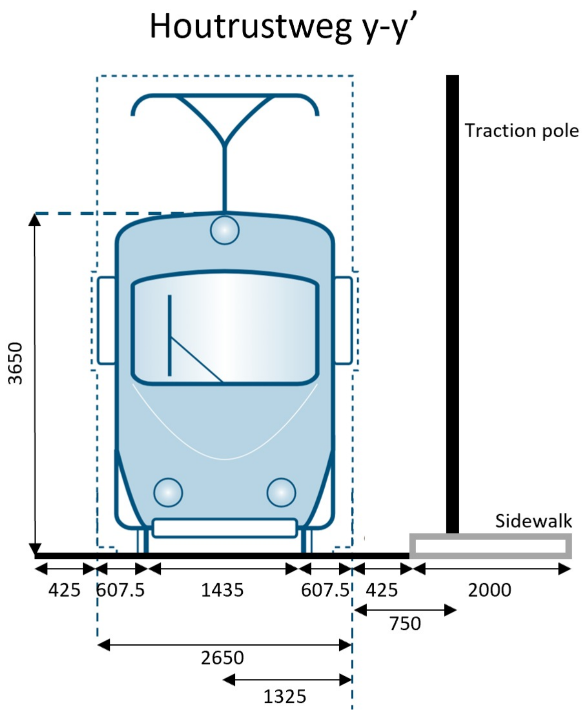


Figure 12.12: Houtrustweg section Y-Y', traction pole placement (Gemeente Den Haag, 2015) and tram characteristics (Hansen et al., 2011).

12.4.5. Kranenburgweg (z-z')

Currently, there is a lot parking lots along the Kranenburgweg, oriented on both sides of the roads. Also, the grassy plain is now not straight

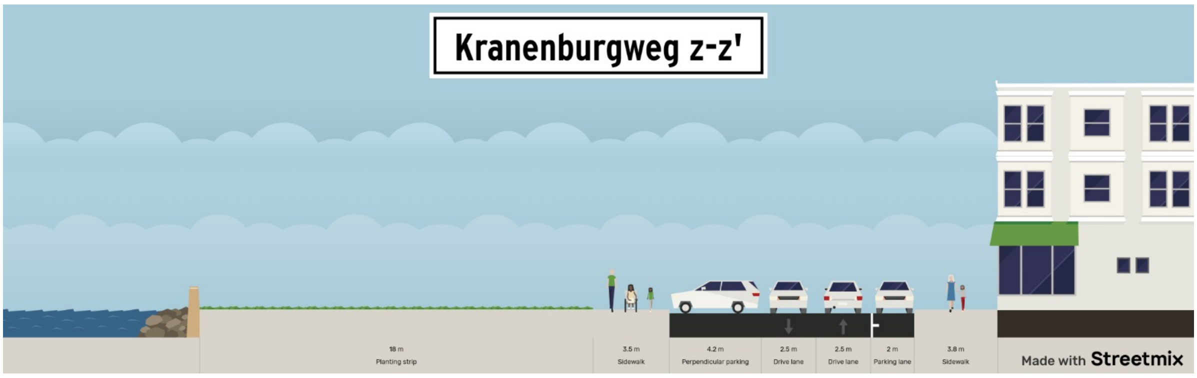


Figure 12.13: Kranenburgweg section Z-Z' in its current situation (Gemeente Den Haag, 2015)

For designing z-z', the current tram track along the Conradkade at the station of the Weimarstraat is used as a reference design. The situation of the cross section is similar here, as there is a street with houses on one side, and a long grassy plain towards the Verversingskanaal on the other side. Note that the parking places here are not conform the guidelines in Figure G.3, the cars in the current situation extend partially over the sidewalk.

Figure 12.14: Kranenburgweg section Z-Z' in the new situation (Gemeente Den Haag, 2015)

A traction pole is now placed in the center of the two tram tracks, as can be seen in the figure below. Using the two tram track areas of 3.5 m, 850 mm space is left over between the trams. The width of the pole is not specified, but a maximum traction pole width of 650 mm is possible to still meet the requirements of section *Requirements*. It is assumed that this maximum width is sufficient for a traction pole.

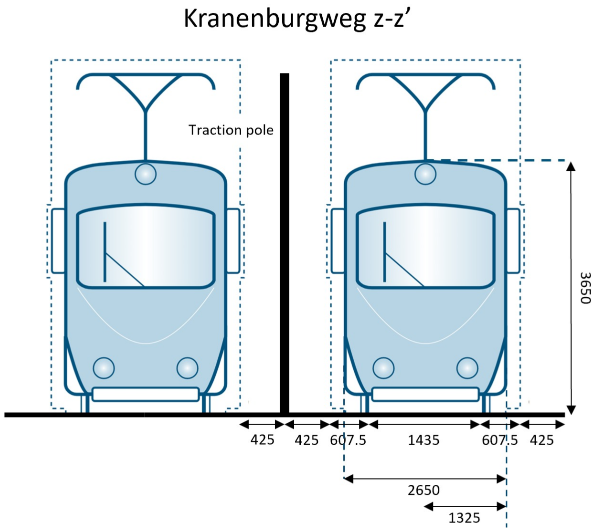


Figure 12.15: Kranenburgweg traction pole placement section Z-Z' (Gemeente Den Haag, 2015)

12.4.6. Points A & E: begin and end point

The tram track is designed for the Regio Citadis, which is a tram that currently operates in The Hague and can move in two directions. The tram track is also designed according to the minimal radius of 23 m (Hansen et al., 2011). Point A is a starting and ending point of the tram in Scheveningen. To create a reversing facility at point A, the tram track will be tuning-fork shaped. A tram that can drive into two directions can move to one track to another track by moving over the single lane of the tuning fork. This turning process is depicted in Figure 12.16.

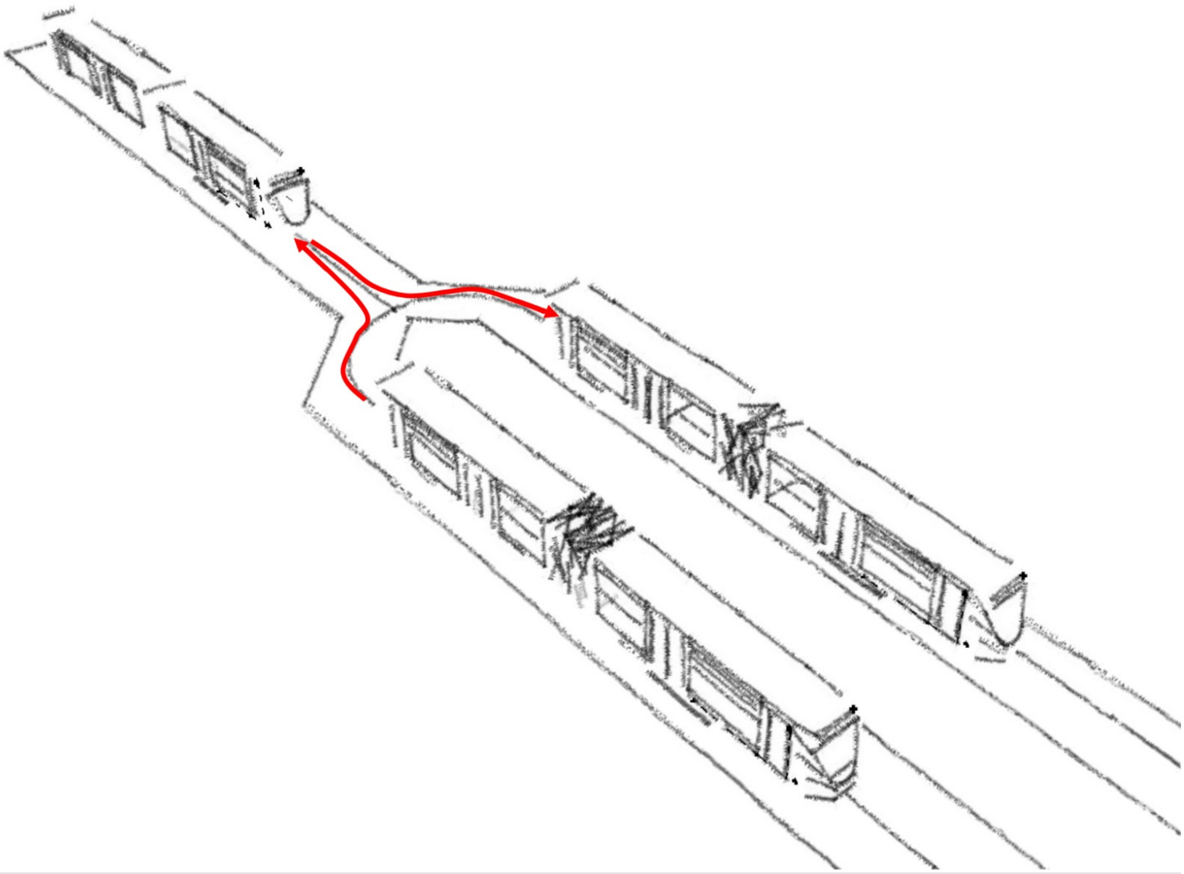


Figure 12.16: Sketch of the tuning-fork at point A, Scheveningen harbour (The red line serves for indicative purposes)

Point E is the other end of the new tram line, which is located at the Noordwal tram stop. The current tram line (1) extends further towards Abtswoudsepark in Delft. Due to the area being densely built around the center of The Hague, there is lack of space for implementing a terminal loop. Hence, it is chosen to place a switch at point E which will allow the tram to switch from tracks thus changing directions. This process is depicted in Figure 12.17.

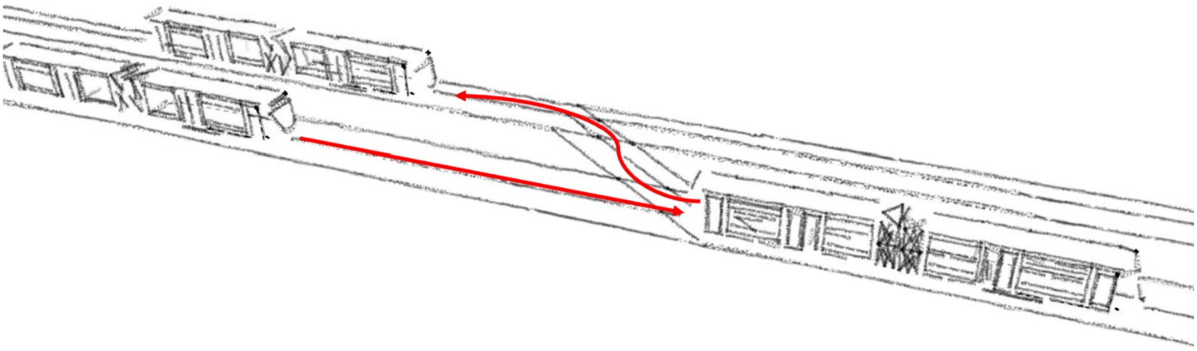


Figure 12.17: Sketch of the switch at point E, Noordwal (The red line serves for indicative purposes)

12.4.7. Point B: Crossing of rails in Scheveningen

Point B is where the tracks split towards the Houtrustweg and Kranenburgweg. Referring to cross sections x-x' and y-y' (See Figure 12.7), the trams are situated on the right side of the streets for nuisance reduction to the neighbourhoods. The trams are ought to drive on the right side of the road due to safety purposes when sharing the road with cars. Combining the two arguments above implies that the tracks are required to cross over each other and has been displayed in Figure 12.18. The tram coming from the fourth harbour continues on the Kranenburgweg and vice versa the tram coming from the center continues on a straight forward.

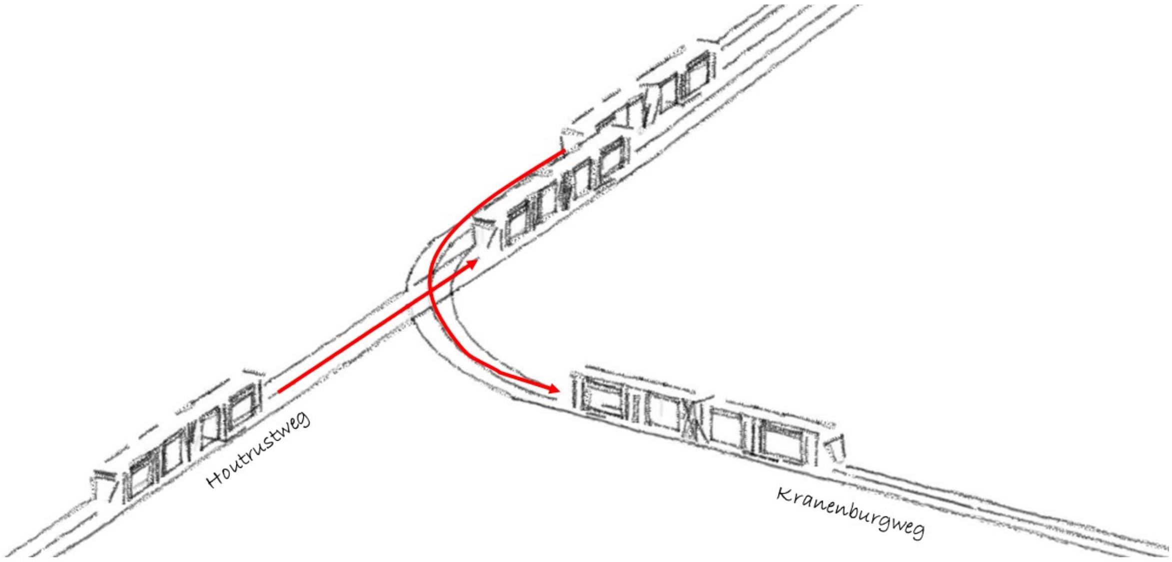


Figure 12.18: Sketch of crossing B

12.4.8. Point C: Intersection Duindorpdam

In it's current situation there are multiple crossroad close to each other at the Duindorpdam, causing several accidents in the past years (De Scheveninger, 2018). When incorporating the tram, this increases the complexity of the current situation, making it even more dangerous. Therefore, it is aimed to improve the safety of the intersections at the Duindorpdam. At an intersection, roundabouts can greatly improve the safety, because the amount of conflict points will decrease significantly (Hansen et al., 2011).

Using Figure 12.19 the current situation can be described. The Houtrustweg connects to the Nieboerweg at 2 different places, namely the 2 bottom circles. This divides the bottom segment in to 2 different intersections, which causes too many obstacles/actions in a small span. The top situation has the same problem, having 2 separate intersections very close to each other. When combining the road layout and the cyclists that are also present, a highly complex situation is formed, which is hard to comprehend for a car driver in a short period of time.

Figure 12.19: Point C (Duindorpdam) in it's current situation. The circles highlight the complex sub-components. Cyclists are depicted with the dotted line. (OpenStreetMap contributors, 2017)

The considered solutions to improve the safety on this crossroad consist out of implementation roundabouts, due to the decrease of the number of conflict points. A single roundabout for the whole crossroad was considered, but this idea was not viable due to the amount of roads that will be connected as on/off ramps for the roundabout. The spatial limitations would cause problems when trying to design this roundabout and connect the current roads in a realistic way. In the second solution, which is implementation of 2 roundabouts, there is enough space to built roundabouts on the 2 current crossroads. Due to using two roundabouts the attachment of surrounding roads is more flexible. In addition the implementation of these roundabouts fits the scope of the design guidelines (See Appendix G.1). Therefore, 2 roundabouts are now implemented as the final design.

The Traffic fluxes of the predicted situation in 2033, retrieved through ZKA Consultants & Planners 2011, are used for designing the new situation at the Duindorpdam. These are displayed in Appendix G.4. The largest traffic fluxes are along the Westduinweg and Houtrustweg, being 1000-1500 veh/h during peak hour. A simple roundabout has a capacity of about 830-1040 veh/h (Hansen et al., 2011). Yet, it is still chosen to implement simple roundabouts with a single lane, because most other directions have negligible intensities. Another reason to opt for a simple roundabout is the fact that all surrounding roads are single lanes for each directions, which would be the bottlenecks when the capacity of the roundabout would be increased. The location of both roundabouts is chosen based on spatial boundaries of the area, with regards to the current roads, buildings and water basins.

New design Duindorpdam. Figure 12.20 shows the new situation for the Duindorpdam, when implementing the tram track and roundabouts at this location. In this figure, a few operational details have to be described in order to understand the functioning of the roundabouts. The roundabouts are both single land roundabouts and function like any other roundabout, driving counterclockwise. Because cyclists and pedestrians are allowed to cross the roundabout both ways it would be unsafe to grant the cyclists priority over the cars, because car drivers do not always expect cyclists to come from 2 directions on a roundabout. For this reason, the cars are granted priority over the cyclists and the pedestrians. however, to make crossing easier, cyclists and pedestrians are able to cross half of the road at a time, using the center strip. Some road markings are sketched to indicate the priorities in this new situation. The detailed rules for road markings stated in (verkeersregels en verkeerstekens), 1990) are not implemented in this design due to time limitations, but they should be implemented for a final design. It is expected for pedestrians, to not walk around a large roundabout, because pedestrians like to walk the minimum distance to cross the streets. Therefore, a pedestrian crossing is implemented at the top of both roundabouts. This pedestrian crossing is not implemented between the roundabouts, because the cars already have little time and space to move between the roundabouts.

Figure 12.20: Total overview new situation cross section C (OpenStreetMap contributors, 2017)

Design of the roundabout Houtrustweg. The design guidelines for roundabouts by Hansen et al. 2011 were taken into consideration, these can be found in Appendix G.1. In Figure 12.21 the dimensions of the roundabout are displayed. An inner radius of 15 m is chosen, because this creates a roundabout in the larger range for of the design guidelines. A larger roundabout creates the possibility to connect the different roads more perpendicular to the circle, which creates a safe overview of the roundabout for cars driving onto the roundabout.

For safety purposes, the bicycle lane crosses the tram track in perpendicular manner. This avoids that cyclists get their wheels stuck between the rails of the tram track. The perpendicular crossing of the tram track and the bicycle lane could only be implemented on the south side of the roundabout. Therefore, the bicycle lane does not entirely circle around the roundabout, instead it is designed as a two way cycling lane on the bottom part of the roundabout, while all directions remain reachable.

The Houtrustweg does have a lane which is shared by the trams, cars and cyclists. However, for cyclists it is not wished to cycle along the tracks. Hence for people that are required to reach Scheveningen, Duindorp or the Norfolk Terrain, there are options to cycle across the Nieboerweg and Kranenburgweg (See Figure 12.22). The bicycle lane connection towards Houtrustweg does however remains available for people who live along the Houtrustweg. A warning sign about the rails should be placed for cyclists entering this road.

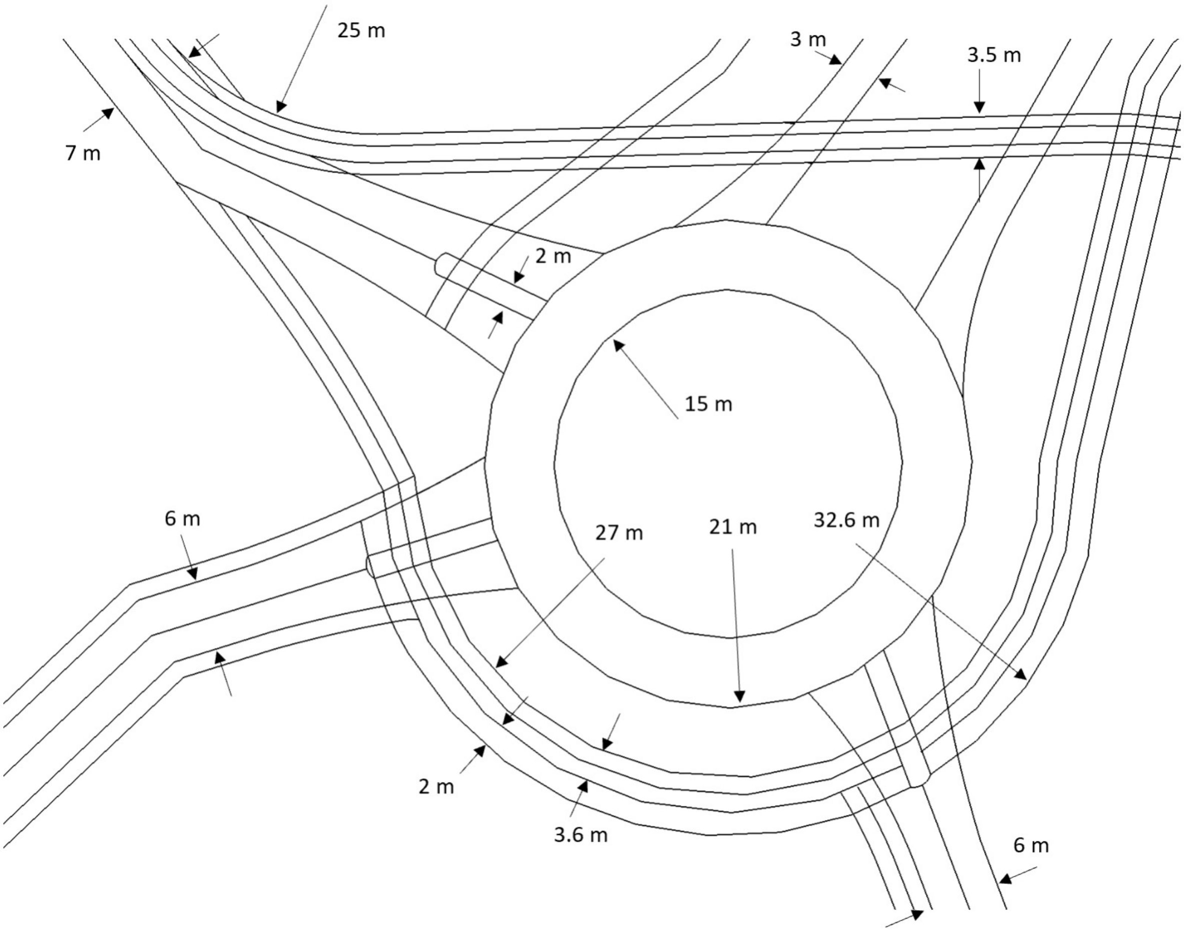


Figure 12.21: Roundabout overview Houtrustweg at point C

Design of the roundabout Kranenburgweg. Due to the spatial limitations, a smaller inner radius is implemented for this roundabout. As the roads are already more perpendicularly orientated, a larger radius would also not be necessary to connect the roads. The tram track crosses this roundabout through the middle.

The cycling path across the Kranenburgweg is two directional, and placed on the right side of the road next to the parking lanes before section X-X' starts (See Figure 12.8).

Interaction with the tram. When a tram reaches the Duindorpdam, both roundabouts need to be cleared from traffic, in order to let the tram pass safely. This can be realised by using traffic lights at all roads connected to both roundabouts that will turn red, only when a tram is about to arrive. A traffic jam could accumulate due to this process, but since the tram is assumed to not drive very frequently, the negative impact of this is deemed minimal. Another way to decrease the amount of times that the roundabouts will have to be closed, is by planning the operations of the tram in such a way, that 2 trams will encounter each other on the Duindorpdam. If this is a possible solution will be elaborated in Section 12.5, Tram operations.

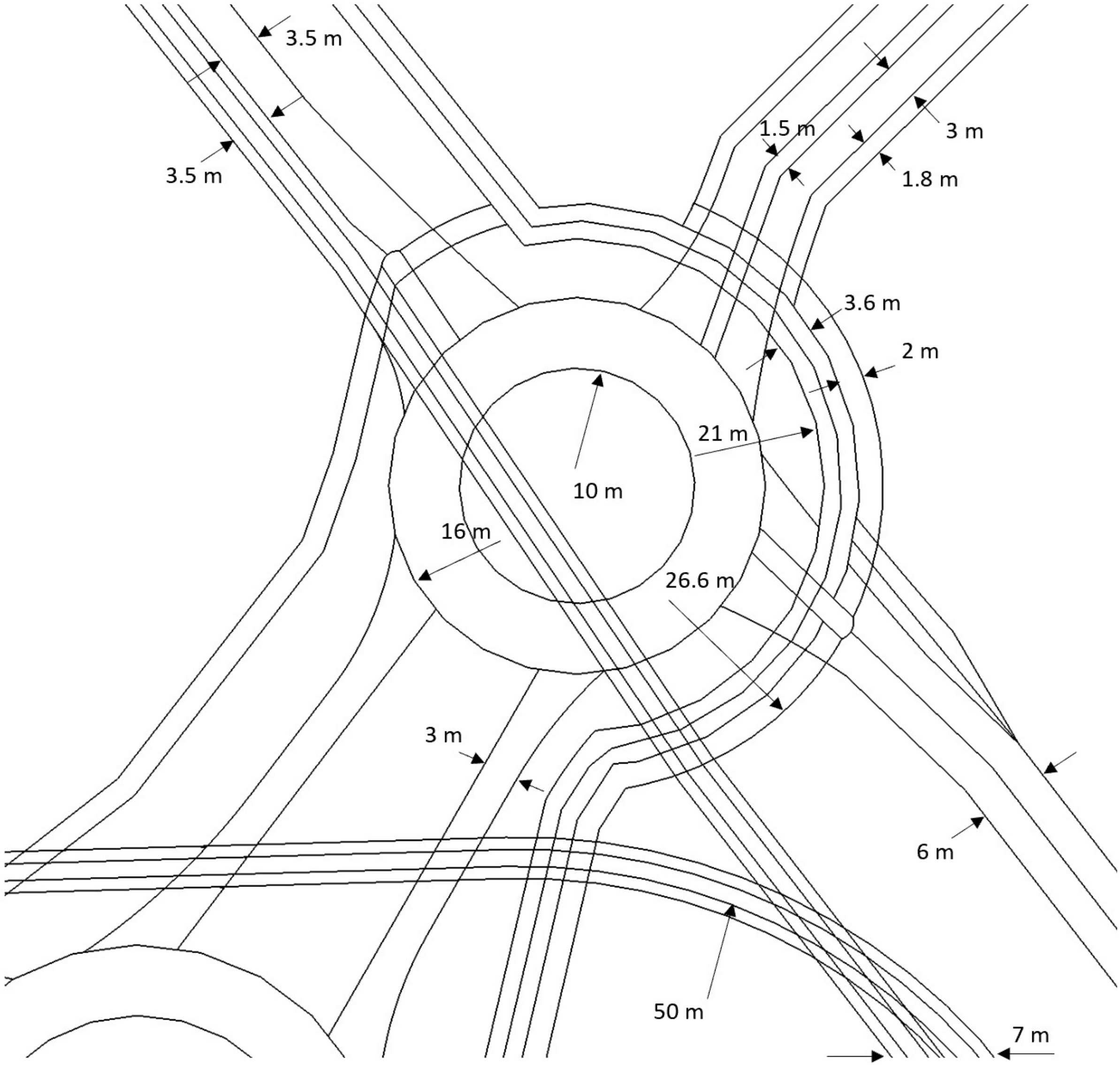


Figure 12.22: Roundabout overview Kranenburgweg at point C

12.4.9. Point D: Connection between new and current tram track

In point D the new tram track is connected to the current tram track of line 11. When making the track design for this point two boundary conditions were taken into account which were to be avoided: 1) Restaurant Brasserie De Laer and 2) The cross road. The tracks are implemented after the cross road to avoid making it too complicated. For the design of the tram track it was chosen to link the tram tracks with an S curve, for which a bend radius of 87.6 m was used on both sides. Because this creates a new intersection between the tram track and a road, traffic lights and coherent road markings and -signs are implemented for a safe cross over. As the current intersection does not have traffic lights, these traffic lights will only hold traffic when a tram arrives.

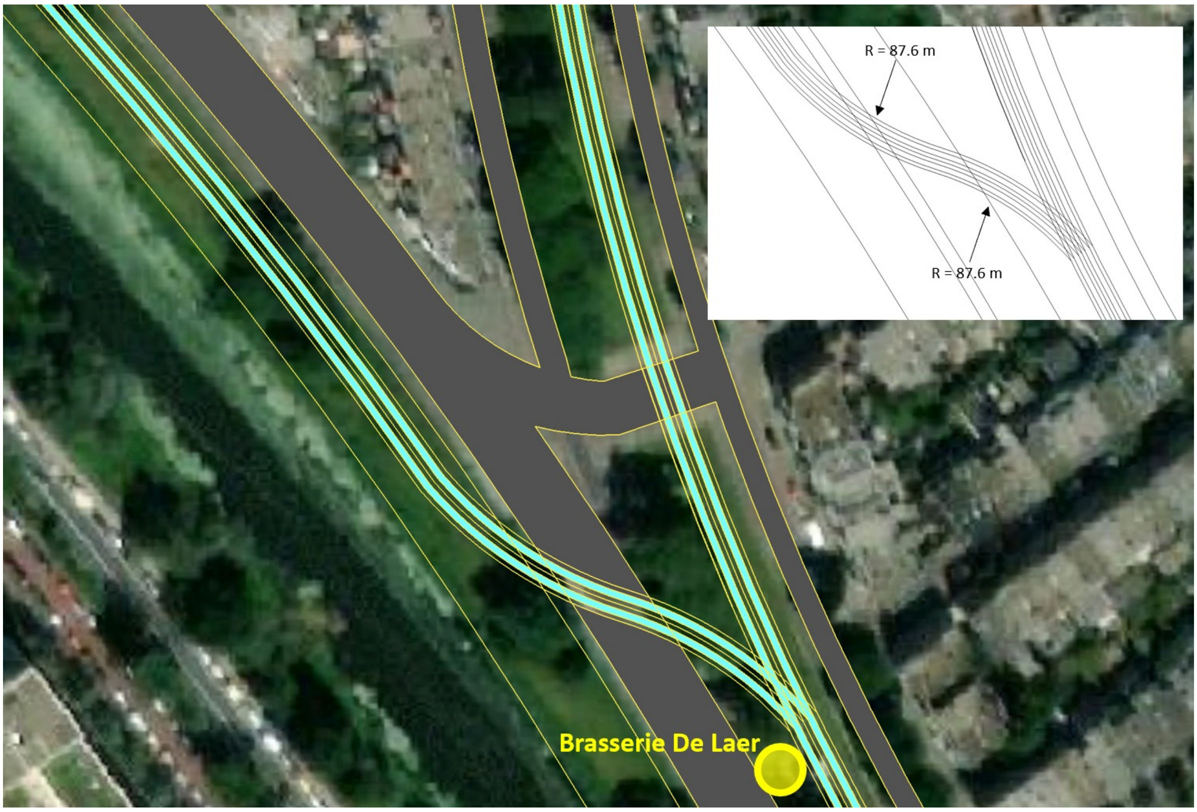


Figure 12.23: Intersection D in the new situation (OpenStreetMap contributors, 2017)

12.5. Tram operations

From the market analysis followed that 70 % of the 720 daily passenger would like to go to The Hague. This is a total of 504 passengers. The capacity of the Regio Citadis accommodates for 84 seats and 130 standees. A design goal for the tram is to provide all the tourists with seating. At least 6 trams are required to provide 504 passengers with seats. During the maximum rate of 8.4 travelers/minute, this means that in 20 minutes there could be 168 travelers wanting to board the tram. This fits within the capacity of the tram using the standees spaces. Yet, the travelers do have the option of waiting for a next tram with available seats.

To prevent interference with the local traffic as much as possible during peak hours, the tram will start moving from the cruise terminal at around 09:00. It is important to fit the new tram into the operations of the current tram network. Therefore, the first stop after connecting both tram tracks is analysed. This analysis is done based on a normal Monday morning schedule, where the departure times at the tram stops around point D where retrieved from HTM personenvervoer NV 2020. This showed that tram 11 is the only tram at the current tram track. A schedule is designed to fit the new tram into the current schedule, with an interval of 20 minutes, as shown in Figure 12.24. This interval is chosen, because with this interval it takes 2 hours to move every tourist from the cruise ship to The Hague by tram.

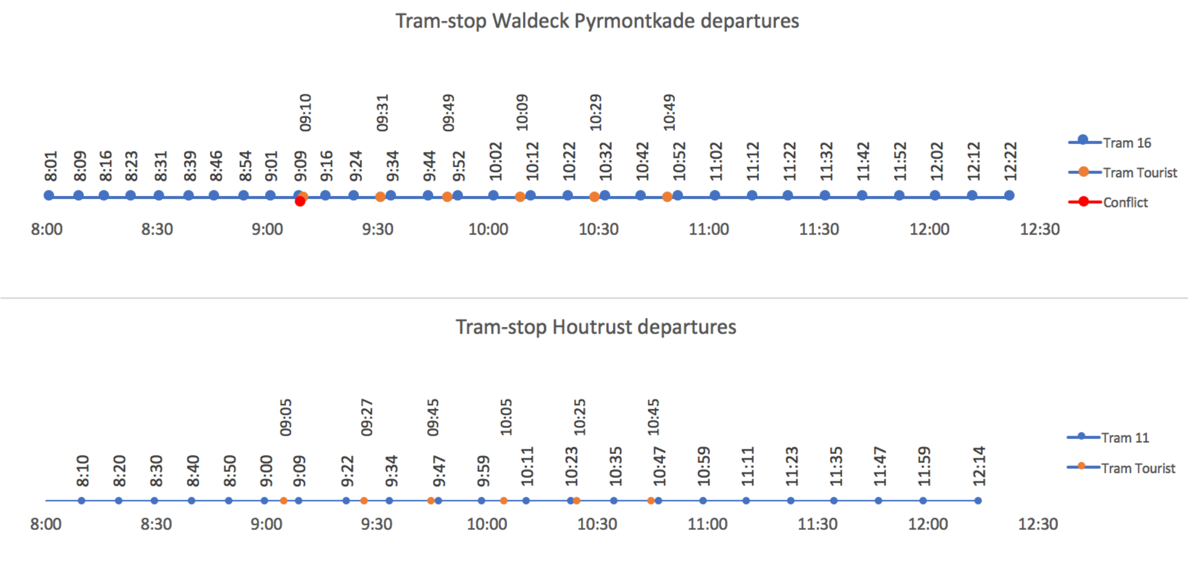


Figure 12.24: Tram stop departures at Houtrust and Waldeck Pyrmontkade

The next step is to determine how long it takes the tram to move from the Fourth Harbour to the Noordwal tram stop, and all the interesting points between. This will lead to the departure schedule at the Fourth Harbour stop, and determine the amount of trams that is necessary to meet this schedule. To acquire a rough estimation of the time it takes to drive on the new tram track, a design speed of 50 km/h is assumed for the tram, whilst adding 30-60 seconds for acceleration and deceleration. This gives an indication of the duration to drive over the new track. To determine the time it takes to drive on the old track, the data of tram 11 and tram 16 are used. It is impossible to overtake these trams, so the new tram is assumed to take about the same time here. The total time to travel between Scheveningen and Noordwal is estimated to be 19 minutes, including 1 turning action (+1 minutes) and 2 (un)loading actions (+1) at point A and E. A few other time losses are taken into account at the Duindorpdam and the connection of tracks. bigbreak Only 2 trams are required to realise the 20 minute schedule, with a travel time of 20 minutes between the Fourth Harbour and Noordwal. When one tram leaves, the other will leave 20 minutes later, and 40 minutes after the first departure, the first tram will be back to go go again.
No extra stops are added on the new tram track, because that would increase the travel time between start and end point, and this would also result in the need for more than 2 trams to operate. No addition of tram stops can be explained by the fact that this tram has a main purpose of bringing its passengers

towards the city center of The Hague and Fourth Harbour in Scheveningen. The stops on the old trajectory on the other hand are used, because there are other trams on these tracks too. Because the tram cannot overtake these trams, it is chosen to use these stops instead of waiting for trams 11 and 16 to move.

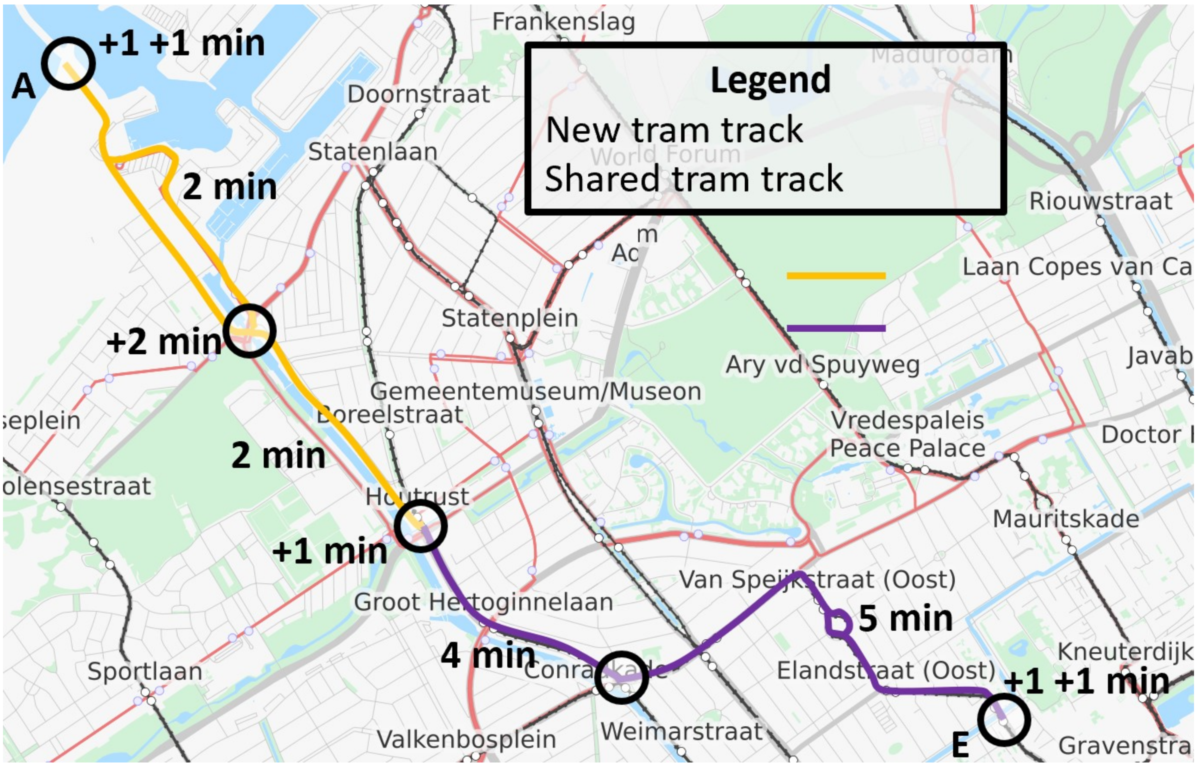


Figure 12.25: Tram trajectory with travel times (OpenStreetMap contributors, 2017)

The tram switches from line 11 to line 16 at the intersection between the Laan van Meerdervoort and the Waldeck Pyrmontkade (See Figure 12.25). Another timeline is now created to make sure there is no conflict with the schedule of line 16 at the Waldeck Pyrmontkade. As shown in Figure 12.25, adding 4 minutes of travel time from the leads to the times of departure at the Waldeck Pyrmontkade tram stop. The times of departure of the tourist tram overlap at one point with line 16, as seen in Figure 12.25 there is only one conflict point at 9:09. To solve for this conflict point, a waiting time of 1 extra minutes is calculated to move the time of the stop to 9:10.

When the first 6 trams have left with a frequency of once every 20 minutes, the frequency during the day will decrease to be once every hour. This frequency could be increased if found necessary (during summer holidays or big events) to the designed frequency of once every 20 minutes. Practise will find out what frequency will be implemented during the day. When a cruise ship is about to leave, or around the evening peak hour, but not in the evening peak hour, the tram frequency will be increased to once every 20 minutes again. These alternate frequencies are not further elaborated as they are not set values. Although, as shown above in Figure 12.24, any frequency lower than 20 minutes should fit on the tram tracks. Last but not least, the operation of the tram will interfere with intersections at points C (Duindorpdam) and D (connection). At these intersections, traffic lights are added to stop cars from moving onto the intersections when a tram arrives. Vice versa traffic lights for the trams are added.

This stops all other traffic for safety reasons. This could have huge impact on the traffic flows, but because the tram does not operate in peak hours, and not very frequently. As a consequence, the actual impact should be minimal. On average the tram will cross point C and D twice every 20 minutes (both directions). The traffic lights are placed at the entry roads before both roundabouts. The traffic lights will be off when there's no tram approaching. When a tram approaches they will hit red, remaining traffic will be able to leave the roundabouts, once the roundabouts are cleared from traffic, the trams will be able to pass point C. An interesting situation is the shared carriageway at Kranenburgweg, because trams will not be able to pass if cars get red light on this road. The solution to this issue is to let these cars onto the roundabout, and let them leave the roundabouts before this tram crosses over.

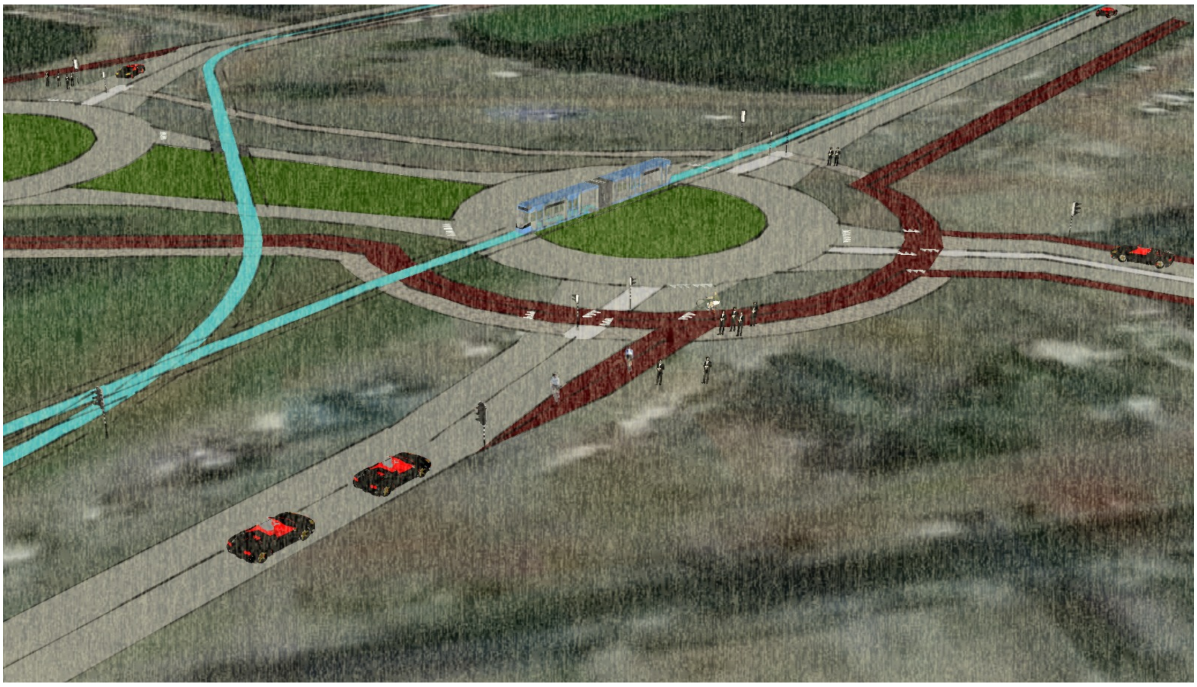


Figure 12.26: impression of point C, including traffic lights (OpenStreetMap contributors, 2017)

12.6. Conclusion

For the infrastructure part of the master plan, a design was produced for a tram line which improves the public transport towards the coast line and an alternative for the current situation at the Duindorpdam. It was found that the most logical way to do this was to place one way tracks on the Houtrustweg and Kranenburgweg (Point A - C in Figure 12.7) and two-way tracks along the Kranenburgweg (Point C - D in Figure 12.7). For different leading cross-sections per segment it is evaluated how the tram tracks is implemented and how the street characteristics are alternated to fit the tram track in. For the points A, B, D & E, a rough idea was sketched on how they could be designed. On the contrary, point C was designed more thoroughly since it is identified as a highly complicated cross road in the current situation. This resulted in the design of two roundabouts at section C, which may at first seem like it has many conflict points with the tram. With the tram interaction described under Section 12.4.8, it becomes evident that the roundabouts will be fully cleared before the trams cross point C. The designs listed above were all dimensioned conform the requirements written in Section 12.3 and the boundary conditions defined by the current situation.
The schedule of the new tram is implemented in a manner that does not cause conflicts with current tram lines 11 & 16. No extra stops were added to the tram track in the design, due its purpose serving as shuttle for connecting mainly (cruise) tourism in both directions. Only 2 trams are required to operate with a frequency of 3 trams/h, as the route from start to end point takes approximately 20 minutes. Because the tram operates outside of rush hours in general, it is deemed more likely that the clearance of the roundabout will not lead to traffic jams surrounding the area.

12.7. Discussion

The discussion is divided up in 2 lists. The first list contains points of discussion about the final infrastructure design. The methods that are used are quite straight forward, therefore not elaborated in this discussion. The second list will contain ideas for further research.

- The tram track in the design is fixated on the passenger analysis concerning the boat tourism. However, to make this plan more viable for acceptation by the municipality the tram could take into consideration using the combination of the old and new track for enabling a connection with public transport to the hinterland. The placement of the tracks would not have to be altered for this implementation, it would be a matter conducting a new passenger analysis for the tram capacity and operations.
- With Duindorpdam being a relative busy road (See Appendix G.4), roundabouts could could not be the most optimal solution for the traffic flow. Adding traffic lights to the roundabouts to clear them when a tram comes also does not add to the throughput of traffic on the Duindorpdam. Because the goal was to increase the safety on this specific location, whilst implementing the tram track, the design has become what it is in its current form.
- Adding a sidewalk on the upper part of the roundabouts at Duindorpdam, but no cycling path could lead to cyclist using the sidewalk as a shortcut. This leads to the unsafe situation of cyclists crossing the tram track in a way that the wheels could get stuck in between the rails. On the other hand, removing the sidewalk could lead to pedestrians illegally crossing the roundabouts as a shortcut. Therefore, the sidewalk is still implemented, clear warning signs could persuade cyclists to comply to the rules.
- In addition to the sidewalk being used as shortcut by cyclists, the clearance of the roundabout for tram usage is possible for fast flowing traffic which overlaps with cyclists and cars. However, point C will most likely not be fully cleared of pedestrians when the trams start crossing. Hence, the pedestrians should be warned/prevented of crossing the tracks when a tram crosses.
- It is not desirable to have cyclists on the tram tracks at the Houtrustweg, which is why they are encouraged to drive along the Kranenburgweg when they want to reach the coastline. For the cycling path between the Nieboergweg and the Houtrustweg, this would imply that this can function as a shortcut for people living at the beginning of this part of the Houtrustweg. For other citizens that are required to just reach Duindorp or a little further along the Houtrustweg, alternative junctions into Duindorp can be found/incorporated along the Nieboerweg (e.g. Tesselse straat and Zwaluwestraat).

- Another point of discussion at the Duindorpdam is from the perspective of the tram driver. When coming from The Hague, the tram will cross the Duindorpdam towards the Houtrustweg. This is unnatural, since the Houtrustweg is orientated left from the Kranenburgweg, where the other track is. This is a point of attention for tram drivers that should be considered.

Recommendations for further research and projects related to infrastructure are listed below:

- It would be interesting to explore opportunities to use the tram for a more broad crowd. An extended market analysis would be necessary to identify the specific market that could be reached, after which the frequencies could be adjusted to fit for a broader demand.
- In the current plans of the municipality multiple goals are mentioned with regards to the traffic in Scheveningen. In our design we have tackled two which fell into our scope, these include:
    1. Improving the public transport towards the coast.
    2. Improving the cross sections and local conflict points.

  Given the project deadline, our scope could not cover all the points. Further projects could tackle some of the remaining (and tackled) goals of the municipality:
    1. Improvement of public transport *along* the coastal area.
    2. Improvement of the pedestrian and cycling routes along the newly designed tram track.
- Other transport types than trams could be considered to connect the Fourth Harbour with the city center of The Hague. A comparative method (e.g. cost benefit analysis, multi criteria analysis) could give a better insight in the pro's and con's of other transport modes.

13 Conclusion

In this chapter the conclusion concerning the integral aspect (Chapters 1-7) of this project is presented. This is followed up by a concise conclusion on the second phase (Chapters 8-12). All sections in the second phase have their own, more detailed, conclusions written in the corresponding section.
This report started with a motivation, mainly based on the (future) problems of the harbour presented by Dr. Ir. Waterman. In order to define the problem, a problem analysis was performed in Chapter 2, consisting of spatial and social analyses. The conclusion that followed from the problem analysis and Dr. Ir. Watermans work, led to the problem statement presented in the summation below:

- The dikes and quay walls do not provide a sufficient flood protection against the water levels.
- The harbor does not provide access for mini cruise ships and does not meet the capacity to dock yachts.
- The harbor lacks the possibility for water sport events in a way to become globally relevant.
- The present water-lens does not sufficiently hinder the salt intrusion to the hinterland.
- The current infrastructure situation does not provide enough capacity by car (persons and goods) and infrastructure for disclosing traffic from The Hague.
- The current waterway system is not sufficient for shipping.

A design objective to tackle the problem statement, was then defined as:

”Expanding the harbour while increasing the flood defence, capacity, nautical functionality and improving its connection to the main (aquatic) infrastructure of the nearby environment while safeguarding its historical culture.”

To reach this objective, multiple concepts were developed. The modified approach to the traditional hydraulic engineering design method was used, as elaborated in Section 1.2. The creative aspect, stimulated by this approach can be traced back to the fact that the boundary conditions and requirements were defined a after creation of the concepts. This translated into extraordinary sub-components of the designs:

- Moving the fishing industry transport from the road to the new waterways via the aquapuncture. This is not a conventional way of solving the traffic situation.
- The concept of a tidal park in the second harbour. The fourth harbour is created for extra harbour capacity, losing the requirement of capacity, the tidal park idea came into existence.
- The Peaky Blinders tunnel based on the canals in Birmingham. In this concept, the tunnel would be located underneath a residential area, which would not be considered when taking into account boundary conditions.

- The Haagvlakte which requires a lot of investment costs and would change the coastal view of Scheveningen.

From the 10 initial concepts, through the verification and a MCA the most suitable one was chosen.
By applying the requirements and boundary conditions (verification), it was concluded that 4 of the 10 initial concepts were not sufficient. The remaining 6 concepts (from this point considered alternatives), were then subjected to a MCA. The global overview of this MCA can be found in Figure 13.1. It is important to note that the criteria mentioned in Figure 13.1 are constructed out of multiple sub-criteria.

The Less is More alternative was then elaborated on the topics of the quay wall, breakwaters, port layout, watermanagement and infrastructure. This resulted in more detailed designs for these sub-components. From these sub-components, each corresponding conclusion describes the design parameters. To avoid repetition of the conclusions drawn in Chapter 8, a more global conclusion is drawn in this section.
Overall the objective as stated above is mostly met by integral implementation of the separate designs from Chapter 8, minus the parts that were not elaborated due to time restrictions.

Criteria / Design	1. Less is more	. Reunion of North and Sout	4. Tidal park Scheveningen	6. Sea Farm	8. Haagvlakte	10. Inside out
Infrastructure	2.3	4.4	3.15	2.9	3.4	2.7
Cultural Heritage	2.125	1.75	1.375	0.875	1.375	1.25
Water sports	2.85	3	3	2.5	3	3.5
Flood defences	4.3	2	3.8	4.3	3.1	2.5
Livability	4.5	3.9	2.7	2	4.3	1.8
Nature preservation	5	1.275	2.325	5	3.225	5
Technical feasibility	4.25	3.75	2.5	4.5	3.5	3.5
Building with nature (4th harbour)	2	1	3	3	5	1
Investment costs	3.75	3.5	2.75	4	2.25	3.5
Totaal:	34.325	27.825	27.85	32.325	33.4	27.25

Figure 13.1: MCA results

14

Discussion & Recommendations

In this chapter a discussion and recommendation following from the integral aspect of the project are elaborated. The discussion notes what could have been improved and learning points. The recommendation will be towards the municipality for further development and future projects within the scope. For detailed discussion per sub-component, see Chapter 8.

14.1. Discussion

- The conceptual designs, created in chapter 3, could have been divided into designs per tackled problem. A method like that would be similar to that of the Morphological Chart, which is generally used by the Industrial Design faculty. The Morphological Chart would have increased the level of reproducibility of the designs. At the start of the design process, this alternative method was also discussed. At the beginning of the project, the emphasis was on the integral design aspect of the solutions which led to picking the current approach instead. In retrospect the Morphological Chart would have been more logical.
- The designs were created without initial themes, which made designing them harder due to the lack of starting points and direction. Instead, picking a theme initially per concept could have created a concrete vision per design. Subconsciously this aspect was taken into consideration when developing the concepts, else no theme would have been possible.
- The assessments of the alternatives could have been done by performing a Cost-Benefit Analysis (CBA) instead of the MCA. The CBA would gain more insight in the monetary values of the project. Due to lack of time, and lack of detail in the concepts, an MCA was conducted instead.
  - We could have specified a budget through communication with the stakeholders. This could have supplemented the CBA, however due to our specification of the client, costs were not a highly significant factor in this project.
- The initial plan of this project group was to work on a plastic catchment plan for rivers in Bali. however, due to the outbreak of the Covid-19 virus, this project could no longer continue. This has caused a last minute switch of plans, where the well-prepared Bali project had to be dropped and this project replaced that. Due to this last minute rush, no company or governmental instance was approached as a client, and 2 of the 3 supervisors were approached rather late.
- A general discussion point when using an MCA is that when a different scale of scoring is used, it may lead to a different outcome, due to the level of detail. Another recognised MCA weakness is that some criteria may overlap. After feedback, most overlap was removed and clear definitions when scoring should minimize this impact.

14.2. Recommendations

- It is recommended to implement the proposed designs:
  - For the protection of the second harbour, the heightening of the quay walls surrounding the area is recommended. This heightening of the quay walls will be implemented in the form of a block on top of the current edges of the harbour.
  - For the extension of the breakwaters to the 10 meter depth contour it is recommended to implement a caisson vertical wall breakwater on a rubble mound foundation. A bullnose is recommended to reduce the mean overtopping rate such that a lower crest level suffices.
  - For the guaranteed water depth, it is opted to use the tidal window in order to save dredging costs. A possible layout of the harbour including the docking of a mini cruise and basins for fishing ships and yachts is proposed.
  - For the enlargement of the water lens, an infiltration pond near Duindorp is recommended. The artificial infiltration is supplemented by water from WWTP Houtrust.
  - For infrastructure, it is recommended to implement a tram between the Fourth Harbour and the city center of The Hague. The redesign of the Duindorpdam is also recommended, whether or not the tram will come.
- The components stated in Chapter 7, that were not elaborated in Chapter 8, should be elaborated in a detailed design.
- The components of alternatives that were not considered from Chapter 8 onward could still form viable solutions. It is recommended to keep these solutions in mind and elaborate these options to gain more insight before implementing one.
- Communication between stakeholders is recommended, as this is a broad project with significant impact.
- The project planning of the designs has not been incorporated in this project scope. The importance to arrange the project planning in a logical way is should be acknowledged. A few ideas:
  - Implement the tram before the cruise ship tourism comes.
  - Constructing the tram tracks and the pipeline at the same time along the Houtrustweg.
  - Build the fourth harbour at the same time as the breakwaters and the quay walls, in order to minimize the time that the port is shut down due to construction works.

A

Current situation and nearby future

In this Chapter, the current situation and nearby future of the Scheveningen harbour area is described. It begins with a description of the domain and boundaries of the plan area in Section 2.1.1.

A.1. Current development plans

A.1.1. Urban plan De Zuid

Since 2008 a new urban plan is developed for the southern part of the Scheveningen Harbour. The space around the third harbour was previously occupied by the Norfolk line. In 2005, the transport company decided to leave the Scheveningen Harbour and space became available for a housing project. Urban plan De Zuid consists of 6 sub-plans that will be delivered in stages. In the total plan, around 400 apartment homes will be created. In between the apartment buildings, a neighbourhood of family homes is created with restricted traffic. There is also room for some penthouses that have a view on the beach, sea and harbour.

Figure A.1: Apartments of urban plan ZuidHaven.

Februari 2008, the first part of De Zuid called ZuidDuin was delivered consisting of 85 apartments. The next phase is called ZuidHaven and is located along the Houtrustweg similarly to ZuidDuin and consists of 62 apartments. ZuidKade, ZuidHof and ZuidBaai are next phases of the project and these parts are located more towards the third harbour. The final phase of the project is creating the sub-plan called ZuidZicht, which will be situated at the location of the temporary theater called Zuiderstrandtheater.

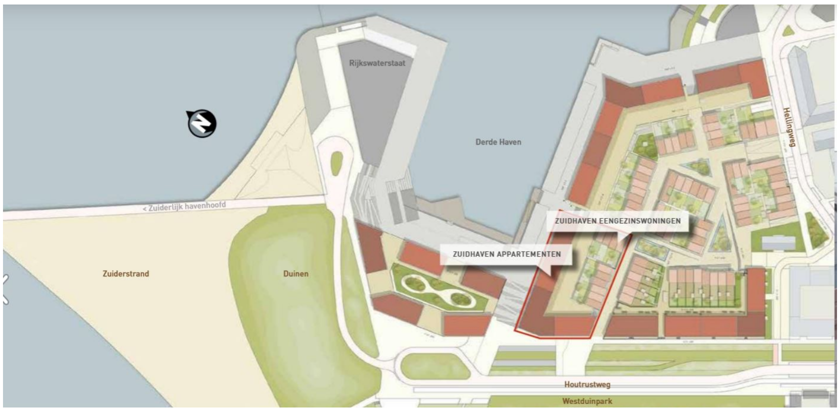


Figure A.2: Overview of urban plan DeZuid.

A.1.2. Northern breakwater

At the northern part of the Scheveningen Harbour, a plan has been developed by the Viscluster (local companies) to redevelop this area and accommodate new business locations, hospitality and recreational space. The central concept of this location is that Scheveningen finds it’s origin in fishery. The plans will be delivered around 2022.

A.2. Watersports in Scheveningen

Municipality The Hague has the ambition to manifest itself as main watersport center. It has plans to invest in training facilities for athletes. In this way a new generation watersport athletes are trained for participating the Olympic Games. The Hague also plans to host main watersport events such as the ISAF World Sailing Championship in 2022. These events will help the city to get global attention, which will help businesses in the city and will boost the economy.

In recent year Scheveningen was one of the stopover cities of The Ocean Race, a yacht race held every three or four years. The Scheveningen Harbour has been pitstop of the race in 2015 and finish in 2018. In 2022 The Ocean Race will come to The Hague for the third time. This time it will be the finish and start point of one of the stages of the race. In the future, the fourth harbour of Scheveningen should remain an attractive start- and finish for The Ocean Race. The event suits the profile of Scheveningen as main sea sailing center of North Western Europe.

A.3. Nature reservation

The development plans of the renovation of the Harbour area took into consideration two types of regulations for the Natura areas: 1) Flora and Fauna act and 2) the Nature Protection act (Natura 2000).
For the Flora- and Fauna wet it has to be specified what species have to be protected. These can be found in the mitigationplan of scheveningen.
The area concerned in the development plan lies at edge of 2 protected nature areas, 'Westduinpark and Papendal' and 'Meijendel and berkenheijde' (See figure A.3). This area is part of the Natura 2000, which contains all the protected nature areas in the European network. For the protected areas and surrounding area major boundary conditions are relevant. The Natura 2000 areas are appointed accordingly to the guidelines of European Bird- and Habitat directives. These guidelines concern the types and Habitats of birds.
In an environmental impact report, it should come forward if a plan or project has impacts on the protected areas. When it is not possible for a plan or project to exclude significant impacts a 'passende beoordeling' has to be incorporated, which can include mitigation measurements. The following questions should be answered:

- What are the preservation goals for the Natura 2000 area?
- Will these goals be achieved or will mitigation measurements be necessary?
- What effects does the plan have on the species- and habitat types? Taking into consideration that a plan in an area surrounding the protected area can have external work.
- Are there activities that will influence the species and habitat types?
- Will there be change in the natural characteristics?

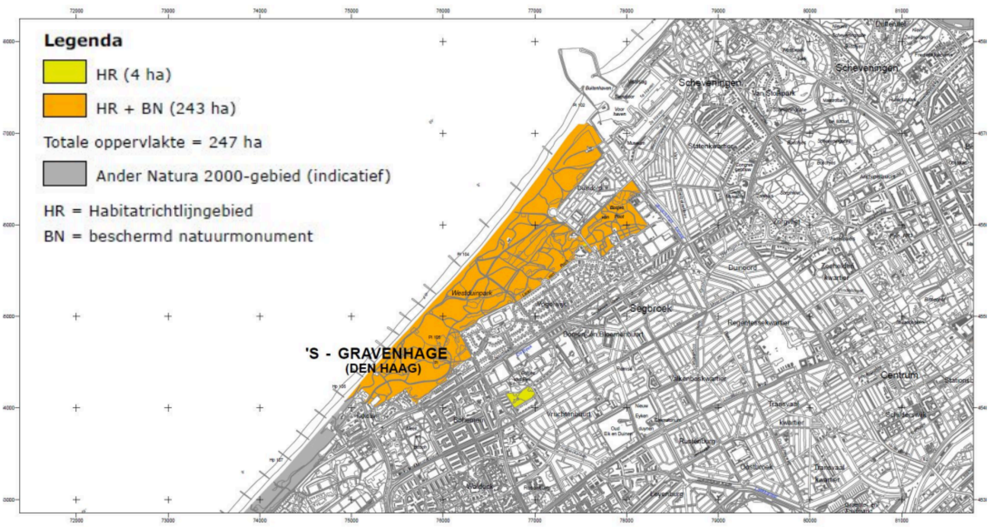


Figure A.3: Natura 2000 areas around project development area.

A.4. Terrain Analysis

The terrain of the project area is quite complex. After a field trip, it was observed that a lot of ground can not be used at the moment because it is still occupied for construction by companies like Boskalis. The development plans related to Urban plan de Zuid are in full course of action, which therefore should be incorporated in the planning of the fourth harbour.

When the general information of the project was given, it was specified by Voorendt that mainly the flood safety of the second harbour is not sufficient. This can be explained with the fact that the specific harbour has flooded twice in the last years. The KNMI has also warned several times that the quay walls are not sufficient for protecting the harbour against floods. A map of the Netherlands has therefore been utilised, which displays the height relative to a fixed horizontal axis. It is the ANH (Actueel Hoogtebestand Nederland) map.

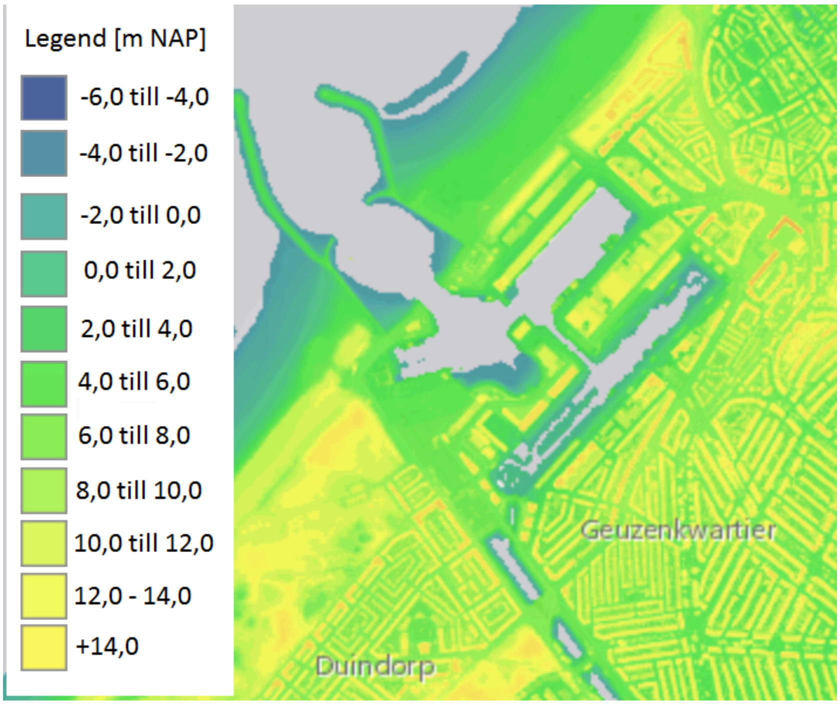


Figure A.4: Elevation map of Scheveningen. The second harbour is located at 2.5m compared to the first harbour at 3.5m

From Figure A.4 it becomes evident that the second harbour is clearly situated lower than the first and third harbour. This explains the fact that the second harbour has flooded twice in the past (and nearly flooded even more).

B

Cultural Heritage

Local fishing has always been a part of Scheveningen even when it did not have its own harbour. The name Scheveningen was first used in an act in 1357 by its inhabitants requesting count's favor. This is due to the increased demand of fish that made the fishermen want to settle in the area. At that time, the ships would be carried on to the beach due to the absence of a harbour. After a large storm had demolished the ships in 1894, the then Minister of Water Management Cornelis Lely insisted on the construction of a harbour in Scheveningen. The harbour of Scheveningen was opened in 1904, after the construction had finished. After had been extended with two extra harbours over the years and has shaped the sight of Scheveningen ever since. It has also provided employment opportunities in the region which meant that the harbour is deeply ingrained in the lives of many inhabitants. Most of the characterising company buildings are located at the second harbour and the Norfolkterrain and are rich of fishing-related history. These company buildings will vanish over the course of time and will be replaced.

This process will result in the loss of the cultural historic of the harbour but due to its importance in the inhabitants' lives, it is important to pass on its identity to the younger generation. A large group of inhabitants feel the need to do this and therefore start initiatives. For example, the resident's organization of the harbour quarter has various initiatives for the preservation of the cultural historic that vary from the placement of new art objects and transferring existing art objects to different locations within the harbour. A number of these objects can be found in Figures B.1 and B.2. Also, an online forum is created to discuss the cultural pride of the harbour of Scheveningen. Earlier mentioned art objects are digitized there by the many amateur made movies by inhabitant.

Figure B.1: Old anchor visible in the harbour

Figure B.2: Old ship's propeller visible in the harbour

These examples indicate how important the cultural historic is for the inhabitants and thus also for our project. The design of the project must therefore have the support of the locals which can only be obtained if the expansion of the harbour is an integral part of the older harbours as well. This will become an important aspect the design phase and must be taken into account accordingly.

The cultural heritage of the harbour's surrounding area is so important that many neighbourhoods their characteristics and urban design are protected by either the municipality or the government. These neighbourhoods are listed in Figure B.3 where the ones in red and blue are being protected respectively by the government and municipality. In this case the characteristics of the following three neighbourhoods are being taken into account that enclose the Scheveningen harbour: Duindorp, Statenkwartier, Scheveningen-dorp and Vogelwijk.

Figure B.3: Protected neighbourhoods in The Hague (Gemeente Den Haag, 2015).

Duindorp

At the location of Duindorp lied a fisher's village that originates from the 14th century. However the impoverishment of the village and surrounding area led to protests against the local municipality of The Hague. This resulted in a plan for the construction of Duindorp as it is today. This took place between 1915 and 1930 and the floor plan is nearly rectangular. The houses are relatively simple and do not have a complex facade because these are designed as social renting houses. From Figure B.4 it can be seen that the houses have a flat roof and are relatively low. The streets seem rather spacious because most of them are turned into one way streets.

Figure B.4: Duindorp

The inhabitants of Duindorp have a strong affiliation with their neighbourhood, Westduinen, Stille strand, the harbour and the fishing industry.
For them it is important that the harbour and its area remain unchanged in aesthetics and ambiance. If this is not done properly in their eyes, they are willing to take action with numbers as they are very connected with each other.

Statenkwartier

Statenkwartier protected by Rijk since 26th of july 1996 because its character does not lie within the individual objects but within the urban design of the neighbourhood with its avenues, squares and preserved ensembles. The large amount of green and the spatial profiles of the streets are typical for this neighbourhood. Although later adjustments have been added, the homogenous image of Statenkwartier has been preserved for most part. Its typical characteristics are listed below and can be seen in Figure B.5.

- A cityscape that consists of premises from the early 20th century that together form an intact ensemble.
- The patterns of the streets are clearly visible as they contain long sight axes and square crossings. This gives the neighbourhood its own identity.
- The historical and sceneric value of the Scheveningseweg and its relation with the Van Stolkpark.

Figure B.5: Urban design of Statenkwartier

Scheveningen-dorp

The village of Scheveningen has been under municipal protection since the 26th of February 2003. It can be divided into four sub districts as shown in Figure B.6 and their respective characteristic are listed below the figure:

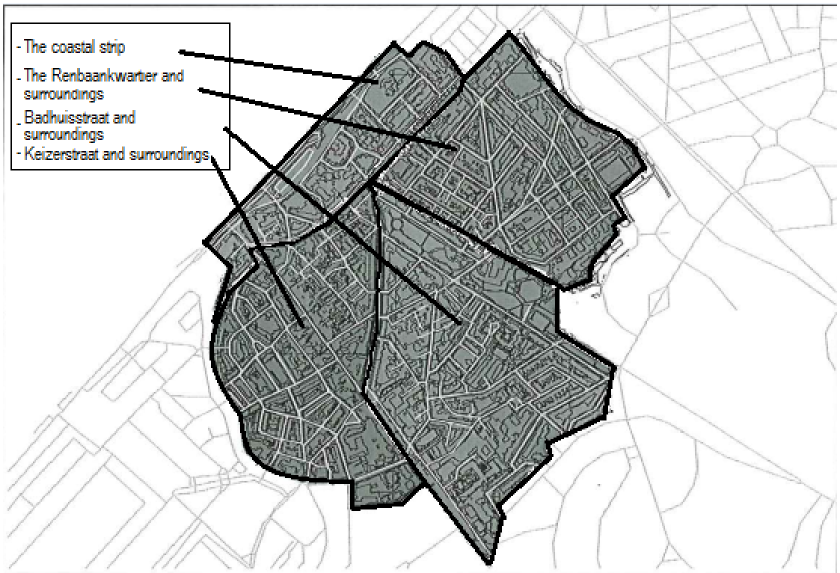


Figure B.6: The four sub-districts within Scheveningen-dorp

1. **The coastal strip**
    - The preserved urban design of the old seaside resort from around 1900 which is prolifically present between the Keizerstraat and Jongeneelstraat.
    - The new boulevard that has been renovated and the architects tried to mimic the intimate and village character of Scheveningen.
2. **The Renbaankwartier and surroundings**
    - The homogenous character of the houses in terms of architecture including the typical diagonal main access roads.
    - he many courts and streetcourts that gives the neighbourhood its village character as can be seen in Figure B.7.

Figure B.7: Streetcourt in Scheveningen

3. **Badhuisstraat and surroundings**

- The so called high architectonic quality of the houses for example along the Haringkade.
- The building image of winding street course which makes an impression of an intimate village.
- The historic meaning of the Badhuisstraat and the Keizerstraat

4. **Keizerstraat and surroundings**

   - Has a predominantly village character with mainly low rise buildings and a winding street course.
   - The Keizerstraat is the core of Scheveningen from where the village expanded.
   - The fisherman's houses in the Zeilstraat and Ankerstraat.
   - The Dr. De Visserplein and surroundings consisting of buildings that are sober but well maintained.

Vogelwijk

Vogelwijk under municipal protection since the 11th of february 2002 to preserve its character which can best be described as a garden district. This becomes clear when looking at an overview of the neighbourhood as it shows a lot of green between houses but also a lot on the streets. Its typical characteristics of have been listed below:

- Buildings that are small villa's and semi-detached houses positioned at avenues and streets with a winding course.
- The abundantly present green both in gardens as in the streets and squares as can be seen in Figure B.8 in the Appendix.
- The ensemble value given by the landhouses of A.J. Kropholler on the Kwartellaan. See Figure B.9 in the Appendix.
- The sceneric value of the Westduinpark and the Bosjes van Poot.

Figure B.8: Urban design in the Vogelwijk

Figure B.9: Ensemble of houses in the Vogelwijk

Conclusive and summarized cultural historic

The most important neighbourhoods are the Statenkwartier and Scheveningen dorp as they enclose the harbour. The structure of the houses and setup of the streets show great differences between those two neighbourhoods. Where the Statenkwartier has a spacious design, the Scheveningen dorp is rather small and intimate. It is therefore important to take both characteristics into account when redesigning the area. The side of the harbour adjacent to the Statenkwartier can therefore have avenues and wide streets. On the contrary, the side of the harbour adjacent to Scheveningen village should have small streets to keep its intimate and old seaside resort characteristics. This is mainly done to keep a homogeneous design and stimulate a smooth transition from neighbourhood to harbour.

Cultural heritage in harbour design

Now that it has been determined what the cultural heritage is and why it is so important for the region and surroundings of the Scheveningen harbour. It is time to determine how to implement and process this into the harbour's design. As cultural heritage is not something that can be grasped into numbers it is impossible to create a requirement of it. However, as it must be implemented in the design it will be used as a criteria for the Multi Criteria Analysis in Chapter 6. In this analysis, every alternative will be given a score on how well it performs in terms of preservation of cultural heritage. The determination of these scores will be explained at a later stage in the chapter of the Multi Criteria Analysis.

C
MCDA

C.1. Weight factors
Elaboration of the weight factor values used in the MCDA

C.1.1. Weight factors
The weight factors per criterion are determined by setting the investment costs as baseline measurement. From this notion the major categories were compared towards the base criterion, which resulted in the same factor for infrastructure, economic development, flood defence, live ability and technical feasibility. The cultural heritage was rated considerably lower (0.5) due to the impact of the designs not interfering on major scale with the criteria it was subjected to. The nature preservation criterion concerns mainly the Natura 2000 area which renders it important. However the interests of the most influencing stakeholders are more related towards the primary criteria stated above. The water sports center had a proper location in every design, room for adaptability differed therefore this criterion was set lower than the baseline.

C.1.2. Sub-criteria weight factors
- **Infrastructure**
  The two most important sub-criteria are traffic flow improvement and accessibility of the harbours as followed from the stakeholder analysis. Whereas the public transport and internal reachability are not wishes from stakeholders and are therefore nice to have.

- **Cultural heritage**
  The integration of urban design of the harbour into its surroundings is the most important criterion as this catches the eye more than the visibility of various cultural objects.

- **Water sports**
  The amount of space for water sports facilities is slightly more favorable than that of reachability of open sea. The municipality of The Hague has as vision to be a leader in water sports and to train athletes for the Olympics. It is a must to have enough space available for the facilities for their training to realize this vision. However the sub-weight factors are almost similar as the reachability of the sea is also important in terms of safety of water athletes. They must be able to reach the open sea safely without possible dangers from mini-cruises or other vessels.

- **Economic development**
  The capacity of the fourth harbour is the most important sub-weight factor as contributes the most directly to the economic development. Whereas the effects of tourism and hospitality branche are less obvious.

- **Flood defences**
  The adaptability is the most important factor as this considers measurements that can be taken to improve the flood protection of the port. Aesthetics is the least important factor as it cannot be

assigned to any operational functionality and the proposed flood protection systems are not large enough to form a landmark such as the Maeslantkering.

- **Livability**
  The noise and visual nuisance sub-criteria are more important as they are permanent hinder for the inhabitants in contrary to the temporary hinder of construction. Out of the two permanent nuisances, the noise nuisance is the most important. This is due to the fact it is present near the beach and possibly within houses and therefore inhabitants cannot escape this nuisance. Whereas visual nuisance can be removed by either not looking at the nuisance or by blocking the nuisance with the use of curtains.

- **Investment costs**
  The sub-weight factors aquapuncture and salinity intrusion have the lowest values as solutions to those problems require the least investment costs. Infrastructure just has the highest weight factor because its investment costs are the highest as the construction of a tunnel and widening of roads are rather expensive.

- **Nature preservation**
  It is more desirable to preserve nature than to compensate for the loss of it because it saves time and money.

- **Technical feasibility** All four sub-criteria have the same value as it is obligatory that all solutions be possible to realize.
  Building with nature Sub-criteria are not available as it contains only one criteria.

C.2. Appendix B.2: Scores

Elaboration of the MCDA scores

Infrastructure

- **Public transport**
  The only concept incorporating public transport is concept 1, therefore it is the only one that satisfies this criterion. Concept 4 contains a water taxi from the harbour to the peer and has therefore received a medium score.

- **Traffic flow**
  Concepts that incorporate the 6 km long tunnel towards Kijkduin score well. Concepts that direct traffic flows to the northern side score medium. Concept 4 does not take into consideration possible bottlenecks therefore it scores lower, so does concept 1 due to not improving the traffic flows as much as compared to the other concepts.

- **Accessability of harbours**
  Concept 1 is the least accessible due to only improving the infrastructure. The most accessible designs (3 8), they make use of tunnels that disclose the traffic entirely separated from the current traffic in Scheveningen. The concepts that incorporate the use of the Strandweg (4 6), were set at medium.

- **Internal reachability**
  The internal reachability is mainly improved in concept 3, which incorporates a bridge and tunnel (thus a wide range of traffic types). This is followed by concept 8 there a sluice is placed, which could function as passage. The internal reachability is not prioritized in any of the other concepts, hence they score lower.

Cultural heritage

- **Integration of urban design**
  Concept 1 is based on preserving most of the system as it functions and looks in the current situation. The tidal turbine in concept 4 does blend in the least into the current situation, therefore it scores the lowest. Concept 8 does not blend into the current situation as well, however it is

built seawards and not in the current harbour area. The other concepts scored at medium for this criterion.

- **Visibility cultural objects**
  Concept 6, 8 and 10 score a little lower due to the breakwater layouts being altered. The other systems do not lead to a visible change in cultural heritage and score higher.

Water sports

- **Amount of space for the water sports center**
  Concepts 1, 3, 4 and 8 score relatively high due to scoring due to being situated on the northern breakwater. The differences made in the scores were due to the amount of space for expansion. Concept 10 scores lower due to having no focus on watersports.

- **Reachability of open sea**
  Concept 1 scores the highest due to the watersports center being located among the breakwater. Concepts 8 and 10 are not focused on the watersports center, however do have space for it. Other concepts the passing of full breakwater and if a sluice were incorporated in the scoring.

Economic development

- **Tourism (attractiveness)**
  The concepts scoring the highest on tourism attraction are concept 4 and 10. With 10 having a lower towards tourism. The other concepts have no real addition towards tourism except for the aquapuncture (which are therefore rated similarly.

- **Capacity of the fourth harbour**
  The capacity of the fourth harbour is represented inaccurately in the designs. However, from boundary conditions and the current layout of concepts 1, 3 and 10 score low. whereas concept 8 scores higher due to overdesign and far seaward extension.

- **Hospitality branche (space and location)**
  The hospitality branche has been located along the new fourth harbour in every concept, therefore the results were similar to the capacity. The space for the hospitality branche differed per concept (i.e. how much of the harbour edges were surrounded by land).

Flood defences

- **Aesthetics**
  Quay walls heightening leads towards a preservation of the current layout and therefore concepts with such a flood defence scores high. Hydraulic structures such as a sluice is aesthetically nice to engineers but less for inhabitants. Concepts with this solutions therefore score low. Concept 8is deemed aesthetically nice since it incorporates a lot of nature in its design of the fourth harbour.

- **Operational obstruction**
  Concepts that have a sluice or bridge incorporated score lower under this criterion as opposed to those that don't require an extra operation such as heightening of the quay walls. Therefore the concepts 1, 4 and 6 score high whereas concepts 3,8 and 10 score low.

- **Adaptability**
  The adaptability of the system was evaluated under this criterion: if the heightening of the quay walls has been executed and a sluice can be additionally implemented later on at two locations. If an extra sluice is added in the first location at the Dr. Lelykade, a high score on the adaptability will be obtained. Whereas the second location is in between the two breakwaters and scores medium on this criterion. The concepts 1 and 6 therefore score high and concepts 4,8 and 10 score medium. Concept 3 has a low score because the only possible adaptation for the flood protection of the system is to heighten the quay walls.

Livability

- **Visual nuisance**
  Concept 3 scores low due to the incorporation of a pedestrian bridge that must have quite a

significant height to ensure vessels to pass under it. Also concept 6 scores low due to the partial extension of the fourth harbour to the north which makes it more visible for inhabitants compared to if it was located in the south. Concept 8 and concept 10 are large constructions that change the visuals of the entire area, however concept 8 is extended more seawards and incorporates more nature. Therefore it scores a little higher than concept 10.

- **Noise nuisance**
  Concepts where the nuisance is far away from neighborhoods or underground score high, therefore concepts 4,6 and 10 score low respectively due to their tourism and infrastructure. Concepts 3 and 8 score relatively high due to the implementation of a tunnel and concept 1 scores high because the infrastructure problems are solved with by trams.

- **Duration/hinder of construction**

Investment costs

- **Aquapuncture** The aquapuncture is the most expensive in concept 4 due to requiring a tunnel and canal to be dug through a neighbourhood. The construction (or renovation) of a ship-lock will cost more than removing one, hence concept 1 is ranked higher than other concepts where it was opted to remove the scour-sluice. The pipeline infrastructure is very similar in all solutions, therefore it played a minor role.

- **Salinity intrusion** The quay walls will by estimate cost more than incorporating small sluices, with larger sluices this may be around equal. Due to the sand that has to be placed concept 8 will be the most expensive and large sluices, concept 10 follows due to the breakwater alteration. The concepts with large sluices follow up on those mentioned above in the cost aspect.

- **Flood protection** The quay walls will by estimate cost more than incorporating small sluices, with larger sluices this may be around equal. Due to the sand that has to be placed concept 8 will be the most expensive and large sluices, concept 10 follows due to the breakwater alteration. The concepts with large sluices follow up on those mentioned above in the cost aspect.

- **Infrastructure** The least infrastructure is placed in concept 1, this will be the cheapest alternative. The tunnel towards Kijkduin is the most expensive, hence concept 2 and 8 score the lowest. Other concepts score marginally lower than concept 1.

Nature preservation

- **Preservation and compensation** Concepts that do not alter nature score the highest on nature preservation. In concept 3 the harbour extension takes a part away from the Natura 2000 area, therefore it scores lower. The tunnels constructed towards Kijkduin score lower than full preservation, and higher than concept 3. Nature is only compensated in concept 8 on large scale, therefore it scores maximum.

Technical feasibility

- **Flood protection**
  The most challenging design of feasibility is concept 8, opposed to other concepts where a sluice is installed at the Pijp which score high on this criterion. Heightening of quay walls around the second harbour and large sluices between the breakwater require more conditions, score between those criteria.
  Incorporating a sluice between the breakwaters is less technically feasible than heightening of the quay walls and a sluice at The Pijp. Incorporation of a small sluice is very feasible.

- **Aquapuncture**
  The operational water management at concept 1 may be complicated compared to the other concepts. Concept 4 scores low due to the tunnel that has to be constructed, which goes through a lot of physical boundaries.

- **Infrastructure**
  The 6 km long tunnels to Kijkduin along the dunes are the least feasible in this regard, the tram

incorporation of concept 1 is the most feasible. The tunnel towards Hubertusplein in Concept 3 is more feasible, than the 6 km tunnels therefore it was rated higher. The broadening of the strandweg will be very feasible, therefore it scores relatively high.

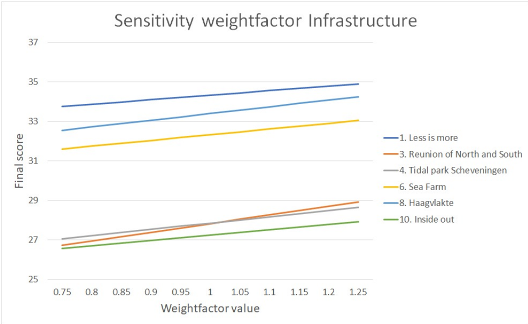
- **Accessibility of construction site**
  Concept 1, 3, 6 and 8 will have reachable construction sites through the current infrastructure. Concept 10 scores low because a large construction must be made within the breakwaters. Concept 4 has a medium score because on one hand for the placement of the turbine there is a lot of space available on the quay walls and the incorporation of the tunnel around the Hendrikskade which will be inside a densely populated neighbourhood.

Building with Nature

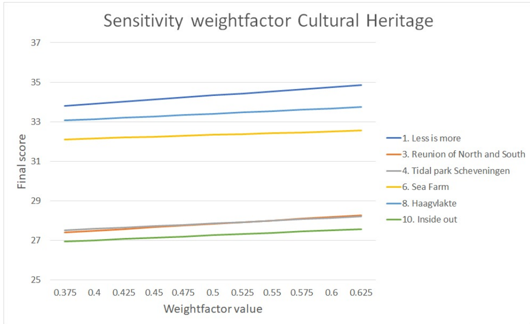
- Concept 8 has the highest score as it incorporates part of the coast into the system by giving it the function of a breakwater. Concepts 4 and 6 have a medium score because they also incorporate nature into the system but not at such a large scale as in concept 8. Concepts 1,3 and 10 have the lowest scores because there is no incorporation of nature into the system.

C.3. Results of sensitivity analysis

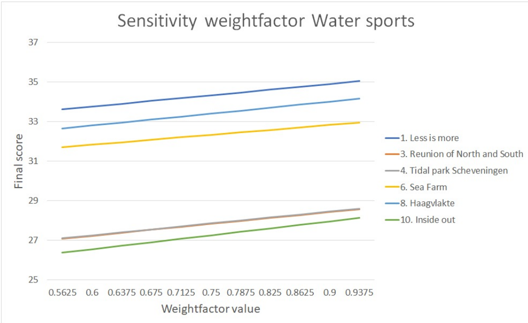
Figure C.1 gives the results of the sensitivity analysis for the MCDA. For all criteria of the MCDA, the corresponding weight factor was slightly adjusted while keeping the other weight factors unchanged. The Figure presents the resulting total score for each concept against the weight factor.



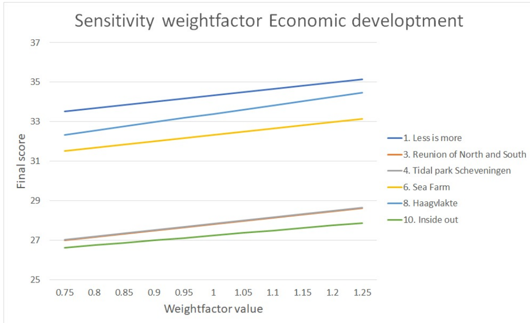
(a) Infrastructure



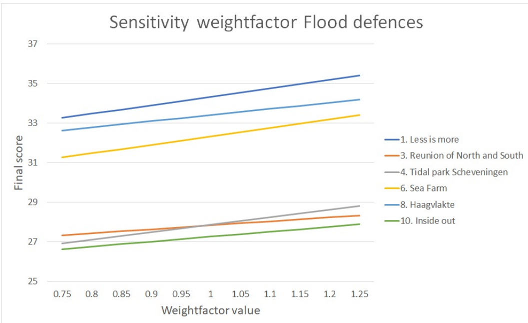
(b) Cultural heritage



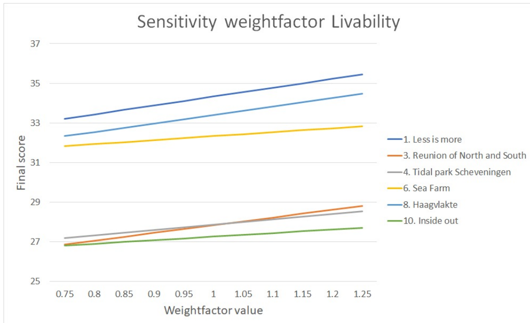
(c) Water sports



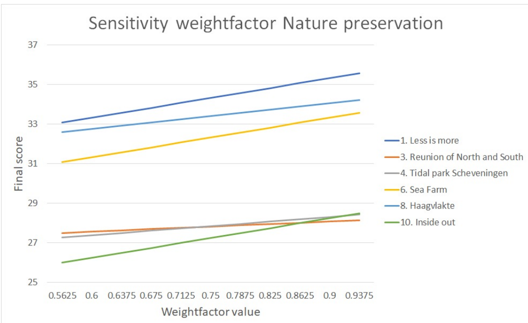
(d) Economic development



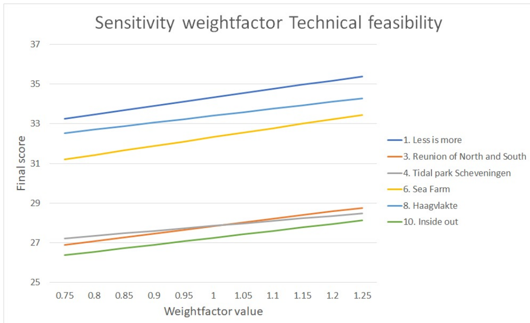
(e) Flood defences



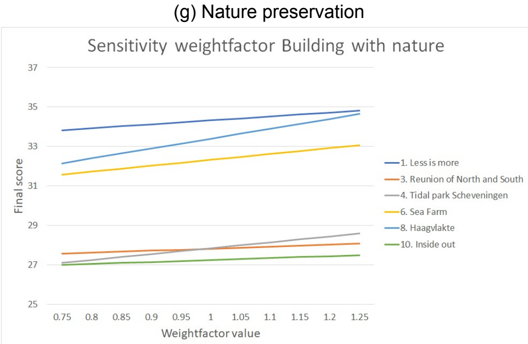
(f) Livability



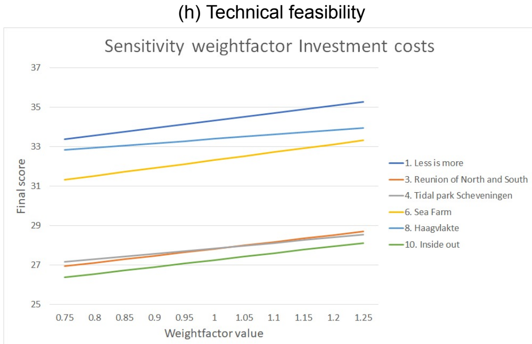
(g) Nature preservation



(h) Technical feasibility



(i) Building with Nature



(j) Investment costs

Figure C.1: Results of the sensitivity analysis of the weight factors of the MCDA.

D

Quay wall design

D.1. Appendix: Sea Level Rise

An increase in the relative sea level rise is sure to happen, which implies it needs to taken into account as well. Unfortunately, the data set containing the water elevation does give enough information to state certain characteristics about the sea level rise. Therefore different sources need to be consulted to quantify the rise after which it can be applied to the required height of the quay wall.

The research institution Deltares investigated the sea level rise over the course of time. Their employees were not exclusive in believing that the sea level rise happened with a constant rate. A new research article, published by Deltares researchers themselves however, states that the growth might even increase in the next couple of years. Some time ago certain estimations were made regarding the sea level rise until the year of 2100. These estimations are called the Delta scenarios. Recent research shows that the assumptions and boundary conditions from that time might not be true anymore. This mainly has to do with the melting land ice and glaciers on Antarctica and the (stochastic) emission of greenhouse gasses. Deltares consistently assumed three possible scenarios for the future:

- The first scenario is the initial scenario, which as stated before is called the Deltascenario. It assumes a small increase in temperature, but does not take into account a correct estimation for the melting of ice at Antarctica.
- The second scenario is called the RCP4.5 scenario. The scenario assumes that the agreement made in Paris is met and does account for the melting of the ice. The global temperature increase in 2100 will equal 2 degrees Celsius.
- The last scenario is called the RCP8.5 scenario. It does account for the melting of the ice but it assumes the agreement made in Paris is not met. It therefore involves a higher concentration of greenhouse gasses in the atmosphere resulting in a global increase of 4 degrees Celsius.

Figure D.1 visualizes the estimations from Deltares.
Deltares did not supply the mathematics, assumptions and boundary conditions behind these scenarios. For that reason combined with the uncertainties regarding sea level rise it was decided to visually choose the point of gravity to the right of Figure D.1. Note that a certain amount needs to be subtracted because this figures defines the elevation in the year of 1995 as reference.

$$\Delta d_{sea} = 1\mathrm{m}$$

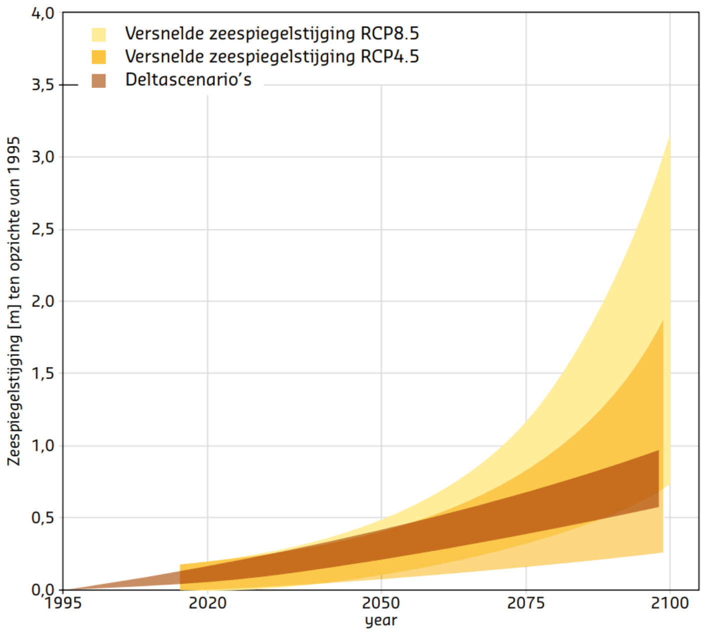


Figure D.1: The results from the three scenarios visualized in one figure. (Deltares, 2018)

D.2. Appendix: Waves in the port

D.2.1. Waves

The surface adjacent to the quay wall is subject to waves coming from the port as well. The excess water needs to be able to leave the top platform of the quay wall otherwise the water will pile up hindering its functional abilities. Unfortunately no data is available regarding wave heights in the port themselves. The Hydraulic Structures Manual however states (page 97) formulation allowing the estimation of wave heights in the absence of data. The equations are based upon research performed by Charles L. Bretschneijder. Equation D.1 and D.2 compute the so called dimensionless wave height and dimensionless wave period respectively. The variables having a tilde on top are the dimensionless variants of the regular variables.

$$\tilde{H} = \tilde{H}_\infty \left(\tanh(0.343\tilde{d}^{1.14}) \cdot \tanh\left(\frac{4.41 \cdot 10^{-4} \tilde{F}^{0.79}}{\tanh\left(0.10\tilde{d}^{1.14}\right)} \right) \right)^{0.572} \tag{D.1}$$

$$\tilde{T} = \tilde{T}_\infty \left(\tanh(0.10\tilde{d}^{2.01}) \cdot \tanh\left(\frac{2.77 \cdot 10^{-7} \tilde{F}^{1.45}}{\tanh\left(0.10\tilde{d}^{2.01}\right)} \right) \right)^{0.187} \tag{D.2}$$

$$\tilde{H} = \frac{gH_{m0}}{U_{10}^2}, \tilde{T} = \frac{gT_p}{U_{10}}, \tilde{H}_\infty = 0.24$$

$$\tilde{F} = \frac{gF}{U_{10}^2}, \tilde{d} = \frac{gd}{U_{10}^2}, \tilde{T}_\infty = 7.69$$

The wave heights are thus dependent upon two variables; the wind velocity and wind direction. In Figure D.2, both of these variables are plotted in a histogram. As no data from Scheveningen is available, the measurements from Hoek van Holland and IJmuiden were taken. These locations lie both north and south with respect to Scheveningen, and are coastal zones as well. As such it is justified to define these measurements as representative when combined. The wind distribution and wind velocity in Scheveningen will be estimated by taking the average values of the time series from Hoek van Holland and Scheveningen.

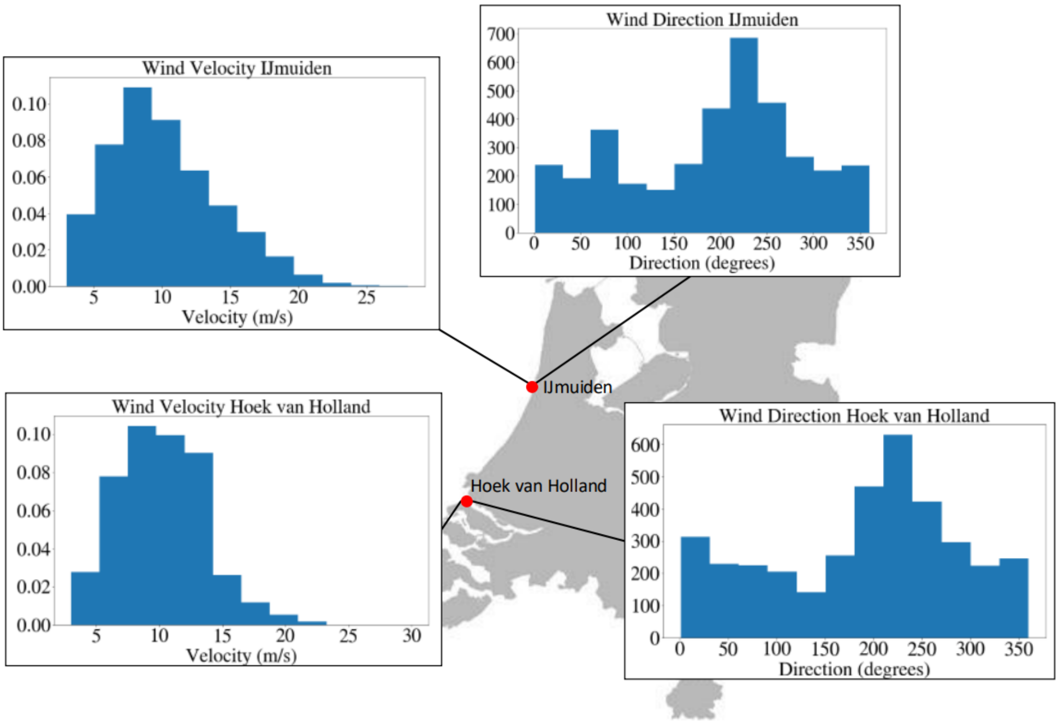


Figure D.2: The histograms of the data which is used in order to model the wind distribution in the port (KNMI Uurgegevens).

The data sets containing the wind directions indicate them in degrees. KNMI indicated the meaning of the directions in a brochure. Important in the design of the waves, is the fact that only a certain range of the direction contributes to waves reaching the to be redesigned quay wall. This contributing range of directions also results in different possible fetches.

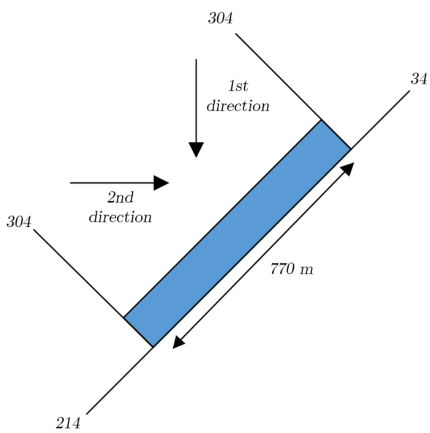


Figure D.3: The possible directions which induce waves towards the quay wall at the Doctor Lelykade. The given values correspond to the reference axis defined by KNMI.

The wind directions thus directly correspond to fetches, for which the greatest fetches are the ones parallel to the quay wall. Using trigoniometrics, the directions in degrees can be translated to the fetches. When this is done, an empirical distribution function of the fetch is found. When the depth is then found using "Navionics", all the variables gathered which are required in Equation D.1. To compute the depth as one single value, the different depth values in the port will be averaged spatially. This depth can be immediately read from the Navionics app. It apparently has a value of 5.3 meters. Using programming software, a Monte Carlo simulation will be done which uses a random picking function to simulate several wave heights. All the wave heights can then be analyzed in order to finally deduce the one for which the quay wall shall be designed.

Modelling the fetch distribution
In order to model the fetch distribution, the data set containing the wind directions in Scheveningen is used. Note that it was not directly measured but estimated from the data corresponding to IJmuiden and Hoek van Holland. Defining the boundaries for the geometry of the water body in combination with trigoniometrics allows the calculation of the fetches.

The calculation of the fetches was done using Python, the codes in the Appendices indicate the trigoniometrics used to calculate the fetches.

The Monte Carlo simulation is based upon Equation D.1. In this form however the wave height itself is still implicitly found in the equation. Specific algebraic operations allow the direct computation of the wave heights.

$$H = \frac{U_{10}^2}{g}\tilde{H}_\infty \left(\tanh(0.343\tilde{d}^{1.14}) \cdot \tanh\left(\frac{4.41 \cdot 10^{-4}\tilde{F}^{0.79}}{\tanh\left(0.10\tilde{d}^{1.14}\right)}\right)\right)^{0.572} \tag{D.3}$$

The simulation uses both the wind velocity data set and the newly created fetch one. Using a random picking function Equation D.3 can be used to model $N = 100000$ wave heights as functions of the stochastic variables wind velocity and fetch. Figure D.4 shows the results of the simulation.

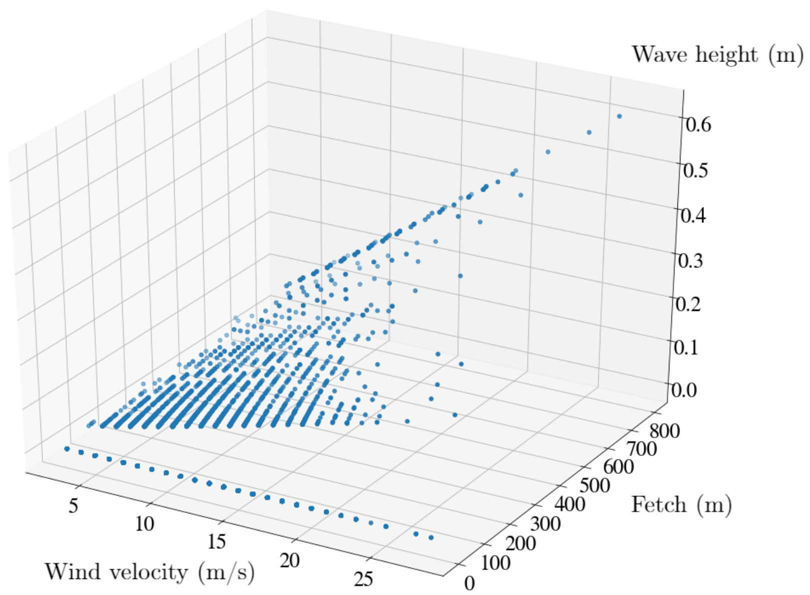


Figure D.4: The results of the Monte Carlo simulation. The horizontal axes indicate the wind velocity and fetch in meters per second and meters respectively. The wave height is shown on the vertical axis in meters.

The wave heights themselves can now be processed further such that the corresponding return periods can be calculated. Again, this is a design step involving logic instead of pure mathematics. One should consider the frequency in which the quay wall might be overtopped due to the waves in the port. If one allows this to happen on a daily basis, the port might get overtopped with high amounts of water in the scale of months. Therefore it is chosen that the wall is overtopped once in 200 days. The corresponding wave height is 0.2 meters. Because the wave height is distributed evenly over the equilibrium water level, half the wavelength needs to be added to the height of the quay wall.

$$\Delta d_{waves} = 0.10\text{m}$$

D.3. Appendix: Doctor Lelykade and Second Port characteristics

Different sources were used to obtain (soil) characteristics regarding the corresponding area. The required elements are for example the average depth of the second port, average elevation of the quay wall in its current state. Many measurements were made available, of which the spatially most representative ones were used in this research (Dinoloket, 2020).

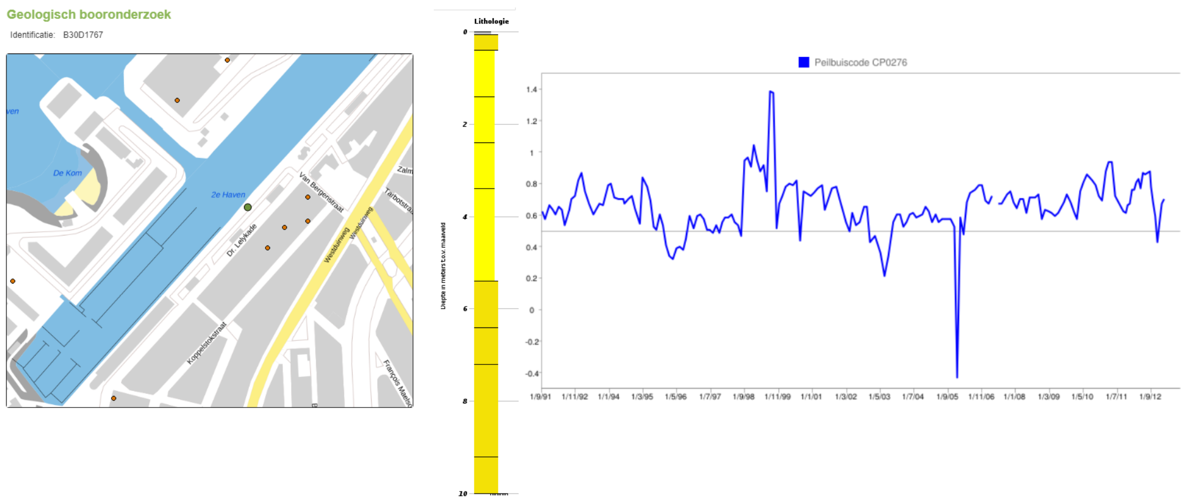


Figure D.5: On the left side the location is given at which the measurements were taken. The middle figure indicates the soil layers of the site. To the right the ground water table is given in blue as a function of time between the years 1992 and 2013. The average elevation of the groundwater table beneath the Lelykade equals 0.5 meters above NAP.

The site characteristics show that the layer only consists of rather find sand material. This implies that only three different densities are necessary for the design of the sheet pile wall:

$$\gamma_d = 16\text{kN/m3}$$

$$\gamma_w = 20\text{kN/m3}$$

$$p = 10\text{kN/m3}$$

These values were also obtained from the material of the BSc course Soil Mechanics. In the soil profile, one can clearly see that there is little variation in the soil profile (regarding the layer types). It is therefore justified to assume one homogeneous soil type.

According to the AHN, the quay walls around the second port are not equal in height. One might therefore consider the coordinate corresponding the lowest elevation and assume this one as normative. In this case an export of the data has been request. The deficit height can then be integrated over the walls. Latter option is economically more accurate but requires more data. In the case of absent consequent data, certain heights can be computed using the map, after which linear interpolation allows the estimation of the elevations between the computed coordinate heights.

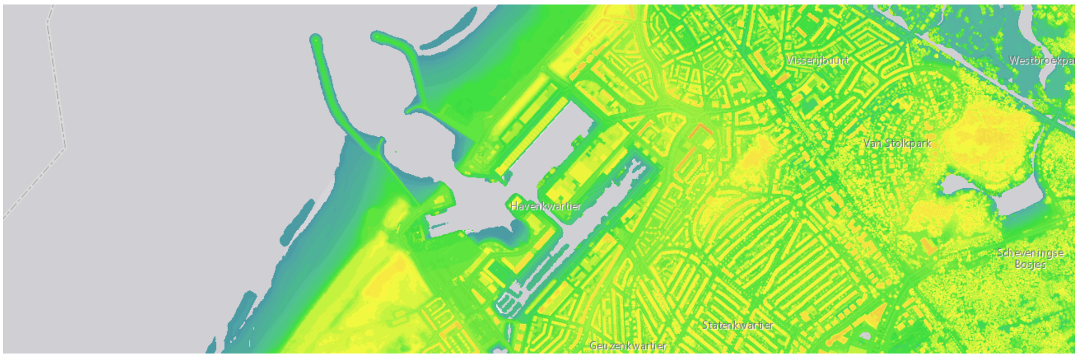


Figure D.6: Surface elevations of the area of Scheveningen Haven. (https://www.ahn.nl/ahn-viewer)

After research, the small grid data turned out to be inaccessible, so the AHN map was used in order to select different coordinates after which interpolation is performed. The height of the quay wall can be computed by using Equation 8.1. The quay wall positioned on the Doctor Lelykade has a length of 770 meters. Figure D.7 shows the elevation profile of the Doctor Lelykade.

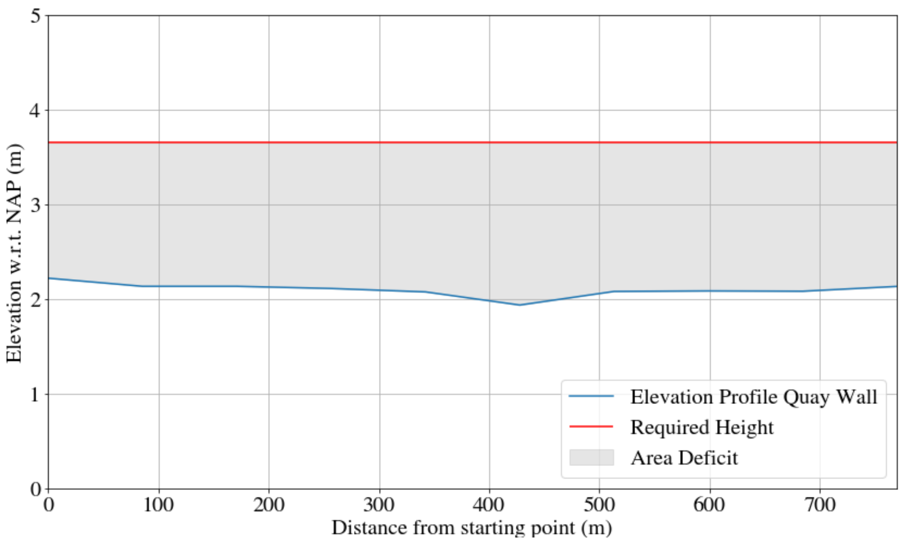


Figure D.7: Elevation profile of the quay wall shown in blue. The required height is shown in blue. The light grey area indicates the extra area necessary to comply with the new return period.

D.4. Appendix: Concept strength calculations

D.4.1. Concept 1

The sheet pile wall consists of multiple beams which are connected in the long axis. The forces on the sheet pile wall can therefore be approximated as if the wall were a beam. This simplifies the problem to a 2 dimensional one. The main forces which will act on the sheet pile are the following ones:

- Soil and water pressure from the landward side of the quay wall horizontally against the sheet pile wall.
- Water pressure which acts upon the sheet pile wall form the side of the port.
- The soil and water pressure from below the water body inside the port which also acts horizontally on the wall.
- Vertical pressure forces from potential machinery situated on top of the quay wall.
- The anchoring force on the quay wall. According to Blum (HS Manual p. 329) the anchor may be modelled as a support. The bottom of the wall may be modelled as a hinged support as well.

Figure D.8 indicates the manner in which the system can be schematized. This is only to give an impression and not absolute. It is however the first concept, but the figure aims at giving an impression regarding how the quay wall can be designed. Later on in the document, more details concerning this concept are given.

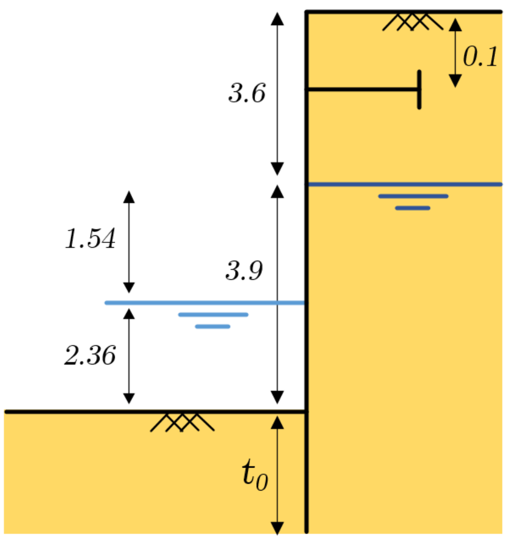


Figure D.8: The mechanical schematization of the sheet pile wall, and the manner in which the element is 'fixed' to the earth, but can however rotate.

The complex mechanics behind the soil structure interactions, means the course CTB2310 Soil Mechanics needs to be consulted in order to calculate the horizontal forces from the soil acting upon the sheet pile wall. The horizontal pressure forces upon the wall can be computed according to the following algorithm (Lecture 26):

1. Determine the vertical stresses σ_v using either the saturated or unsaturated weight.
2. Determine the pore water pressures p.
3. Determine the effective vertical stresses by subtracting the pore pressure from the vertical stresses: $\sigma_v' = \sigma_v - p$.
4. Determine the horizontal effective stresses by using the K_a, K_p method.
5. Determine the total horizontal stress by adding the pore water pressure.

The different pressure values $\sigma_{a,1}, \sigma_{a,2}$ and σ_p can be computed by using the algorithm given in CTB2310, which was specified before. The following definitions are therefore necessary:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi}, K_p = \frac{1+\sin\phi}{1-\sin\phi}$$

In the calculation of these ϕ is defined as the internal angle of friction. For soils like sand most often a value of 30 degrees is used. When this numerical value is substituted in the definitions, the active and passive coefficient can be calculated. Regarding the surcharge q, one has to take into account the activities/materials on top of the quay wall. When one utilizes Google Street View, it can clearly be seen that car traffic exists on top. According to the Quick Reference used at CITG, the load that corresponds to this situation equals 20 kilo pascals (Wagemans, 2004). The total soil pressures and the effective stresses can now be expressed in terms of the excavation depth t_0.

$$K_a = \frac{1}{3}, K_p =, q = 20\text{kPa}$$

$$\sigma_{a,1} = 20 + 3.6 \cdot 16 = 77.6kN/m^3 \rightarrow \sigma'_{a,1} = 77.6kN/m^3$$

$$\sigma_{a,2} = 77.6 + 20(3.9 + t_0) \rightarrow \sigma'_{a,2} = 77.6 + 20(3.9 + t_0) - 10(3.9 + t_0) = 116.6 - 10t_0$$

$$\sigma_p = 20t_0 \rightarrow \sigma'_p = 20t_0 - 10t_0 = 10t_0$$

The total horizontal stresses can then be found by multiplying by the respective K coefficients and by adding the pore pressure. Afterwards, the quay wall is balanced by active forces and the passive forces. The excavation must therefore be great enough in order for the passive area to build enough counter force. Now that all the forces are known in terms of t_0, t_0 can be calculated by computing the

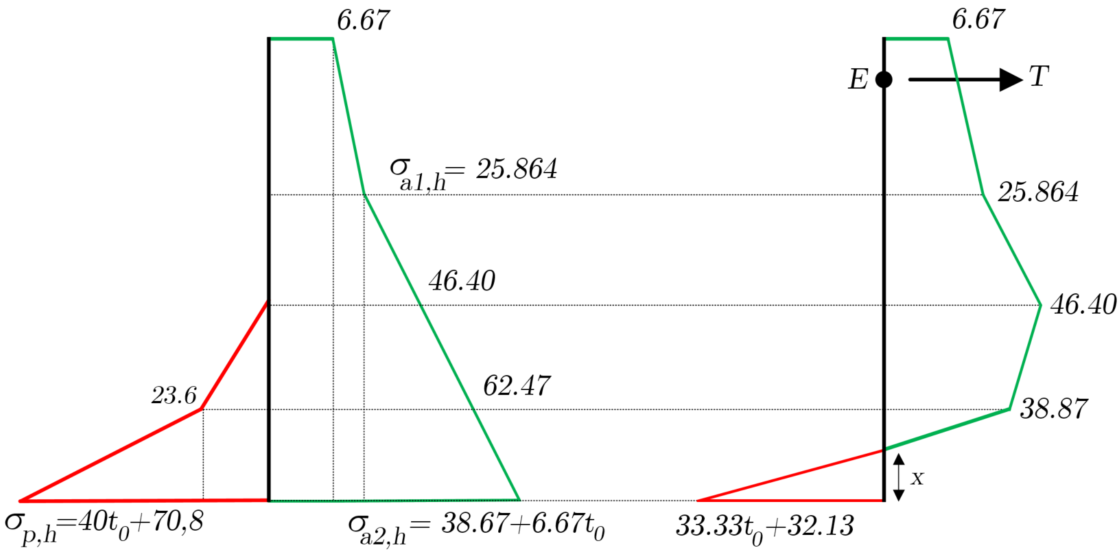


Figure D.9: The left image shows the total horizontal stresses on the sheet pile wall. These were used to compute the resulting forces on the wall. The left image shows the simplified force scheme where the pressures are summed up. The tension force is shown as well in the right image.

equilibrium of moments around the anchor point. Naturally, the perpendicular distances from the forces to the anchor must be computed. The the only unknown which has to be calculated is x, which can be computed by considering the fact that the force triangles corresponding to the dimension t_0 have similar geometry. The ratios between corresponding elements should thus be equal. One can then formulate the following equation:

$$\frac{33.33t_0 - 38.67}{33.064} = \frac{x}{t_0 - x}$$

$$(33.33t_0 - 38.67)(t_0 - x) = 33.064x$$

$$x = \frac{33.33t_0^2 - 38.67t_0}{33.33t_0 - 5.606}$$

$$x = \frac{15t_0(1111t_0 - 1289)}{16665t_0 - 2803}$$

Important is that for triangular distributed loads the line of gravity of the force is situated at one third from the base of the triangle. After computing the distance t_0 which results in rotational equilibrium, the horizontal force balance can be utilised in order to compute the force in the anchor. The equilibrium of moments can be formulated as follows:

$$+\sum M_E = 0 \tag{D.4}$$

$$-24.012 \cdot 1.7 - 34.5 \cdot 2.3 - 100.87 \cdot 5.45 - 60.06 \cdot 6.1 - 78.03 \cdot 7.68 - 27.85 \cdot 8.19$$

$$-(0.5(t_0 - x)33.064)(9.86 + 0.33(t_0 - x))$$

$$+((33.33t_0 - 38.67)0.5x \cdot (9.86 + (t_0 - x) + \frac{2}{3}x) = 0$$

Solving for the embedded depth t_0 results in:

$$t_0 = 2.6\text{m}$$

Now that the vertical dimension of the sheet pile wall is known, the horizontal force balance can be applied in order to calculate the required tension force in the anchor.

$$\sum F_h = 0$$

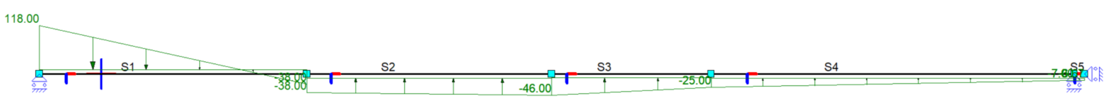
$$-24.102 - 34.5492 - 100.87 - 60.06 - 78.03 - 27.848$$

$$-0.5(t_0 - x)33.064 + (33.33t_0 - 38.67) \cdot \frac{1}{2}x + T = 0$$

$$T = 108\text{kN/m}$$

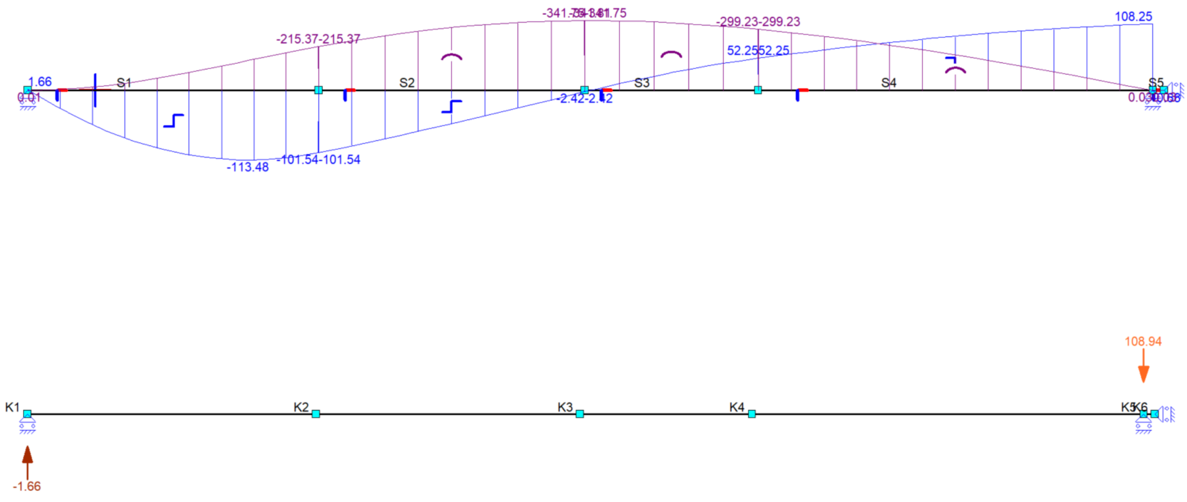
All forces acting upon the sheet pile wall are now known. To determine the greatest bending moment in the wall (to dimension the profile of the sheet pile wall), the M-lines and V-lines need to be constructed. This can be done by 'cutting' the wall in segments and calculating the internal moments/forces. Aside that, from structural mechanics the following relationships are known:

$$\frac{dV}{dx} = -q_z, \frac{dM}{dx} = V \xrightarrow{substitute} \frac{d^2M}{dx^2} = -q_z$$



This maximum bending moment will be used to determine the required elastic section modulus of the wall. From structural mechanics the following equation is known:

$$\sigma_{max} = \frac{M_{z,max}}{W_z} \tag{D.5}$$



The equation can be used after one chooses an appropriate steel with a corresponding yield stress. This is often an iterative process, therefore the cheapest one will be used as the initial guess. This is an S235 profile, having a yield strength of 235 $\mathrm{N/mm2}$.

$$W_{z,req} = \frac{M_{z,max}}{\sigma_{max}} = \frac{341 \cdot 10^6}{235} = 1451063mm^3 = 1451cm^3/m$$

D.4.2. Concept 2

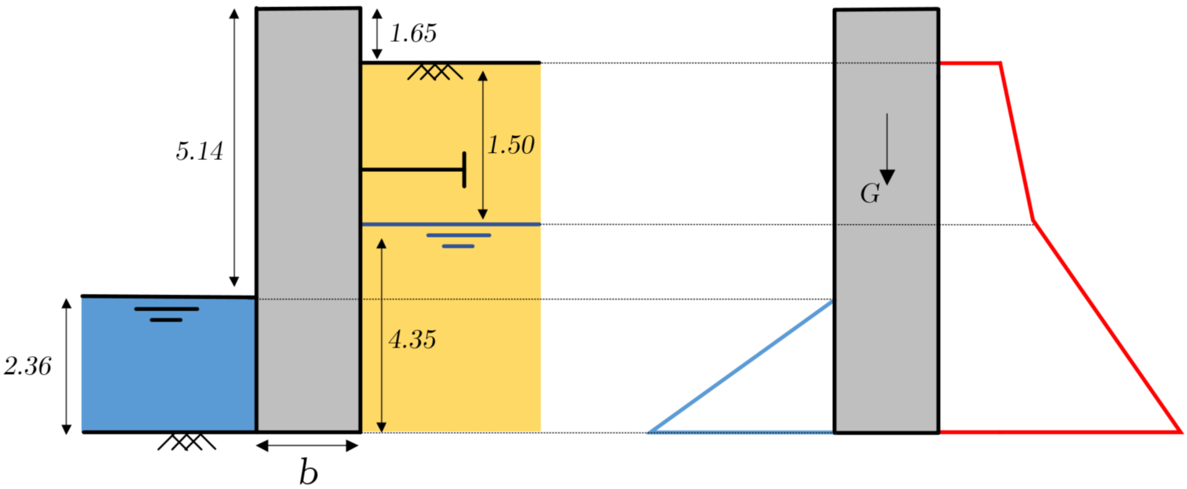


Figure D.10: A schematic overview of the situation in which the quay wall is extended using a gravity structure.

Figure D.10 shows the situation of the gravity structure, along with the not yet calculated soil pressures and forces. Following general structural mechanics and the Hydraulic Structures Manual, five criteria are relevant when considering a structure like this. The calculations were done in Google Spreadsheets, so the output will here consistently be represented using tables. The iterative process mainly lied in choosing an appropriate width b, which is shown as a parameter in Figure D.10. The width was initially chosen to be 3 meters, which turned out to be in conflict with the rotational stability requirement. The final width therefore became 4 meters.

Horizontal Stability

The first requirement appeals to the horizontal stability. From a structural/geotechnical perspective this implies that the friction force between the structure and the soil needs to be greater than the sum of the horizontal forces on the structure.

$$\sum H < f \sum V \tag{D.6}$$

H and V are defined as the horizontal and vertical forces respectively. They are calculated based on the weight of the structure and the soil/water pressures acting from the side. Note that most of the mechanical values are given in terms of unit depth, but when this contradicted certain requirements, the structures were assumed to have a depth of 10 meters. Table D.1 shows the general characteristics of the table.

Characteristics of the structure		
Width b	4.0	meter
Height h	7.5	meter
Density ρ	2500.0	kg/m3
Volume (per unit depth)	30	m3/meter
Mass (per unit depth)	75000	kg/meter
Depth of one unit	10.0	meter
Volume of one unit	300	m3
Mass of one unit	750000	kilograms
Weight of one unit	7357500	N
	7357.5	kN
Weight per per unit depth	735.75	kN/m

Table D.1: Characteristics of the table

The right side of the structure again is defined as the active soil pressure side. This is correct because in the current configuration the structure will tend to rotate anti-clockwise.Table D.2 shows the relevant numerical values for the environment.

The characteristics of the environment can then be used to compute the lateral forces and moments on the structure. Table D.3 indicates these values.

Everything is now known in order to monitor whether the structure satisfies the first requirement, using Equation D.6. Because the output of the equation (which is now turned to a fraction) is smaller than one, the right hand side of Equation D.6 is greater than the left hand side. the first requirement is therefore met.

$$\frac{\sum H}{f \cdot \sum V} = \frac{1788.102}{0.5 \cdot 735.75} = 0.48$$

Rotational Stability

The second requirement enforces that the structure does not rotate/tilt due to high moments. This is done by ensuring that the work line of the gravity coincides with the core of the structure, which is easily defined here given the rectangular form (for now). The requirement in mathematical form is obtained from the Hydraulic Structures Manual:

Characteristics environment		
Depth water left	2.36	meter
Depth water right	4.35	meter
Depth dry soil right	1.5	meter
Density water p	10.0	kN/m3
Density dry soil ρ_d	16.0	kN/m3
Density wet soil ρ_w	20.0	kN/m3
Surcharge quay wall q	20.0	kN/m3
Active soil coefficient K_a	0.33	-
Passive soil coefficient K_p	3.0	-
Friction coefficient f	0.5	-
C_b piping	12.0	-

Table D.2: Characteristics of the environment

Soil pressures and forces			
water pressure left under		23.6	kN/m2
Total pressure right mid		44	kN/m2
Total pressure right under		131	kN/m2
Eff. vert. pressure right under		87.5	kN/m2
Eff. hor. pressure right under		29.1	kN/m2
Total hor. pressure right up		6.66	kN/m2
Total hor. pressure right mid		14.6	kN/m2
Total hor. pressure right under		72.6	kN/m2
Total hor. pressure left under		23.6	kN/m2
Total hor. force left		27.8	kN/m
	on a unit	278.5	kN
Hor. force right up		-16	kN/m
Hor. kracht right down		-189.9	kN/m

Table D.3: Soil pressures and forces on the structures

$$e_R = \frac{\sum M}{\sum V} \leq \frac{1}{6} b \tag{D.7}$$

The newly introduced variable e_R is defined as the eccentricity. Geometrically it is the perpendicular distance between the resulting vertical force upon the structure and the vertical through the center of gravity. One can see the small difference between the two, proving why the initial value for b of 3 meters was not sufficient.

$$e_R = \frac{461.88}{735.75} = 0.62m$$

$$\frac{1}{6} b = 0.67m$$

$$e_R < \frac{1}{6} b$$

Vertical Stability
The third requirement tests whether the soil can resist the vertical forces. Important here is that the friction between the soil particles suffices and that there is no tension, as soil can not deliver this kind of force. This implies that there is a maximum force (soil strength) and a minimum force (no tension). Both can be formulated respectively as the Hydraulic Structures Manual states:

$$\sigma_{k,max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{bl} + \frac{\sum M}{\frac{1}{6} l b^2} \tag{D.8}$$

$$\sigma_{k,min} = \frac{F}{A} - \frac{M}{W} = \frac{\sum V}{bl} - \frac{\sum M}{\frac{1}{6} l b^2} \tag{D.9}$$

The maximum bearing capacity can be determined following the theory of Brinch Hansen, which results in a maximum bearing capacity of 400 kPa. The minimum forcing must be 0 kPa, as this implies no tension. When the values are substituted into the equations, one obtains the following acting pressures (again a unit depth of 10 meter was chosen). The requirements are thus met.

$$\sigma_{k,max} = \frac{7357.5}{4 * 10} + \frac{|-461.88|}{\frac{1}{6} \cdot 10 \cdot 4^2} = 201.25kPa < 400kPa$$

$$\sigma_{k,min} = \frac{7357.5}{4 * 10} - \frac{|-461.88|}{\frac{1}{6} \cdot 10 \cdot 4^2} = 166.62kPa > 0kPa$$

Piping (internal backward erosion)
The phenomena piping may cause the soil under the structure to collapse because of the head difference around the gravity structure. This can occur when the soil underneath the structure is permeable and when the waterway is shorter than the lengths defined by Bligh or Lane (Hydraulic Structures Manual). Bligh defined the following requirement:

$$L \geq \gamma \cdot C_B \cdot \Delta H \tag{D.10}$$

The constant γ herein is defined as the safety factor, which is equal to 1.5 according to the Hydraulic Structures Manual. C_B here is defined as the soil constant and ΔH is defined as the head difference.This leads to a required waterway of approximately 36 meters, which is significantly larger than the current 4 meter.This would require the construction of one or multiple sheet piles in order to increase this length L by 32 meters. One sheet pile would require a length of 16 meter whereas the application of two piles would requires lengths of 8 meter. This is quite large. Options would be to increase the width of the

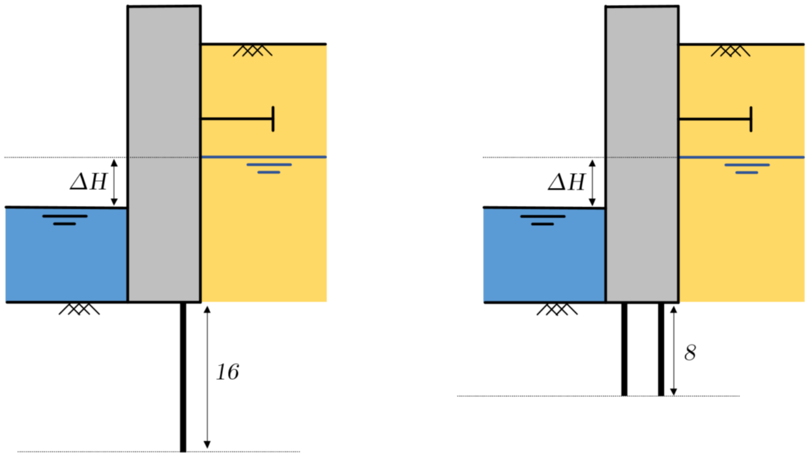


Figure D.11: Possible measures to ensure that piping would not occur in the situation in which a gravity structure is constructed.

structure, but this results in higher costs and less basin size capacity. These required are quite large so this is a disadvantage of this system. Figure D.11 shows how these piles can be applied.
Again, the Matrixframe software was used to compute the internal forces in the structure. Especially the bending moments in the bottom of the structure might be great implying the essence of steel reinforcement. Figure D.12 shows the output of the software.
The Hydraulic Structures Manual states the following two expressions for the design value of the compressive strength and the tensile strength respectively (these values are also given in the manual as a function of the concrete class in Table 35-1):

$$f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_C} \tag{D.11}$$

$$f_{ctd} = \frac{\alpha_{ct} \cdot f_{ctk,0.05}}{\gamma_C} \tag{D.12}$$

This may imply (when strength of the concrete itself is not sufficient) that the cross section would need reinforcement, this will however only be calculated if the concept is chosen as the final design.

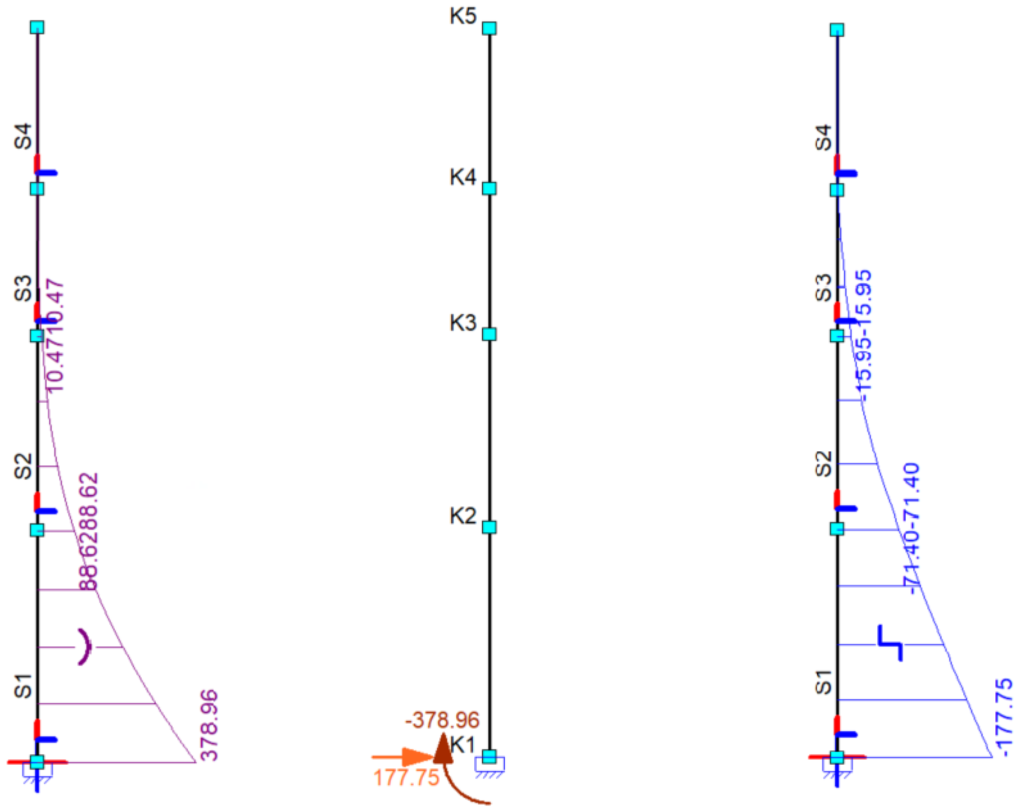


Figure D.12: The the different internal force diagrams for Concept 2. The image to the left shows the moment line, the middle one shows the reaction forces, and the right image shows the shear force line.

D.4.3. Concept 3

The way in which the system is structurally schematized is important in the design of the structural elements, one must also closely consider the forces which are to be taken along. The quay wall itself will be assumed to be loaded vertically by q_s because of human operations and weight. The quay wall will be considered under the influence of a wind load q_w which causes the column to bend clockwise. The water level in the port will be assumed at Lowest Astronomical Tide so that wind can also act on the compression pile itself with the greatest distance. The wind acting as a force upon the wall is converted as a bending moment upon the compression pile. Figure D.13 shows an overview of this mechanical scheme.

The water pressure does not have to be taken into account when considering the bending moments in the pile, as it works fully around the pile. Therefore both sides compensate for one another. It does however deliver a compressive force in the horizontal direction, for which the pile will be checked. At first, the bearing capacity of the pile will be determined. Using Dinoloket, different Conus Penetration Tests (CPTs) can be requested as data which can be read using software made available on their website as well. Figure D.14 shows the different locations where CPTs were done.

BRO delivers more informaton, therefore that specific type of data is preferred. Regarding the fact that the compression pile would be situated in the water, therefore the CPT ending on 35 will be used.

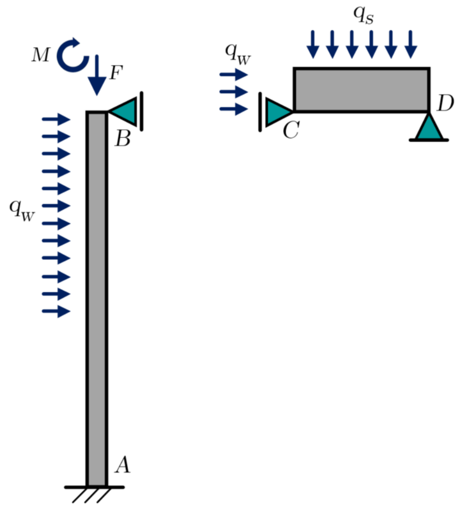


Figure D.13: Schematic mechanical overview of the situation in which the new quay wall rests on top of a compression pile.

As mentioned before, the visualizations of the tests can only be done in the software from Deltares and Dinoloket. This results in only visual representations of the CPTs, this is unfortunate because now no programming software can be used in order to compute the values corresponding to the Koppejan method. The maximum bearing capacity of the soil-pile system can be formulated using the following expression according to the Hydraulic Structures Manual.

$$F_{r;max} = F_{r;max;tip} + F_{r;max;shaft} - F_{r;max;nk} \tag{D.13}$$

Where:

$$F_{r;max;tip} = A_{tip} \cdot p_{r;max;tip}$$

and:

$$F_{r;max;shaft} = O_{p;avg} \int_0^{\Delta L} p_{r;max;shaft} dz$$

The three terms correspond to the force in the tip, in the shaft and negative friction force, which actually only needs to be taken into account in the serviceability limit state. The maximum tip resistance $p_{r;max;tip}$ can be found according to the Koppejan method which was also adopted in the Dutch Standard NEN 6743. The way in which the q_{avg} values are defined can be found either in the manual or in the NEN documents.

$$p_{r;max;tip} = \frac{1}{2}\alpha_p \beta s \left(\frac{q_{c;I;avg} + q_{c;II;avg}}{2} + q_{c;III:avg} \right) \tag{D.14}$$

$$p_{r;max;shaft} = \alpha_s \cdot q_{c;z;a} \tag{D.15}$$

The way in which the column will be designed will go according to the following process:

1. At first the loads upon the column will be defined. This will be done per meter depth at first.
2. The column will then modelled in MatrixFrame in order to calculate internal forces and the support forces.
3. The vertical reaction in point A is the force which needs to be delivered by the pile soil system. This force will then be taken as the threshold for the bearing capacity.

The modelling of the forces will go from top till bottom, as is usual in structural engineering. No probabilistic method is used till this point, therefore a semi-probabilistic method will be used, which will include the utilisation of safety factors.
An assumption is that the plateau will not be the failure mechanism, because this was assumed to be associated with the compression pile. The plateau will therefore not be designed on the structural

Figure D.14: Available conus penetration tests locations. One can see that there are multiple which might be representative for the potential location for the compression pile. Important is that data starting with 'S30' was made available by DINO, while data starting with CPT was made available by BRO.

scale (yet), as this is also a concept. What is relevant for the column, is that half of the weight and the surcharge of the quay wall is transferred to the compression pile as an eccentric force. This causes a compression force and a bending moment at the top of the wall.

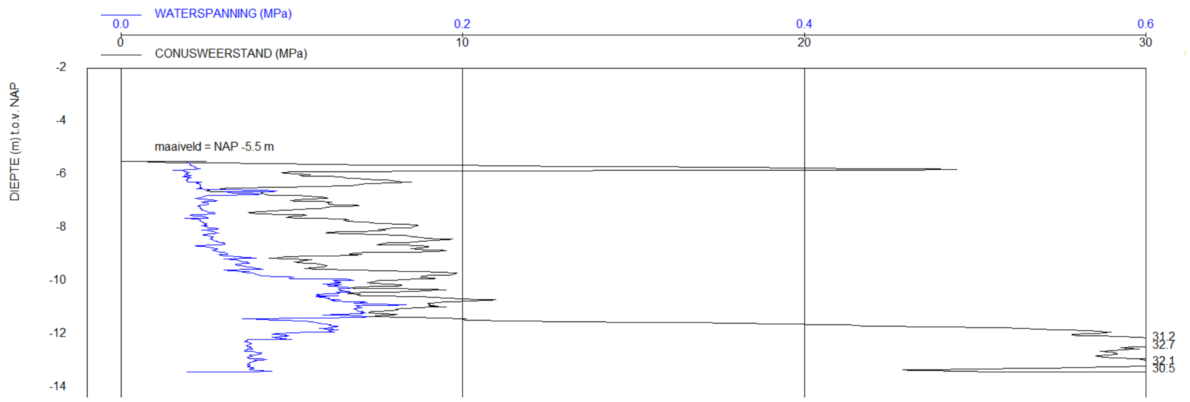


Figure D.15: CPT done at the specified location.

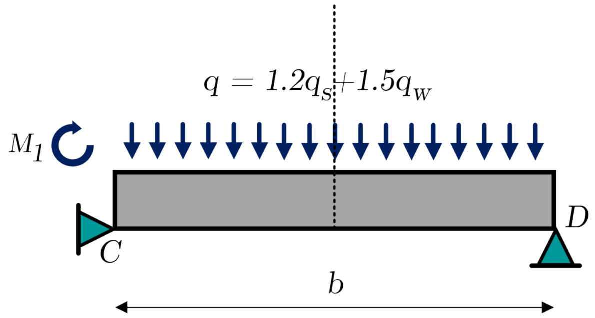


Figure D.16: Overview of the forces acting upon the plateau.

The loads were substituted in Matrixframe and the linear elastic calculation was performed. The obtained diagrams are shown in Figure Please note that the self weight of the compression pile has not yet been taken into account as this is a function of the geometric properties of it. This makes the design also an iterative process, because the self weight does act upon the soil pile system. Table D.4 shows the values which were used as input for the Matrixframe model.
The bending moment upon the column, is quite large. This generally implies the essence of steel reinforcement within the cross section. This will however only be designed in the final stage when the concept is chosen. The bearing capacity however is a boundary condition so this characteristic will already be analyzed.

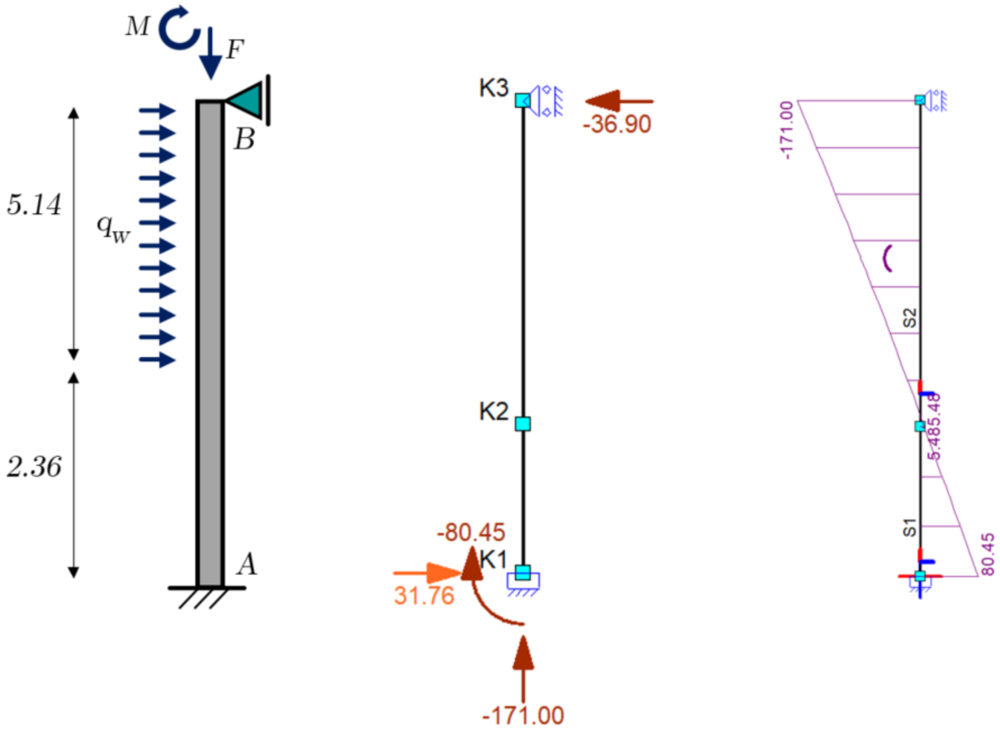


Figure D.17: Overview of the forces acting upon the plateau.

Material Characteristics			
Width b plateau	4.0	m	
Height h height	1.65	m	
Surcharge	20.0	kN/m2	
Density Concrete γ_c	25.0	kN/m3	
Total Load	85.875	kN/m2	
Permanent Loads Safety Factor	1.5	-	
Variable Loads Safety Factor	1.2	-	
Lateral Wind load on the plateau			
q_z	1.0	kN/m2	
F_z	1.65	kN/m	
M_z	1.36	kNm/m	
Loads on the compression pile			
Compression force	171.75	kN/m	per meter plateau
Bending moment (eccentricity)	171.75	kNm/m	per meter plateau
q_z	1.0	kN/m2	

Table D.4: Input values for the calculation of the compression pile.

When using the Koppejan method, two parameters need to be known beforehand. These are the pile diameter and the elevation level of the bottom of the pile. This again causes the design to be an iterative process. Figure D.15 will again be used in order to compute bearing strength capacity. As mentioned before, no data set was obtained which means that the visual method must be applied. Two initials values will be used for the elevation height and the diameter:

$$D_{eq} = 0.3m, h_{bottom} = -8meter$$

Figure D.18 shows the results of a successful integration, followed by the computations in Table D.5.

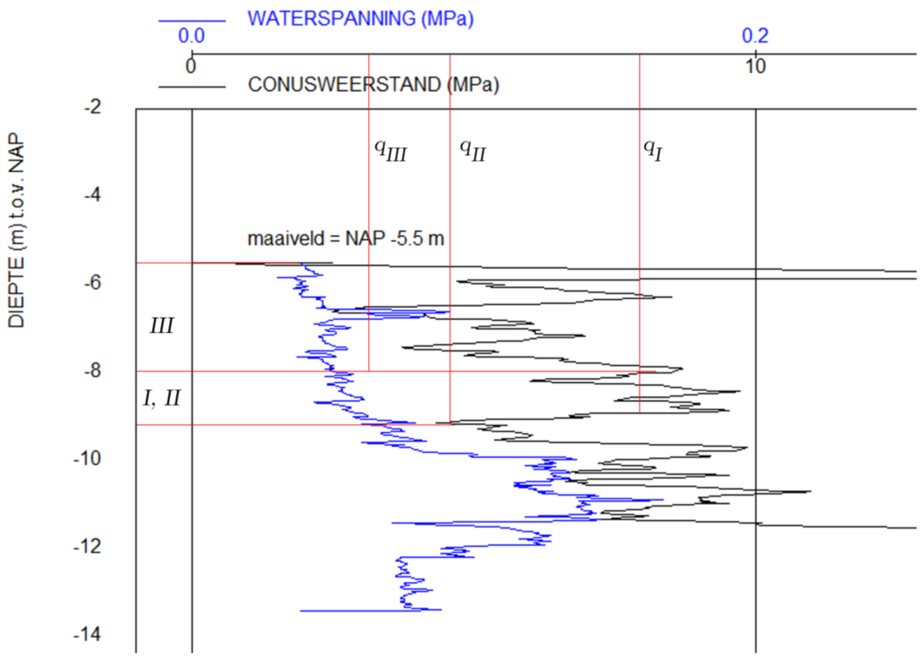


Figure D.18: The Koppejan method visualized in the CPT figure.

Maximum Pile Resistance					
α_p	0.70	-	$p_{rmaxtip}$	**1796.6**	**kN/m2**
β	0.80	-	α_s	0.01	-
s	0.70	-	$p_{rmaxshaft}$	0.050	MPa
Diameter	0.30	m		50	kN/m2
Elevation Height Tip	-8.0	m			
Elevation Bottom of section 1	-9.2	m	F_{rmax}	**130.98**	**kN**
Elevation Top of section 3	-5.6	m	F_{rshaft}	**223.5**	**kN**
q_1	7.833	MPa			
	7833	kN/m2	F_{rmax}	**354.5**	**kN**
q_2	4.5	MPa	W_{pile}	**26**	**kN**
	4500	kN/m2			
q_3	3.0	MPa			
	3000	kN/m2			

Table D.5: Values generated with the Koppejan method

$$F_{r,max} = 354 > 171 + 26 = 197kN$$

This implies that the compression pile will fail due to a bearing capacity deficit. Whether the pile needs compression will be computed when the concept is chosen as the final design. This also holds for the buckling capacity.

D.5. Costs Quay Wall

An important aspect in choosing in what manner to increase the height of the wall, is by considering the costs per method. In this specific concept, only a new quay wall is built in front of the current existing one. A relationship exists between the retaining height of a quay wall and the total costs of it (Gijt, 2011). Figure D.19 shows a scatter plot of different quay wall project and the corresponding costs.

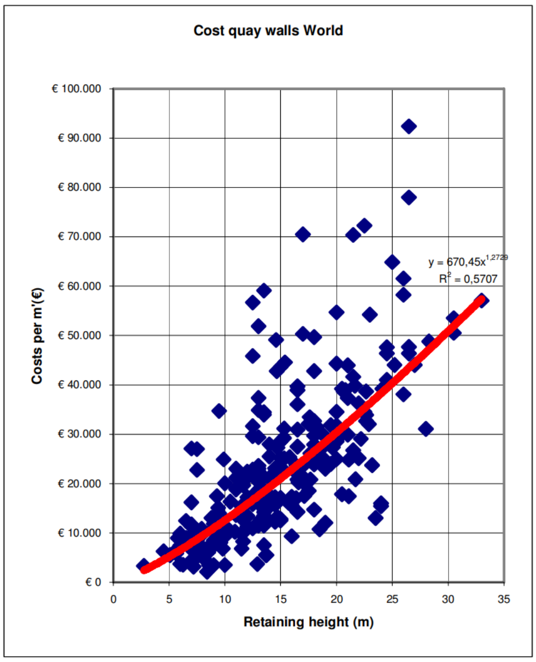


Figure D.19: A relationship between the retaining height of a quay wall and the costs to construct that specific wall (Cost of quay walls including life cycle aspects, 2018).

Hence, the costs of the new quay wall can be described be applying equation D.16.

$$C_{total}(h) = 670.45h^{1.2729} \tag{D.16}$$

In the equation, C_{total} stands for the total costs per meter wall, whereas h represents the retaining height. The two constants were derived using regression fitting. The costs can be estimated as follows:

$$C_{total}(7) \approx 670.457^{1.2729} = 7981 \text{ euros per meter}$$

$$C_{total} = 7981 \cdot 770 = 6.15 \text{ million euros}$$

E

Breakwater design

In this appendix the analysis of the design storm for the breakwater design will be discussed. The dataset was obtained from Rijskwaterstaat and consists of 21 years of wave and water level data with intervals of 10 minutes. Significant wave heights measurements are from the EuroPlatform which is located to the South of Scheveningen. The water heights are from a measurement location inside the Scheveningen harbour. In addition, wind data was obtained from measurement locations at Hoek van Holland and IJmuiden.

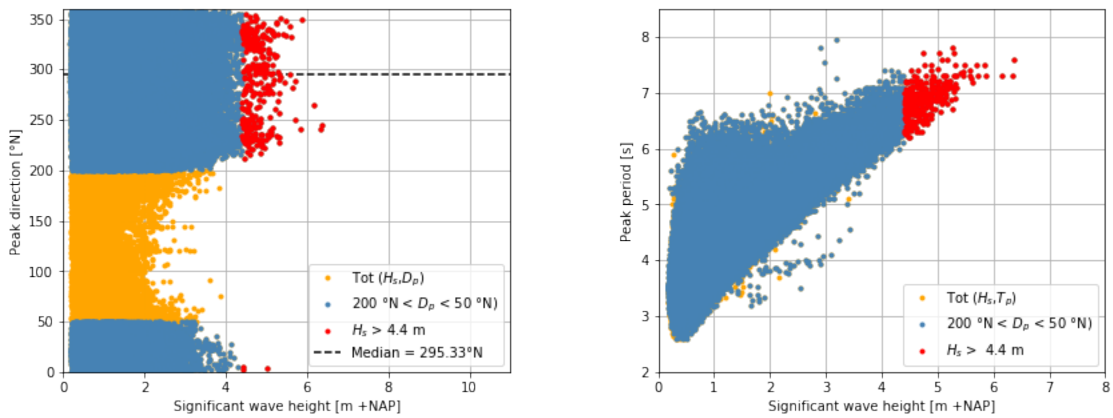
E.1. Extreme value analysis for storm surge and significant wave height

First of all, the raw data of the significant wave height and water levels were imported using python and data cleaning was performed. The data should be in the right form with the data as index such that the data can be filtered correctly. The raw data had multiple duplicate time measurements and missing values which were removed using the Pandas library.

E.1.1. Filtering the wave dataset

To do an extreme value analysis, it is very important to have dataset of homogeneous and independent storm events. This means that the data should originate from similar meteorological events. To obtain this, the dataset should be filtered first.

To filter the wave height measurements, the wave climate at the location should be examined. Figure E.1 (a) gives a plot of the significant against wave direction. It can be seen that the largest waves come from a direction 200 °N to 50 °N. Outside this range, the large waves are not present. Since we are interested in the extreme significant wave heights, we filter the dataset such that only waves from the dominant direction are present corresponding to waves from the North-West. In addition to the filter based on the dominant wave direction, we can also investigate whether the waves are wind waves or swell waves. Figure E.1 (b) gives a plot of the significant wave height against the peak period. Swell waves generally have a long period and low wave height and will thus show up in the lower right hand side of Figure E.1 (b). It can be seen that this region has almost no data points which means that swell waves are likely not present at the measurement location. The next step is to choose a threshold value and select only the waves with significant wave height larger than this threshold. The threshold value should be high enough such that only the largest storms are selected, however not too high since it is better to have more data points in the filtered data set for fitting an extreme value distribution which will be done later. The threshold that was chosen for the given data was $H_{threshold} = 4.4\ m.$ The waves with significant wave height larger than the threshold are indicated in red.



(a) (b)

Figure E.1: Plots of (a) peak direction against significant wave height and (b) peak period against significant wave height. The median of the dominant wave direction (from 200 °N to 50 °N) is indicated with a dashed line, dominant D_p = 295.33 °N.

Another way to see if we are dealing with swell or wind waves by calculating the deep water wave steepness H_0/L_0 for the waves with height larger than the threshold. Here, the deep water wavelength follows from the dispersion relation. Wind sea waves are generally steeper than swell waves. Figure E.2 shows the wave steepness against significant wave height. Swell waves generally have steepness $H_0/L_0 < 0.025$ and wind sea waves $H_0/L_0 > 0.025$. It can be seen that the largest waves all have $H_0/L_0 > 0.025$ and thus are wind waves. It can thus be concluded that it is now a homogeneous dataset.

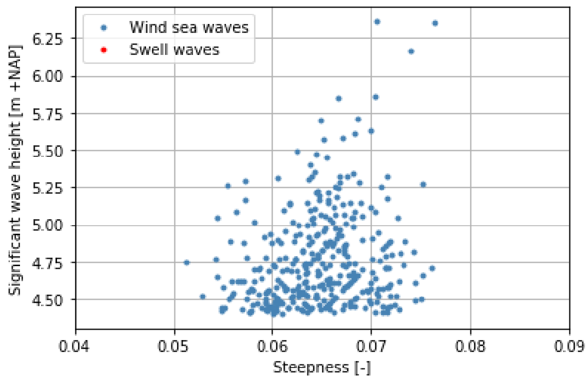


Figure E.2: Filtering the data set based on the waves steepness H_0/L_0.

As stated before, a threshold of $h_{threshold} = 4.4\ m$ was chosen to filter the dataset and get storm observations. This method is called peak-over-threshold (PoT). The number of storm per year N_s is an important indicator. The dataset with storm observations should be large enough to get a good fit for the extreme value distribution. As a rule of thumb, one should choose a value $2 < N_s < 10$. Using the threshold $H_{threshold} = 4.4\ m$, 70 storm observations are present in the 21 years of data. The peaks were selected such that only one peak can occur in 3 days. This prevents getting more than one observation from the same storm and thus of the same meteorological event. The chosen threshold results in the number of storm per year of $N_s = 3.33$ which is a good result. Figure E.3 gives a plot of the wave height peaks above the selected threshold.

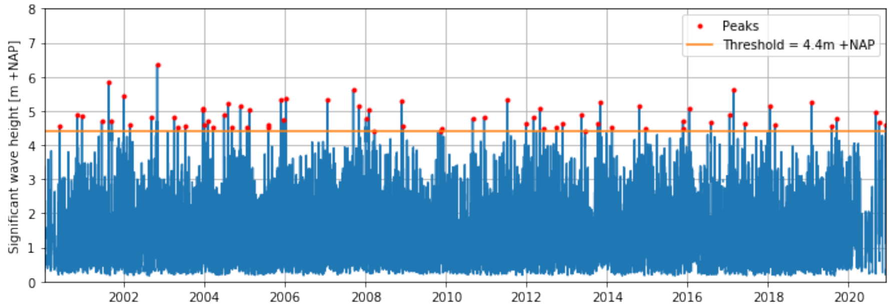


Figure E.3: Peak-over-threshold, using a threshold of level $4.4\ m$. This gives $N_s = 3.33$.

To obtain storm surges, the water level measurements should be filtered such that only the storm surge of independent storms are present. Firstly the tidal elevations should be subtracted from the water level measurements. At the same measuring location in the Scheveningen harbour also the tidal elevation is predicted and can be obtained in the same raw data file. To obtain storm surges, a threshold $h_{threshold}$ should be chosen simmilarly to the threshold for the significant wave height. Using the same rule of thumb, a threshold of $h_{threshold} = 1.4\ m$ was chosen which results in 90 storm surges and $N_s = 4.29$. The storm surge peaks above the selected threshold presented in Figure E.4. It can be seen that the largest storm surge occurred at the 6th of December in 2013. This was during the Cyclone Xaver (Sinterklaasstorm in Dutch) that caused multiple floodings in northern Europe. The storm surge occurred simultaneously with spring tide, resulting in even higher water levels.

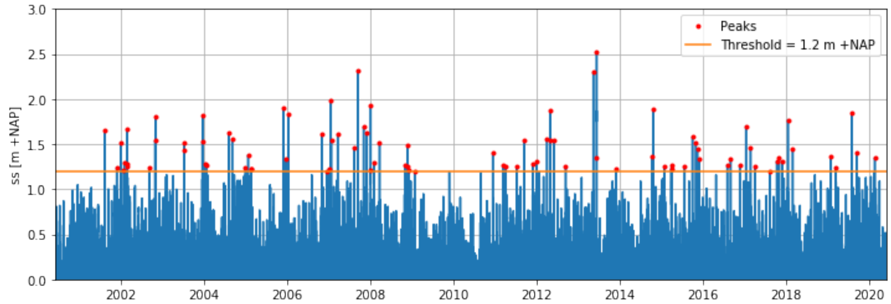


Figure E.4: Peak-over-threshold, using a threshold of level $1.2\ m$ + NAP. This gives $N_s = 4.29$.

E.1.2. Extreme value distribution fit

After filtering the dataset and obtaining the storm observations, we can proceed to fit the extreme value distribution. For the peak over threshold method, it can be shown that the extremes follow the Generalized Pareto Distribution (GPD). To fit the GPD to the extreme wave heights and storm surges, the data should first be ranked from smallest to highest values. The probability of exceedance P_i and probability of non-exceedance Q_i can then be calculated based on the rank numbers $i = 1, 2, .., N$.

$$P_i = \frac{i}{N+1} \tag{E.1}$$

$$Q_i = 1 - P_i \tag{E.2}$$

The return period of datapoint i can now be calculated using the number of storms per year N_s.

$$R_i = \frac{1}{Q_i \cdot N_s} \tag{E.3}$$

Fitting the GPD was done in python using the scipy package. For both the storm surge and wave height, the distribution is evaluated for return periods up to $R = 1000$ years. Figure E.5 gives the fitted GPD distribution for the significant wave heights with the observations. It can be seen that the significant wave height corresponding to the 1000 year return period is approximately $H_{m0} = 7.1\ m$.

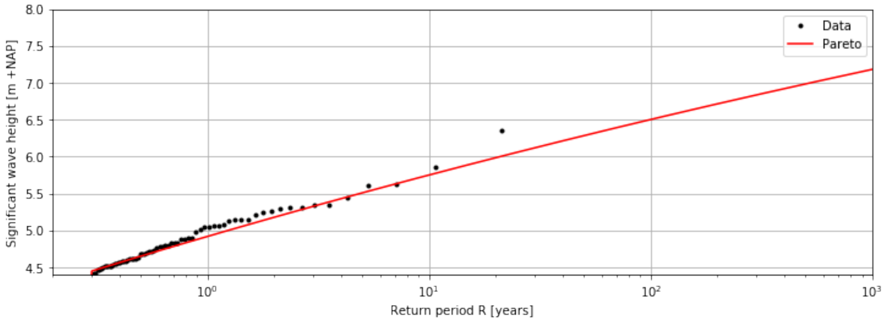


Figure E.5: Observed storm wave heights and corresponding return periods together with the fitted extreme value distributions.

Figure E.6 presents the fitted GPD for the storm surge in the Scheveningen harbour. From the plot it can be seen that the storm surge with 1000 year return period is approximately $SS = 2.9\ m$.

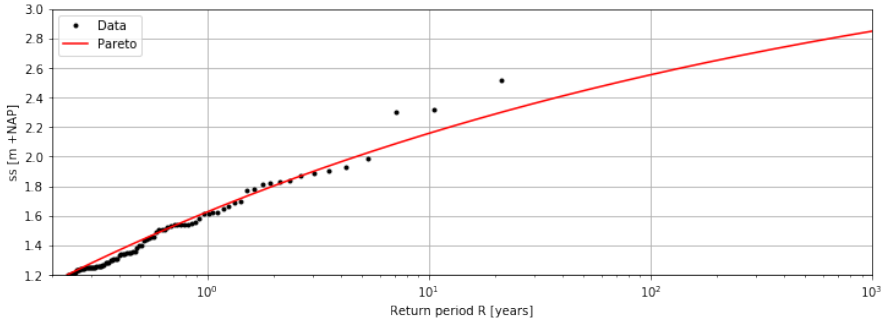


Figure E.6: Observed storm surges and corresponding return periods together with the fitted extreme value distributions.

In Table E.1 the wave height and periods for various return periods are given. These return periods were chosen for the design criterion for different failure modes. The wave period is estimated by fitting a curve through the scatter plot as given in Figure E.7. In this way, the wave periods corresponding to the significant wave heights can be computed. The period corresponding to the 1000 year wave height of $H_{m0} = 7.14\ m$ then becomes $T = 8.41\ s$.

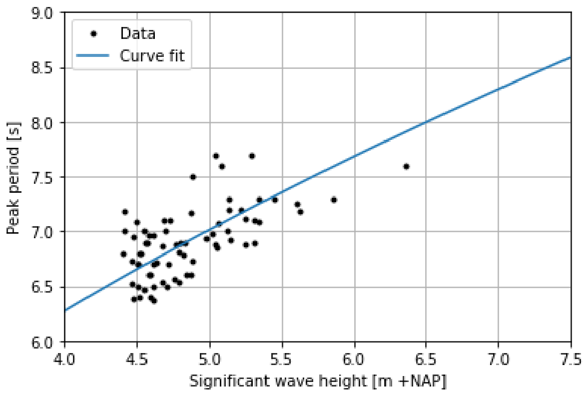


Figure E.7: Correlation between wave period and wave height.

R [yr]	Significant waveheight [m]	Wave period [s]
1	4.88	6.96
500	6.95	8.29
800	7.08	8.36
1000	7.14	8.41

Table E.1: Significant waveheight and wave period for different return periods.

E.2. Tidal levels and sea-level rise

Tidal levels were estimated from water levels in the Scheveningen harbour, where the dataset again originates from Rijkswaterstaat. From the time series, the tidal levels were determined and are given in Table E.2.

Tidal Level	**Level [m, NAP]**
LAT	-1.04
MLWS	-0.79
MLWN	-0.62
MHWN	0.77
MHWS	1.48

Table E.2: Tidal levels

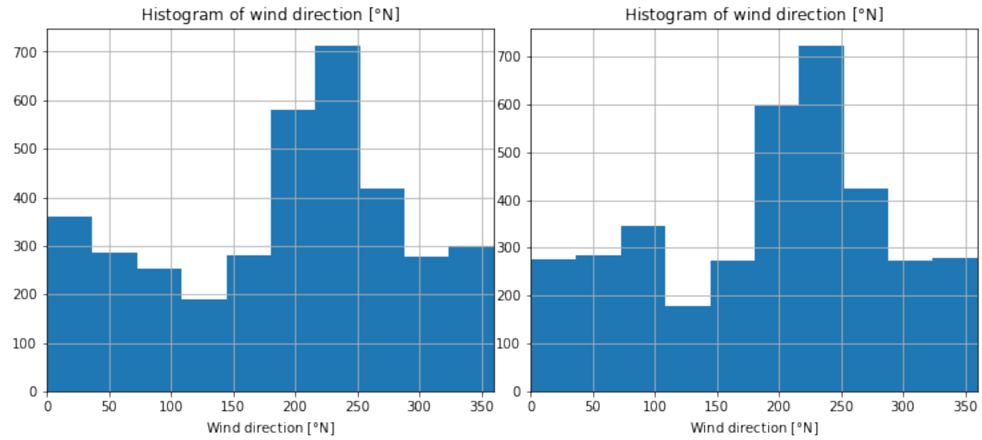
In Table E.3, the total water level at the toe of the structure is given for different return periods. Here SS is the storm surge as determined by the extreme value analysis in the previous section and the tidal level that is used is MHWS. For sea level rise, scenarios from Deltares were used as described in Appendix D. Here it was estimated that the sea level rise would be approximately 1 meter at the end of the design life of the structure. It should be noted that these predictions are highly uncertain.

R [yr]	SS [m, NAP]	Tidal Level [m, NAP]	SLR [m, NAP]	**Total [m, NAP]**
1	1.62	1.48	1.00	**4.10**
500	2.77	1.48	1.00	**5.25**
800	2.82	1.48	1.00	**5.30**
1000	2.85	1.48	1.00	**5.33**

Table E.3: Water Levels

E.3. Wind intensity and wind direction

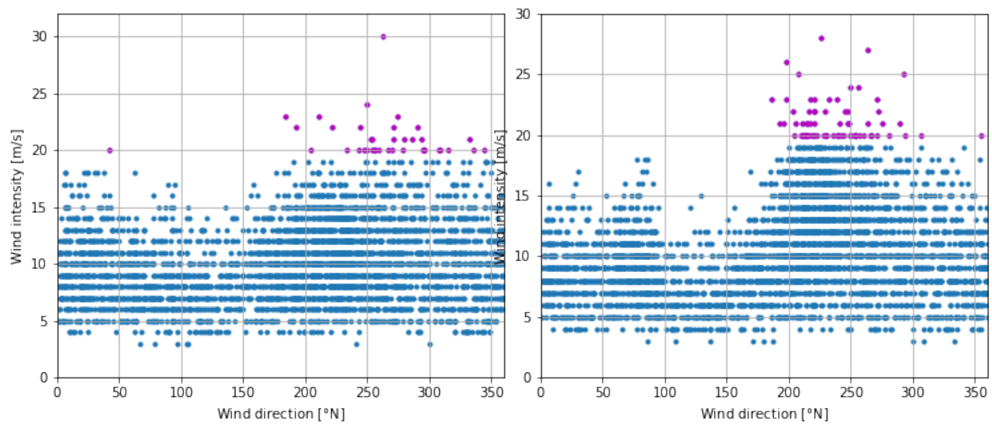
For the transformation of the offshore wave conditions to nearshore wave conditions, the wind intensity and wind direction should be given as input in SwanOne. For this reason, wind data obtained from Rijkswaterstaat was used. In Figure E.8, histograms of the wind directions are given for the locations Hoek van Holland and IJmuiden for measurements from 01-06-2010 up to 01-06-2020. From these plots, it can be seen that wind is most of the time comes from the North-West.



(a) Hoek van Holland (b) IJmuiden

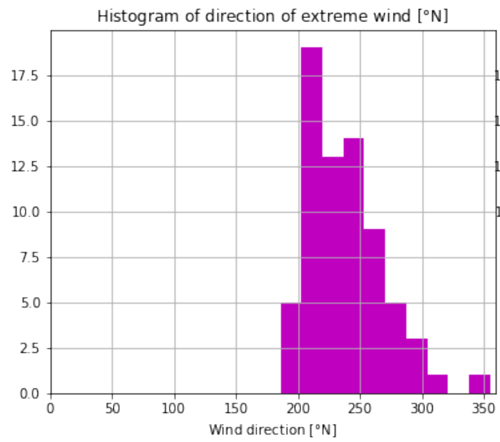
Figure E.8: Histograms of wind direction for a dataset with daily measurements from 01-06-2010 up to 01-06-2020.

Since we are mostly interested in storm conditions, we filter the dataset to include only wind intensities above a certain threshold. The threshold was chosen to be $20\ m/s$. Figure E.9 shows the wind intensity against wind direction. The datapoints with intensity above the threshold are indicated in purple. We are interested in the wind direction of the storms so a histogram of wind directions is given in Figure E.10. From this plot it can be seen that the storms are coming from the South-West.

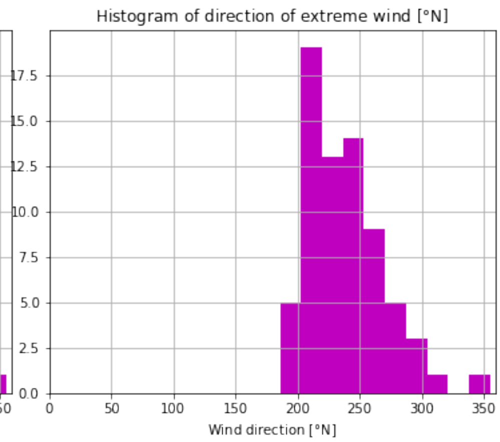


(a) Hoek van Holland (b) IJmuiden

Figure E.9: Scatterplots of wind direction vs wind intensity for a dataset with daily measurements from 01-06-2010 up to 01-06-2020. The extremes, with measurements of wind intensity above 19 m/s, are indicated purple.



(a) Hoek van Holland



(b) IJmuiden

Figure E.10: Histograms of wind direction of storms (daily average wind intensity above 19 m/s).

It was found that the extreme wind intensities have a mean of $21.29\ m/s$ for IJmuiden and $21.05\ m/s$ for Hoek van Holland. The average wind direction is $237.51°N$ for IJmuiden and $261.26°N$ for Hoek van Holland.

E.4. Nearshore wave conditions

For each failure mechanism and return period (for SLS and ULS) a different nearshore wave condition is normative. The offshore dataset that was analysed in previous chapter will be converted to nearshore conditions in this chapter. For this, the SwanOne software will be used. The offshore data at the Europlatform location is to the south of the site at Scheveningen. Using Navionics, a simplified bottom profile is made, which is presented in Figure E.11. At the breakwater section, the depth will be approximately 10 m. As discussed before, the design water level consist of MHWS, storm surge and sea level rise. The normative situation is near the end of the design life, since then the sea level rise will be largest.

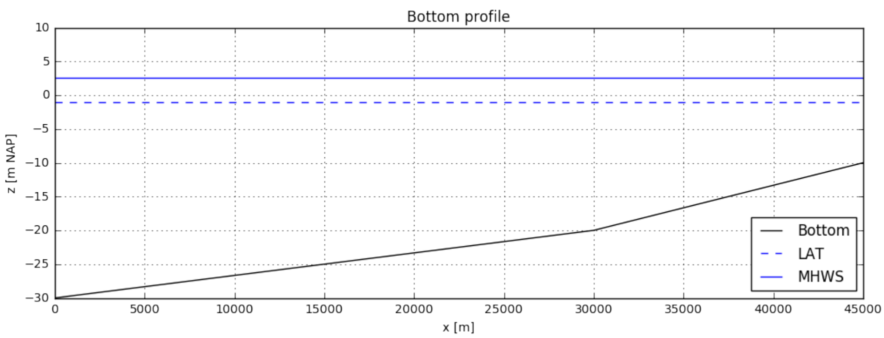
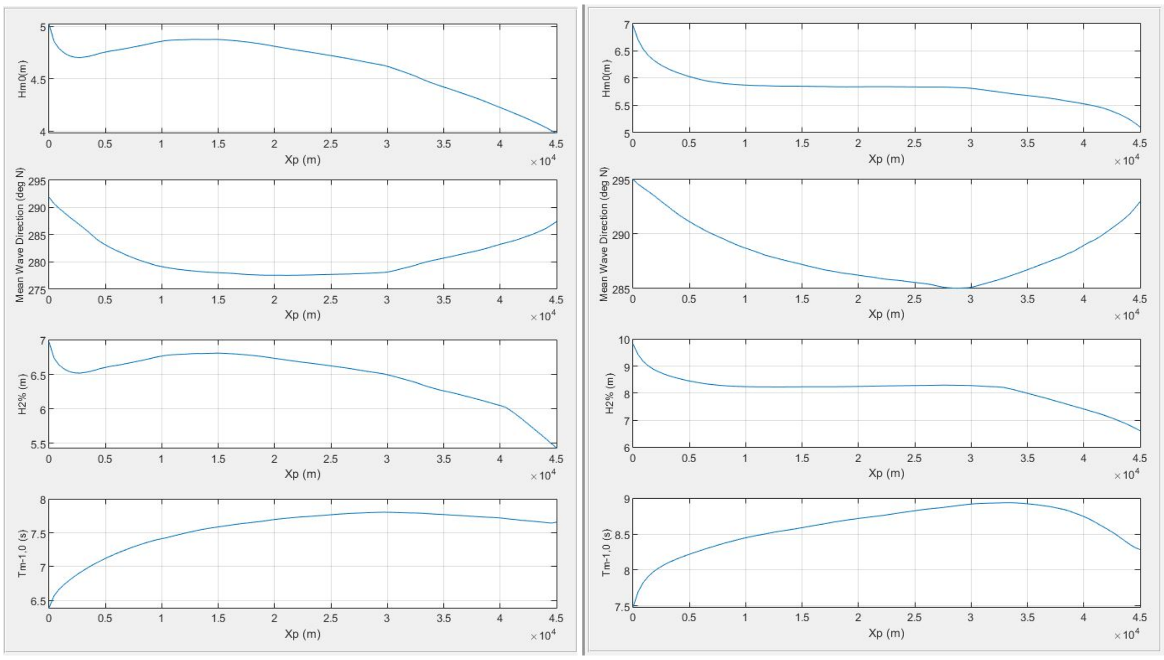


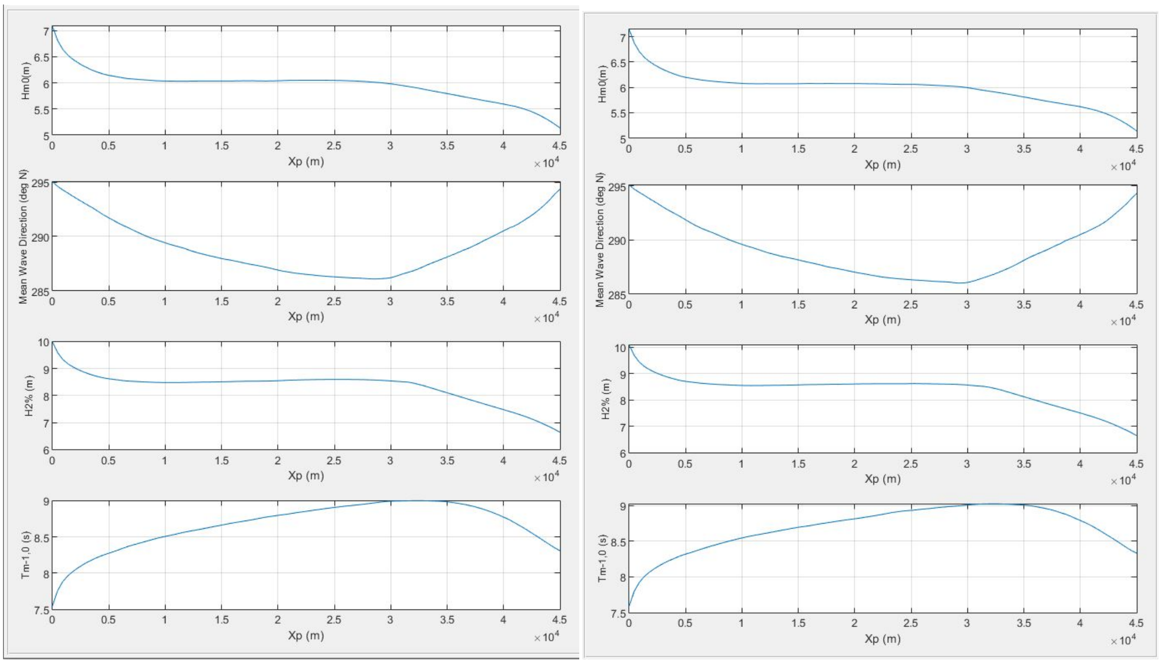
Figure E.11: Bottom profile.

Using the above bottom profile, with $\alpha = 135°N$. The boundary conditions, the water depth w.r.t. CD are used for each return period as presented in Table E.3. The wave height and period for each return period are given in Table E.1. Using a mean wave direction $\phi = 295.33°N$, as found from the data set and presented in Figure E.1. The wind intensity and wind direction are taken to be $21.29\ m/s$ and $261.26°N$ respectively since these give the most critical situation. Ray-plots of the offshore-nearshore transformation are given in Figure E.12 and Figure E.13.



(a) R=1 (b) R=500

Figure E.12: Ray plots for return periods $R = 1$ and $R = 500$.



(a) R=800 (b) R=1000

Figure E.13: Ray plots for return periods $R = 800$ and $R = 1000$.

F

Port design

F.1. Guaranteed water depth

The access channel must provide a sufficient water depth for all vessels to navigate through safely. The water depth must have certain buffers inside as there are many external factors that can positively but also negatively influence the water depth. The factors that will be taken account with are shown in Figure F.1.

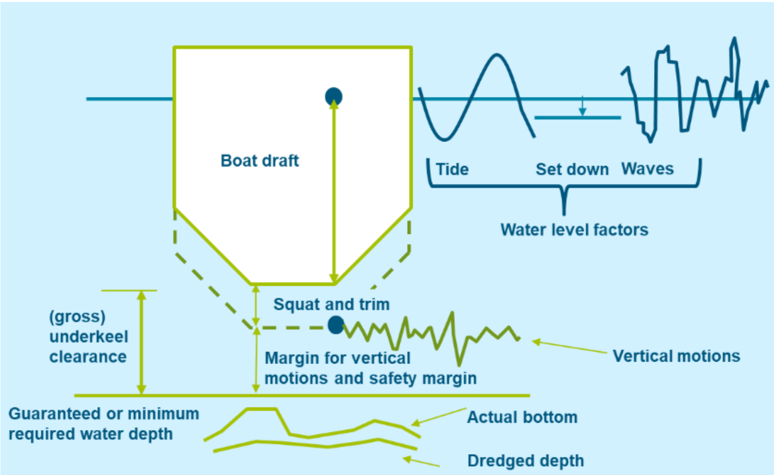


Figure F.1: Cross section of a vessel in the water. Figure from the lecture notes of Ports and Waterways.

The calculation of the water depth is based on a deterministic formula (Lansen, 2019) and is listed below:

$$d = D + h_T + s_{max} + z + h_{net} + T \tag{F.1}$$

For which its variables are elaborated below:

d : Guaranteed depth [m]
D : Draught of the design vessel [m]
h_T : Tidal elevation below reference level [m]
s_{max} : Maximum sinkage due to squat [m]
z : Vertical amplitude due to waves [m]
h_{net} : Safety margin or net underkeel clearance [m]
T : Dredging tolerance [m]

As can be seen from the figure, many variables are present such as draught, water level and dredging factors. But also ship related factors such as squat, trim, heel and wave response are included. The tidal elevation below reference level is equal to the amplitude due to the tide with reference level of MSL. This results in a tidal elevation of 0.78 meter as specified in E.2 in Appendix E. It was stated in the requirements that the water within the breakwaters is sheltered which means that the significant wave height lies between zero and one meter (Lansen, 2019). Therefore, assuming a worst case scenario, the vertical amplitude due to waves is assumed to be 1 meter. The safety margin or gross underkeel clearance is 0.5 meter and the dredging tolerance is 0.75 meter (Lansen, 2019).

s : Squat [m]
C_b : Blockage coefficient [-]
k : Blockage coefficient [-]
V_s : Vessel speed (relative to water) [knots]

The block coefficient (C_b) is a coefficient that describes the volume of water pushed away due to the ships shape. This can be seen in Figure F.2 and the higher the coefficient, the larger the volume of water pushed away by to the vessel.

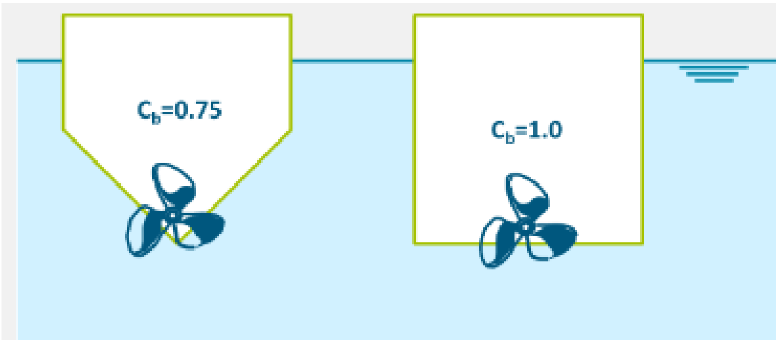


Figure F.2: The blockage coefficient . Figure from the lecture notes of Ports and Waterways (Lansen, 2019).

Passenger ships such as mini cruises tend to have a block coefficient between 0.6 and 0.7 (Solutions, principship). The largest blockage coefficient of 0.7 is used as it is opted for a conservative approach. This is because a larger blockage coefficient results in a higher squat and larger required water depth. For this approach the largest width and draft are chosen which are 25 metres and 6,5 metres respectively. The ship's cross-sectional area follows from these two variables and equals 162.5 m^2. The area of the channel consists of the water height and the depth calculated in the previous section because the channel is assumed to be squared which is allowed if the channel width is much larger than the water depth. The water depth is to be assumed at first but can be iterated with the equation for the guaranteed water depth. Lastly, the vessel speed equals 3.2 m/s which coincides with 6.22 knots (H. Ligteringen, 2000). Now every parameter is determined and the squat of the vessel can be calculated:

$$s = \frac{0.7}{17.4} \cdot \frac{6.5 \cdot 25}{130 \cdot h}^{0.76} \cdot 6.22^2 \tag{F.2}$$

This results in the following equation for the guaranteed water depth which contains the water depth on both sides:

$$d = 6,5 + 1.25 + \frac{0.7}{17.4} \cdot \frac{6.5 \cdot 25}{130 \cdot h}^{0.76} \cdot 6.4^2 + 1 + 0.5 + 0.75 \tag{F.3}$$

Iteration has resulted in a guaranteed water depth of 10.4 metres.

F.1.1. Tidal window

For the passage of a mini cruise it is very handy as their arrivals are planned beforehand and the intensity of mini cruises arriving in the port is rather low. In general arriving mini cruises do not have to wait because of an occupied berthing location. The assumption that the vessel makes perfect use of the tidal window by entering and leaving the access channel at it's highest tidal elevation is not made. This is because the mini cruise has a certain time that it wishes to remain in the port which may not coincide with the tidal elevation. Also there is always the possibility of delay due to unforeseen circumstances. If this is the case, then the highest tidal elevation could be missed. Besides the vessel has a certain time that it stays in the harbour and has the desire to leave when it is ready.

For the calculation it must be known what the mean high water and mean low water is. These two variables also depend on the tilt of the earth. During a spring tide the mean high water is at its highest and the mean low water is at its lowest. Whereas during the neap tide, the mean high water is slightly lower and the mean low water is slightly higher than that during the spring tide. As the most conservative approach is applied, the MHWN (Mean High Water Neaptide) and the MLWS (Mean Low Water Springtide) are used.
The MHWN equals 1.81 m +CD which is the water level due to the tidal motion which is diurnal and therefore assumed to have a period of 12 hours. The mean sea level (MSL) equals 1.03m +CD which is necessary to calculate the amplitude of these tidal waves. Their amplitude equals the difference between the MHW and MSL which is 0.78 metres. This results in the following equation for the water level: The tide can be described with a cosine function as follows:

$$\begin{aligned} x(t) &= a \cdot \cos\pi + \frac{t}{6} \cdot \pi + MSL \\ x(t) &= 0.78 \cdot \cos\pi + \frac{t}{6} \cdot \pi + 1.03 \end{aligned} \tag{F.4}$$

The water level given by the previous equation has been plotted with the characteristic values of MHW,MSL and MLW.

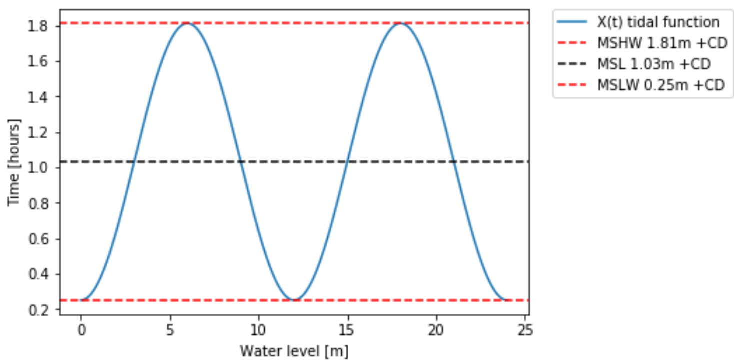


Figure F.3: Water level caused by the tide as a function of the time.

The tidal window must now be calculated to determine when vessels are able to navigate on the channel. At first the maximum waiting time and service time must be known which can be done with the following equations:

1. The maximum service time is 8 hours.
2. The mooring time of the cruise is 2 hours.

The maximum waiting time is equal to the summation of the maximum service and mooring time which in this situation is equal to 8 + 2 = 10 hours.
Now it is desired to calculate when it is possible and when not for vessels to enter the access channel. This can be done by using the equation for the water level and set it equal to its own equation after a time step equal to the maximum waiting time:

$$\begin{aligned} x(t) &= x(t + maximumwaitingtime) \\ 0.78 \cdot \cos\pi + \frac{t}{6} \cdot \pi + 1.03 &= 0.78 \cdot \cos(\pi + \frac{t+10}{6} \cdot \pi) + 1.03 \end{aligned} \tag{F.5}$$

The guaranteed depth without the use of a tidal window as calculated in the previous section was 10.40 metres. However this will be reduced with 1.55 meter resulting in a new guaranteed depth of 8.85 metres.

F.2. Dredging costs

The costs for a wet earthwork suction dredger is 6 euro's per cubic metre according to (der Horst, 2019. However, according to the site of the the costs lie between 20 and 25 euro's (Noorderkwartier-Hoogheemraadschap,2014). Both are reliable sources but neither tell anything about the size of the project. Although the more volume that must be dredged, the smaller the costs per cubic meter become. Thus it is assumed that the price of 6 euro's per cubic metre regards a massive project such as the Maasvlakte whereas that of the HHNK regards for smaller local projects. The price range that lies more to the price of 6 euro's per cubic metre will be used in this project namely between 10 and 15 euro's per cubic metre. This is because a rather substantial amount must be dredged from the access channel. For the calculation of the costs for dredging, it must first be known what the total volume of soil requires dredging. The water depth as a function of the length of the access channel can be seen in Figure F.4.

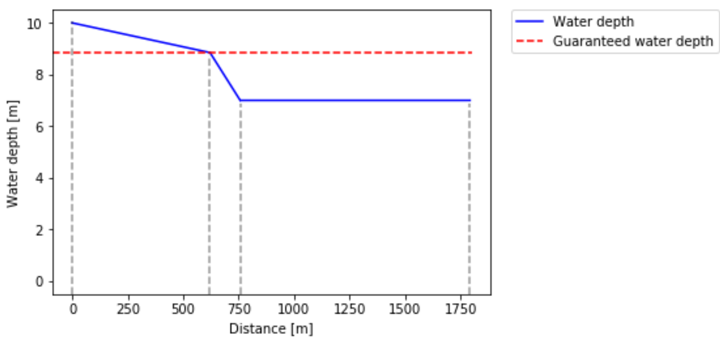


Figure F.4: The water depth in the access channel and turning circle.

At a distance of 0 metre in Figure F.4 is the starting point of the access channel which is at the 10 metres depth line. From this point, the access channel has already a sufficient guaranteed water depth over a length of 620 metres. This means that no dredging has to be performed within this part of the access channel. From this point onward the water depth can be seen declining when moving onshore, until a distance of 757 metres has been reached. The water depth is constant until the length of 1792 metres has been reached. Namely, this length is equal to the required access channel length plus turning circle. Thus, the water depth is below the guaranteed water depth over a length of 620 metres and 1792 metres which means dredging must be performed. The volume of soil that must be dredged is equal to the depth times the length times the width. Although the width is not entirely constant over the length of the access channel, it can be assumed as a constant rectangular access channel. This is justified as the assumption is valid since the water depth is much smaller than the channel width (depth « width). This results in the following calculations:

$$Volume = Depth \cdot Length \cdot Width$$

$$Costs = Volume \cdot dredgingcostperm^3$$

$$Volume1 = 0.5 \cdot (8.85 - 7) \cdot 136 \cdot 115 = 14467m^3.$$
$$Volume2 = (8.85 - 7) \cdot 1034 \cdot 115 = 219983.5m^3.$$
$$Costs = (14467 + 219983.5) \cdot 10 = 2.34 \cdot 10^6\text{euro}$$
$$Costs = (14467 + 219983.5) \cdot 15 = 3.52 \cdot 10^6\text{euro}$$

The investment costs that are saved will therefore lie between 2.34 and 3.52 millions euro's.

F.3. Basin dimensions

The formula to calculate the total quay length will be used to determine is obtained from the lectures of (J.P. Bos and Verhagen, 2018) and is given below:

$$L = \frac{Q \cdot (1+s) \cdot f_1}{r \cdot h} \tag{F.6}$$

L : Quay length [m]
Q : Total peak daily discharge in the ports[ton/day]
r : Main unloading rate per vessel per hour [ton/hr]
h : Number of unloading hours in a day [-]
l : Main vessel length [m]
s : Space between the vessels [m]
f_1 : Irregularity factor for the vessels [-]

The variables and their values are given in the table below and are obtained from the municipality of The Hague and FAO advice (W.A. Johnston,1994).

Total daily peak discharge	Space in between vessels	Irregularity of vessels	Unloading rate	Unloading hours per day
4000 tonnes	5 m	1.1	10 t/hr	8 hr/day

$$L = \frac{Q \cdot (1+s) \cdot f_1}{r \cdot h} \frac{(4000 \cdot (1+5) \cdot 1.1)}{10 \cdot 8} = 330 \text{ meters}$$

F.4. Marina

The municipality of the Hague wishes to expand the capacity however the exact number was not yet specified and therefore an estimation was made. The municipality is expecting a steep rise in activity within the harbour itself as the amount of visitors will rise from 3.5 million to 5 million (of The Hague, 2016). The expectation is powered by their vision as they aim to improve the livelihood of the harbour including that of the marina. To stimulate this, it is desired to have many places available in the marina for trespassers. Above this the amount of inhabitants around the harbour is rising quickly due to many housing constructions. This all has led to the decision to create 400 new berthing places in the marina which is slightly higher than the capacity of the current marina which is currently 350 of which 50 are for trespassers (of The Hague,2016).
The marina will have a finger peer construction This board has a width between 1.0. Above this, there is also a buffer space between vessels of one metre as safety measures.The length of each gang board depends on the length of its design vessel and this is a somewhat iterative process.

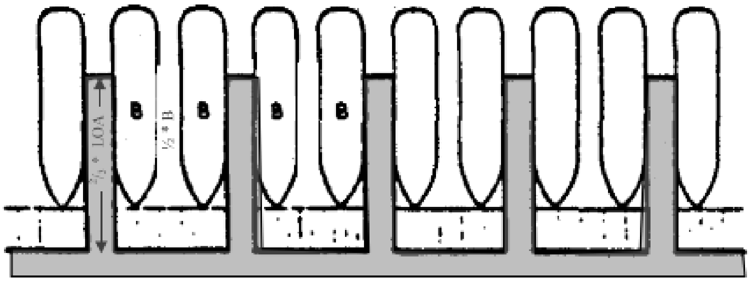


Figure F.5: Layout of the moored yachts in the marina. Figure from the lecture notes (J.P. Bos and Verhagen, 2018).

In order to visualize the layout aesthetically, it is required to know the amount of yachts within a certain size category that must be present. A table with this information is given below and the dimension values have been taken from the lecture notes (J.P. Bos and Verhagen, 2018). The percentages per size category must be exactly defined by the stakeholders as now an estimation has been made:

Size categories [m]	%	Yachts	Length [m]	Width [m]
<4	20	80	4	2.0
4 - 5	40	160	5	2.5
5 - 6	25	100	6	3.5
6 - 8	10	40	8	4.0
>8 - 15	5	20	15	4.5

The minimum width of the basin was determined however the length not yet and an estimate was made for this. As can be seen in the Figure F.6 the length of the basin in determined by the length of all area's between 2 and 5 of which each area's length was determined. Between each pier which was one meter wide, a space of 1.5 vessel length was reserved as turning space. This in combination with the vessel dimensions from the table above has resulted in a basin width of 200 metres.

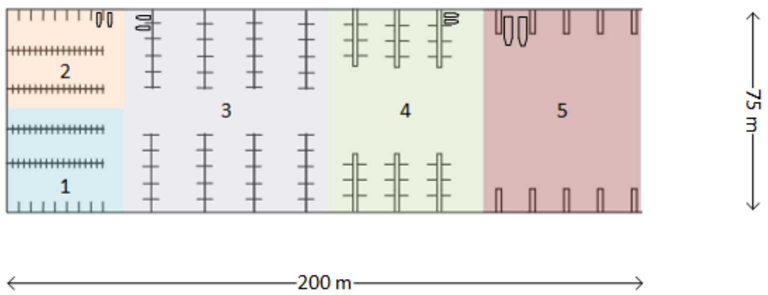


Figure F.6: Layout of Marina

F.5. Cruise docking

The pier construction that will be present to provide the docking possibility for the mini cruise can be seen in Figure F.7.

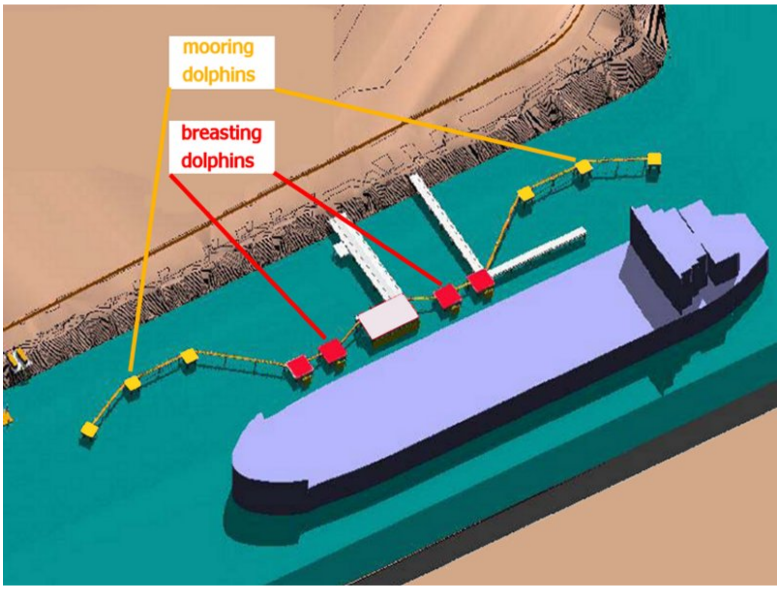


Figure F.7: Visual representative of the mooring of a mini cruise inside the breakwater. Figure taken from Oceanandairtechnology

As can be seen from Figure F.7 breasting and mooring dolphins are present to ensure that the cruise stays put. These dolphins are shown in Figures **??** and F.8 including their functions elaborating on why they are used are given below:

(a) Breasting dolphin

(b) Mooring dolphin

Figure F.8: Breasting dolphin and Mooring dolphin

Breasting dolphin

Its functions are to resist horizontal load caused by the physical stopping of the berthing cruise and to provide berthing and in some cases mooring equipment. As the vessel approaches the breasting dolphin it still has a certain velocity towards the dolphin which results in contact. The kinetic energy is absorbed by the fender that is placed on top of the breasting dolphin.

1. Resist horizontal load caused by the physical stopping of the berthing cruise.
2. Provide berthing and mooring equipment.

Mooring dolphin

The functions of the mooring dolphin are to provide space for bollards of Quick Release Hooks holding the cruise's mooring lines, to resist horizontal forces and to provide mooring equipment (M.Z. Voorendt,2019). Once the vessel is slowed down it will be tugged to the bollards using mooring lines which resist the horizontal forces that occur for example due to waves and the tide.

1. Provide space for bollards or Quick Release Hooks holding the cruise's mooring lines.

2. Resist horizontal forces.
3. Provide berthing and mooring equipment.

F.6. Laterally loaded pile foundation

The calculations to determine the dimensions and the strength of the breasting dolphin start by determining the load that acts on the vessel. For this Table 29-3 from the Manual of Hydraulic structures (M.Z. Voorendt, 2019) is used to obtain the design value for the force:

vessel type	length ℓ (m)	mass m [a] (ton)	force F_{dx} [b,c] (kN)	force F_{dy} [b, c] (kN)
small	50	3 000	30 000	15 000
average	100	10 000	80 000	40 000
large	200	40 000	240 000	120 000
very large	300	100 000	460 000	230 000

[a] the mass (1 ton = 1 000 kg) is the total mass of the ship, including the ship structure, load and fuel.
[b] the given values are related to a velocity of about 5 m/s and include the effects of additional hydraulic mass.
[c] if relevant, the effect of a ram bow (*bulbsteven*) should have been taken into account

Table 29-3 Berthing forces per sea-going vessel class, according to Eurocode 1 (NEN-EN 1991-1-7+C1+A1)

Figure F.9: Berthing forces. Figure obtained from Table 29-3 from the Manual of Hydraulic Structures (M.Z. Voorendt, 2019)

.

The mini cruise has a design length of 160 metres which is in between the large and average vessel type. The values for the average and large vessels are used to extrapolate the values for the mini cruise following a linear growth. Note that this might deviate from reality and either more data or equations are required to determine the exact forces imposed by the mini cruise. Linear extrapolation will result in the following values for the forces:

$$F_{dx} = \frac{240000 - 80000}{200 - 100} \cdot 60 + 80000 = 176000kN.$$
$$F_{dy} = \frac{120000 - 40000}{200 - 100} \cdot 60 + 40000 = 88000kN.$$

According to the Manual of Hydraulic Structures (M.Z. Voorendt, 2019), the values from Figure F.9 can be multiplied with 0.5 if it concerns vessels within a port or harbour. As this is the case in this project this results in a $F_{dx} = 88000$ kN and $F_{dy} = 44000kN$. The force parallel to the navigation direction of the vessel is equal to F_{dx} and is used to design the breasting dolphin with. The vessel and therefore also the force acting on the fender will be at an angle of 15 degrees to the perpendicular of the fender. This can also be seen in Figure F.10. This results in a force acting perpendicular on the fender of $F_x = cos(15) \cdot 88000 = 85000kN$ and a force parallel of $F_y = sin(15) \cdot 88000 = 22776kN$.

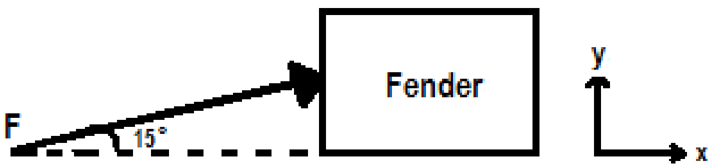


Figure F.10: Vessel's force acting on the fender

The breasting dolphin including its lateral load can be schematized with Blum's method. The idea behind this method is that the pile is displaced by the applied force, except at the lower end where a displacement towards in the opposite direction occurs as the pile rotates around a point just above its deepest point. The soil's reaction can be replaced by the concentrated force (R3) also known as Ersatzkraft. A schematic for this method can be seen in Figure F.11.

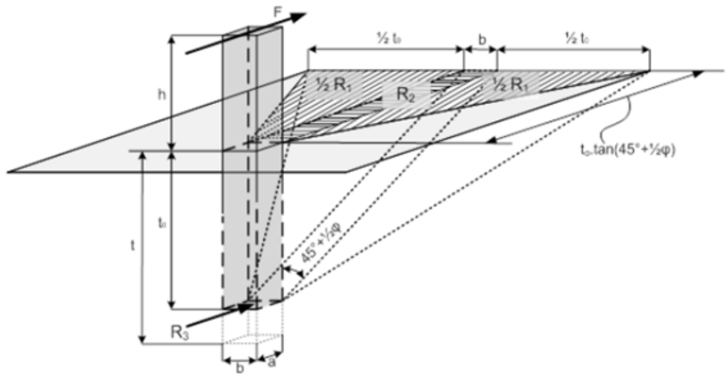


Figure F.11: Schematization of the acting forces including corresponding arms.

For which the variables are defined as following:

F : Load [kN] (F.7)

R_1 : Resultant force of the soil wedges next to the soil wedge directly behind the pile (two half pyramids). [kN] (F.8)

R_2 : Resultant force of the soil wedge directly behind the pile (a triangle with width b). [kN] (F.9)

R_3 : Substitute force (Ersatzkraft).[kN] (F.10)

A : Width of the pile in the load's direction. [m] (F.11)

B : Width of the pile perpendicular to the load. [m] (F.12)

H : Length of the unsupported part of the pile.[m] (F.13)

T_0 : Theoretical embedded depth [m] (F.14)

T : Practical embedded depth which is equal to 1.2 $\cdot T_0$.[m] (F.15)

ϕ : Angle of internal friction.[degrees] (F.16)

(F.17)

The schematisation of loads and arms is shown in Figure F.12 and it contains the following variables
:

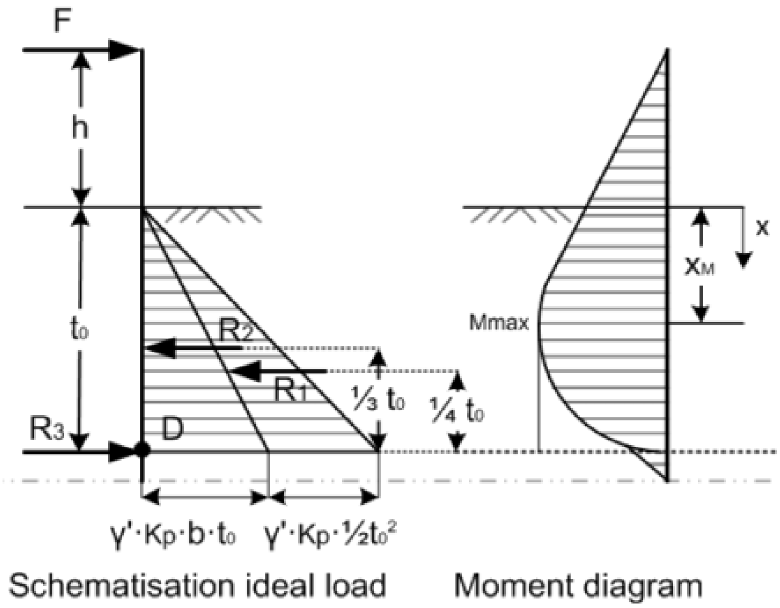


Figure F.12: Schematization of the acting forces according to Blum including its arms.

F_{max} : Maximum load that can be resisted by the soil. [kN]
R_1 : Resultant force of the soil wedges on both sides next to the soil wedge directly behind the pile (two half pyra
R_2 : Resultant force of the soil wedge directly behind the pile (a triangle with width b). [kN]
R_3 : Substitute force (Ersatzkraft). [kN]
b : Width of the pile perpendicular to the load. [m]
h : Length of the unsupported part of the pile. [m]
t_0 : Theoretical embedded depth. [m]
K_p : Passive soil pressure coefficient. [m]
δ : Wall friction, δ = -$\frac{2}{3} \cdot \phi$. [degrees]

Taking the momentum equation around point D as can be seen in Figure F.12, it results in the Ersatzkraft (R3) being cancelled out due to an arm of 0 metre. Therefore only the external applied force on the structure and the two forces by the soil (R1 and R2) remain in the momentum equation. Solving that equation results in the following expression for the maximum load that the soil can resist:

$$
\begin{aligned}
M_0 &= 0 \\
-F_{max} \cdot (t_0 + h) + R_1 \cdot \frac{1}{4} \cdot t_0 + R_2 \cdot \frac{1}{3} \cdot t_0 &= 0 \\
F_{max} \cdot (t_0 + h) = (\frac{1}{6} \cdot \gamma \cdot K_p \cdot t_0^3) \cdot \frac{1}{4} \cdot t_0 + (\frac{1}{2} \cdot \gamma \cdot K_p \cdot b \cdot t_0^2) \cdot \frac{1}{3} \cdot t_0 & \\
F_{max} \cdot (t_0 + h) = \frac{1}{24} \cdot \gamma \cdot K_p \cdot t_0^4 + \frac{1}{6} \cdot \gamma \cdot K_p \cdot b \cdot t_0^3 &
\end{aligned}
\tag{F.18}
$$

$$\rightarrow F_{max} = \gamma \cdot K_p \cdot \frac{t_0^3}{24} \cdot \frac{t_0 + 4 \cdot b}{t_0 + h} \tag{F.19}$$

The maximum force that the soil can resist must be larger or equal to the occurring maximum force which is 85000 kN. Now as the F_{max} is a function of both the embedded depth (t_0) and the width (b) of the pile it has been iteratively modified until the condition was met. This has resulted in an embedded depth of 26 meters which is a depth of 31.2 meter in reality as $t = 1.2 \cdot t_0$ and a width of 4 meters.

$$K_{p,h,\sigma} = \frac{cos^2(\phi - \alpha)}{cos^2(\alpha) \cdot \left[1 - \sqrt{\frac{\sin(\phi-\delta)\cdot\sin(\phi+\beta)}{\cos(\alpha-\delta)\cdot\cos(\alpha+\beta)}}\right]^2} \tag{F.20}$$

- α = β = 0

In order to calculate the maximum force with Equation F.21, the $K_{p,h,\sigma}$ and also the internal friction ϕ must be known. The last variable can be calculated with help from the given values in Figure F.14 which contains the friction coefficient. The value of this coefficient depends on the soil profile which was rather troublesome. The soil profiles provided by (Dinoloket,2020) at the designated location within the breakwater were rather old (1968) and the soil profiles only went to a soil depth of 2 metres. It is unsafe to use such a soil profile over large depths. Therefore three soil profiles from various locations outside the breakwater have been used to determine the friction coefficient. The soil profiles can be found in Figure F.13. It can be noticed that most part of the soil profile consist of medium sand and some parts of more coarser sand. Therefore from Figure F.14 the values from silty to medium sand is used but the upper value which is more closely to the coarser sand. So the corresponding value would be between 0.35 and 0.45, however the value of 0.45 will be used. It can be noted that some clay and peat is present but this is a rather small part of the entire depth of the soil profile and that this is mostly present near the bed floor. The downside to this is that the soil profile does not exceed 7 meter depth. New soil profiles with a larger depth could be performed in order to remove any uncertainty with regard to soil strength.

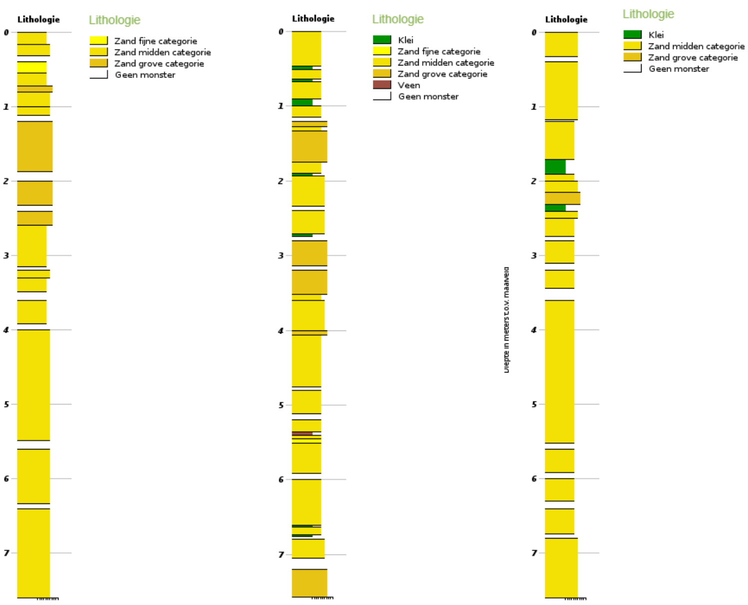


Figure F.13: Soil profiles obtained from DINO loket.

Interface Materials	Friction Coefficient, f
Mass concrete on the following foundation materials:	
Clean sound rock	0.70
Clean gravel, gravel--sand mixtures, coarse sand	0.55 to 0.60
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45
Fine sandy silt, nonplastic silt	0.30 to 0.35
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50
Medium stiff and stiff clay and silty clay	0.30 to 0.35

Figure F.14: Table of the friction coefficient per soil type. Figure obtained from Table37-1 of the Manual of Hydraulic Structures (M.Z. Voorendt, 2019
).

The γ' is the effective volumetric weight and is obtained from $\gamma_{sat} - \gamma_w$ which is 28- 10 kN/m^3 = 18 Kn/m^3. Also the height of the breasting dolphin above the bed floor, which is term 'h' in Blum's schematisation, is equal to the water depth plus 2 meters (which is the height of fender above water level). This results in a value of 'h' of 12.4 meter.
The maximum momentum occurs for which the derivative of the momentum equation equals zero ($\frac{\delta M}{\delta x} = 0$) which can be seen in Figure F.15 and this must be calculated for when the maximum allowable lateral force is applied (F_{max}).This results in the following equation:

$$\begin{aligned} M_x &= F \cdot (h + x) - \gamma \cdot K_p \cdot (x + 4 \cdot b) \cdot \frac{x^3}{24} \\ x_m^2 \cdot (x_m + 3 \cdot b) &= \frac{t_0^3}{4} \cdot \frac{t_0 + 4 \cdot b}{t_0 + h} \\ x_m^2 \cdot (x_m + 3 \cdot 4) &= \frac{26^3}{4} \cdot \frac{26 + 4 \cdot b}{26 + 12.4} \\ \rightarrow x_m &= 15.07 metres \end{aligned} \tag{F.21}$$

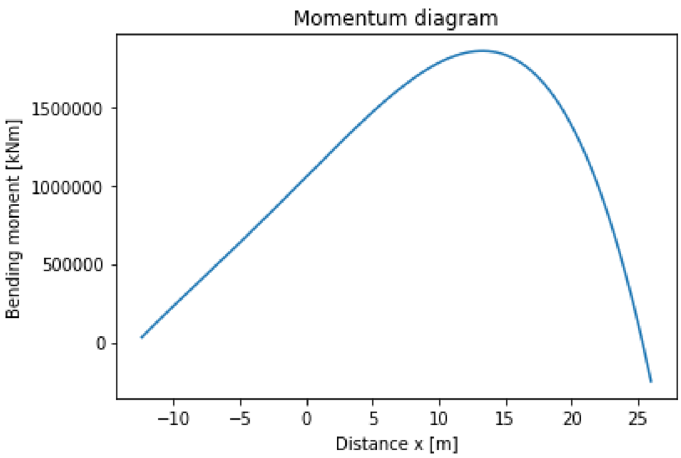


Figure F.15: Momentum diagram of the pile

Now the maximum bending moment can be calculated with the following equation:

$$\rightarrow M_x = F \cdot (12.4 + 15.07) - \gamma \cdot K_p \cdot (15.07 + 4 \cdot 4) \cdot \frac{12.54^3}{24} = 1.93 \cdot 10^6 kNm$$

The maximum shear force can be obtained from this curve and equals 4.04 $\cdot 10^5$ kN. This is also equal to the Ersatzkraft as the derivative of the moments goes to zero which implies that the shear force equals 0 kN. Note that in reality the shear force should be a constant from the top of the pile until the bed floor as there is only the vessel force acting on the pile. Although it is not the case in Figure F.16 it still swirls around the applied force and seems like a proper estimation. As this shear curve was estimated from the derivation of the moment curve, it implies that the moment curve is not exactly linear in this region whereas in reality this is the case.

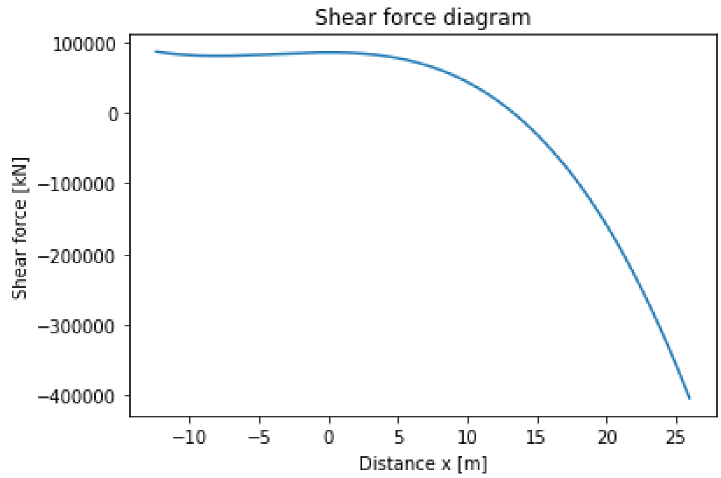


Figure F.16: Shear force curve over the length of the pile

F.6.1. Cross-sectional dimension of pile

For design calculations values of characteristics European standards are used which for concrete is taken from the Eurocode 3 Design of steel structures which are as follows:

material property ↓ / steel quality →	**S235**	**S275**	**S355**
yield stress f_y at 20 °C [N/mm^2]	235	275	355
tensile strength f_u [N/mm^2]	360	430	510
Young's modulus	E = 210 000 N/mm^2		
Shear modulus (*glijdingsmodulus*)	$G = \frac{E}{2(1+\nu)} \approx 81000$ N/mm^2		
Poisson's ratio in elastic stage	ν = 0,3		
Coefficient of linear thermal expansion	$\alpha = 12 \cdot 10^{-6}$ per °C (for T ≤ 100 °C)		

Figure F.17: Properties of steel. Figure obtained from Table 36-1 from the Manual Hydraulic Structures (M.Z. Voorendt, 2019
).

These values for the yield stress f_y are also its design values (f_{yd}) because $f_{yd} = \frac{f_y}{y_{m0}}$ as y_{m0} = 1.0 for resistance of cross-sectionals for all classes. Since there are three different steel qualities that can be used it is an iterative process, however at first the steel quality S235 will be used s this is most common.

The previously mentioned pile of the breasting dolphin will be made from steel and must be able to withstand the present forces which according to the limit state is fulfilled if the resisting design variable is larger than the maximum occurring design variable. The calculation methods for the structural elements partially depends on the characteristics of the chosen profile such as width over thickness ratio. The profile and width over thickness ratio determine if it deforms in a plastic, semi-elastic or elastic manner which states if the plastic theory or elastic theory must be used. The classification cross-sections for steel profiles are shown in Figure F.18:

type of plate	type of load	class 1 (plastic)	class 2 (semi-plastic)	class 3 (elastic)
flange rolled I-profile	pressure	$b/t_f \leq 10$ all HE-profiles	$b/t_f \leq 11$	$b/t_f \leq 15$
web rolled I-profile	bending pressure	$h_w/t_w \leq 72$ $h_w/t_w \leq 33$	$h_w/t_w \leq 83$ $h_w/t_w \leq 38$	$h_w/t_w \leq 124$ $h_w/t_w \leq 42$
walls rolled square tube	pressure	$b/t \leq 33$	$b/t \leq 38$	$b/t \leq 42$
wall cylindrical tube	pressure or bending	$d/t \leq 50$	$d/t \leq 70$	$d/t \leq 90$

Figure F.18: Classifications of steel. Figure obtained from Table 36-2 from the Manual Hydraulic Structures (M.Z. Voorendt, 2019
).

A wall cylindrical tube is used as profile because this is commonly used when bending moments are present. According to Figure F.18 a web rolled I-profile could also have been used, however this profile is not desirable for the pile foundation. The diameter of the pile was previously calculated at 4 metres, however the thickness is yet to be determined but set at 0.8 metres for now. This results in width over thickness ratio of $4/0.8 = 5$ which is class 1. This means that the profile will deform in a plastic way and plastic theory can be applied.

Bending moment In order for the pile to be able to resist the occurring bending moments, the unity check which is $\frac{M_{ed}}{M_{c,Rd}} \leq 1.0$ must be satisfied.

- $M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{y_{m0}}$

  The f_{yd} and the y_{m0} are already known only the W_{pl} can be calculated with the following equation:

$$W_{pl} = \frac{d^3 - d_{inner}^3}{6} = \frac{4^3 - (4 - 2 \cdot 0.8)^3}{6}$$

$$\rightarrow W_{pl} = 8.36m^3 \tag{F.22}$$

$$M_{c,Rd} = \frac{8.36 \cdot 235 \cdot 10^3}{1.0} kNm = 1.97 \cdot 10^6 kNm$$

The maximum present moment had previously been calculated in the pile foundation and was equal to $1.93 \cdot 10^6 kNm$. Now the unity check can be calculated:

$$UC = \frac{M_{ed}}{M_{c,Rd}} = \frac{1.93 \cdot 10^6}{1.97 \cdot 10^6} = 0.98$$

These profile dimensions suffice against the present bending moment because the unity check is lower than 1.

Shear The shear force that acts on the hollow cylindrical tube must be smaller than the shear force resistance thus $\frac{V_{Ed}}{V_{pl,Rd}} \leq 1.0$ must be satisfied. The maximum shear force that is present is equal to 4.04 $\cdot 10^5$ kN whereas the design plastic shear resistance is given by the following equation:

$$A_v = \frac{1}{4} \cdot \pi \cdot (d^2 - (d - 2 \cdot t)^2) = 8.04m^2$$

$$V_{pl,Rd} = \frac{A_v(\frac{f_{yd}}{\sqrt{3}})}{\gamma_{m0}}$$

$$V_{pl,Rd} = \frac{8.04(\frac{235*10^3}{\sqrt{3}})}{1.0} = 1.09 \cdot 10^6 kN$$

$$UC = \frac{V_{Ed}}{V_{pl,Rd}} = \frac{8.5 \cdot 10^4}{1.09 \cdot 10^6} = 0.05$$

Torsion It is possible for the vessel to hit the fenders in an angle and therefore imposing torsion on the pile. Therefore it must also be proven that the torsional moment induced by the force is smaller than the torsional moment capacity. The torsional moment is considered to be a part of two internal effects namely:

$$T_{Ed} = T_{t,Ed} + T_{w,Ed}$$

Which are the following:

T_{Ed} : Total torsional moment
$T_{t,Ed}$: Internal Saint Venant torsion
$T_{w,Ed}$: Internal warping torsion

Calculations regarding the strength due to torsion will be performed using the yield criterion by Von Mises which must be smaller than the yield stress of steel. This is shown in the equation below:

$$\sigma_{vgl,Ed} = \sqrt{\sigma_{max}^2 + 3 \cdot \tau_{max}^2} < f_{yd} \tag{F.23}$$

In this equation, the σ_{max} is the result of internal warping torsion and τ_{max} of the internal Saint Venant torsion and must satisfy $\frac{\sigma_{vgl,Ed}}{fy_d} \leq 1.0$.
However, Eurocode 3 (NEN-EN 1993) allows simplified calculations which for closed profiles such as a wall cylindrical tube, the total torsional moment is completely obtained by the internal Saint Venant torsion. This assumption is also supported by (Hoogenboom,2019). According to these notes the exact expression for Torsion for a wall cylindrical tube is the as follows:

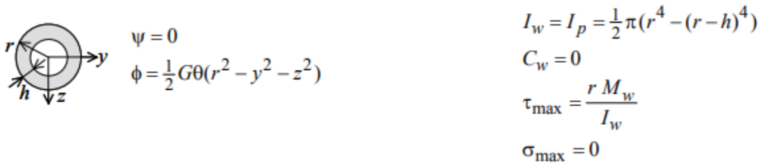


Figure F.19: Shear stresses due to Saint Venant. Figure taken from (Hoogenboom,2019)

$$I_w = 0.5 \cdot \pi(r^4 - (r-t)^4) = 0.5 \cdot \pi(2^4 - (2-0.8)^4) = 21.88 m^4$$
$$\tau_{max} = \frac{r \cdot M_w}{I_w} = \frac{0.5 \cdot 4 \cdot 85000}{21.88} = 16091.03 kN/m^2$$
$$\sigma_{max} = 0$$
$$\sigma_{vgl,Ed} = \sqrt{\sigma_{max}^2 + 3 \cdot tau_{max}^2} = \sqrt{0 + 3 \cdot 16091.03^2} = 1.11 \cdot 10^5 kN/m^2$$

Combination of bending moment and shear The effect of shear in combination with a bending moment has considerable effect on the bending moment resistance is and should only be taken into account if the shear force is less than half the plastic shear resistance. Therefore before performing any calculations, it should be checked whether the effect on the bending moment resistance by the shear force can be neglected or not according to the Manual Hydraulic Structures (M.Z. Voorendt, 2019):

$$V_{ed} = 85.000 kN$$
$$V_{pl,Rd} = \frac{A_v \cdot (\frac{f_y}{\sqrt{3}})}{y_{m0}} = \frac{A_v \cdot (\frac{f_y}{\sqrt{3}})}{y_{m0}} = 1091181.7 kN$$
$$V_{ed} < 0.5 \cdot V_{pl,Rd} = 85000 kN < 1091181.7 kN$$

From this calculation it results that the effect of shear in combination with a bending moment does not have a considerable effect on the bending moment resistance.

F.7. Water sports

There must also be space reserved for the possibilities to practise water sports as it is a demand of the municipality of The Hague that Scheveningen becomes a key player within the water sports domain. The exact amount of space required for the realisation of the training facilities is to be determined by mutual agreement by various parties such as the municipality of The Hague and the Watersportverbond.The training facilities must be officially accredited by the NOC*NSF that the facilities fulfill all requirements that are necessary to offer an ideal training location for professionals in which training, studying and living can be well combined. In the current situation there is only a sail center for professionals which is officially accredited by the NOC*NSF. Whereas the Watersportverbond is an orgazination that contains many more water sports such as canoeing, windsurfing, kitesurfing and surfing. Training facilities for these water sports that are officially accredited are not yet present in Scheveningen. It is therefore required for the concerned parties important to discuss for which water sports training facilities must arise and is mainly dependent on the vision plan of the NOC*NSF.
As mentioned in the boundary conditions, knowing exactly what type of and the size of training facilities must be present will not be dealt with. It is however important to provide space for the berthing of water sports vessels and training facilities including the possibilities of expansion.

F.8. Zoning plan

The zoning plan concerns the layout of the harbour in which the land- and water-based facilities required for the harbour extension, access channel and basins are graphically shown. These required facilities and their area and dimensions are determined and shown in the Appendix whereas that of the access channel and basins have already been defined in earlier sections. This has resulted in the following sketch.
The area and dimensions of the on shore facilities have been derived from the requirements and from the amount of tonnes of cargo that must be processed. The amount of cargo has a daily peak of 4000 tonnes per day which is used to obtain the following values:

Land-based facilities	**Quantitative requirement**	**Area [m^2]**
Market or auction hall	5 m^2 / t	20.000
Processing facilities	-	350
Administration building	-	30
Ice factory	4 m^2 per ton per day	16000
Ice storage	1 m^2/t	4000
Cold storage	1 m^2/t	4000
Slipway and boat lift	10m^2 per crane	40
Net repair facilities	-	500 m^2
Sanitary facilities	-	15

Now the chosen values for each item will be elaborated as the requirements did not specify an exact value for all items yet:

- The processing facilities and administration building was specified in the requirements to be between 25 and 1000 m^2. For this case a value of 350 m^2 was used because the harbour has quite a lot of fishing and is a decent sized fishing port with mostly high sea vessels. However, it is far from the largest in the world let alone in The Netherlands and this also applies to the amount of fish that is being processed. Therefore a value of just below average was taken.

- The ice storage was given to be between 0.5 and 1.0 m^2/t and therefore a value of 1.0 was used because it regards a rather small ice storage (J.P. Bos and Verhagen,2018). As this also holds for the ice factory a rather large value of 4 m^2 per ton per day was used.

- Net repair facilities were given to be between 50 and 1000 m^2. Similar to that of the processing facilities. However, as the harbour itself does not have repair facilities (of The Hague,2016) it was opted for 500 m^2 instead of 350 m^2.

The land based facilities are placed such that required operations run as efficient as possible. Therefore it is chosen that one side of the harbour is destined for returning vessels and one side for leaving vessels:

- On the quay walls on the seawards side vessels will berth that are returning from their trip. At this side, the auction hall and processing facilities will be located. This is so that the distance for the fish to travel from vessel to facilities is as small as possible.
- On the quay walls on the landwards side vessels will berth that will start their trip. At this side, the ice factory and ice storage are located. This is so that the distance the ice has to travel to the vessel is as small as possible.
- The administration building is placed next to the marina including the sanitary facilities. This building is however rather close to the harbour and can possibly be located more land inwards. This depends on the plans of the stakeholders with the area next to the marina.
- The slipway, boat lift and net repair facilities will be present on the quay wall on the land inwards side and next to the ice storage and ice factory. Although not present in the sketch.

3D overview sketches

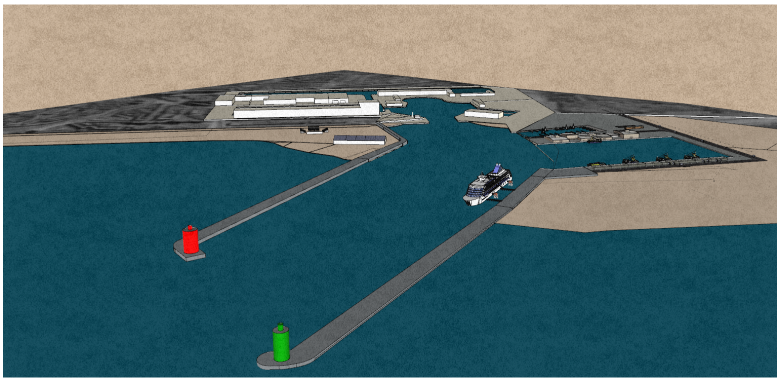


Figure F.20: 3D Sketch of the entire port of Scheveningen including the fourth harbour.

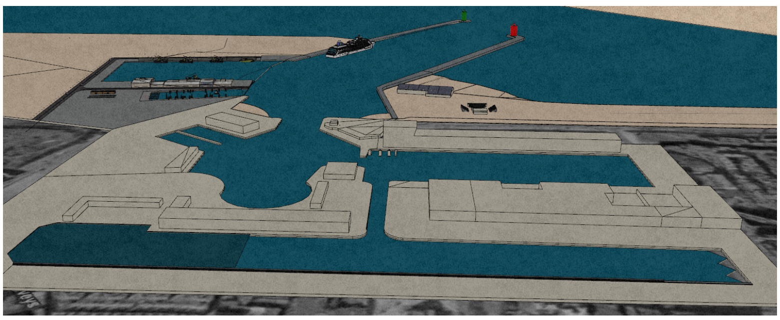


Figure F.21: 3D Sketch of the entire port of Scheveningen including the fourth harbour.

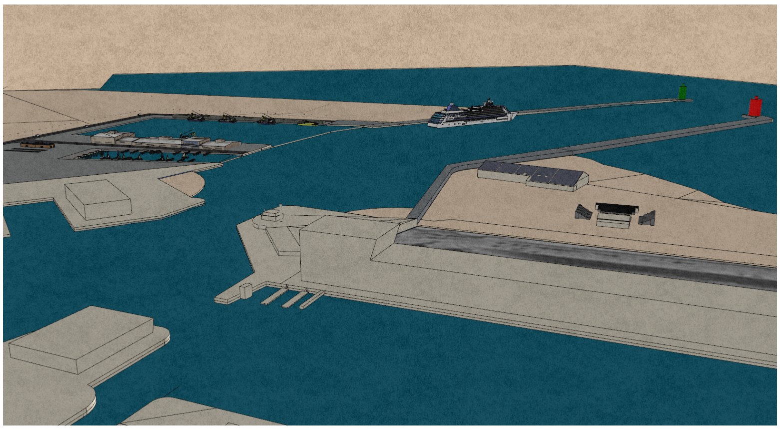


Figure F.22: 3D Sketch of the entire port of Scheveningen including the fourth harbour.

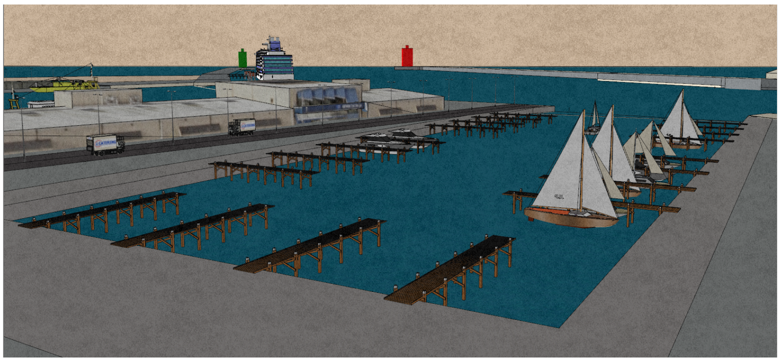


Figure F.23: 3D Sketch of the entire port of Scheveningen including the fourth harbour.

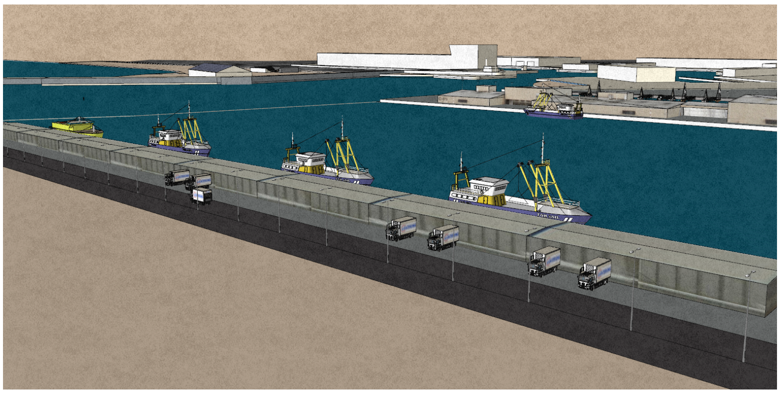


Figure F.24: 3D Sketch of the entire port of Scheveningen including the fourth harbour.

G
Appendix

G.1. Design criteria

The design criteria listed below were used as guidelines for the design of the roundabouts in Section 12.4.8.

Ontwerpvariabelen [m]	enkelstrooks-rotonde	tweestrooks-rotonde
binnendiameter	13 – 30	24- 80
buitendiameter	25 - 40	44 - 108
rijbaanbreedte	5 - 6	9 - 11
breedte overrijdbare strook	[redacted]	5 *)
afrondingsboog toerit	10 - 14	10 - 100
afrondingsboog afrit	10 - 14	12 - 100
rijstrookbreedte toerit, afrit	3,0 - 4,0	3,0 - 4,5
breedte middengeleider	2 - 5	1,6 - 18
afstand oversteekplaats	5 - 10	5 - 10

*) indien met passeerbaan

Figure G.1: Preferred design criteria for roundabouts (Hansen et al., 2011)

G.2. ZKA consultancy tram variant

The plan of the ZKA consultancy for the future tram line is shown in the Figure below.

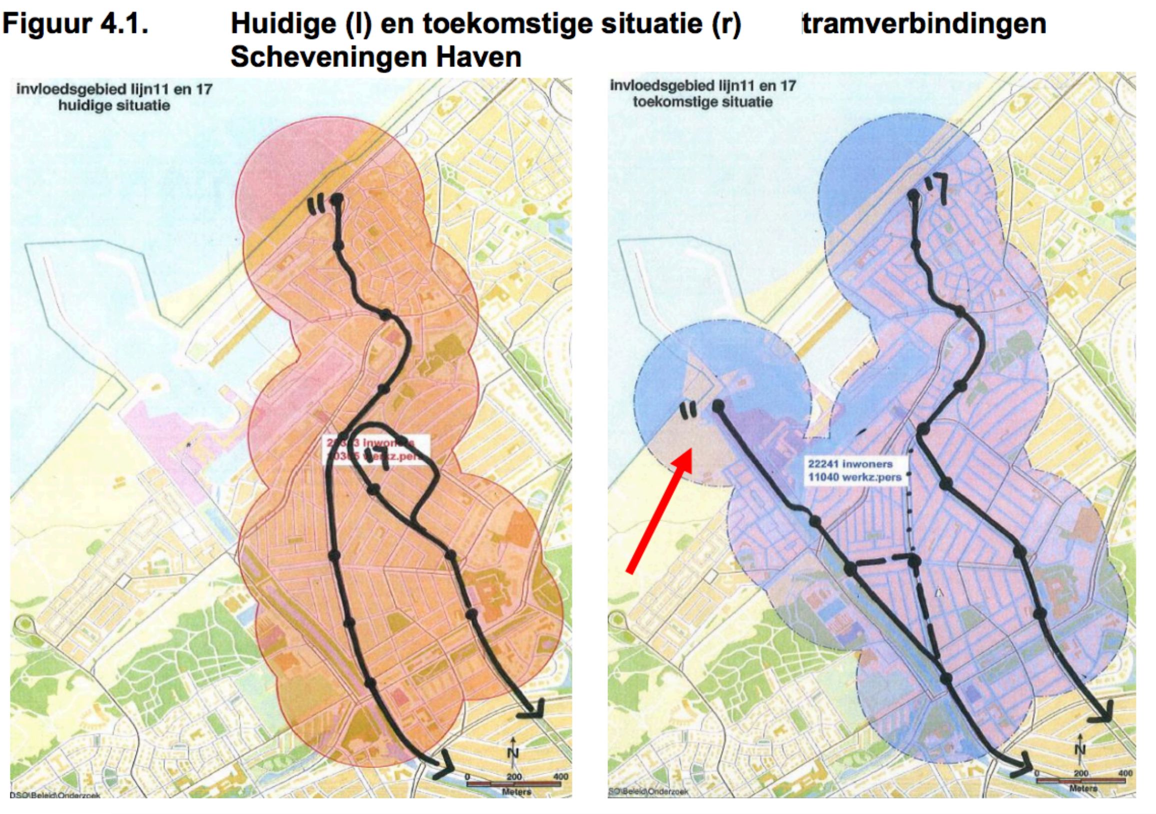


Figure G.2: Tram plan by ZKA consultants (ZKA Consultants & Planners, 2011)

G.3. Parking lot table

The parking lot length is given as a function of width and parking angle in this appendix. It can be seen that for angles of 60 and 70 degrees, a length of 4.20 m is mostly sufficient.

parkeerhoek [graden]	breedte parkeerplaats [m]					
	2,25	2,30	2,35	2,40	2,45	2,50
60	4,00	4,00	4,00	4,00	4,00	4,00
70	4,50	4,20	4,00	4,00	4,00	4,00
90	6,75	6,35	5,95	5,65	5,45	5,40

Figure G.3: Parking spot length table as function of width and parking angle (Hansen et al., 2011)

G.4. Duindorp intensities

The traffic intensities are displayed per road section in vehicle equivalents per hour during peak hours.

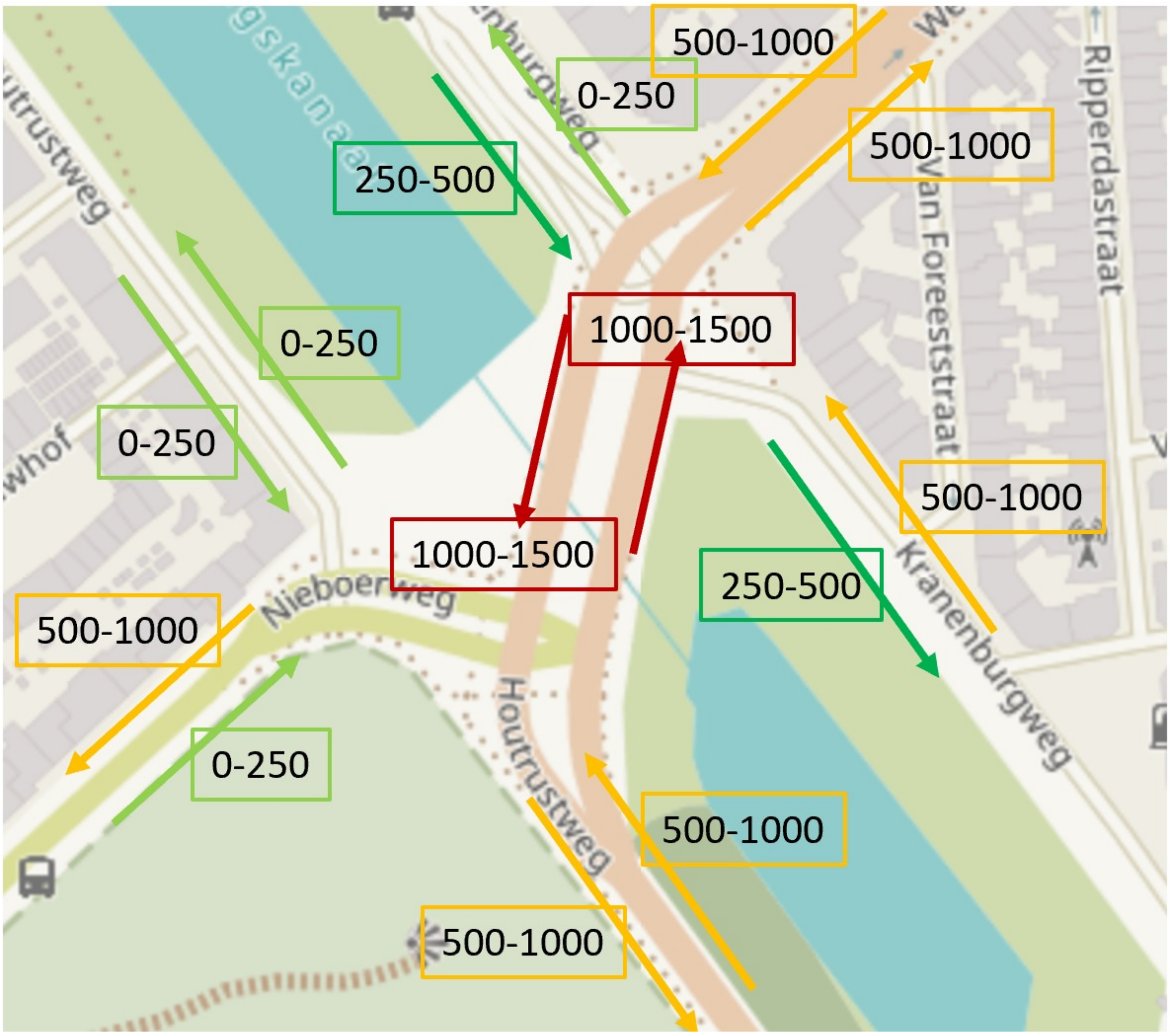


Figure G.4: Intensity map, adapted from ZKA Consultants & Planners 2011

Bibliography

[1] Algemeen Hoogtebestand Nederland (AHN). Ahn viewer. 2020.

[2] Besluit kwaliteitseisen en monitoring water, bijlage III. -. 2009.

[3] Tom Bruce, Jentsje Van Der Meer, Tim Pullen, and William Allsop. Wave overtopping at vertical and steep structures. In *Handbook of coastal and ocean engineering*, pages 411–439. World Scientific, 2010.

[4] D. Butler and J.W. Davies. Urban drainage, 2nd edition. 2000.

[5] RIVM WUR CBS, PBL. Sea level: Dutch coast and worldwide. 2016.

[6] CETMEF CIRIA, CUR. The rock manual. *The use of rock in hydraulic engineering, CIRIA, CUR, CETMEF, C*, 683, 2007.

[7] De Scheveninger. Gemeente gaat westduinweg in 2019 echt aanpakken. `https://www.de-scheveninger.nl/2018/09/06/gemeente-gaat-westduinweg-echt-aanpakken/`, 2018.

[8] Delfluent services. Factsheets wwtp houtrust. 2020.

[9] Delft Bluewater. Dbw gietwater uitermate geschikt voor de glastuinbouw. 2013.

[10] Deltares. Mogelijke gevolgen van versnelde zeespiegelstijging voor het deltaprogramma. September 2018.

[11] Department for Communities and Local Government: London. Multi-criteria analysis: a manual. 2009.

[12] A. Q. C. Van der Horst. Constrution technology, lecture notes cie4170, 2019.

[13] Dinoloket. Ondergrondmodellen bro regis ii v2.2. `https://www.dinoloket.nl/ondergrondmodellen`, 2020.

[14] Dunea. Duingebieden. 2020.

[15] D.F. Elger, B.A. Lebret, C.T. Crowe, and J.A. Roberson. Engineering fluid mechanics, 11th edition. 2015.

[16] Faculteit Civiel Techniek en Geowetenschappen. Gezondheidstechniek. 2013.

[17] Gemeente Den Haag. Bestemmingsplan scheveningen haven. 2013.

[18] Gemeente Den Haag. kadernota straten, wegen en lanen. 2015.

[19] Gemeente Den Haag. Verkeersvisie scheveningen. 2016.

[20] Gemeente Den Haag. Voorstel van het college inzake verkeersvisie scheveningen 2025. 2016.

[21] Gemeente Den Haag. Monitor toerisme 2017. 2018.

[22] Gemeente Den Haag. Ontwerpeconomische visie den haag. 2019.

[23] Gemeente Den Haag. Toelichtende omschrijving gemeentelijk beschermd stadsgezicht scheveningen-dorp, 2020. URL `http://www.monumentenzorgdenhaag.nl/sites/default/files/beschermd-stadsgezicht/toelichting/toelichting%20gemeentelijk%20beschermd%20stadsgezicht%20Scheveningen%20Dorp.pdf`.

[24] De Gijt. Cost of quay walls including life cycle aspects. 2011.

[25] Yoshimi Goda. The design of upright breakwaters. Technical report, ICCE 1992, 10 1992.

[26] Yoshimi Goda. *Random seas and design of maritime structures*. World scientific, 2010.

[27] H. Velsink H. Ligteringen. *Ports and Terminals*. 2000.

[28] I.A. Hansen, P.C.A. Opstal, and P.B.L. Wiggenraad. Geometrisch ontwerp van wegen en spoorwegen. 2011.

[29] Hartsuijker. Toegepaste mechanica deel 2: Spanningen, vervormingen, verplaatsingen. March 2010.

[30] P. Hoogenboom. Aantekeningen voor wringing. 2019.

[31] HTM personenvervoer NV. Dienstregeling. `https://www.htm.nl/dienstregeling`, 2020.

[32] Ingenieurs Bureau Oranjewoud. Scheveningen achtergrond document water. 2013.

[33] H.J. van den J.P. Bos and Verhagen. Breakwater design, lecture notes cie5308, February 2018.

[34] Z. Kapelan. Urban drainage. 2020.

[35] KNMI. Dagwaarden neerslagstations. 2019.

[36] N. Koeman-Stein, W. Appelman, R. Creusen, M. Paalman, B. Raterman, and W. Voogt. Wateraanbod glastuinbouw haaglanden. 2014.

[37] A.J. Lansen. Ports and waterways, lecture notes cie4330, May 2019.

[38] K.G. Bezuyen M.Z. Voorendt, W.F. Molenaar. Lecture notes caissons, February 2011.

[39] W.F. Molenaar M.Z. Voorendt. Manual hydraulic structures. February 2019.

[40] W.F. Molenaar M.Z. Voorendt. Hydraulic structures general lecture notes. 2020.

[41] Hoogheemraadschap Hollands NoorderkwartierHoogheemraadschap. Baggerwerkzaamheden oosterdelgebied. October 2014.

[42] Municipality of The Hague. Technische vragen van diverse fracties over stukken over plannen scheveningen haven. 2016.

[43] Municipality of The Hague. Technische vragen van diverse fracties over stukken over plannen scheveningen haven. 2016.

[44] Office of Rail Regulation. Guidance on tramways. 2006.

[45] OpenStreetMap contributors. Planet dump retrieved from https://planet.osm.org . `https://www.openstreetmap.org`, 2017.

[46] PIANC. Harbour approach channels design guidelines. Technical Report 121, PIANC, 2014.

[47] K. W. Pilarczyk. *Geosynthetics and Geosystems in Hydraulic and Coastal Engineering*. 2000.

[48] Provincie Zuid-Holland. Vergunning meijendel. 2010.

[49] PSO. Monitor toerisme 2010 - 2014. 2015.

[50] Randstadrail. De voertuigen. 2006.

[51] MAN Energy Solutions. Basic principles of ship propulsion.

[52] P.J Stuyfzand. Hydrochemistry and hydrology of the coastal dune area of the western netherlands. 1993.

[53] Katsutoshi Tanimoto, Tadahiko Yagyu, and Yoshimi Goda. Irregular wave tests for composite breakwater foundations. In *Coastal Engineering 1982*, pages 2144–2163. 1982.

[54] J.W. Van der Meer. *EurOtop*, 2nd edition, dec 2018. URL `www.overtopping-manual.com`.

[55] HWJ Van Dijk. Invloeden van oppervlakte-infiltratie ten behoeve van duinwaterwinning op kruidachtige oevervegetaties. 1984.

[56] B. van Weeren, J. Beersma, Hakvoort H. Peerdeman, K. and, and M. Talsma. Nieuwe neerslagstatistieken voor korte tijdsduren. 2018.

[57] Reglement verkeersregels en verkeerstekens). -. 1990.

[58] MZ Voorendt. Design principles of multifunctional flood defences. 2017.

[59] et. al. W.A. Johnston. Freezing and refrigerated storage in fisheries. 1994.

[60] Wagemans. *Quick Reference Guide*. 2004.

[61] Wareco Engineers. Grondwaterstand gemeente den haag. 2020.

[62] Ronald E Waterman. Integrated coastal policy via building with nature. 2010.

[63] Xie. Scour patterns in front of vertical breakwaters. Technical report, 1981.

[64] ZKA Consultants & Planners. Achtergrondrapport verkeer scheveningen-haven. 2011.