Brouwershaven

Is there a necessity to adapt the harbour constructions in the harbour of Brouwershaven, or to secure them against the reduced tide in the Grevelingen lake ?

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By

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In partial fulfilment of the requirements for the degree of

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Preface

This Master's thesis report presents my work in partial fulfilment of the requirements for the degree Master of Science in Hydraulic Engineering at the Delft University of Technology. I researched the necessity to adapt the harbour constructions in the harbour of Brouwershaven, or to secure them against the reduced tide in the Grevelingen lake? The request for this research came from the Gemeente Schouwen-Duiveland and was carried out in cooperation with Aquavia.

First of all I would like to thank my graduation committee consisting of Jarit de Gijt (TU Delft), Tiedo Vellinga (TU Delft) and Jules Verlaan (TU Delft) for their guidance during my master thesis. Also special thanks to Gilbert van Noorden who supervised the first part of my research in cooperation with Aquavia. Furthermore I want to thank the people who work at the archives of Schouwen-Duiveland and Middelburg for the help in the search for specifications and technical drawings. As well as to Rijkswaterstaat for their information on the plans of the new reduced tide in the Grevelingen lake. I'm also thankful to all my colleagues at Aquavia for their input into the research and for creating a nice environment to work on my thesis. At last I would like to thank my parents for the encouragements and unconditional support to pursue my dreams.

I hope that you enjoy reading this report.

Fons De Vlieger

Sas van Gent, 10/07/2018

Summary

After the big flood in 1953 the Grevelingendam and the Brouwersdam were built as a part of the 'Deltawerken'. By constructing these dams the Grevelingen was separated from the North Sea, which created the largest salt water lake in Europe. Several decades later it was discovered that during hot summers the deeper areas of the lake were leaking oxygen. This leads to a massive mortality of the fauna and flora living in these depths. Since this area is spreading to the shallow areas it was decided by Rijkswaterstaat to bring back a reduced tide into the Grevelingen lake.

The idea is to bring this reduced tide back by constructing a sluice caisson or tidal power plant into the Brouwersdam. This tidal range was determined in a way that the fauna and flora on the islands could remain. Another problem that arises with this reduced tide is that it is unknown what the consequences are for the harbours around the Grevelingen lake and their structures. Brouwershaven specifically gets its income from the harbour and its tourism. This made the Gemeente Schouwen-Duiveland ask to investigate the consequences of a potential reduced tide in its harbour. This led to the following research question:' **Is there a necessity to adapt the harbour constructions in the harbour of Brouwershaven, or to secure them against the reduced tide in the Grevelingen lake?'.**

This research was started by investigating the different boundary conditions such as:

• Wind	1,54 m/s Southwest
• Occurring water levels	+0,7 m NAP and -0,5 m NAP
• Not exploded explosives	Not taken into account
• Soil structure	Exists mainly of clay and peat, with a thick sand layer at -16 m NAP
• Profile of the harbour bottom	Design level of the harbour bottom at -2,75 m NAP
• Shipping	Limiting factors: ship draught of 2 m and length of 14 m
• Flow rate through the guard lock	In case of tidal power plant: 0,154 m/s In case of sluice caisson: 0,0719 m/s

The new part of the harbour was designed after the closure of the Grevelingen. This is why the option was to check the stability of the structure in this part of harbour. At the end of the calculation it turned out that there was no danger for the structures to become unstable by the reduced tide. However, there is a statistical probability that the scaffoldings as well as the quay wall will be flooded once in a hundred years. The bigger problem that was found was the accessibility of the harbour. The harbour is now only accessible for ships with a draught of 2 m at a water depth of 2,5 m. Which at a lower water level would cause problems to safely enter and manoeuvre in the harbour.

In the search for a solution a brainstorm session was held with the construction company 'Aquavia'. With the help of a multi criteria analysis (MCA) it was found that the best solutions were:

- Construction a new harbour in front of the guard lock
- Creating a new function for the existing harbour and shifting the harbour function to a new location in front of the guard lock
- Demolition of the sills in the guard lock and dredging the harbour to a deeper level

In consultation with 'Gemeente Schouwen-Duiveland' it was decided to design the first and the last bullet in more detail.

The first variant that was dealt with was that of the demolition of the sills in the guard lock and the dredging of the harbour. The idea here was to lower the bottom of the harbour and the guard lock to at least a level of -2,75 m NAP, which produces a volume of 5143 m^3 of material such as silt to be dredged away. Which includes the possibility of:

- Finding not exploded explosives
- The quay walls of the oldest part of the harbour becoming unstable.

Also the stability of the guard lock construction after removing the sills had to be checked. This unfortunately was not executed due to the lack of technical data and drawings of the reinforcement. Finally an estimation of 300.000 EUR was made to realise this variant.

The idea for the second variant is to leave the harbour behind the guard lock in the state it is currently in and to construct a new harbour in front of the guard lock. In this way smaller ships can still use the old harbour whereas the ships that cannot enter the harbour anymore can moor in the new harbour as well as even larger ships. In this new harbour then there would also be a place to moor the fishing boats as well as a river cruise ship. Because of strict time scheduling it was decided to only design one of the important structures of the harbour, namely the harbour mole. For this design there were 2 variants to take into account. In the first variant the total mole construction (breakwater + the pier) was made of wood, whereas in the second variant only part of the breakwater was made of wood. The pier, however, was made of concrete. Finally it was estimated that the construction of the new harbour would cost 7 million EUR. Which is a big difference compared to the price estimation of the demolition of the sills in the guard lock. Both variants have their pros and cons. By demolishing the sills and dredging the harbour to a lower level the problem of the harbour is resolved while a smaller/ more optimised version of the other variant could enable more future prospects to be worked out for the harbour by increasing the capacity and attracting new functions to the harbour. This could of course increase the harbour profits.

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Nomenclature

S	The total rise of the water level due to wind action	$[m]$
C_2	Constant $\approx 3{,}5 \cdot 10^{-6}$ to $4{,}0 \cdot 10^{-6}$	$[-]$
d	Water depth	$[m]$
u	Wind speed	$[m/s]$
s_x	Standard deviation	$[unit]$
x_i	The value of a number in the series	$[unit]$
$\bar{x}$	The average of all numbers in the series	$[unit]$
n_x	The number of numbers in the series	$[-]$
K_s	The shoaling coefficient	$[-]$
k	Wave number	$[-]$
d	Water depth	$[m]$
R_c	Freeboard	$[m]$
q	Specific discharge	$[m^3/m\,/s]$
H_{m0}	Wave height	$[m]$
γ_β	Angle of incidence	$[-]$
h_0	Rise of the still water level	$[m]$
L	Wave length	$[m]$
d	Depth	$[m]$
p	Pressure	$[kN/m^2]$
k	Wave number	$[-]$
H_{in}	Incoming wave height	$[m]$
ρ	Mass density	$[kg/m^3]$
g	Gravitational acceleration	$[m/s^2]$
N	Normal force	$[N]$
M	Moment	$[Nmm]$
A	Area	$[mm^2]$
W	Moment of resistance	$[mm^3]$
$f_{t,0,k}$	Characteristic tensile stress into the longitudinal direction	$[N/mm^2]$
$f_{c,0,k}$	Characteristic compressive stress into the longitudinal direction	$[N/mm^2]$
$\sigma_{v,d}$	Design shear stress	$[N/mm^2]$
f_{vk}	Characteristic shear stress	$[N/mm^2]$
k_{mod}	Modification factor	$[-]$
γ_M	Material factor	$[-]$
$q_{c,mean}$	Mean cone resistance	$[MPa]$
$p_{r,max,tip}$	Maximum tip resistance	$[MPa]$
$p_{r,max,shaft}$	Maximum s shaft resistance	$[MPa]$
$F_{r,max,tip}$	Maximum tip load	$[kN]$
$F_{r,max,shaft}$	Maximum shaft load	$[kN]$
α_p	Pile class factor	$[-]$
β	Factor for the shape of the pile's foot	$[-]$
s	Factor influence shape of the cross-section	$[-]$
$\emptyset$	Diameter	$[mm]$
γ_b	Volumetric weight of the reinforced concrete	$[-]$
δ	Angle of friction between pile and soil	$[\text{degrees}]$
K_0	Neutral coefficient of earth pressure	$[-]$
$F_{s,nk}$	Negative shaft friction	$[kN]$
v_s	Speed of ship	$[m/s^2]$

$\sigma_{c;0;d}$	Design compression stress into the longitudinal direction	$[N/mm^2]$
k_b	Breakpoint factor	$[-]$
λ	Slenderness	$[-]$
λ_{rel}	Relative slenderness	$[-]$
l_k	Buckle length	$[m]$
$f_{m;k}$	Characteristic bending stress	$[N/mm^2]$
$f_{t;0;k}$	Characteristic tension stress into the longitudinal direction	$[N/mm^2]$
$f_{t;90;k}$	Characteristic tension stress into the transverse direction	$[N/mm^2]$
$f_{c;0;k}$	Characteristic compression stress into the longitudinal direction	$[N/mm^2]$
$f_{c;90;k}$	Characteristic compression stress into the transverse direction	$[N/mm^2]$
$E_{0,mean}$	Mean MVE longitudinal	$[N/mm^2]$
$E_{0,05}$	5% MVE transverse	$[N/mm^2]$
$E_{90,mean}$	Mean MVE transverse	$[N/mm^2]$
G_{mean}	Mean shear modulus	$[N/mm^2]$
β_c	Initial curvature of the bars	$[-]$
I	Moment of inertia	$[mm^4]$
i	Radius of inertia	$[-]$
ρ_{min}	Minimal reinforcement	[%]
f_{cd}	Design concrete compression stress	$[N/mm^2]$
$f_{t;k}$	Characteristic tension stress	$[N/mm^2]$
$f_{y;k}$	Characteristic yield stress	$[N/mm^2]$
f_{ctm}	Mean concrete tension stress	$[N/mm^2]$

List of Figures

1 Introduction

1.1 Problem with the Grevelingen lake

Through the construction of the Grevelingendam in 1965, and of the Brouwersdam in 1971 the Grevelingen was isolated from the North Sea. Thus the largest saltwater lake of Europe came into existence, i.e. the Grevelingen lake. As the tide and the current in the Grevelingen lake had now disappeared it appeared that during hot summers an oxygen shortage was created in the deeper areas of the lake. The subjoined figure represents this oxygen shortage.

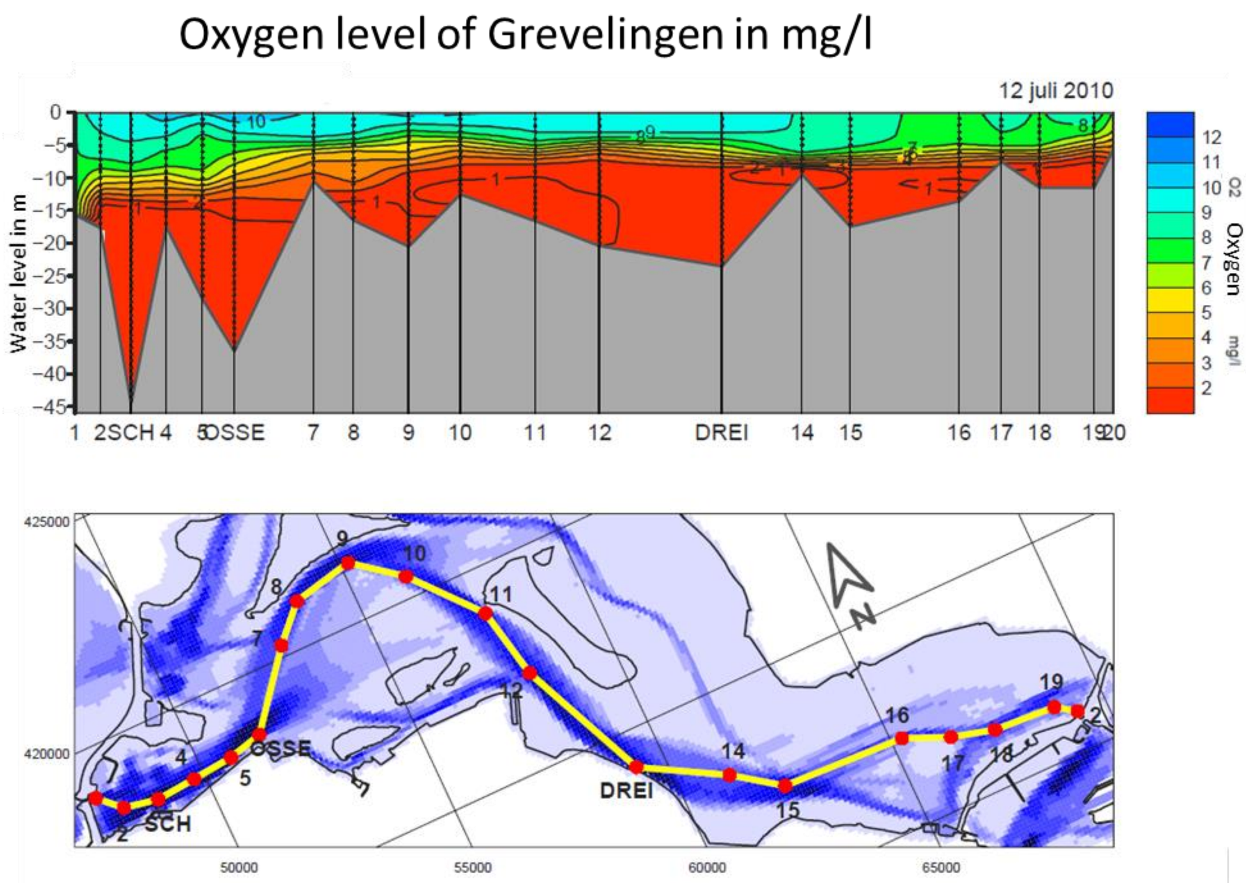


Figure 1: Oxygen level Grevelingen lake[1]

This oxygen shortage leads to a massive mortality in fauna and flora living in these depths. Once these rests have died they start to rotten, which results in a kind of a white layer lying over the bottom and a smell of rotten eggs which can be perceived at the water surface. In recent years it was established that the area suffering from oxygen shortage has been increasing, also affecting more shallow areas. Photographs showing the results of this effect have been established in figure 2. This of course also has economic consequences on the surrounding municipalities and recreative resorts, as through this situation this region becomes less attractive to tourists, and more specifically, to divers.

[1] presentation 'Getij op de Grevelingen', Rijkswaterstaat Zee & Delta, slide 2

Figure 2: Photographs of fauna and flora mortality and rotting material at the bottom of the Grevelingen lake[2]

[2] Photos taken by Bas van der Sanden

1.2 Sluice caisson/ tidal power plant

In 2014 the abovementioned cause made the Dutch government decide to start the project called ‘Bringing back the tidal process restores water quality in the Grevelingen and Volkerak-Zoom lake’. This document can be found in appendix 1.

To solve this it was decided to create a sluice caisson in the Brouwersdam. Thus the Dutch authorities of *Rijkswaterstaat* (hereafter simply referred to as ‘Rijkswaterstaat’) maximally aim at creating a natural tide in the Grevelingen lake as shown in the subjoined figure.

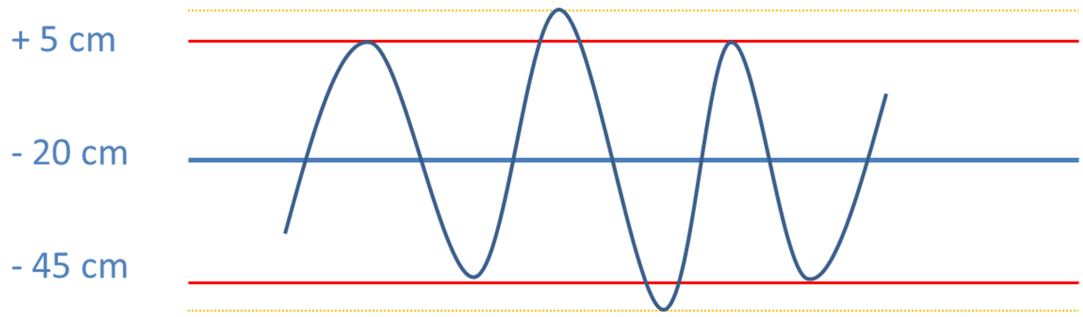


Figure 3: Desirable tidal course in the Grevelingen lake [3]

In figure 3 Rijkswaterstaat indicated that the tide within the red line boundaries will certainly be reached, still discussing the possibility of the slipping’s attaining the yellow line boundaries at this moment as these could possibly create a problem in the pushback of the sweet water bubbles under the various small islands covering the Grevelingen lake. If these sweet water bubbles vanished, the existing fauna and flora equally would, resulting in the loss on these islands of, amongst others, the rare ‘groenknolorchis’.

Regarding this Rijkswaterstaat established a specification of questions including its claims, desires and boundary conditions connected to the project of the sluice caisson construction, which can be seen in appendix 2. This survey shows how the average water level in the Grevelingen lake should remain at -20 cm NAP, permitting the tide to deviate 25 cm from this level, creating the need for a sluice caisson construction in the northern part of Brouwersdam, as can be seen in figure 4.

Figure 4: Photographs of the bottom of the Grevelingen lake[4]

[3] Presentation ‘Getij op de Grevelingen’, Rijkswaterstaat Zee & Delta, slide 13
[4] www.maessenweb.nl/archives/getij-energie-in-de-etalage-van-de-bv-nederland

By means of this questions' specification companies are now allowed to make a project for the sluice caisson construction of a possible tidal power plant. Either of these constructions will affect the tide created in the Grevelingen lake differently. This effect can be seen in figure 5.

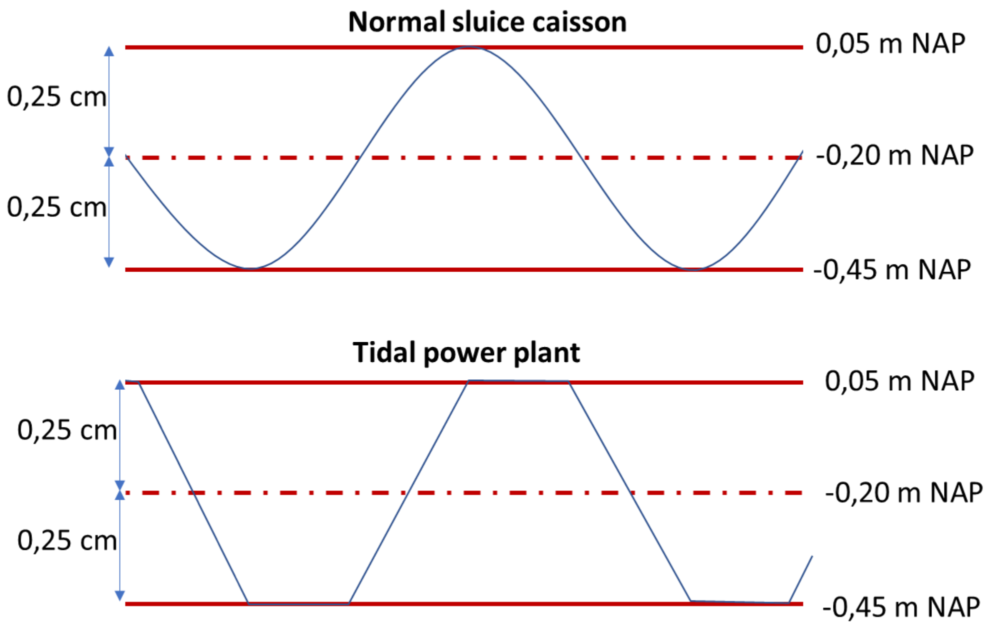


Figure 5: Tidal impact in the Grevelingen lake when constructing either a sluice caisson or a tidal power plant

Through constructing the common sluice caisson the tide desired by Rijkswaterstaat can be imitated perfectly. In order to reach an optimal functioning of the tidal power plant a maximal difference in water level is desired over the Brouwersdam. This can be attained by keeping the water in the Grevelingen lake at the most extreme levels for a longer period of time, thus attaining the abovementioned tide. By decreasing the run-through time and by enlarging the water level differentiation the water will flow in and out of the Grevelingen lake, as well as of the harbour of Brouwershaven.

Even knowing that Rijkswaterstaat is not really consenting the idea to have the tidal power plant keep the water in the Grevelingen lake for a longer span of time, the subject leaves room for discussion. For Rijkswaterstaat has detected the following important advantage with the tidal power plant :

By installing turbines in the sluice caisson openings the flowing through diameter for the water to flow freely is reduced. In order to have the same water quantity flow in and out of the Grevelingen lake the tidal power plant will have to be constructed on a larger scale than the common sluice caisson. This can be seen in figure 6. For Rijkswaterstaat is now afraid that regarding the increase of the sea level the common sluice caisson, after having been used for some 30 years, will no longer be capable of flushing the water of the Grevelingen lake sufficiently. Which will create a tendency to have the average level on the Grevelingen lake increased. As the tidal power plant is constructed on a larger scale it is possible to decide to extract the turbines out of the sluice caisson openings after 30 years, thus automatically creating a larger sluice caisson as can be seen in the bottom image of figure 6. Choosing for this solution the water level of the Grevelingen lake could be kept at -0,20 m NAP for a longer span of time.

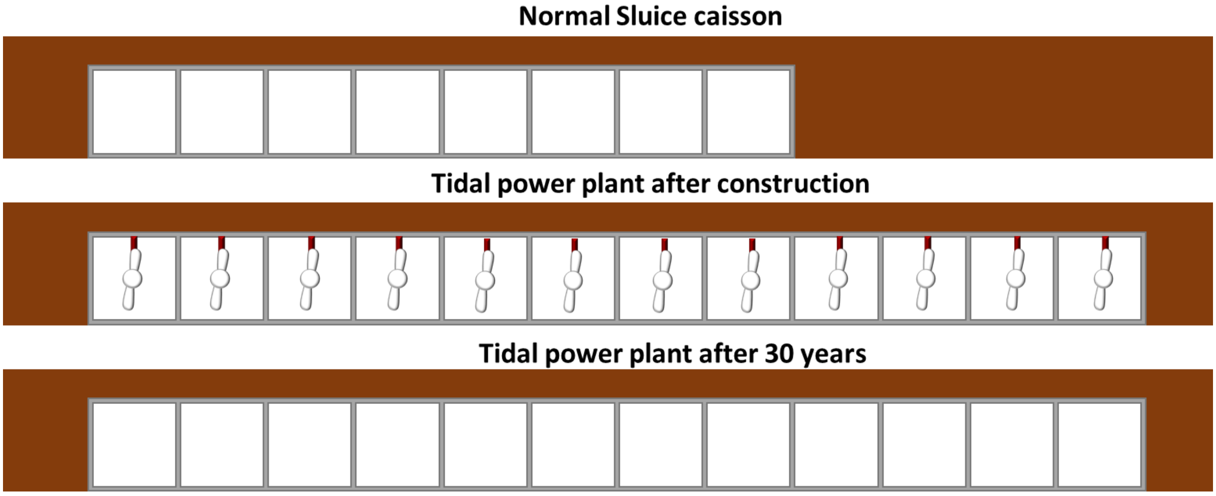


Figure 6: Diagrams of the tidal power plant advantage

As we already showed in this paragraph another organisation already investigated the ecological consequence of the return of the tidal process in the Grevelingen lake. Discussing this with Rijkswaterstaat it was stated that so far no surveys have been conducted to reveal what the consequences could be that the harbours situated around the Grevelingen lake might have. The municipality of Schouwen-Duiveland was asked to conduct this survey for the harbour of Brouwershaven. Brouwershaven is situated on the island of Schouwen-Duiveland, which lies between the Grevelingen lake and the Oosterschelde.

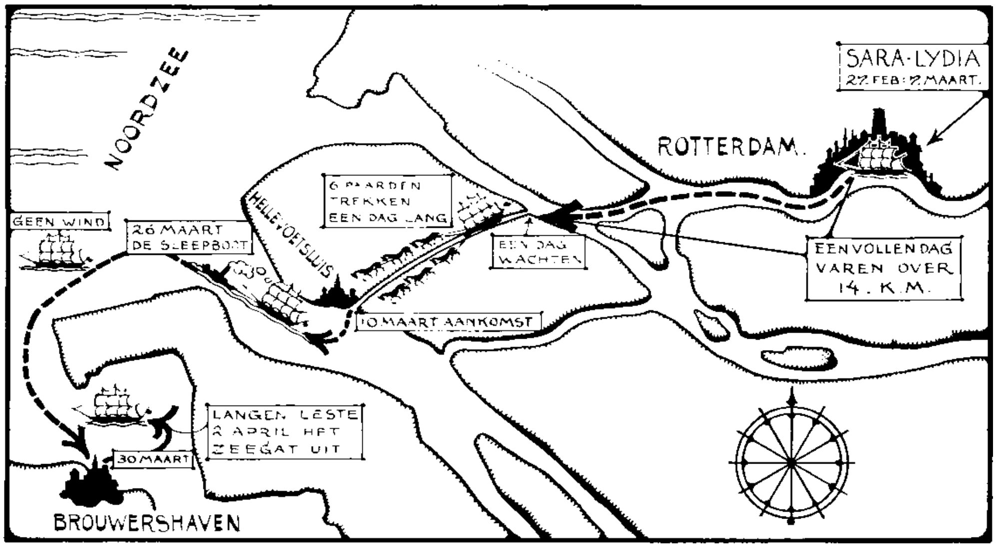
Figure 7: Site of the research project

1.3 History of Brouwershaven[5]

Figure 8: Map of Brouwershaven 1545 Jacob van Deventer

During the second half of the Middle Ages Jan van Renesse bought 41 hectares of land on Schouwen. On this estate a scouring sluice was built, together with a real harbour, in those days called ‘Brijdorpsluis’. Around 1285 count Floris V was asked permission by the Cleaszoon brothers as to build a village around the harbour. Quickly after this the first houses and barns were built, after which also started the construction of the gothic church. The most important revenues of Brouwershaven mainly were fishing and trading wine, beer, wood, stone, beets etc., which resulted in a rapidly growing town. Thus the quaysides became increasingly long and the number of warehouses enhanced. In 1403 the town of Brouwershaven was granted city rights. However, as these rights never received into the ‘Staten van Zeeland’, Brouwershaven remained to be called a “smalstad” until the present day. Until the nineteenth century Brouwershaven experienced many ups and downs. The size of the ships increased for example, which caused these to be no longer capable of entering the harbour. This resulting in these ships choosing for larger harbours such as Rotterdam. As a consequence Brouwershaven threatened to decay. Because the ‘Goereese Gat’ and the ‘Briele Maas’ increasingly silted up during the nineteenth century Rotterdam became unaccessible for large and heavily loaded sailing ships. As a consequence of this Brouwershaven flourished anew as a transhipment port. This resulted in increasing business life through the introduction of pilotage, tax administration and a large workshop for the construction of buoys, which were used for tracing out the fairway towards Brouwershaven. Even passengers on their way to India embarked only in Brouwershaven. This was also caused by the fact that in most cases the journey by land was more pleasant than traveling by inland waterways. In case of adverse wind the latter could easily take many days as presented in figure 3.

[5] www.stadsraadbrouwershaven.info/de-geschiedenis-van-brouwershaven/
www.digitaalbrouw.nl/geschiedenis/smalstad.htm



In het jaar [illegible] had het zeilschip „Sara Lydia" vijf weken noodig, om van de Boompjes in open zee te komen. Brouwershaven lag aldus halfweg Rotterdam – Java.... Geteekend door Adriaan van der Plas

Figure 9: Presentation of the traveling time span from Rotterdam to Brouwershaven by ship[6]

The increase of passengers also resulted in a rapidly growing number of pubs in Brouwershaven, i.e. 26 in total. Some of the larger houses were even reconstructed and turned into hotels as well. However, the creation of the 'Nieuwe Waterweg' near Rotterdam put an end to this period of prosperity.

On 1 February 1953 also Brouwershaven suffered from the inundation disaster, which brought about a lot of damage and made 3 victims here. In 1957 at first, and as a consequence of this disaster, the guard lock at the harbour's entrance was built. This guard lock was meant to block the high water levels during storm surge situations and was rapidly degraded to become a secondary flood defence in 1970 through the construction of the Grevelingendam and the Brouwersdam, which were part of the 'Deltawerken', having to secure the Zeeland delta region against high seawater levels.

After the inundation disaster the harbour regained its vitality, amongst other things through laying out a new yacht harbor and stimulating tourism. Until today this still is the economic backbone of Brouwershaven.

Through the construction of the sluice caisson complex or tidal power plant, as described in the previous chapter, the harbour will be exposed to a 50 centimetres' tide in the future. This means that the following exceptional water levels can possibly occur :

- +0,3 m NAP once every 10 years
- +0,5 m NAP once every 100 years
- +0,7 m NAP once every 1000 years

However, it is not known what the consequences would be for the harbour (constructions) of Brouwershaven this time.

[6] Dr.ir J.G. de Gijt, CIE5313 Lecture: History of quay walls and principle cross sections, slide 10

2 Problem description

During the latest 47 years the water surface in the Grevelingen lake has been kept at a fixed level already. This is why most harbour constructions in the harbour of Brouwershaven have been built respecting this water level. The introduction of a reduced tide of half a meter could constitute a problem for the well-functioning of the existing harbour (constructions).

The goal of this report is to try and answer the following question : **' Is there a necessity to adapt the harbour constructions in the harbour of Brouwershaven, or to secure them against the reduced tide in the Grevelingen lake ?'** We do so by checking whether the various harbour constructions and the harbour itself will need either potentially necessary adaptations or will still be capable of meeting their formerly established requirements, desires and boundary conditions. As a final result we will present a project explaining how to adapt the harbour to the reduced tide.

3 Requirements, boundary conditions and desires

In order to establish the programme of requirements, desires and boundary conditions various subjects were investigated. These subjects will be tackled rapidly in this chapter, after which we present a survey of the requirements, desires and boundary conditions.

3.1 Wind

In order to have a general view of the prevailing wind direction over the Grevelingen lake we made use of the wind data of the Ouddorp measuring station. These data and their locations are presented in figure 5. The compass rose tells us that the wind's main direction is southwest. Its average speed throughout the year is 3kts or 1,54 m/s.

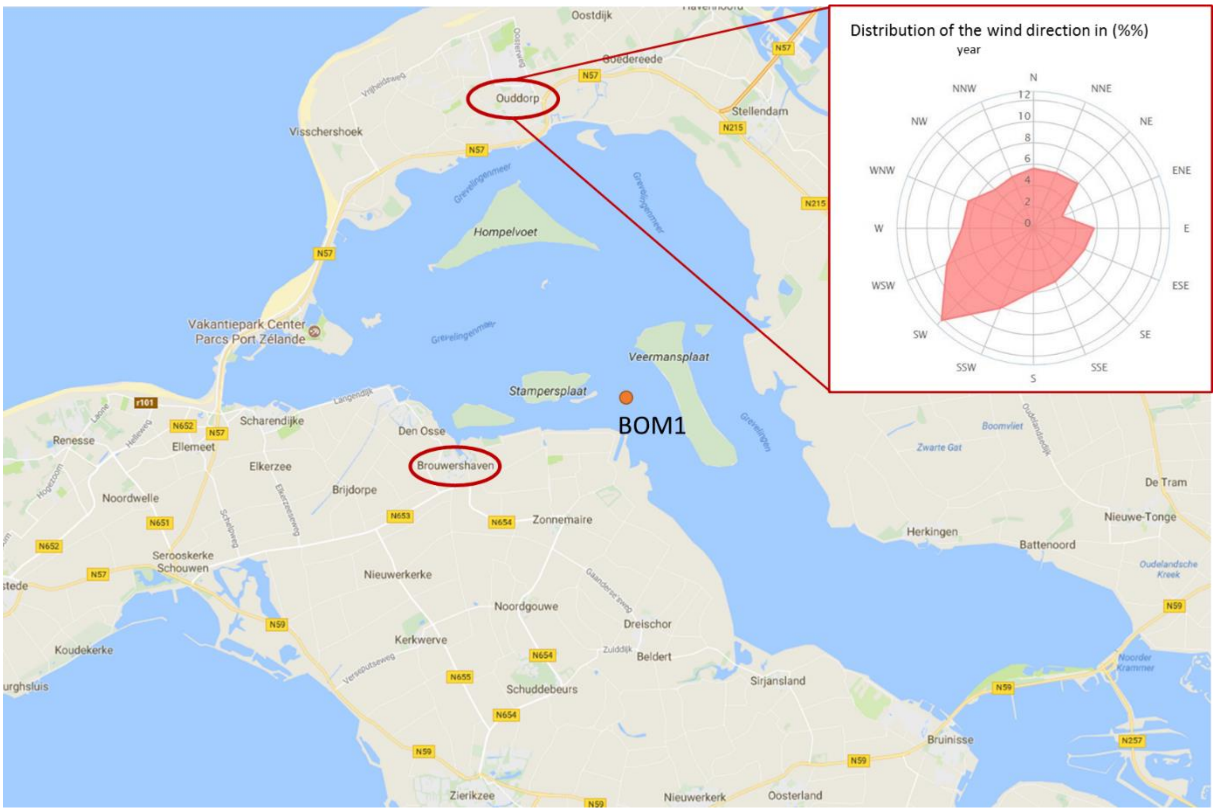


Figure 10: Location measurement Ouddorp (Compass rose)[7]

3.2 Occurring water levels

In order to determine the occurring water levels in the harbour of Brouwershaven various influencing factors were investigated. The effect of these factors will be treated in the subjoined chapters.

3.2.1 Determine the reduced tide

To be able to determine the reduced tide in the harbour of Brouwershaven a number of programmes were written in the python programming language. The complete description of the composition of these programmes, as well as the programme code itself can be found in appendix 3.

[7] https://nl.windfinder.com/windstatistics/ouddorp

3.2.1.1 The data

As starting data we chose to start with the North Sea water level for the year 2016 in the surroundings of Brouwersdam. We could find this level on the website of Rijkswaterstaat[8] under the denomination of 'brouwersdam buiten'. A minor part of these data is presented in figure 15. In order to obtain these data an every 10 minutes' measurement was conducted during the complete year. We started to extract some important data from these measurements, such as the average low and high tide water levels. These data were obtained using the programme from appendix 3.

- Average low tide : -0.89 m NAP
- Average high tide : 1,48 m NAP
- Average water level : 0,29 m NAP
- Average tide : 2,43 m

```
20160101 0000 20160131 2350 10
brbu WT_S_2
           1
01-jan-2016 00:00          -57
01-jan-2016 00:10          -61
01-jan-2016 00:20          -66
01-jan-2016 00:30          -70
01-jan-2016 00:40          -68
01-jan-2016 00:50          -71
01-jan-2016 01:00          -64
01-jan-2016 01:10          -60
01-jan-2016 01:20          -61
01-jan-2016 01:30          -65
01-jan-2016 01:40          -64
01-jan-2016 01:50          -64
01-jan-2016 02:00          -64
01-jan-2016 02:10          -62
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Figure 11: Data of the water levels in front of Brouwersdam

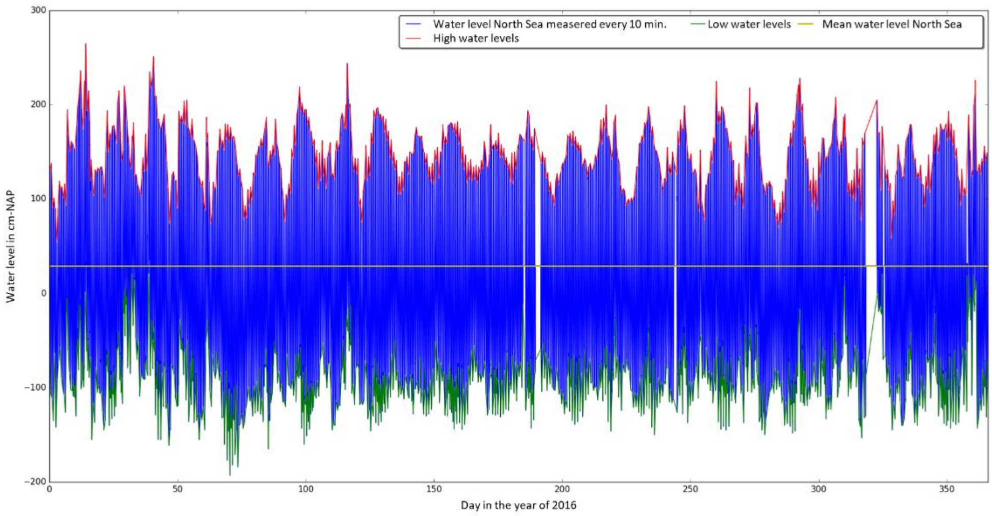

Figure 12: Presentation of the tide in 2016 for Brouwersdam

As is shown in the figure 16 above white areas appear in the tide data. These were periods when the measuring equipment did not function and thus no measurements were established. As this could possibly provoke problems for the well-functioning of the programme we chose to complete these data with the average water level. Then the data series obtained was used to determine the reduced tide in the Grevelingen lake and the harbour of Brouwershaven in two different situations. Concretely :

- The construction of the opening in Brouwersdam using a common sluice caisson
- The construction of the opening in Brouwersdam using a tidal power plant

The next step in the programme is determining the reduced tide in the Grevelingen lake. For this we need the dimensions of the sluice caisson construction.

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[8] https://waterberichtgeving.rws.nl/water-en-weer/dataleveringen/ophalen-opgetreden-data

3.2.1.2 The sluice caisson

Rijkswaterstaat already made a project for the sluice caisson construction. This project serves as an example of what this construction could possibly look like. As this also is the only project available at this moment we chose to continue to use this project when determining the reduced tide in the Grevelingen lake. Some of the views and cross-sections of this construction can be found in appendix 4[9]. The subjoined figure presents a survey of the most important parameters.

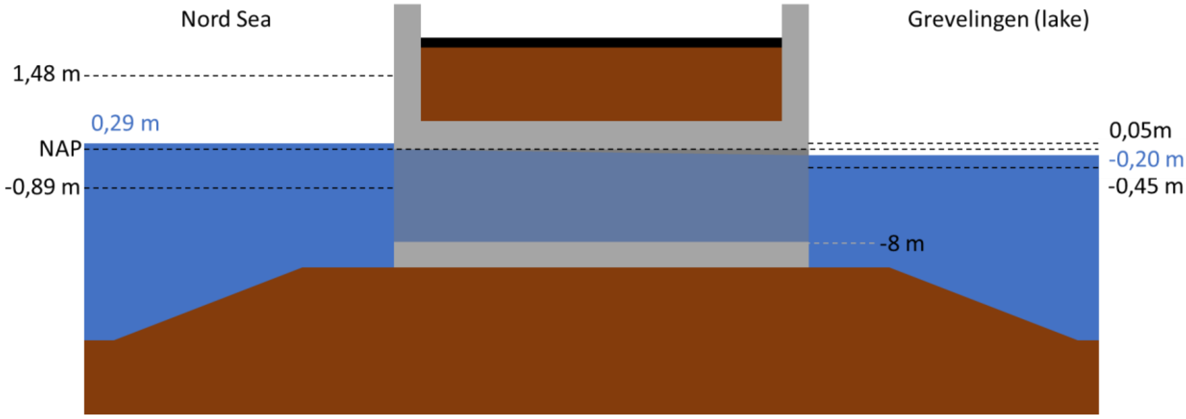


Figure 13: Diagrammatic drawing of the sluice caisson construction indicating the main heights

This construction has its threshold at a depth of -8 m NAP and a transmitting width measuring 120 m. Other important parameters and limits are the following :

- The Grevelingen lake has an average water level of -0,20 m NAP and should not exceed a tide of half a meter (at its maximum)
- The Grevelingen lake has a wet surface of 108 square kilometres

3.2.1.3 Reduced tide with sluice caisson

After this the abovementioned parameters were introduced in the programme in appendix 3, resulting in figure 14. This figure shows the effect of the sluice caisson presented for the first 10 days of 2016 as well as limits that were set to the reduced tide of the Grevelingen lake. What is immediately striking in this figure is that the upper limit of the Grevelingen lake is attained more frequently than the bottom limit. We can also see that the water level for Brouwersdam sometimes fails to sink sufficiently in order to be capable to flush enough water from the Grevelingen lake back to the North Sea.

[9] Presentation ' Getij op de Grevelingen', Rijkswaterstaat Zee & Delta, slide 19

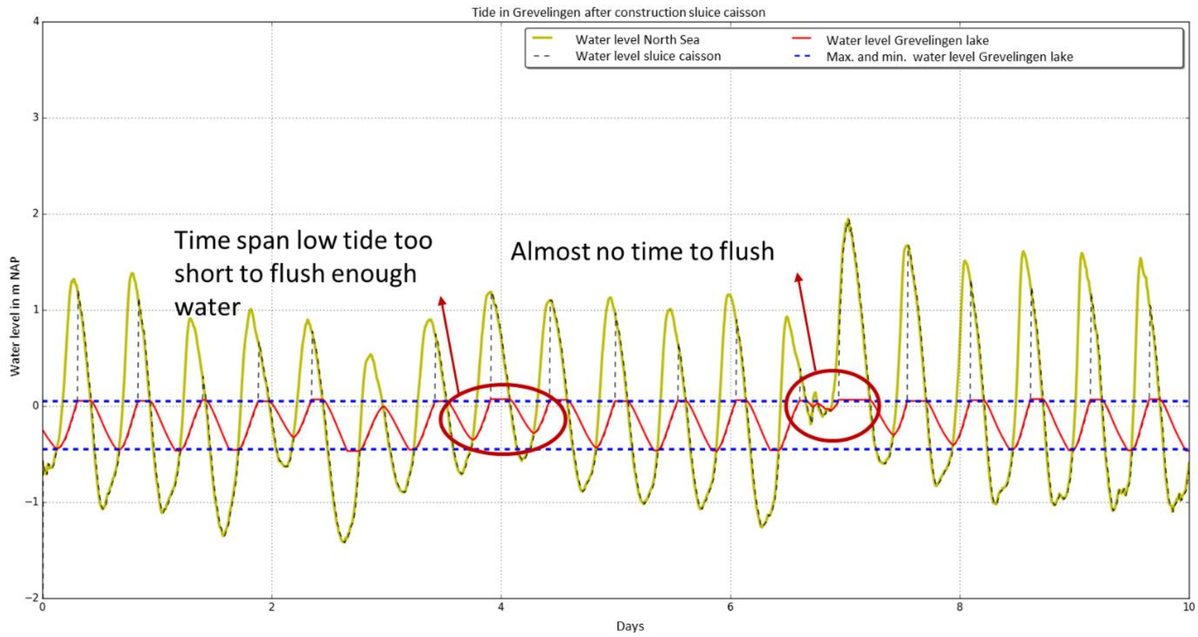


Figure 14: Presentation of the reduced tide in the Grevelingen lake for the year 2016

Now that we determined the tidal course in the Grevelingen lake we can use the same method in order to thus determine the eventual tide in Brouwershaven. This time the depth of the guard lock's sills lies at -2,5 m NAP having a transmitting width measuring 8,9 m. The harbour itself has a wet surface of 0,04 km^2.

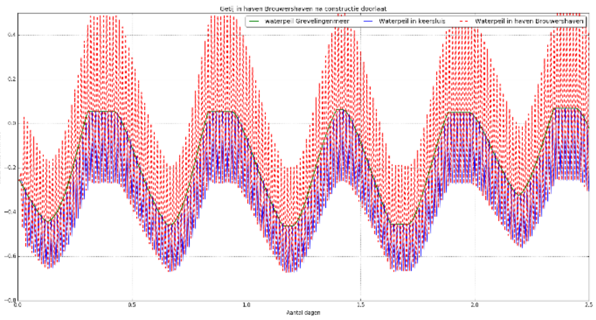


Figure 15: Reduced tide in het harbour of Brouwershaven

Figure 15 shows that the water in Brouwershaven starts to oscillate very strongly around the tide of the Grevelingen lake. This can be linked to the time lapses used for the determination of this tide, which prove to have been taken too largely as well as to the fact that Rijkswaterstaat chooses to create a reduced tide as is presented in figure 3. It was decided to create 'the ideal tide' in the Grevelingen lake in order to be capable thus to present the effect on the water level in the harbour of Brouwershaven. This ideal tide oscillates with 0,5 m around a water level of -0,20 m NAP. In figure 20 we see that the water level in Brouwershaven can easily keep up with the tide in the Grevelingen lake. Only if the diagram is strongly zoomed into a difference appears.

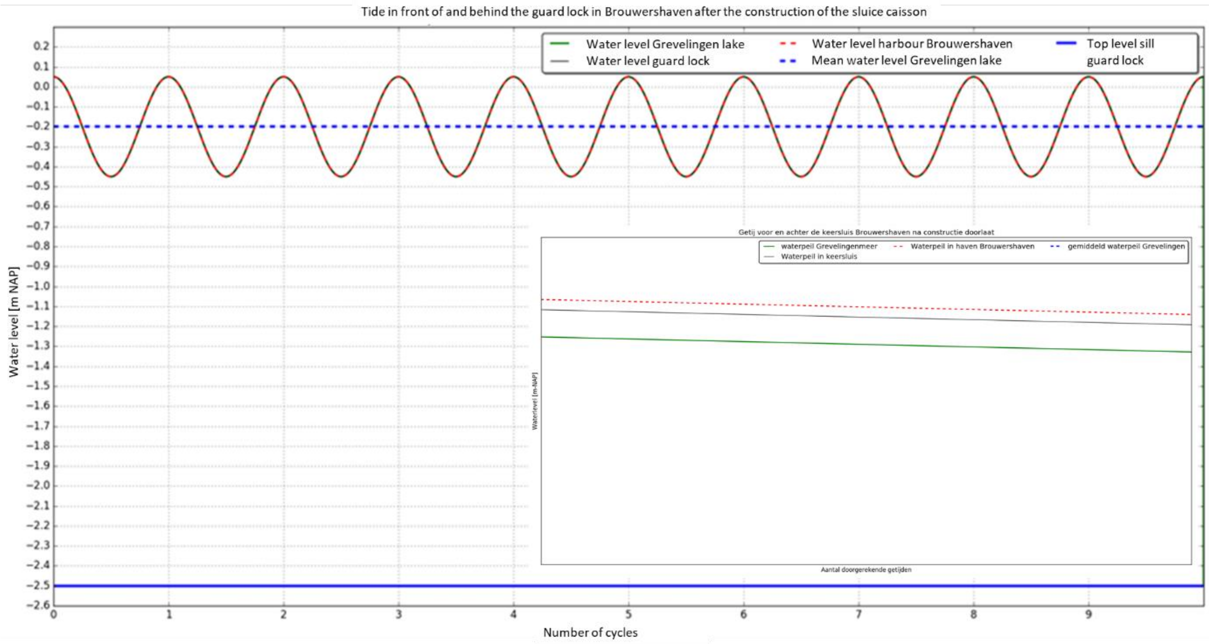


Figure 16: tide in Brouwershaven with common sluice caisson construction

In this document that can be found in appendix 2 it is also described that it is admitted to attain the maximal upper limit of 0,7 m NAP once every 1.000 years. Even in this situation the water level in the harbour of Brouwershaven will be capable of easily keeping up with the tide in the Grevelingen lake. As describes appendix 2 only the bottom limit of -0,50 m NAP is allowed to be exceeded here.

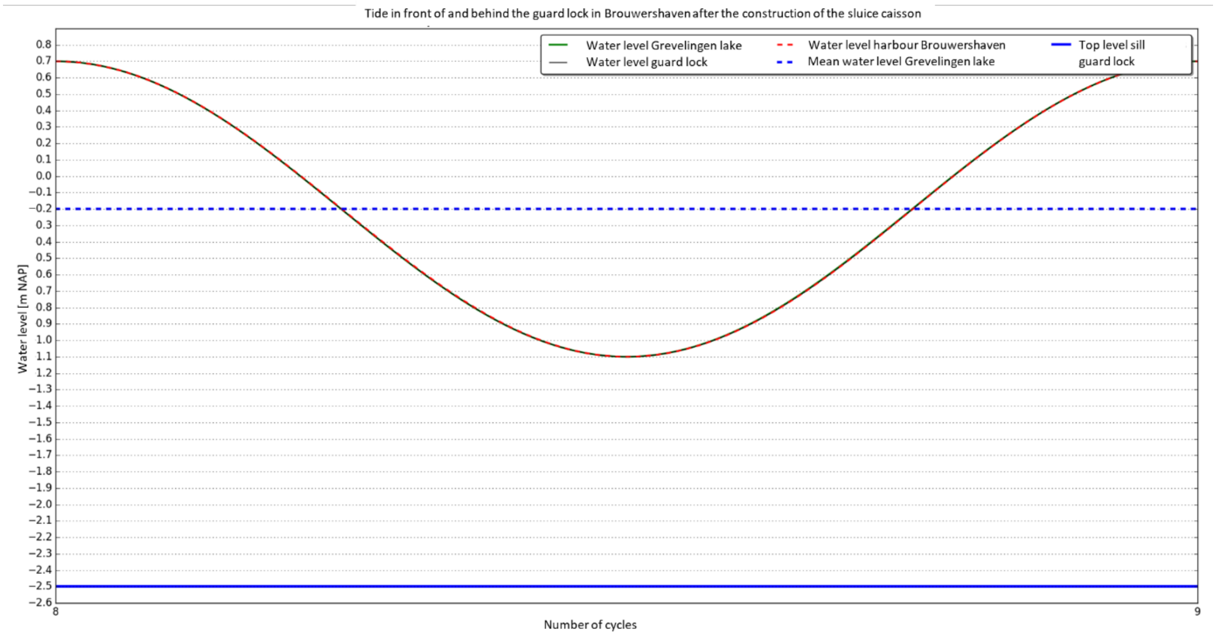


Figure 17: tide in Brouwershaven with construction of sluice caisson and the once in a 1000 years water level

3.2.1.4 Reduced tide with tidal power plant

In order to determine what the tide in the Grevelingen lake and Brouwershaven would look like after the construction of the tidal power plant the previous programme was extended further (appendix 5). This time we precedingly made a model of the average tide in the North Sea for Brouwersdam. After this we watch how the tidal power plant affects the tide. In order to determine the impact the same construction has been used, this time with turbines installed, however. The number of tubes also remains equal as the impact of the turbines on the current is disregarded. A study of variants for a tidal power plant in the Brouwersdam[10] shows that the turbines function optimally with a water level deviation over the tidal power plant of 1,5 m[g]. This explains why we chose to continue to keep the water level in the Grevelingen lake until this deviation is attained. Once this is the case the sliding lids in the tidal power plant can be opened. These then remain opened until the Grevelingen lake reaches its maximal water level.

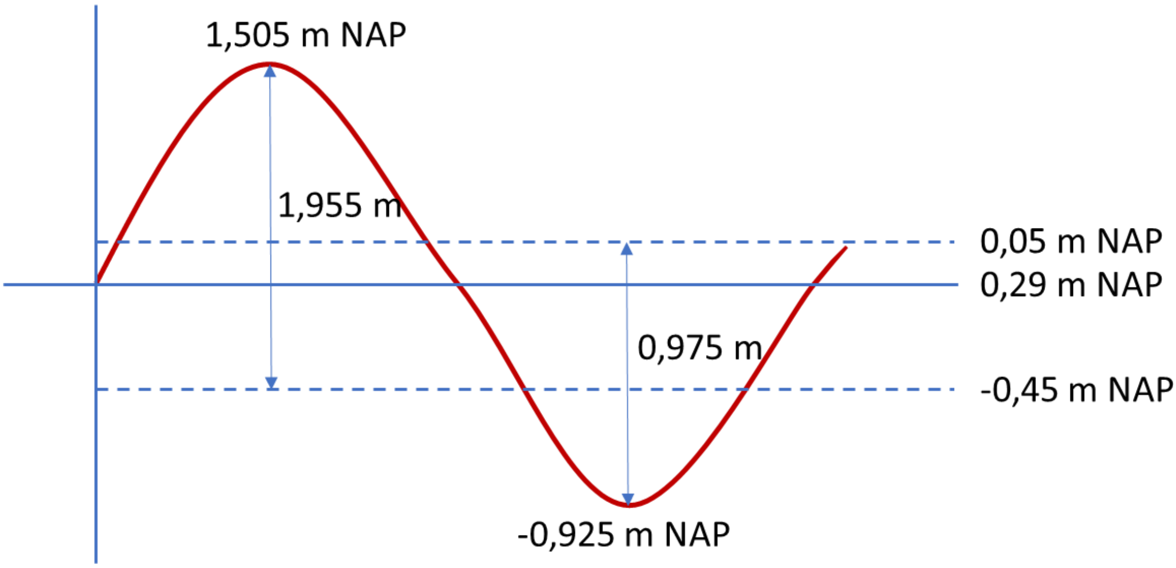


Figure 18: Difference between North Sea high tide and Grevelingen lake low tide and vice versa

When considering the survey of the average tide for the Brouwersdam and the limits attributed to the Grevelingen lake figure 21 shows that during high tide the difference in water level of 1,5 m is attained whereas this is not the case when the tide is low. This is why it was decided not to continue to keep the water level in the Grevelingen lake during low tide. Thus the tide in het Grevelingen lake and the harbour of Brouwershaven is created as presented in figure 22. This figure shows that the reduced tide in the Grevelingen lake never attains the bottom limit. Therefore it was decided to extend the sluice caisson further using 5 complementary tubes having a transmitting capability of 8x8 m each. The result of this extension can be found in figure 23. In this case the bottom limit is effectively attained and thus a reduced tide that oscillates around -0,20 m NAP is attained as well. Also in this situation it turns out that the water level in the harbour of Brouwershaven can keep up with the reduced tide in the Grevelingen lake without any problem.

[10] Leslie Mooyaart & Tom Van Den Noortgaete, Rapport Getijcentrale in de Brouwersdam, Royal Haskoning, bijlage 3

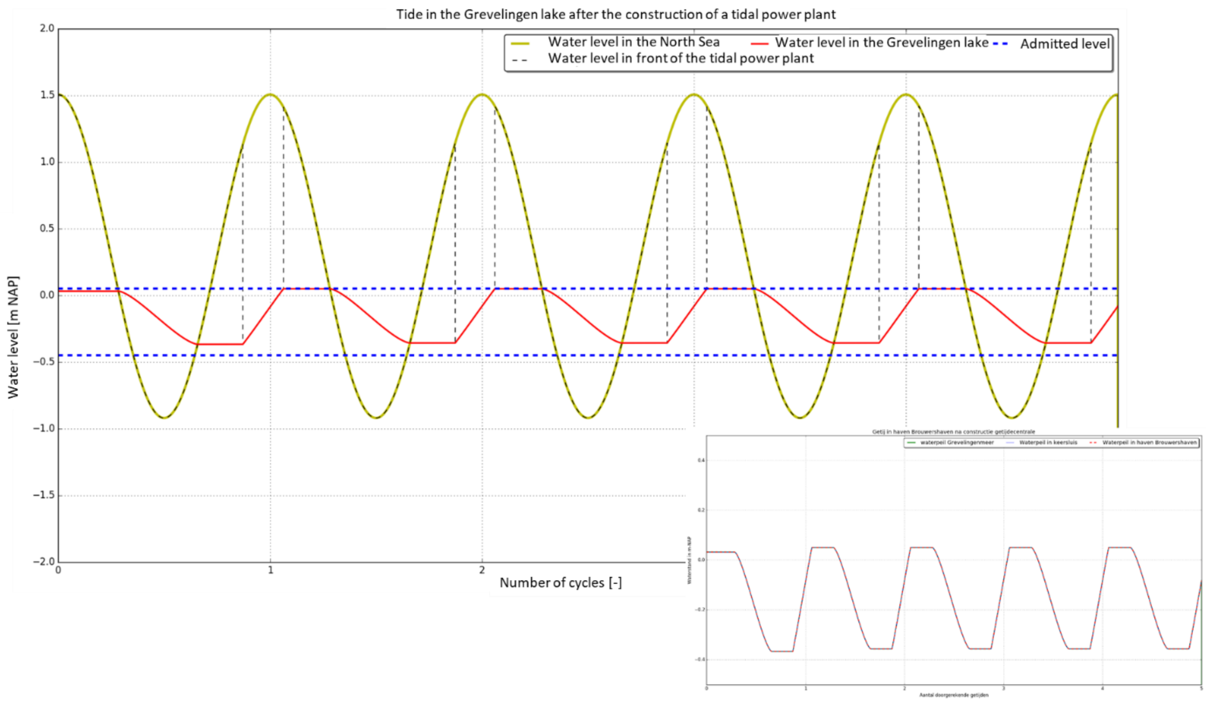


Figure 19: Reduced tide created by tidal power plant

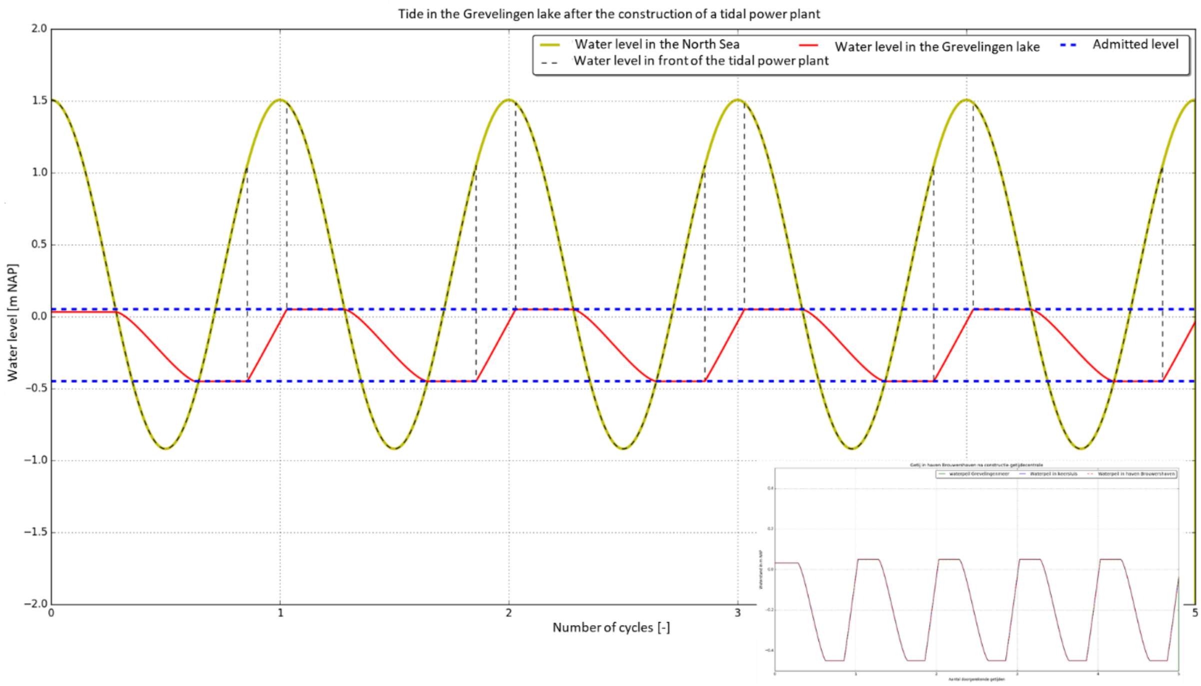


Figure 20: Reduced tide created by tidal power plant with 5 complimentary compartments

The final situation that can be checked is the one in case the sea level would rise.

3.2.2 Sea level rise

No one can any longer ignore the fact of sea level rise. This phenomenon has been going on for years already and will still be there in the coming years. In order to determine how rapidly the sea level in the Netherlands is rising the water level data for Vlissingen from the year 1900 until 2000 were used.

Using python the most appropriate line across these data was drawn. You will find a presentation of this in figure 24.

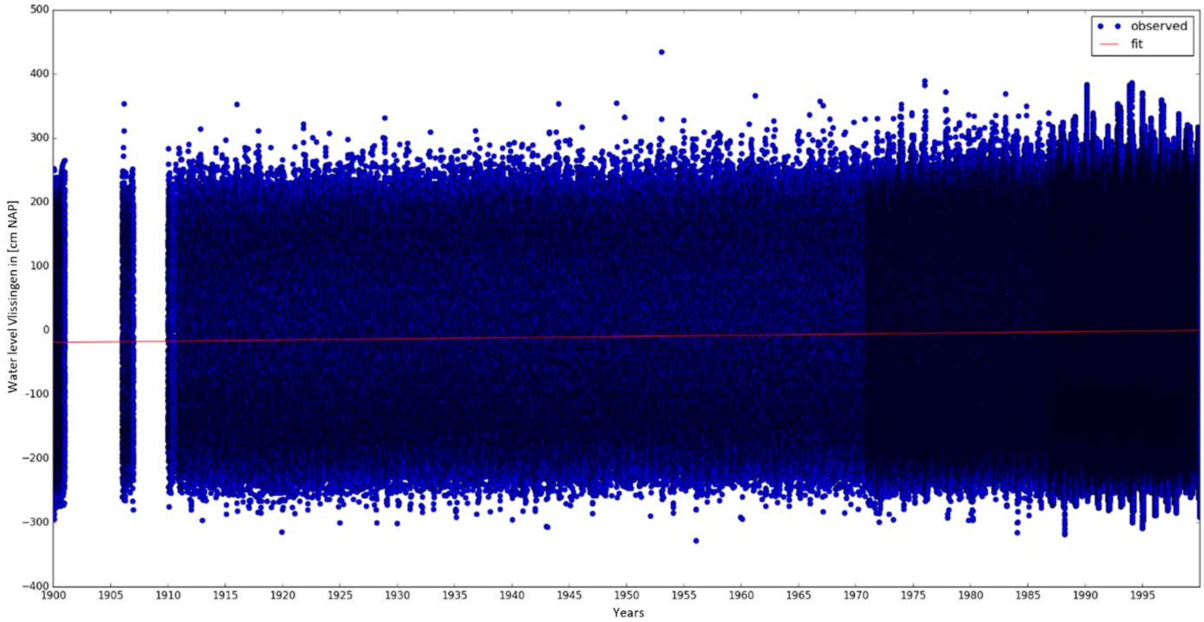


Figure 21: most appropriate line across a 100 years' data

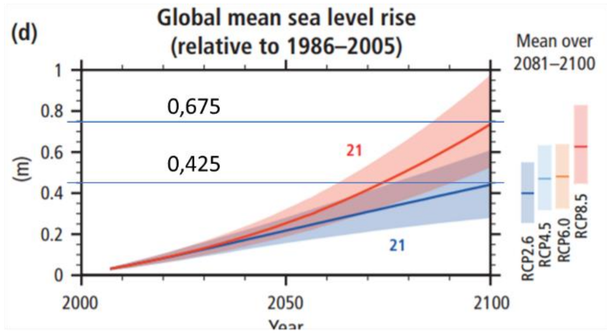
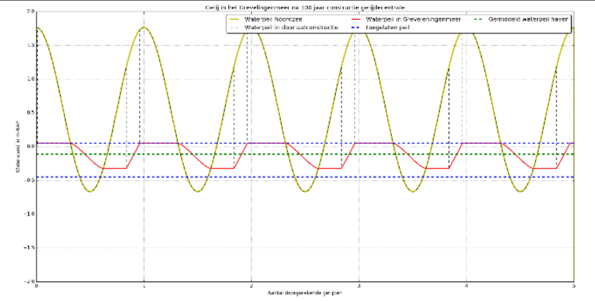
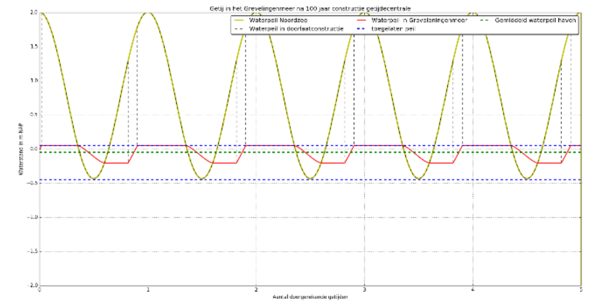


Figure 22: Global mean sea level rise

This line shows that over this period the sea level has risen 18,73 cm in total, which means 0,19 cm per year. When this line is interpolated further until the year 2100 the average sea level turns out to have risen to 17,85 cm NAP by then. If the prediction is taken into account as stated in the 'Climate Change 2014: Synthesis Report'[11] it is found that their models expect a sea level rise that can vary between 1 m and 24 cm. This variation is so large because human impact on global warming and sea level rising as a consequence of this is very unpredictable. This is why it is assumed that the sea level will rise by 55 cm in 100 years from now. Which produces the following situations:

[11] www.ipcc.ch/pdf/assessment-report/ar5/syr/SYR_AR5_FINAL_full.pdf page 59

After 50 years	After 100 years
	
High tide Brouwershaven: 0,05 m NAP Low tide Brouwershaven: -0,32 m NAP Average water level: -0,11 m NAP	High tide Brouwershaven: 0,05 m NAP Low tide Brouwershaven: -0,20 m NAP Average water level: -0,05 m NAP

Rijkswaterstaat does not like these future situations as a weaker tidal process reduces the refreshment of the Grevelingen lake water. The choice here can be to enlarge the sluice caisson construction, which makes it possible to flush the water from the Grevelingen lake back to sea more rapidly. Or Rijkswaterstaat could choose to raise the average water level of the Grevelingen lake. For our further research it was assumed that the sluice caisson will be enlarged in order not to have to raise the average water level of the Grevelingen lake.

3.2.3 Fall and rise of the water level due to wind action[f]

To have an impression of the fall and rise of the water level due to wind action the wind speed occurring during storm was used. These calculations seemed interesting because the measuring station (BOM1) monitoring the water level of the Grevelingen lake is situated almost in the centre of the lake. This is why fall and rise of the water level due to wind action can create higher water levels at the extreme sides of the lake.

Rise of the water level due to wind action

The Rise of the water level due to wind action can be determined using the following formula :

$$\frac{dS}{dx} = C_2 \frac{u^2}{gd}$$

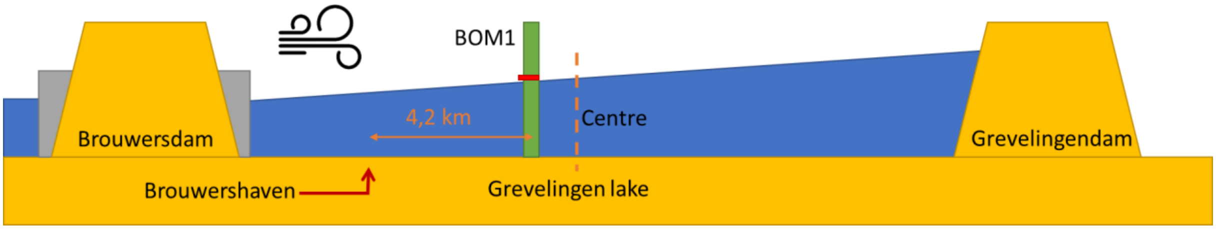
In which the symbols are: (For Grevelingen lake)

- S [m] the total rise of the water level due to wind action
- C_2 [-] constant item $\approx 3{,}5 \cdot 10^{-6}\ to\ 4{,}0 \cdot 10^{-6}$ ($3{,}7 \cdot 10^{-6}$)
- d [m] water depth (5,4)
- u [m/s] wind speed (27,78)

Consequently we see that : $\frac{dS}{dx} = 5{,}46 \cdot 10^{-5}\ \left[\frac{m}{m}\right]$

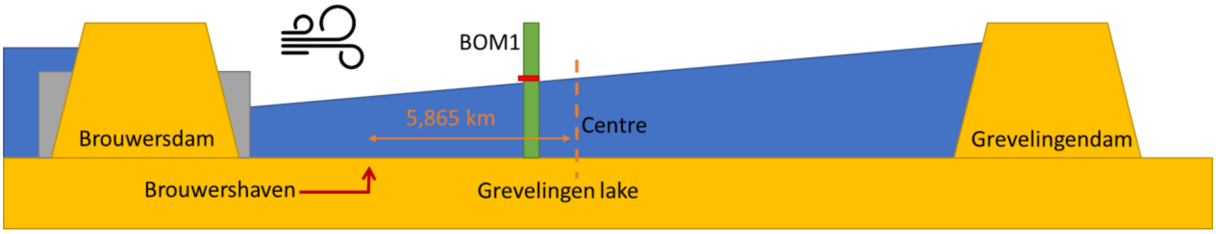
We made this calculation for the following 3 situations :

Open sluice caisson, wind coming from direction Northwest



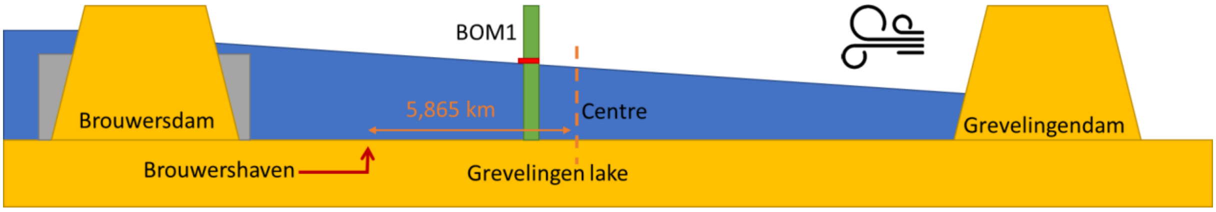
In this situation high wind speeds occur above the lake, whereas the sluice caisson has not been closed yet. Because the sluice caisson is still open and fresh water can enter the lake, rise of the water level due to wind action will take place all over the lake. The impact will be at its maximum at the Grevelingendam but this is not relevant for this research. From the moment the water level at BOM1 attains a level of 0,05 m NAP the sluice caisson will be closed resulting in the above presented water level course. Consequently at Brouwershaven a water level fall of 0,23 m occurs, causing the water level in the harbour of Brouwershaven to fall to -0,18 m NAP.

Sluice caisson closed with a maximal water level at the Grevelingen lake and wind from Northwest



In this situation the sluice caisson has been closed when the maximal water level at BOM1 was attained, after which a Northwest wind started to blow. In this situation the water in the lake switches over around the centre which means that a larger fall can be expected at Brouwershaven. In this situation a fall of 0,32 m occurs, resulting in an eventual water level of -0,27 m NAP.

Sluice caisson closed with maximal water level on Grevelingen lake and Southeast wind



This situation is similar to the one described previously, however, with the wind now coming from the opposite direction. So this time we see a rise of 0,32 m emerging through which the water level in the harbour of Brouwershaven will rise to 0,37 m NAP meaning that the upper limit that is allowed to occur 1x every 10 years will be exceeded.

As these water levels remain situated within the conditions determined by Rijkswaterstaat these cases of rise and fall of the water level due to wind action are not taken into account.

3.2.4 Extreme water levels

As not even a single one of the measured water levels exceeds the levels determined by Rijkswaterstaat the Rijkswaterstaat water levels are used in further stages of our project. I.e. :

- 0,7 m NAP as an upper limit
- -0,50 m NAP as a bottom limit

3.3 Not exploded explosives ('NEEs')[12]

During the second world war the island of Schouwen-Duiveland was confronted with various bombings and artillery attacks. Nowadays one can still be confronted with the consequences of these attacks and bombings in building projects executed on this island. As it is not known where these not-exploded explosives are situated a specific overall map of the island of Schouwen-Duiveland was established in recent years. In this map the areas are indicated where NEEs could possibly be found. For Brouwershaven for example it is even very, or at least more or less likely to find NEEs in the soil.

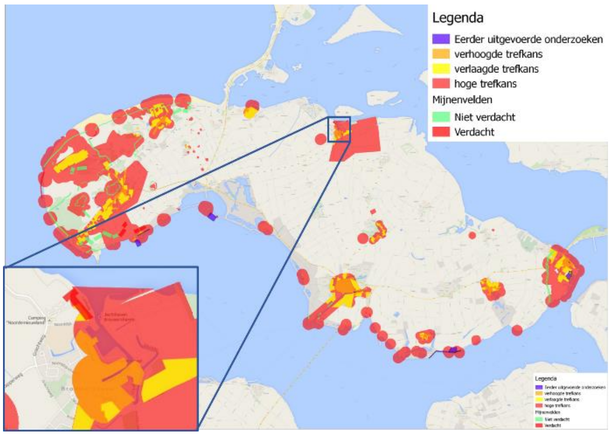


Figure 23: Map indicating areas where NEEs are likely to be found

3.4 Soil structure

An outline of the soil structure was obtained through cone penetration tests. For the ancient harbour these cone penetration tests were executed in preparation of the construction of the new quay wall at the southern side of the harbour in the course of April 2016. The cone penetration tests for the "new harbour" date back from the year 1968 already. This also was the period when plans were established for the construction of the new yacht harbour (new harbour). These cone penetration tests can be found in Appendix 6.

In the figures underneath a cross section of the soil based on the cone penetration tests can be seen. In figure 24 the composition of the soil is represented as it was for the construction of the new yacht harbour. The soil was taken away for these building purposes until a level of -2,75 m NAP was reached. Later on the building depth was changed again into -2,5 m NAP. In these cross sections we

[12] Website archief Gemeente Schouwen-Duiveland

can see that cone penetration test nr. 5 holds most clay, which may have negative effects on constructions to be built. Therefore these will be decisive for checking the existing constructions.

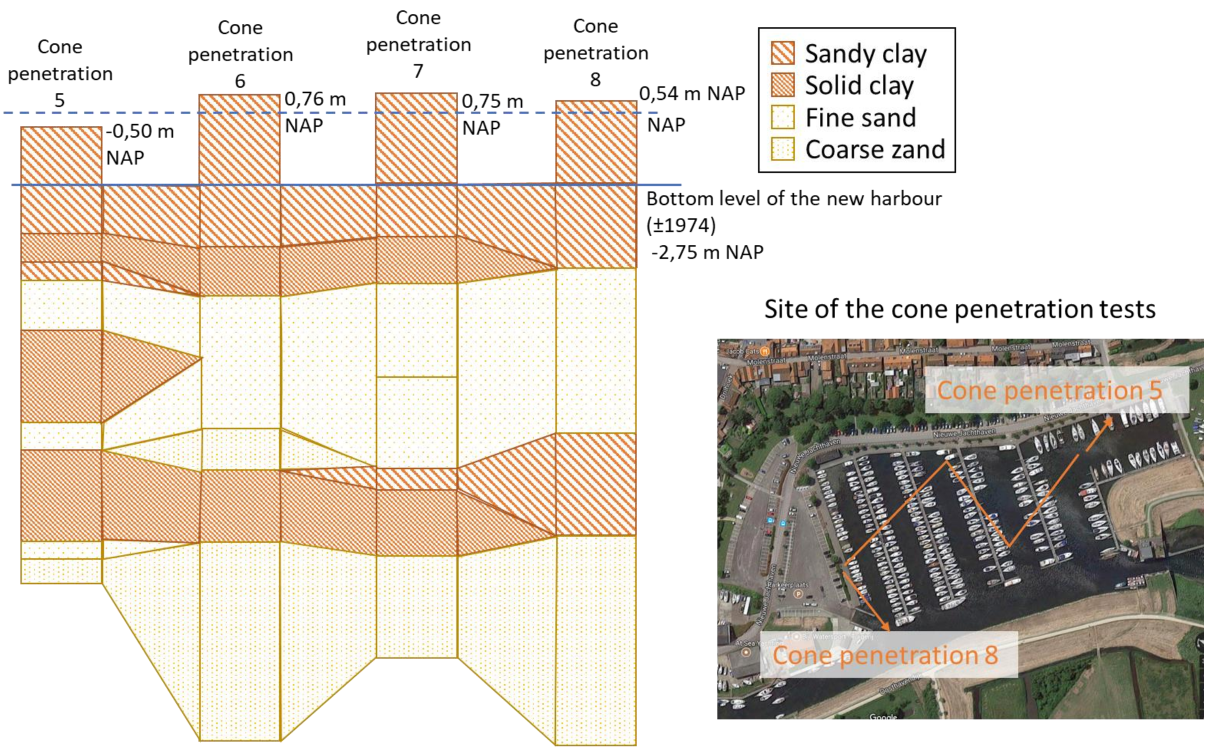


Figure 24: Soil structure through the new harbour of Brouwershaven

The soil cross section along the quay wall on the southside of the harbour can be seen in the following figure :

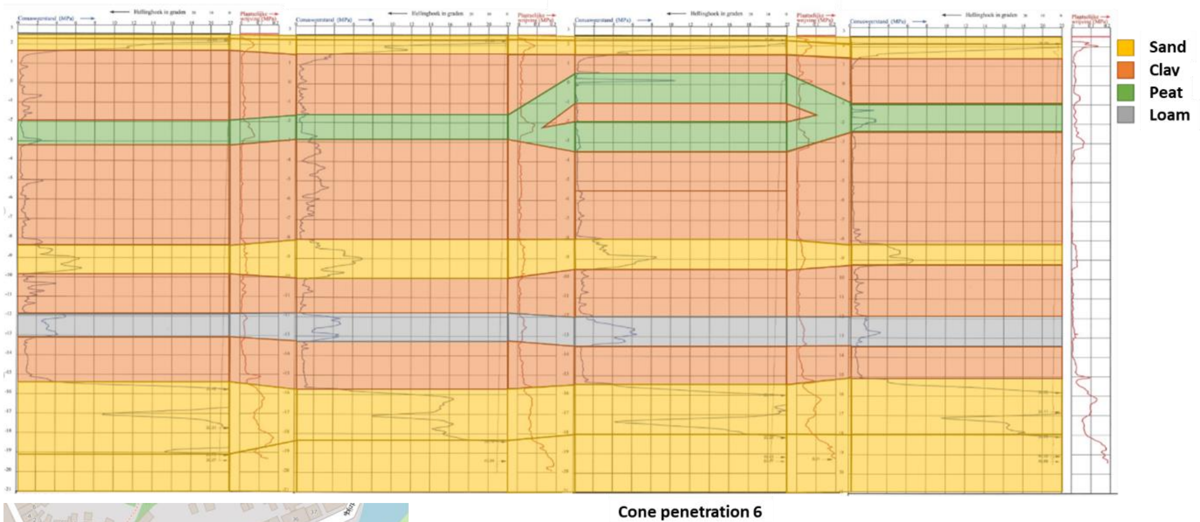


Figure 25: Soil structure for the cone penetration test of the old harbour

The ‘Kadeconstructie Brouwershaven: DO berekening’-report reveals that cone penetration test nr. 6 consists of the worst soil composition [a].

Here the soil has the following basic characteristics:

Laag	Bovenzijde [m t.o.v. NAP]	Grondsoort	γ_{droog} [kN/m^3]	γ_{sat} [kN/m^3]	φ' [°]	C' [kN/m^2]	δ' [°]
1	2,50	Zand vast	19,0	21,0	35,0	0,0	20,0
2	2,20	Zand	18,0	20,0	30,0	0,0	20,0
3	1,50	Klei - schoon, slap	14,0	14,0	17,5	0,0	11,7
4	0,50	Veen - niet voorbelast, slap	10,5	10,5	15,0	1,0	0,0
5	-1,00	Klei - organisch, slap	13,0	13,0	15,0	0,0	10,0
6	-2,00	Veen - matig voorbelast, matig	12,0	12,0	5,0	2,5	0,0
7	-3,50	Klei - organisch, matig	15,0	15,0	17,5	0,0	11,7
8	-5,50	Klei - schoon, vast	19,0	19,0	17,5	13,0	11,7
9	-8,00	Zand - zwak siltig, kleiig	18,0	20,0	27,0	0,0	18,0
10	-9,60	Klei - zwak zandig, matig	18,0	18,0	22,5	5,0	15,0
11	-12,00	Leem - sterk zandig	19,0	19,0	27,5	0,0	18,3
12	-13,50	Klei - schoon, slap	14,0	14,0	17,5	0,0	11,7
13	-15,50	Zand - zwak siltig, kleiig	18,0	20,0	30,0	0,0	20,0
14	-18,00	Zand - schoon, vast	19,0	21,0	35,0	0,0	20,0

Figure 26: Table with ground layer specifications from the report 'Kadeconstructie Brouwershaven'[a]

3.5 Profile of the harbour bottom

The harbour has been used in various depths throughout the years. For example the new yacht harbour originally had a bottom profile as presented in figure 43 in appendix 7. Eventually the depth level used was set at -2,5 m NAP. In 2015 measurements were carried out all over the harbour in order to determine how deep the current bottom profile is. This map shows how the bottom profile sank under the original level of -2,75 m NAP in places like the ancient harbour canal or between scaffolds where intense boat traffic occurs. This can be due to erosion of the bottom profile through turbulence of the ship propellers.

3.6 Shipping

The number one limiting factor for ships wanting to enter the harbour of Brouwershaven is the guard lock at the harbour’s entrance. The lock’s dimensions can be seen in the figure 27. For security reasons and in order to prevent damage to the ships only vessels having a draught of 2 m or less are allowed to enter, the berthing boxes allowing lengths up to 14 m[13]. Bigger vessels having a larger draught can potentially berth at the wharf just outside the guard lock. The table below shows an outline of the number of passers-by and overnight staying

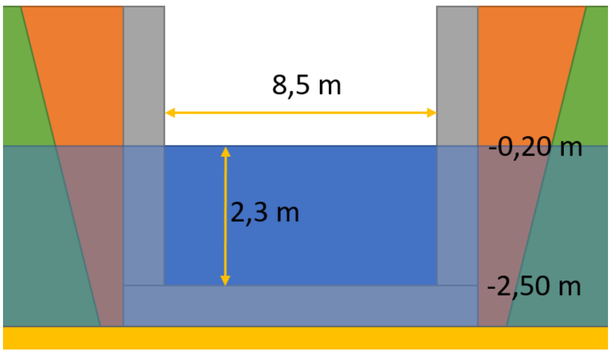


Figure 27: Dimensions guard lock

[13] www.wvbrouwershaven.nl/ligplaatsen

passengers that use the harbour per month. In this table we can see that the harbour is most intensely visited during spring and summer months.

Gem. Jaartotalen

	jan	feb	mrt	apr	mei	jun	jul	aug	sep	okt	nov	dec	totaal
2016	2	2	11	147	486	617	1575	1349	433	170	3	2	4797
2017	0	0	2	176	371	589	0	0	0	0	0	0	1138

Figure 28: Number of passers-by/ passengers staying overnight[14]

3.7 Flow rate through lock with flood gates

Using the programme for simulating earlier the tide in the harbour of Brouwershaven the maximal flow rate through the guard lock can be determined. These are the flow rates assessed for the 2 different options :

	Maximal speed during high tide	Maximal speed during low tide
Common sluice caisson	0,0719 m/s (0,26 km/hr)	0,0719 m/s (0,26 km/hr)
Tidal power plant	0,154 m/s (0,55 km/hr)	0,087 m/s (0,31 km/hr)

3.8 Summary of requirements and boundary conditions

Requirements:

- Harbour should be accessible 24 hr/24 hr during normal weather conditions

Boundary conditions

- Average water level Grevelingen lake and Brouwershaven harbour at -0,20 m NAP
- Common tide in Grevelingen lake between +0,05 and -0,45 m NAP
- Maximal values tide:
  - Never under -0,50 m NAP
  - 1 x per 1000 years max. 0,7 m NAP
- Bearing layer is situated under -18 m NAP

[14] Data from the chief harbour officer of Brouwershaven (Flip de Leeuw)

4 Check of the harbour constructions in the new harbour

A complete check of the various objects can be found in appendix 9 until 12. The following figure presents which constructions were checked. Focus here was laid on the constructions in the new harbour as this harbour was built after separating Grevelingen from the North Sea. Thus the new harbour has never known the sea tide.

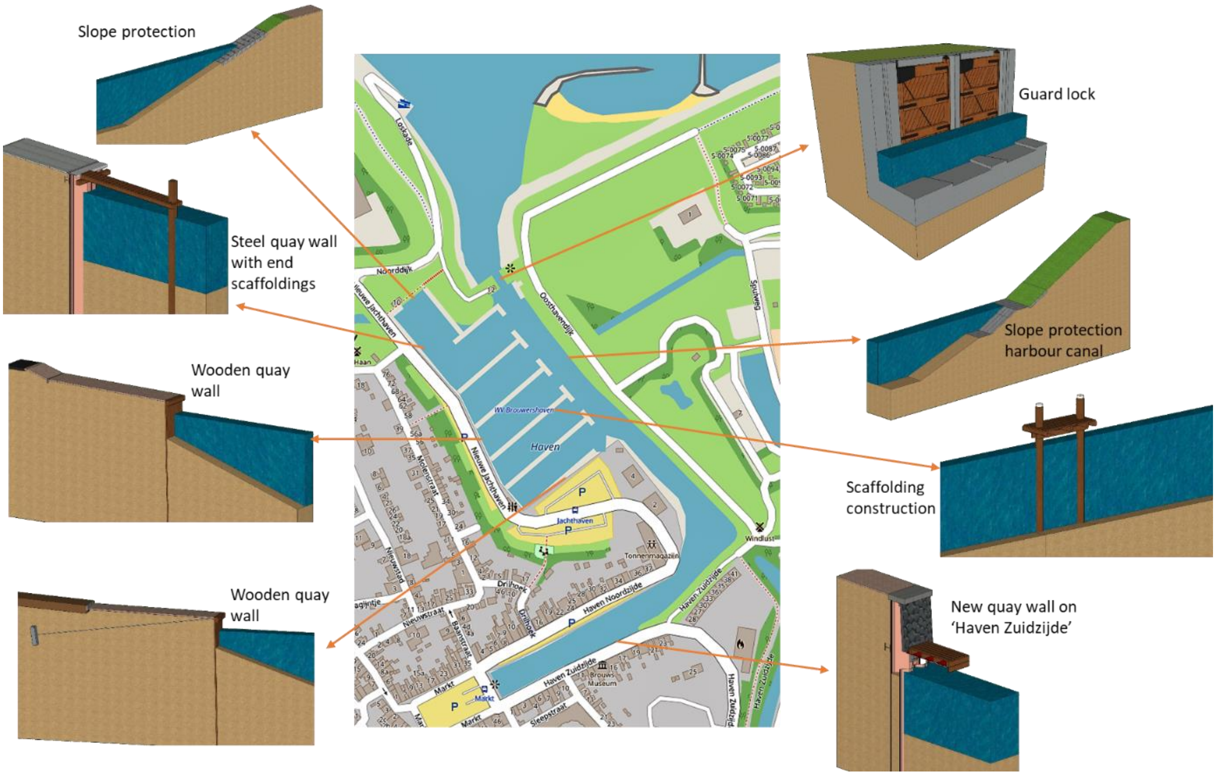


Figure 29: outline checked constructions

The check calculations produced the following results :

Construction	Stability extreme high tide	Stability low tide	General functionality
Wooden sheet piling	o.k.	o.k.	Quay wall getting under water for 30 cm in case of extreme high tide
Steel sheet piling	o.k.	o.k.	Quay wall getting under water for 20 cm in case of extreme high tide
Scaffolds new harbour	o.k.	o.k.	Scaffold walking deck getting under water for 30 cm in case of extreme high tide
End scaffold steel sheet piling	o.k.	o.k.	Also under water level during extreme high tide
Acclivity harbour	/	/	Common tide remains within the area of the slope protection. The water level reaches an even higher level only in extreme cases. As this lasts only shortly damage remains limited.
Acclivity ancient harbour canal	/	/	

Guard lock	/	/	Threshold on the bottom of the guard lock creates a problem for ship traffic at a water level under -0,20 m NAP
New quay wall harbour southside	/	/	Quay wall sufficiently high, during extreme high tide berthing scaffolds getting under water for not more than 5 cm
Harbour bottom	/	/	Similar problem to that of the guard lock as the maintained depth lies at -2,5 m NAP.

5 Multi criteria analysis (MCA)

5.1 The brainstorm session

In search for a solution for the problem presented in chapter 4 a brainstorm session was organised with the collaborators of the company and the chief harbour officer of Brouwershaven, the participants having the following functions :

3x process planner	1x chief harbour officer
1x process planner/executor/calculator	1x project manager
1x calculator	2x executor

In order to inform all participants of the goal of the brainstorm session a short presentation was given prior to the session. A short summary of this presentation can be found in appendix 13. In this presentation firstly a short summary was given of the history of Brouwershaven al well as the background of the research theme. After this the research was continued which had been started already trying to reveal the problems in the harbour that could possibly occur. Eventually focusing the problems emerging during reduced tide and for which a solution should be found. The brainstorm session led to the following solutions presented in the column on the left. Based on this column it was decided to execute a multi criteria analysis resulting in solutions presented in the column on the right. A better description of the chosen solutions follows in the next chapter.

Solution from the brain storm session

1. Demolition of the sills in the guard lock+ dredging of the harbour
2. Construction of the scouring sluice + demolition of the sills in the guard lock
3. Privatisation
4. Buffer
5. Sluice with ship lift
6. Tilting sluice
7. Construction of the new harbour in front of the guard lock
8. Construction of a scouring sluice halfway of the harbour
9. No interventions
10. Closing the harbour
11. Construction of the new harbour in front of the guard lock + giving the old harbour a new function
12. Construction of a conveyor belt
13. Demolition of the sills in the guard lock + redesign of the harbour

Selected solutions for MCA

1. Demolition of the sills in the guard lock+ dredging of the harbour
2. Construction of the new harbour in front of the guard lock + giving the old harbour a new function
3. Closing the harbour
4. Construction of a scouring sluice halfway of the harbour
5. Buffer
6. Construction of the new harbour in front of the guard lock
7. Construction of a conveyor belt

5.2 The solutions

5.2.1 Demolition of the sills + Dredging of the harbour

This solution was presented by the municipality in an earlier discussion with Schouwen Duiveland already and also was one of the first solutions to be presented during the brainstorm session. These reasons as well were one of the incentives why this specific solution was chosen to be elaborated. In this solution the limiting factor in the guard lock, i.e. the sills holding the doors when closed, is taken away. As far as we know this is not a problem as the guard lock can never be closed again as the mechanical parts are lacking. This solution also includes a deeper dredging of the harbour.

5.2.2 Demolition of the sills + Construction of a scouring sluice

This solution is more or less similar to the previous one. The only difference here is that the ancient technique of using a scouring sluice in the harbour is chosen to flush the harbour and to fight silting of the harbour.

5.2.3 Privatisation

As Brouwershaven is the only remaining harbour of the Grevelingen lake being fully owned by the municipality privatisation could be an option. This would exempt the municipality from having to invest in the harbour in order to adapt it for the future tide.

5.2.4 Buffer (turning around the guard lock)

This includes a turnover of the function of nowadays' guard lock. This means that the water is kept in the harbour instead of keeping the water outside the harbour during storm tide.

5.2.5 New harbour outside the guard lock (larger vessels) + Using the existing harbour for smaller vessels

In this solution the existing harbour is kept in its present status, being accessible to smaller ships only. Outside the guard lock a complementary new harbour is built where ships berth that cannot cross the sills in the guard lock. This solution also makes the harbour more attractive for ships that could not reach the harbour in former days.

5.2.6 Scouring sluice halfway of the harbour

In this case the ancient harbour is used as a scouring basin. Here the water collected from high tide is retained until there is low tide in the Grevelingen lake. In this situation the scouring sluice is opened resulting in a cleansing flushing of the new harbour.

5.2.7 Closing the harbour

This solution closes the harbour for recreative shipping and lets decay enter gradually.

5.2.8 New function for the existing harbour (barges) + New harbour outside the guard lock

The existing harbour is closed for recreative shipping and it gets a new function such as floating houses for example. Outside the guard lock a completely new harbour is arranged then to still keep it accessible to recreative shipping.

5.2.9 Conveyor belt (as seen in water attractions)

In these solutions the water is retained at a fixed gauge level by installing a simple wall inside the guard lock. Thus ships are hoisted out of the water over the wall by a kind of conveyor belt and then put back on the water surface at the other side.

5.2.10 Demolition of the sills + reformatting the existing harbour
In this option the sills in the guard lock are demolished as well, whereas only the new harbour is dredged as it had a bottom level being 25 cm lower than the now existing depth. This will include a reformatting of the complete harbour taking into account that larger vessels having a deeper draught berth in the new harbour and smaller ones in the old harbour.

5.3 The criteria
The abovementioned solutions will be tested taking into account the following criteria :

5.3.1 Retention of minimal water depth in the harbour
These criteria refer to the fact that sufficient water depth should be retained in the harbour at all times so as not to create danger nor damage for the usual visitors and passengers in the harbour.

5.3.2 Accessibility to the harbour
This mainly deals with the level of convenience in accessing the harbour and during how many hours per day people can get access to the harbour. The building of certain constructions could limit this.

5.3.3 Financing
It is desirable of course that the building costs of any chosen solution should be kept as low as possible and that this investment could potentially be recuperated through creating complementary revenues.

5.3.4 Esthetics
Nowadays the eye has its claims too. This criterion here determines how much extra effort should be invested in the emanation of the project.

5.3.5 Retention of a secondary water retaining structure
The harbour is now being protected by a storm surge barrier. Question is how import it is that it should be kept functioning in the future project.

5.3.6 Impact of the project on the surroundings
How large is the impact of the solution on the surroundings ? Are there more people coming to this area also producing more stray garbage ? How will nature be influenced by this solution ?

5.3.7 Level of easy maintenance
How easy to maintain is the solution and is the maintenance easy to be done ? These are also elements which can influence maintenance costs.

5.3.8 Level of convenience
Is the solution easy to adopt by harbour users or is it rather considered as an impediment ?

5.3.9 Security
Is the solution safe as well for the teams building it as for the maintaining workers, operators and users ?

5.4 The importance of the criteria
In order to attain an optimal solution for all the parties concerned it is important for them to be involved in this part of the process. This is why a short poll was composed which was presented to

users as well as principals and the Aquavia contractor company. In this poll participants were asked to rank the abovementioned criteria in an order of importance.

5.4.1 Contractor company "Aquavia"

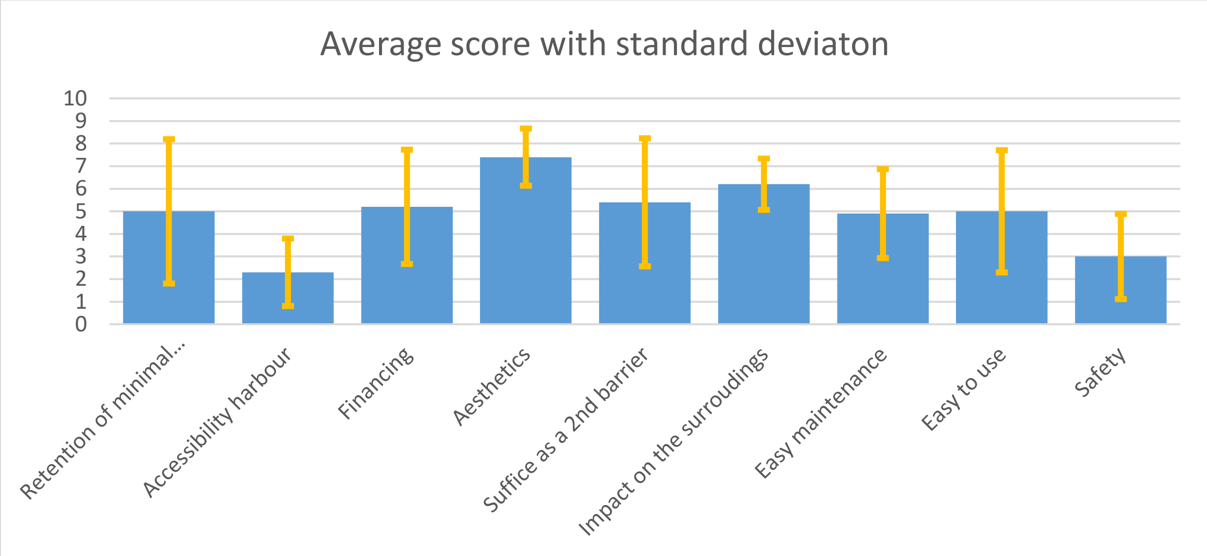
This poll was organised together with the brain storm session in the company, more precisely after the presentation part. Firstly the various criteria were explained, after which one by one the participants were asked whether there were criteria lacking or not. Eventually the poll was filled out requesting the participants to rank the criteria in an order of importance from 1 (the most important criterion) to 9 (the least important criterion). The result of this brain storm session can be found in the following table. In order to get a score that can be used in the MCA the scores of the various persons were added together. We here gave the most impact to the criterion that was considered to be the most important of all.

	person 1	person 2	Person 3	person 4	person 5	person 6	person 7	person 8	person 9	person 10	Average	Total	Score
Retention of minimal depth in the harbour	2	5	2	2	3	7	9	9	9	2	5	50	4
Accessibility harbour	3	4	1	1	1	5	1	3	1	3	2,3	23	1
Financing	5	9	9	4	4	2	4	8	3	4	5,2	52	5
Aesthetics	9	7	8	8	9	8	7	6	5	7	7,4	74	8
Suffice as a 2nd barrier	4	8	6	3	2	9	8	1	8	5	5,4	54	6
Impact on the surroundings	7	6	7	5	6	6	6	4	7	8	6,2	62	7
Easy maintenance	6	3	3	7	7	1	5	5	6	6	4,9	49	3
Easy to use	8	2	4	3	8	4	3	7	2	9	5	50	4
Safety	1	1	5	6	5	3	2	2	4	1	3	30	2

Analysis of the table above shows that the contracting company thinks accessibility of the harbour as well as security to be important. Whereas the least attention is given to the solution's aesthetics and impact on the surroundings. When applying the following formula for standard deviation to the poll the following conclusion can be made:

$$s_x = \sqrt{\frac{\sum(x_i - \bar{x})^2}{n_x}}$$

With: s_x = standard deviation
x_i = the value of a number in the series
$\bar{x}$ = the average of all numbers in the series
n_x = the number of numbers in the series



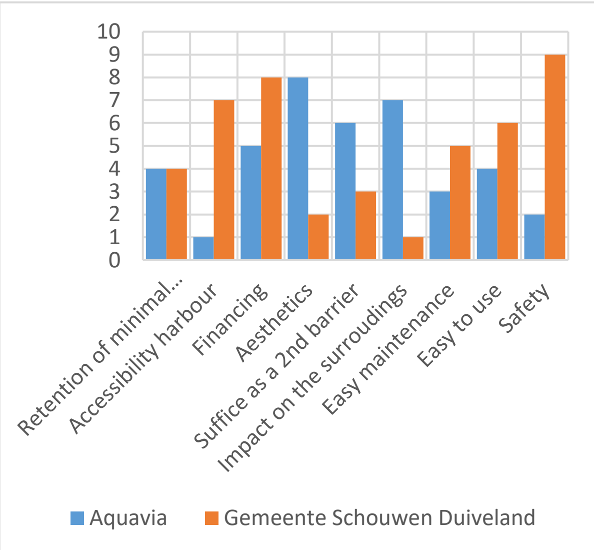
The abovementioned graph quickly shows that esthetics and the impact on the surroundings have a small standard deviation. This means that everyone gave this criterion a similar ranking, i.e. somewhere in the rear. Equally remarkable is that the standard deviation is at its largest for the criteria situated in the middle of the ranking.

5.4.2 The client "Schouwen-Duiveland"

Using the website " enquetemaken.be" collaborators of the municipality of Schouwen-Duiveland who are connected to the harbour of Brouwershaven were asked to attach a score to the various criteria. However, only 1 person reacted and gave the following ranking :

As the response for the poll was very low here (1 person only) we cannot use these scores to

	Schouwen-Duiveland
Retention of minimal depth in the harbour	4
Accessibility harbour	7
Financing	8
Aesthetics	2
Suffice as a 2nd barrier	3
Impact on the surroundings	1
Easy maintenance	5
Easy to use	6
Safety	9



determine the best solution considered from the point of view of the municipality of Schouwen-Duiveland.

5.4.3 The "Watersport club Brouwershaven" users

The same poll was sent to the aquatic sports association of Brouwershaven but unfortunately no reaction was received.

5.5 The multi criteria analysis

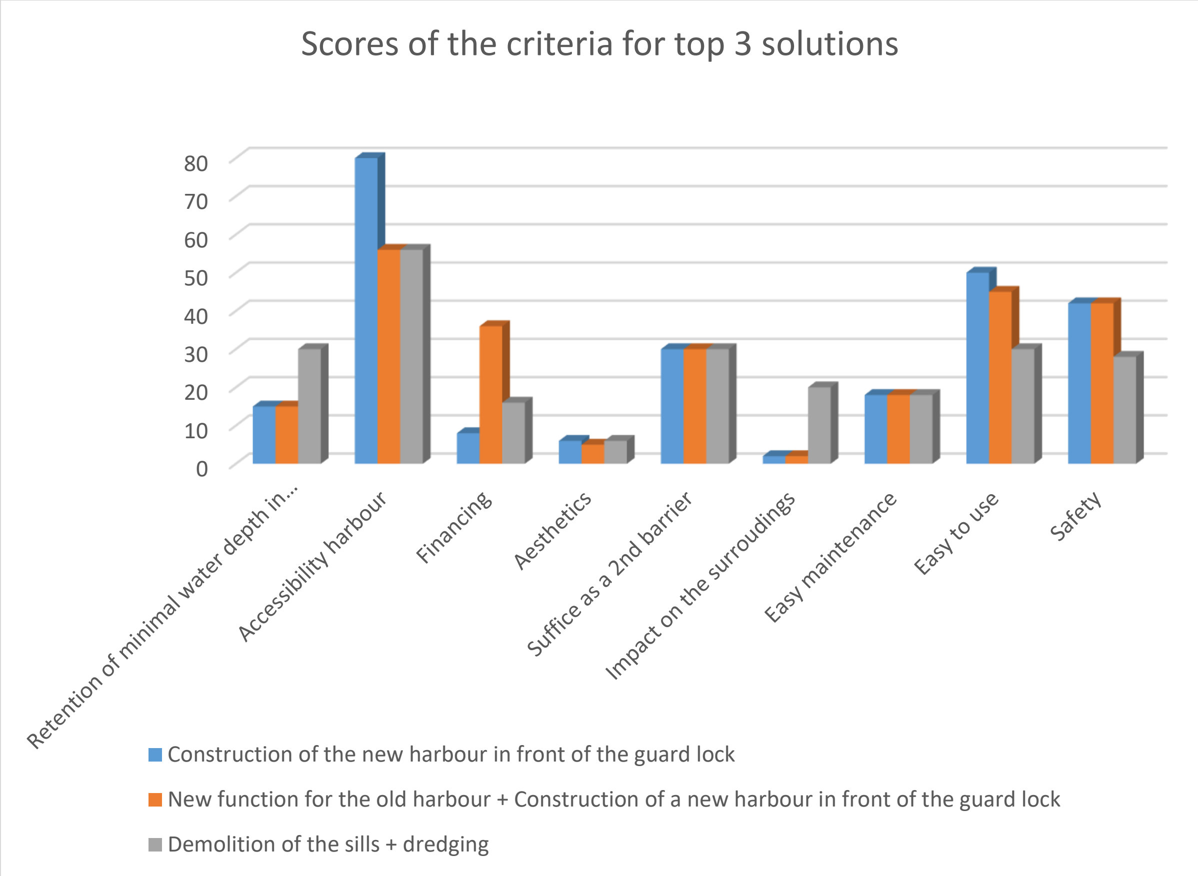
This part as well was executed with the brain storm session members. For every solution it was discussed here how good its scores are for the various criteria. The eventual result of this discussion can be seen in the following table :

	Criteria Score	Demolition of the sills + dredging	New function for the old harbour + Construction of a new harbour in front of the guard lock	Closure of the harbour	Construction of the souring sluice halfway harbour	Turning around guard lock	Construction new harbour in front of the guard lock	Construction conveyor belt
Retention of minimal depth in the harbour	5	6	3	1	6	10	3	10
Accessibility harbour	8	7	7	1	5	3	10	4
Financing	4	4	9	2	5	3	2	2
Aesthetics	1	6	5	1	2	9	6	4
Suffice as a 2nd barrier	3	10	10	10	10	10	10	4
Impact on the surroundings	2	10	1	1	8	9	1	5
Easy maintenance	6	3	3	10	5	5	3	1
Easy to use	5	6	9	1	3	3	10	5
Safety	7	4	6	10	4	5	6	1
Total score		234	249	189	211	223	251	154

The abovementioned multi criteria analysis shows that the following solutions are the best for the problems concerned :

- Construction of the new harbour in front of the guard lock
- Create a new function for the existing harbour and shift the harbour function to a new location in front of the guard lock
- Demolition of the sills in guard lock and dredging of the harbour

The following graph shows us that the abovementioned solutions differentiate from the other because of their high scores in terms of accessibility, being easy to use and safety.


Scores of the criteria for top 3 solutions
80
70
60
50
40
30
20
10
0
Retention of minimal water depth in...
Accessibility harbour
Financing
Aesthetics
Suffice as a 2nd barrier
Impact on the surroudings
Easy maintenance
Easy to use
Safety
Construction of the new harbour in front of the guard lock
New function for the old harbour + Construction of a new harbour in front of the guard lock
Demolition of the sills + dredging

6 Demolition of the sills of the guard lock and dredging of the harbour

6.1 Quantity of material to be dredged

To let ships manoeuvre safely through or in and out of the harbour the bottom of the harbour has to be situated at a level of at least -2,75 m NAP. To determine how much material has to be dredged to attain this the harbour was split up in different areas. From these areas the mean depth and area were estimated as seen in figure 30. After a small calculation it was found that a volume of 5143 m^3 has to be dredged. This intervention cannot be executed without the following risks :

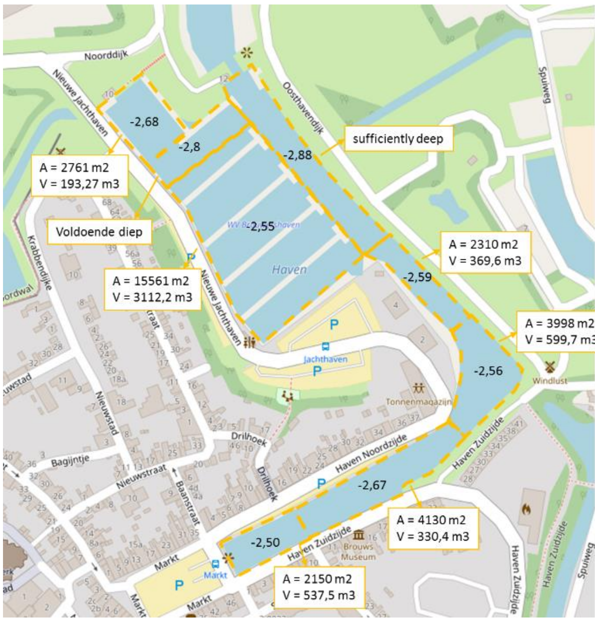


Figure 30: Estimation quantity of material to be dredged

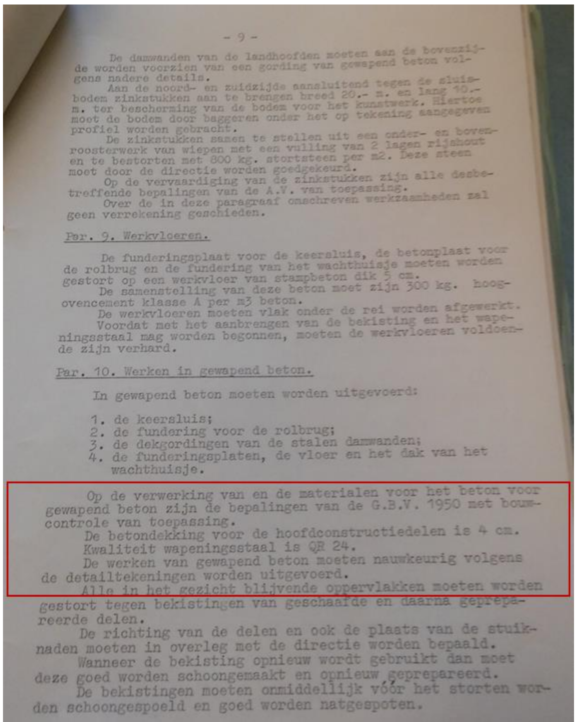
- Encountering not exploded explosives :
  This is not a problem for the new harbour since the originally designed bottom depth was 2,75 m NAP. But this becomes a different story when the old harbour is concerned.
- The old quay walls at the end of the old harbour :
  In this part of the 'old harbour' the old quay walls, which are still there, were built out of basalt blocks and founded on bricks. For this reason it is possible that the harbour at this location cannot be dredged as deep as required, since the quay wall could become unstable.

6.2 Specifications of the lock

In order to find the specifications and design drawings of the lock three different archives were visited and consulted :

- The archive of Schouwen-Duiveland
- The archive of Middelburg
- The National archive

In the first two archives a lot of technical plans and a handout of the specifications to which the design had to comply were found. The dimensions found in the technical plans were then used to reconstruct a 3D model as can be seen in appendix 14. In order to check whether the construction still remains stable without the sills, it is necessary to know what type of reinforcement was used and how this was positioned in the construction. As can be seen in figure 31 it is stated that reinforcement has to be placed exactly similar to what the detail



- 9 -

De damwanden van de landhoofden moeten aan de bovenzijde worden voorzien van een gording van gewapend beton volgens nadere details.
Aan de noord- en zuidzijde aansluitend tegen de sluisbodem zinkstukken aan te brengen breed 20.- m. en lang 10.- m. ter bescherming van de bodem voor het kunstwerk. Hiertoe moet de bodem door baggeren onder het op tekening aangegeven profiel worden gebracht.
De zinkstukken samen te stellen uit een onder- en bovenroosterwerk van wiepen met een vulling van 2 lagen rijshout en te bestorten met 800 kg. stortsteen per m2. Deze steen moet door de directie worden goedgekeurd.
Op de vervaardiging van de zinkstukken zijn alle desbetreffende bepalingen van de A.V. van toepassing.
Over de in deze paragraaf omschreven werkzaamheden zal geen verrekening geschieden.

Par. 9. Werkvloeren.

De funderingsplaat voor de keersluis, de betonplaat voor de rolbrug en de fundering van het wachthuisje moeten worden gestort op een werkvloer van stampbeton dik 5 cm.
De samenstelling van deze beton moet zijn 300 kg. hoogovencement klasse A per m3 beton.
De werkvloeren moeten vlak onder de rei worden afgewerkt.
Voordat met het aanbrengen van de bekisting en het wapeningsstaal mag worden begonnen, moeten de werkvloeren voldoende zijn verhard.

Par. 10. Werken in gewapend beton.

In gewapend beton moeten worden uitgevoerd:

1. de keersluis;
2. de fundering voor de rolbrug;
3. de dekgordingen van de stalen damwanden;
4. de funderingsplaten, de vloer en het dak van het wachthuisje.

Op de verwerking van en de materialen voor het beton voor gewapend beton zijn de bepalingen van de G.B.V. 1950 met bouwcontrole van toepassing.
De betondekking voor de hoofdconstructiedelen is 4 cm.
Kwaliteit wapeningsstaal is QR 24.
De werken van gewapend beton moeten nauwkeurig volgens de detailtekeningen worden uitgevoerd.
Alle in het gezicht blijvende oppervlakken moeten worden gestort tegen bekistingen van geschaafde en daarna geprepareerde delen.
De richting van de delen en ook de plaats van de stuiknaden moeten in overleg met de directie worden bepaald.
Wanneer de bekisting opnieuw wordt gebruikt dan moet deze goed worden schoongemaakt en opnieuw geprepareerd.
De bekistingen moeten onmiddellijk vóór het storten worden schoongespoeld en goed worden natgespoten.

Figure 31: Page out of specifications of guard lock design

drawings show. Unfortunately these drawings could not be found in the various archives. Because of all these uncertainties and since the structure is a secondary flood defence it was decided to not continue to check this option.

6.3 Construction method

In this chapter a method has been applied to get the missing data and a construction method to put this option into practice.

First some research will have to be done to get an impression of how the reinforcement is incorporated in the construction, and more specifically into the floor of the guard lock. After having checked this construction on stability the modification of the guard lock can begin.

The first step is to close the guard lock from the open water. On the side of the Grevelingen lake this can be done by putting a bulkhead in the guard lock at the space reserved for it. Unfortunately this technique is not possible on the other side of the guard lock since the sill that supports the bulkhead has to be removed. That is why on the harbour side it was decided to put a sheet pile wall into the harbour bottom. This will be done using a pontoon with a crane and a pile driver on it since no sheet piles can be driven into the soil at the sides of the harbour, because of the stone cover and rumble on the dikes. The sheet pile wall will connect to the dikes on the sides with the help of a clay dam. This clay will also be used around the sheet pile wall to make the structure watertight.

The next step is to pump the guard lock dry and clean the floor slab. After which the removal of the sills can start. This will be done by removing the concrete layer until it has the same height as the rest of the floor slab. Some part of the reinforcement has to be removed as well as the ashlar doorstops. When finished the concrete on the floor slab will have to be repaired in a way that the remaining reinforcement is sufficiently protected from corrosion.

The last step is to fill the guard lock with water again and remove the sheet pile wall with the clay dams and the bulkhead at the other side of the guard lock.

6.4 Cost estimation

The total overview of the cost estimation can be found in appendix 26. In this estimation the demolition of the sills in the guard lock, the dredging of the harbour as also the construction of the bulkhead to place in the guard lock in case of an emergency was taken into account. On the end of the estimation it was found that these actions would cost 300.000 EUR.

7 Constructing a new harbour in front of the guard lock

Before the design of the new harbour in front of the guard lock can start. An overview of the stream patterns of flood and ebb through the Grevelingen lake was made.

7.1 Occurring forces

7.1.1 Estimating flow patterns of the Grevelingen lake

Deltas are known to form two types of channels, i.e. the dominant flood and ebb channels. These are formed by the water current through the delta caused by ebb and flood. The ebb current concentrates in a set of continuous and deep channels, while the flood current is stronger and more dispersed over shallow channels.[b]

These channels can be determined by the use of shipping maps (appendix 15), which was helpful to estimate the streaming patterns. These estimations were done for the situation before the closure of the Grevelingen lake on the one hand and for the situation after bringing back a reduced tide by the construction of a sluice caisson in Brouwersdam on the other hand.

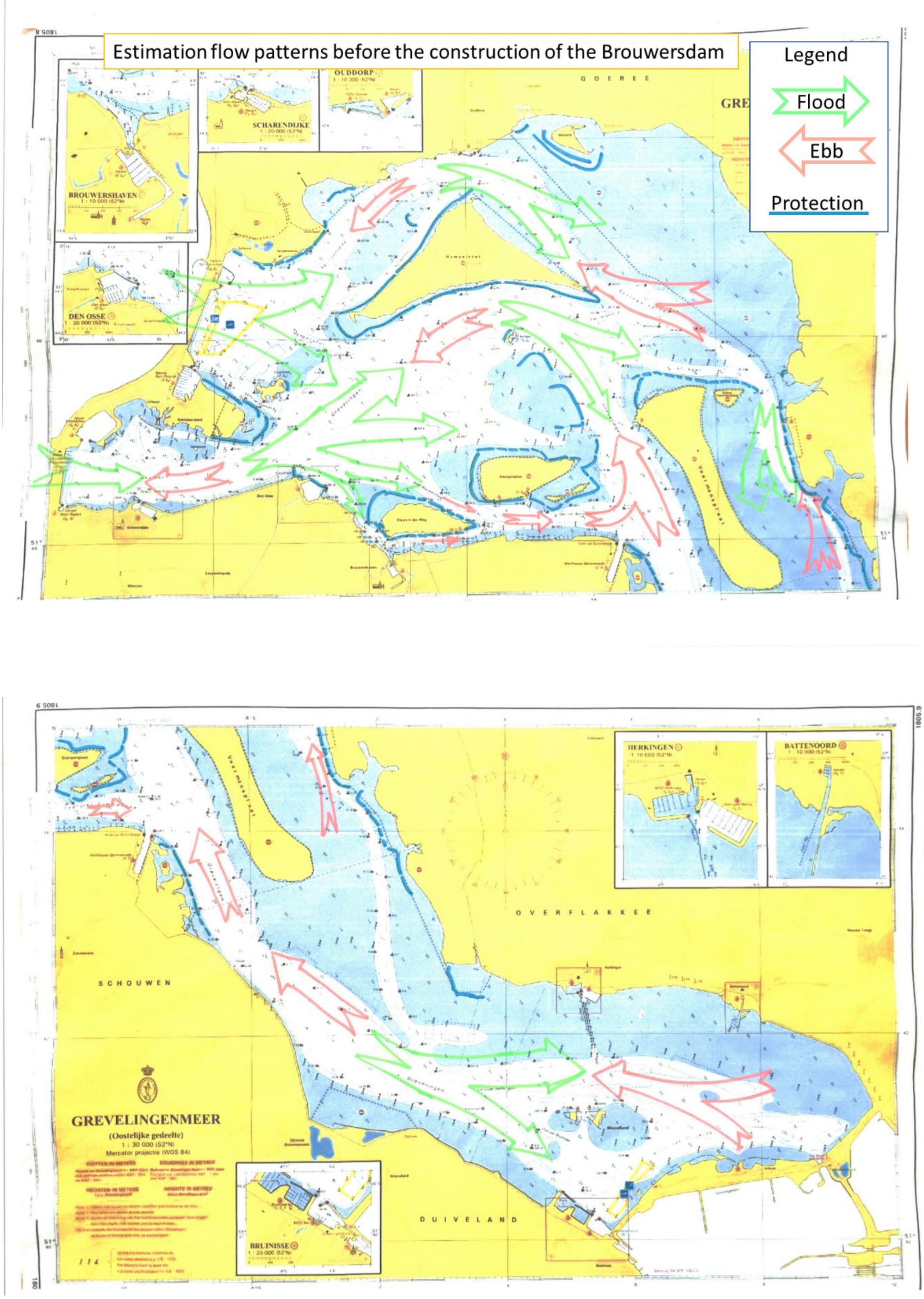


Figure 32: estimated flow pattern before construction of the Brouwersdam

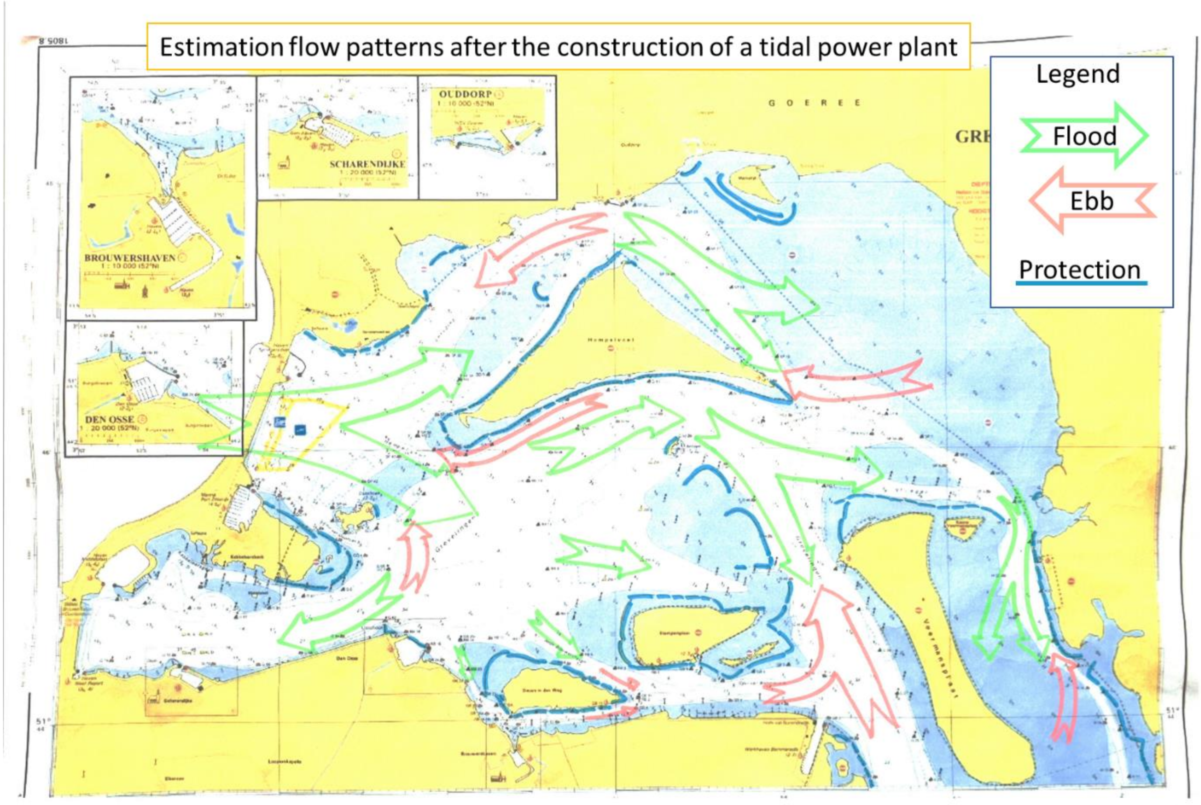
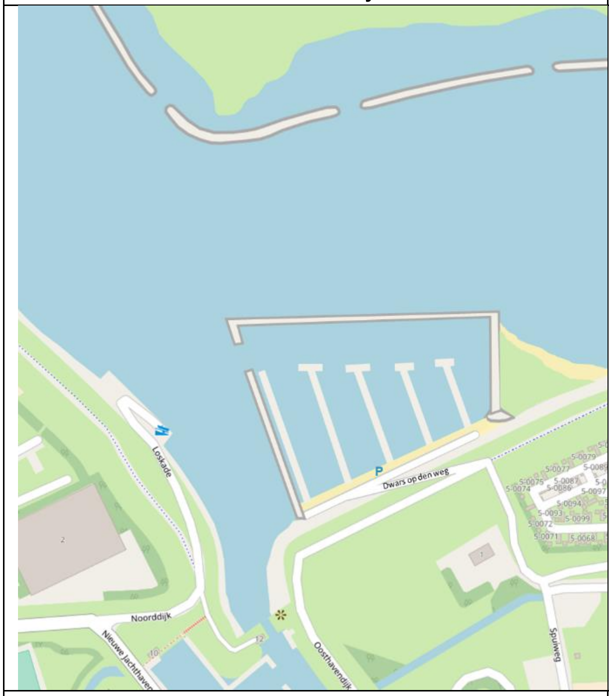
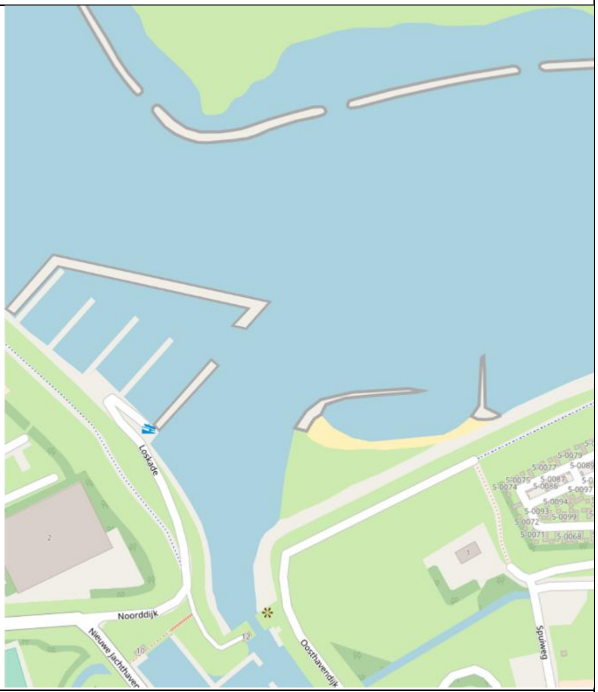


Figure 33: Estimation flow pattern after the construction of a tidal power plant

7.1.2 Designing the new harbour

There are 2 sites where the new expansion can be designed, each with its pro's and con's which will be discussed in the table below.

Harbour on the 'Oosthavendijk'	Harbour on the 'Loskade'
	
Pro's: • Navigation channel can remain in its original site • Use of the already existing cribs Con's: • There are no services available nor a good access road to the harbour.	Pro's: • Services and access road are already present • No loss of an artificial beach Con's: • Dredging a new navigation channel is necessary

Because of the presence of an access road as well as of the necessary harbour services it was decided to elaborate the second project, thus creating the following concept project :

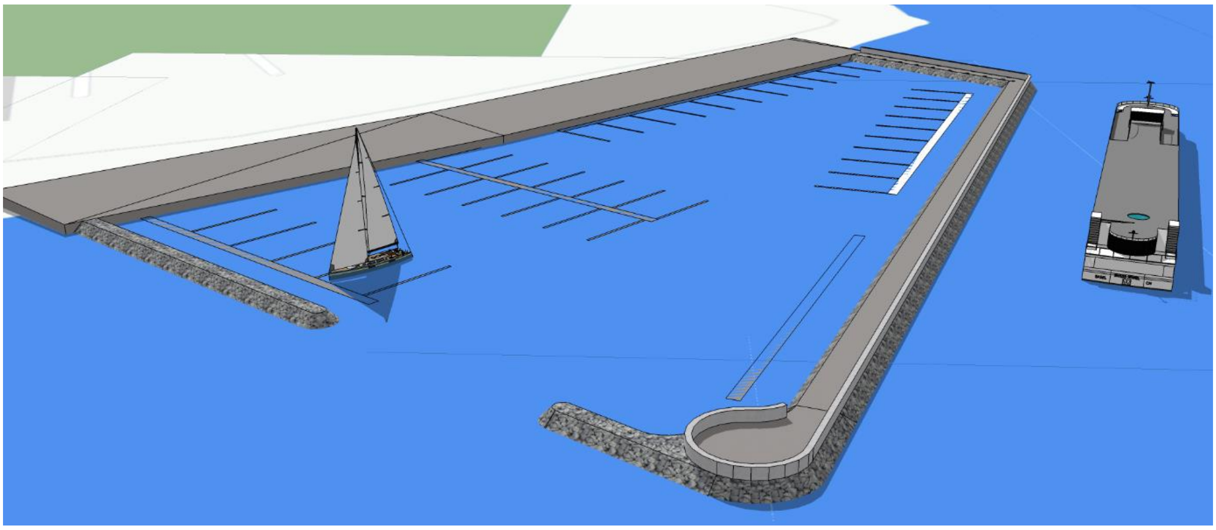
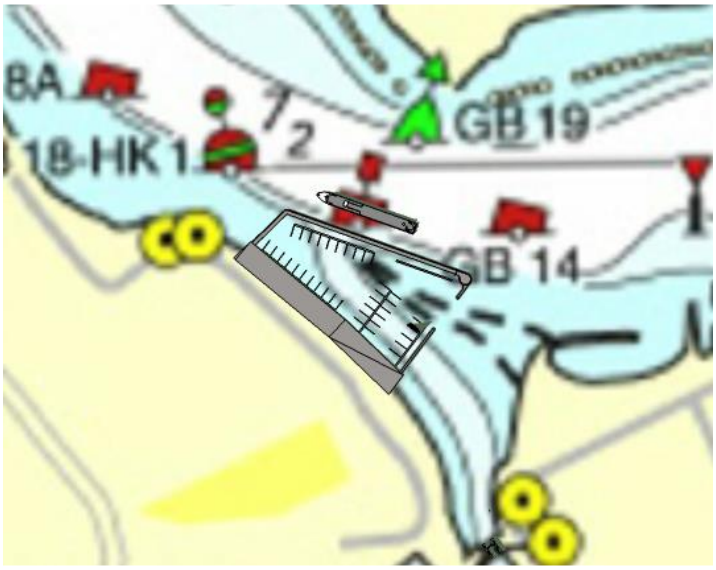


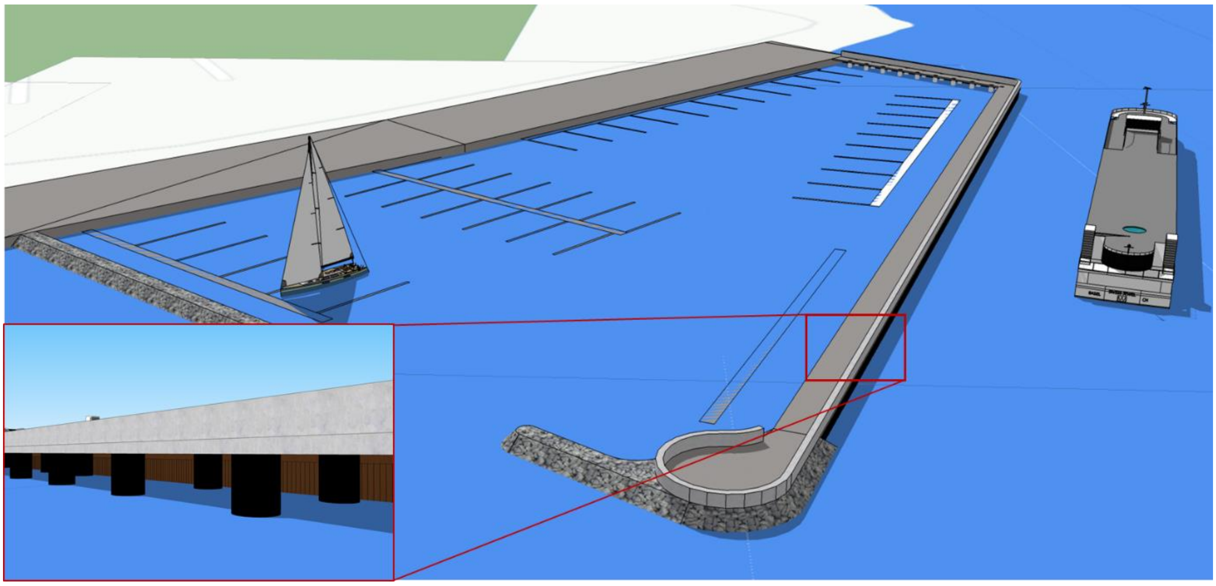
Figure 34: A first impression of the new harbour (the different dimensions were calculated with the help of Report 'Ports and Terminals: Planning and Functional Design')[i].

In this first concept project the existing wharf is extended in order to thus create space for the necessary facilities such as a sanitary utility building etc., as well as parking lots for harbour users. In order to protect the harbour and its users from the waves two moles will be constructed around the harbour out of rubble. In the harbour berths will be made available for the ships for which the existing harbour is not, or has never been accessible. Apart from these berths on the outside of the moles further berthing facilities will be offered for river cruise ships thus providing a promenade on the mole potentially capable of letting pass a small delivery van.

7.1.3 Water depth profile: existing situation

Through projecting the concept project over the bottom sounding chart we can see that the longest mole is running along the fairway. This can provoke problems for the rubble construction as this rubble could after some time disappear into the fairway through the erosion of the outer side of the fairway bend. Also because of this reason we chose to adapt the project for this mole to a mole built of steel foundation tubes with prefab concrete elements on top of them. At the outside this mole will be constructed using wooden wave breaking flaps.





7.1.4 Wave height

To determine the wave height that could arise in the Grevelingen lake, the nomogram from 'Groen and Dorrestein' was used. On the map in figure 35 it can be seen that waves can come from 2 directions at which they can get a fetch length of approximately 6 km. Together with this fetch length a wind velocity of 25 m\s is assumed. The calculation of the wave dimensions can be found in appendix 16 , which gives the following wave dimensions:

- Length : 12,96 m
- Height : 1,7 m
- Period : 3,5 s

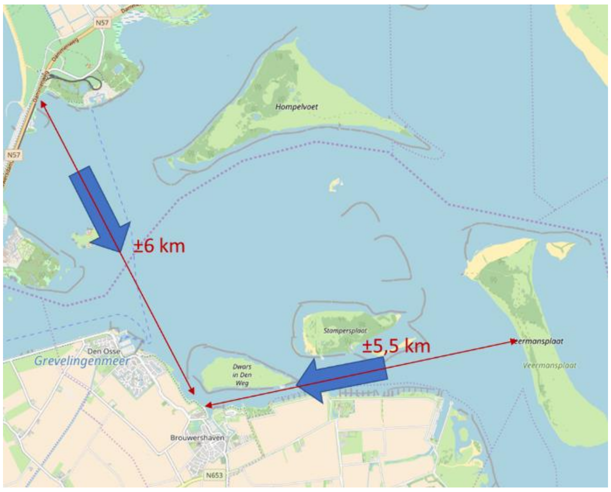


Figure 35: Fetching length over grevelingen lake

7.1.5 Shoaling in front of breakwater

Once the waves arrive at the new harbour they will firstly face the reduction of the water depth to ± 2,6 m NAP. This will influence the wave height by potentially occuring shoaling. We can determine the shoaling factor using the following formula :

$$K_s = \sqrt{\frac{1}{\tanh(kd)(1+\frac{2kd}{\sinh(2kd)}}}$$

This factor has been determined for the lowest water level as well as for the highest, which produced the following results :

	Low tide : -0,5 m NAP	High tide : 0,7 m NAP
Incoming wave height	1,7 m	1,7 m
Occurring water depth (bottom at -2,6 m NAP)	2,1 m	3,3 m
Shoaling factor	0,92	0,93
New wave height	1,56 m	1,58 m

7.1.6 Required height of breakwater[h]

The depth at which the breakwater has to reach can easily be calculated with the following formula:

$$d_{constr} = water\ level - wave\ height \cdot \gamma_{safety} = -0{,}5 - \frac{1{,}56}{2{,}0} \cdot 1{,}2 = -1{,}436\ m - NAP$$

For the determination of the height of the structure formula 5.18 out of the European overtopping manual is used:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = a \cdot e^{-\left(b \cdot \frac{R_c}{(H_{m0} \cdot \gamma_\beta)}\right)^{1,3}} \quad for\ non-breaking\ waves$$

With:

- $a = 0{,}09 - 0{,}01(2 - \cot \alpha)^{1{,}3}$ for $\cot \alpha < 2$ and $a = 0{,}09$ for $\cot \alpha \geq 2$
- $b = 1{,}5 + 0{,}42(2 - \cot \alpha)^{1{,}5}$ with a maximum value of $b = 2{,}35$ and $b = 1{,}5$ for $\cot \alpha \geq 2$
- R_c Freeboard [m]
- q specifieke discharge $\left[\frac{\frac{m^3}{m}}{s}\right]$
- H_{m0} Wave height [m]
- γ_β angle of incidence [-]
  - $\gamma_\beta = 1 - 0{,}0022\beta$ for $0° \leq \beta \leq 80°$
  - $\gamma_\beta = 0{,}824$ for $\beta > 80°$

This manual also states that for a safe use of the promenade the overtopping discharge by waves less the 2 meter high must be less than 5 l/s/m.

7.1.6.1 Deterministic approach

Firstly the height of the construction was assessed deterministically. This was done by calculating the overtopping discharge for different heights of the construction. At the end of the calculation a construction height of 2,526 m NAP was found.

Since the structure will be designed as a straight wall, the alpha in the previous formula is equal to zero. Which gives :

- $a = 0{,}09 - 0{,}01(2 - \cot(0))^{1{,}3} = 0{,}047[-]$
- $b = 1{,}5 + 0{,}42(2 - \cot(0))^{1{,}5} < 2{,}35 \rightarrow b = 2{,}35[-]$

Figure 35 learns that it can be estimated that the wave will hit the structure at an inclination of six degrees, which gives a reduction coefficient of 0,925 [-].

$$R_c = 2{,}526 - 0{,}7 = 1{,}826$$

$$q = a \cdot e^{-\left(b \cdot \frac{R_c}{(H_{m0} \cdot \gamma_\beta)}\right)^{1{,}3}} \sqrt{g \cdot H_{m0}^3} = 0{,}09 e^{-\left(2{,}35 \cdot \frac{1{,}826}{1{,}58 \cdot 0{,}925}\right)^{1{,}3}} \cdot \sqrt{9{,}81 \cdot 1{,}58^3} = 5 \frac{l}{s}/m$$

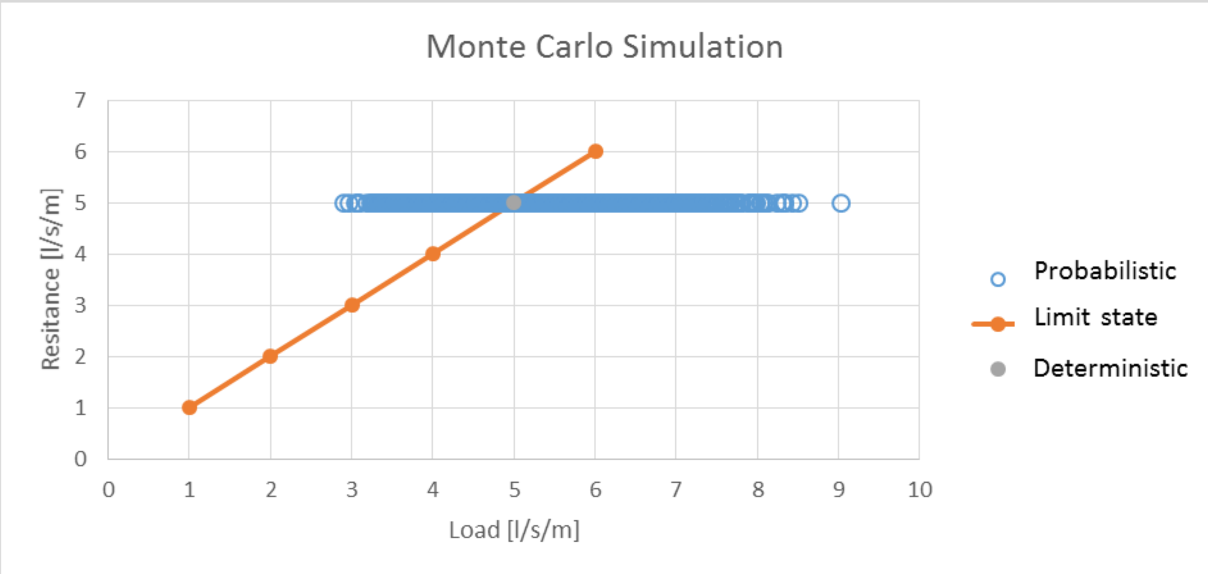
7.1.6.2 Probabilistic approach [e]

Even the deterministic approach proved that a height of 2,526 m NAP would be enough, this will not guarantee that this is a safe situation. Various parameters in the formula have uncertainties. For this reason a level III probabilistic approach was executed. With this method a standard deviation is given to the various parameters that contain an uncertainty, after which a Monte Carlo simulation is performed. This simulation takes a random value out of the deviations of the different parameters and then redoes the calculations performed in paragraph 7.1.6.1. At last the limit state function is used to check whether the outcome satisfies the given criteria. This process is repeated for a 1000 times or more. At the end of the Monte Carlo simulation the probability the structure does not meet the requirement can be found by dividing the number of times the requirement was not met by the number of trails.

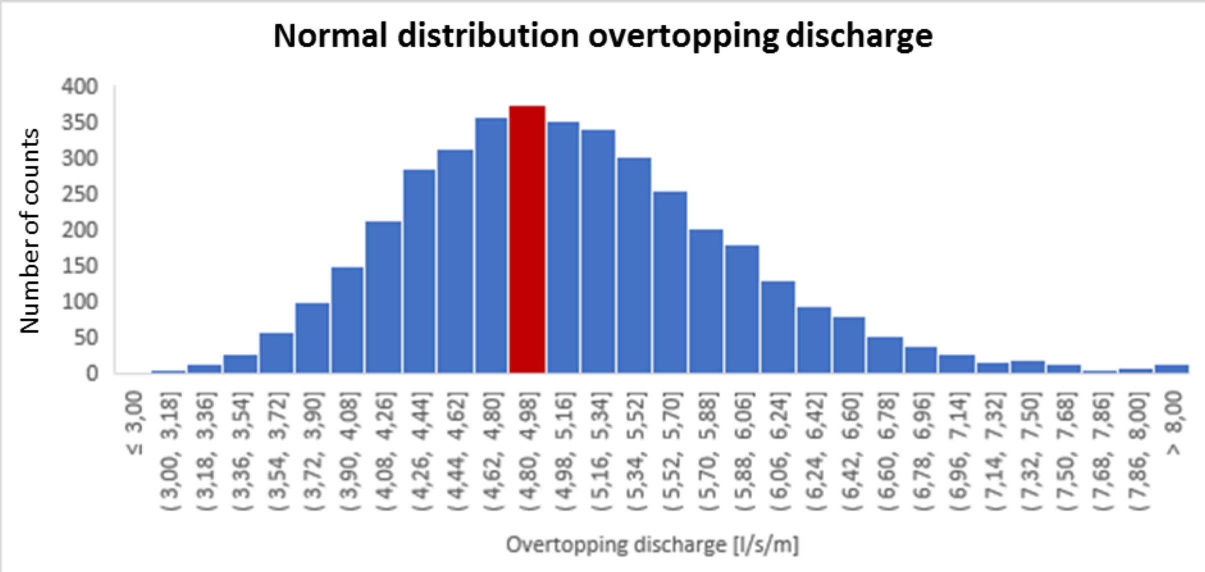
To perform the simulation the following values and deviations were used :

Variable					
Parameter		Distribution	Mean	Deviation	Unit
Bottom depth	d	Normal	2,6	0,2	m
Wave length	L	Normal	12,96	1,5	m
Wave height	H	Normal	1,7	0,03	m
Angle of wave incidence	β	Normal	6	1	Degrees
Gravitational force	g	Normal	9,81	0,001	m/s2
Water level	hw	Deterministic	0,7	-	m NAP
Height construction	h constr	Deterministic	2,526	-	m

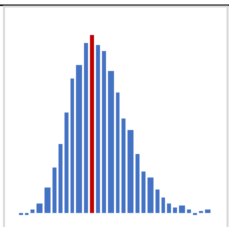
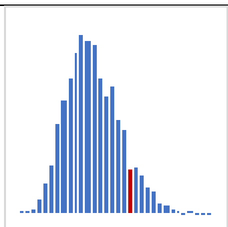
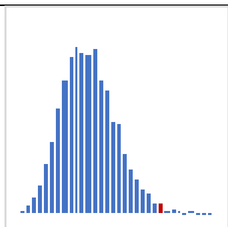
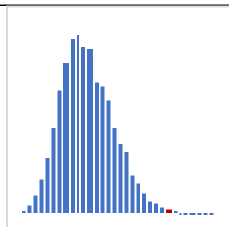
After 3993 simulations the following result was found:



The graph above shows all the simulations that were done against the Limit state function. The dots at the left of the limit state function satisfy the requirements. If now the simulations are ranked by their overtopping discharges the following normal distribution is obtained :



This graph shows that half of the simulations do not meet the requirements of no discharge larger than 5 l/m/s. This is also confirmed when considering the numbers : 2070 of the 3993 simulations do not satisfy the requirements. Which means a failure probability of 52 per cent.

The failure probability during highwater event of a storm	1/2	1/10	1/50	1/100
Height breakwater	2,526 m NAP	2,608 m NAP	2,655 m NAP	2,675 m NAP
Distribution				
Difference with deterministic approach	0	8,2 cm	12,9 cm	14,9 cm

Since the water level of 0,7 m NAP occurs only once in a thousand years the dikes around Brouwershaven and even the Brouwersdam are were designed to withstand a water level that arises once in a 4000 years. It was decided to use the failure probability of the structure of 1/10 so the total probability becomes once in a 10.000 years.

Finally the wave breaking construction will have a height of 3,55 m at which the top of the construction has to be on 2,61 m NAP.

7.1.7 Wave force against breakwater [f]

For the determination of the force of the waves against the structure the method of Sainflou was used. Since the wave does not break in front of the structure, interference will occur with the reflected wave resulting in the still water level in front of the structure rising with a value of :

$$h_0 = \frac{1}{2} k H_{in}^2 \coth(kd) = \frac{1}{2} \cdot 0{,}485 \cdot 1{,}58^2 \cdot \coth\big(0{,}485 \cdot (0{,}7 + 2{,}6)\big) = 0{,}654\ [m]$$

With:

- h_0 :Rise of the still water level [m]
- k :Wave number [-] $k = \frac{2\pi}{L}$
- L :Wave length [m]
- H_{in} :Incoming wave height [m]
- d :Depth [m]

This leads to a maximum water pressure p_1 at the mean water level of:

$$p_1 = \rho \cdot g \cdot (H_{in} + h_0) = 1024{,}8 \cdot 9{,}81 \cdot (1{,}7 + 0{,}654) = 22{,}43\ kN/m^2$$

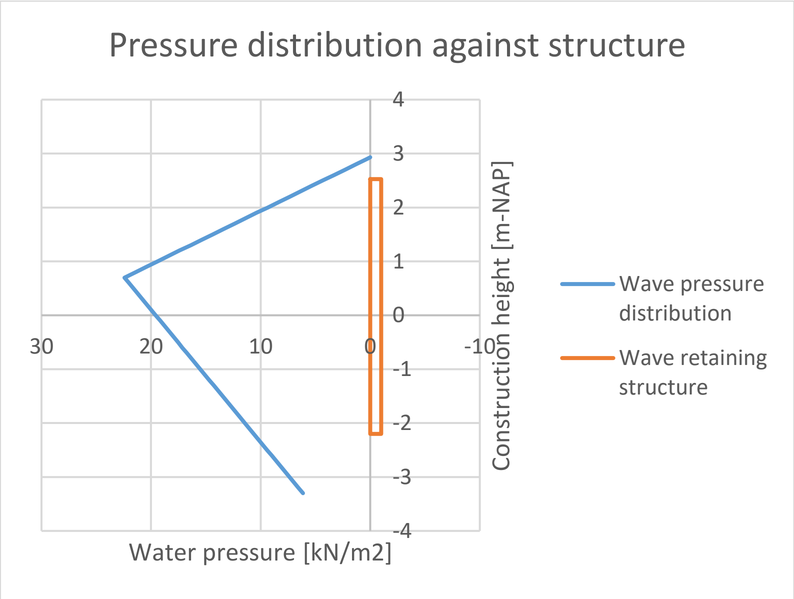
With:

- ρ :Mass density of salt water $[kg/m^3]$

And a water pressure p_0 on the bottom of:

$$p_0 = \frac{\rho \cdot g \cdot H_{in}}{\cosh(k \cdot d)} = \frac{1024{,}8 \cdot 9{,}81 \cdot 1{,}7}{\cosh(0{,}485 \cdot 3{,}3)} = 6{,}15\ kN/m^2$$

It is assumed that the pressure develops linear between these points as can be seen in the graph below. The force on the structure can now be determined by taking the area between the pressure distribution and the structure. This results in a force of 64,23 kN/m.



7.2 Wooden mole

In this variant the whole structure except the foundation piles is designed out of wood. The harbour mole will have to be able to carry the load of a small delivery van. The reason for this is that the mole will also have the function of a pier for the fishing boats and a river cruise ship which needs supplies and want to unload their goods.

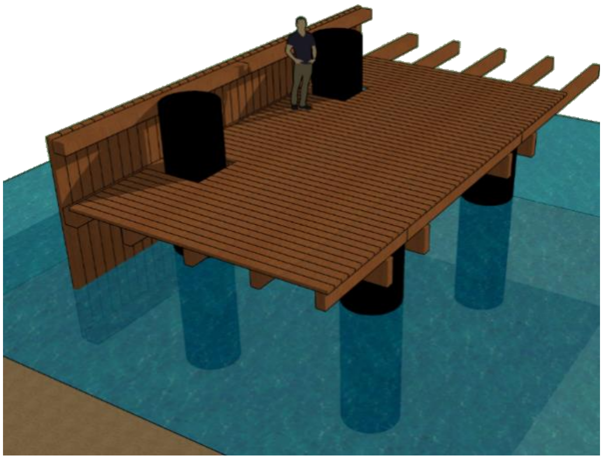


Figure 36: Representation of a wooden mole founded on steel tube piles

7.2.1 Determining the dimensions of the breakwater

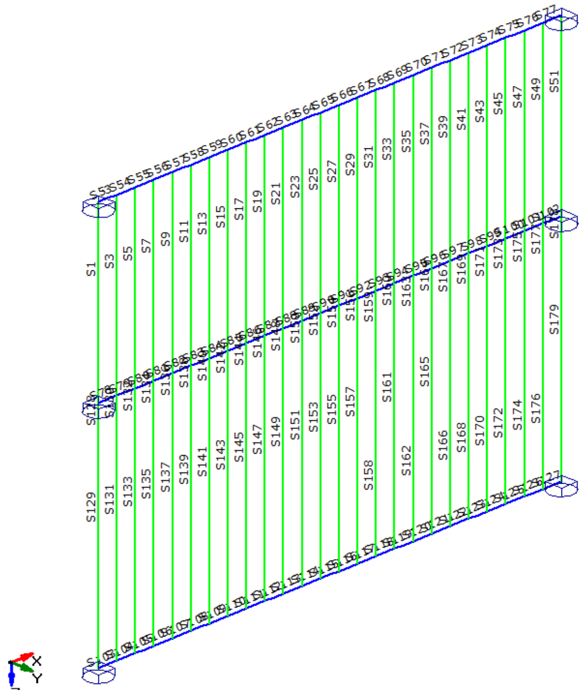
For the first dimensioning of the breakwater different dimensions of the piers in the existing harbour were used. After the first check it could be concluded that these dimensions could not carry the required loads. After optimisation the following dimensions were obtained:

- Cross members:
  - Length: 4400 mm
  - Height: 200 mm
  - Width: 250 mm
- Wooden boards:
  - Length: 3450 mm
  - Height: 80 mm
  - Width: 200 mm

The check of these dimensions was done with the help of a model of the construction that was built in the MatrixFrame programme. This programme helped to determine the internal force, bending of the construction and the forces on the supports of the structure. A figure of this model ca be found in the figure below. Only free rotations around the Z-axis is possible in the supports. This was modelled this way in case the connection between the separate cross beams would fail.

- Position upper cross beam: 2,61 m NAP
- Position middle cross beam: 0,9 m NAP
- Position lower cross beam: -1,36 m NAP
- Centre-to-centre distance of wooden boards: 0,17 m

The loads that were that were introduced on this model were the wave load and the self-weight of the structure.



Figurer 37: Calculation model of wooden break water in MatrixFrame

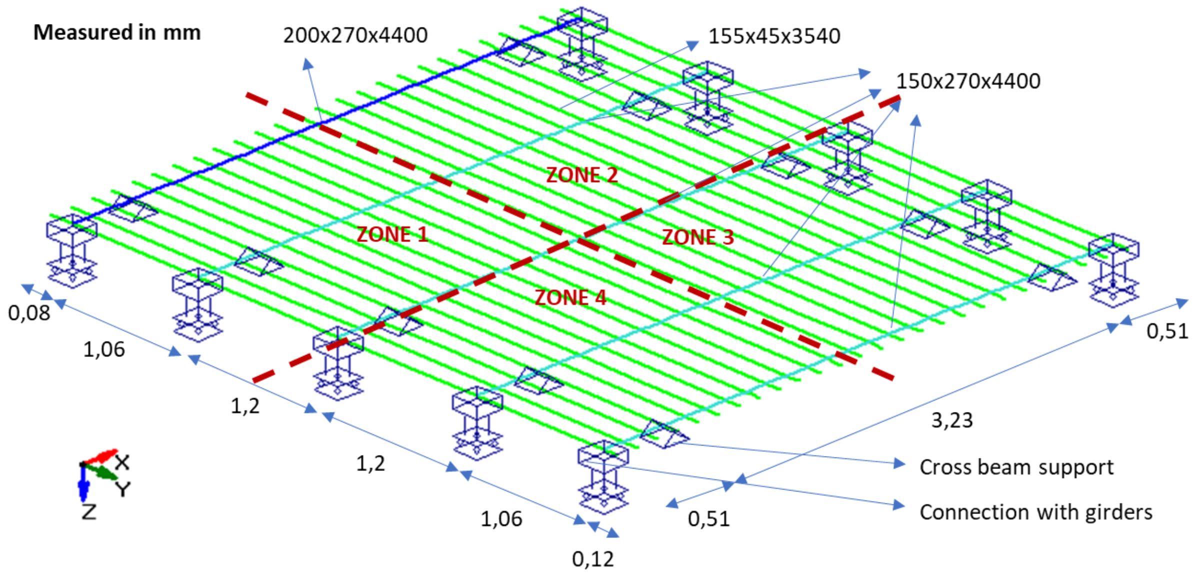
The different force diagrams that came out of MatrixFrame and the checks that were preformed can be found in appendix 18.

7.2.2 Determining the dimensions of the jetty

This jetty must be able to carry the load of a small delivery van since the ships that moor on the side of the jetty should be able to load and unload their goods. The use of the following report is used to get an idea which what load this construction has to bear:

- Report "Ingenieursburo Maters en De Koning: Ontwerp Brug Terneuzen".
  The design in this report had to withstand a the load of a small delivery van. This is based on a vehicle with a weight of 3.5 tons, where the wheels are spaced apart at a distance of 2 and 3.5 m.
- Report "Ingenieursburo Maters en De Koning: Kadeconstructies Brouwershaven"
  In this report a distributed load of 20 kN/m was assumed for traffic.

To determine the dimensions of the different wooden parts of the construction two models were used. A 3D model to determine the dimensions of the structure for a distributed load and a 2D model to determine the dimensions of the structure for a point load. For the calculation of the internal forces in the construction, the construction was split into four zones. These zones can, supplementary to their own weight, be loaded individually, as well as collectively by a distributed load of a van.



The various force diagrams and the checks that were preformed to get the dimensions in the upper figure can be found in appendix 19.

7.2.3 Determining the dimensions of the pile foundation

For the design of the pile foundations it was decided to use steel tube piles. This was done for the following reasons:

- Presence of not exploded explosives: It is an expensive investment to investigate if there really are not exploded explosives in the area were the new harbour will be built. For this reason it is important that this area is kept as small as possible, which eliminates the method to vibrate or hammer the pile into the ground. This would create vibrations in the soil a couple of meters around the pile. Which could activate the explosives that still are in the ground.

- The construction may not fail during a head-on collision with a river cruise ship.

The calculations for designing the pile can be found in appendix 20. As a final design a pile with following dimensions was found:

- Length: 20,11 m
- Diameter: 1000 mm
- Steel thickness: 10 mm
- Design load 5000 kN

At the end of the design calculations it was found that by this depth the interaction between the pile and the soil could transfer 7322 kN. Which means that the pile can bear the total weight of the pier and breakwater since it loads the pile with a weight of 795 kN.

7.2.3.1 Check collision river cruise ship

It must be checked if the harbour mole can resist a collision with a river cruise ship, since the mole will also be used to moor river cruise ships. A list of some river cruise ships that passes near the Grevelingen lake is in appendix 21.

The largest river cruise ship that can enter the Grevelingen lake can be determined by looking at the dimensions of the only lock connecting the Grevelingen lake with the rest of the open water, namely the Grevelingen lock. This lock has the following dimensions:

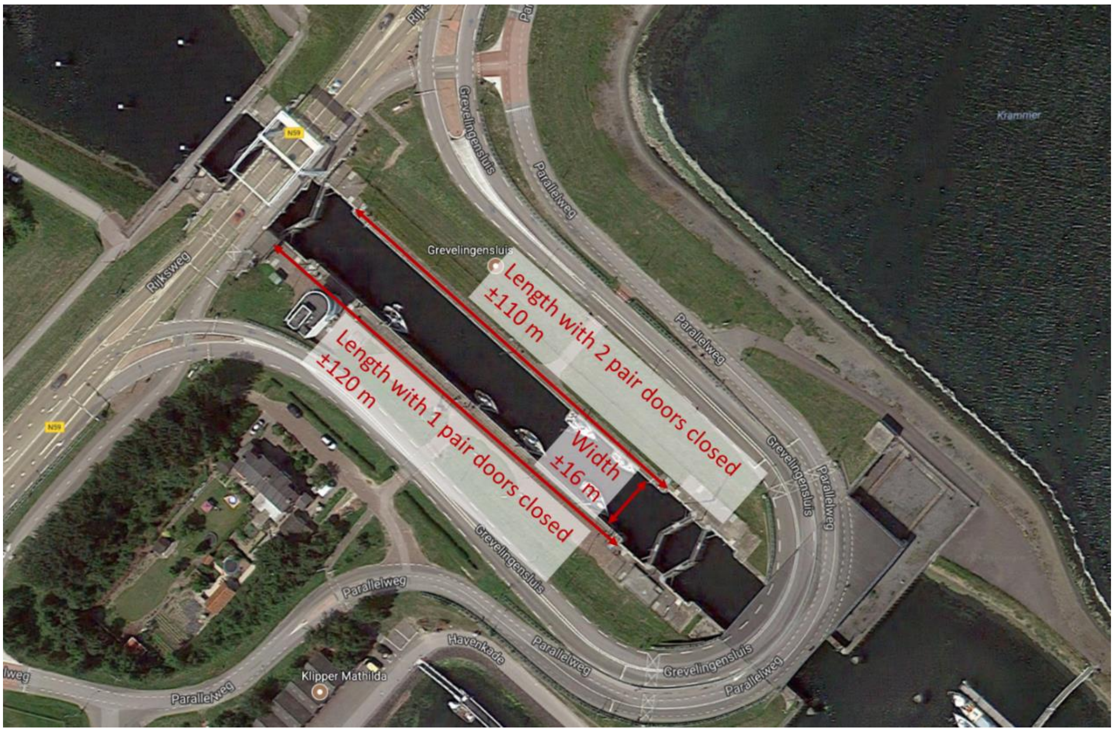


Figure 38: Dimensions of the Grevelingen lock

After checking the dimensions it can be concluded that all of the ships in the list in appendix 21 can enter the Grevelingen lake. Which means that the river cruise ship Bellucci will be the largest ship to moor on the mole.

The mole will have the largest impact when facing a head-on collision of a river cruise ship. The calculation of the energy that the piles have to bear during this collision can be found in appendix 22. It was found that the river cruise ship 'Antonio Bellucci' moving at a speed of 11,67 km/h generated a kinetic energy of 7312 kN. After checking the mole construction it turned out that it is strong enough to bear the bending moment and shear apart, but not at the same time. Further investigation to determine which cost will arise to compensate for the damage of the structures and boats after the mole and the extra cost to make the mole resistance to the head-on collisions will have to be made.

7.3 Concrete mole

In this variant the breakwater is made of 2 different materials. The first part closest to the water is made of wood as in the first variant, while the upper part and the boulevard is made of a concrete L-shape. This construction will also be founded on steel tube piles.

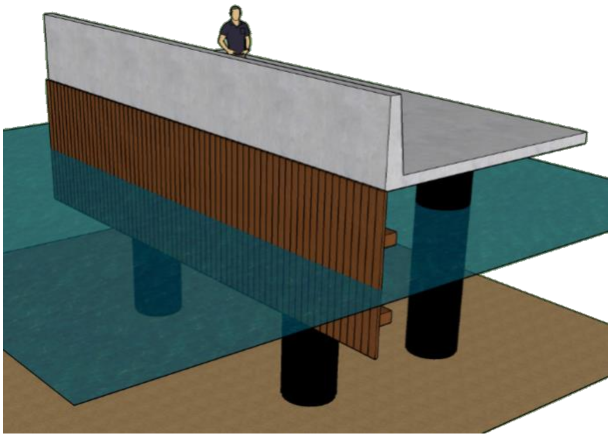
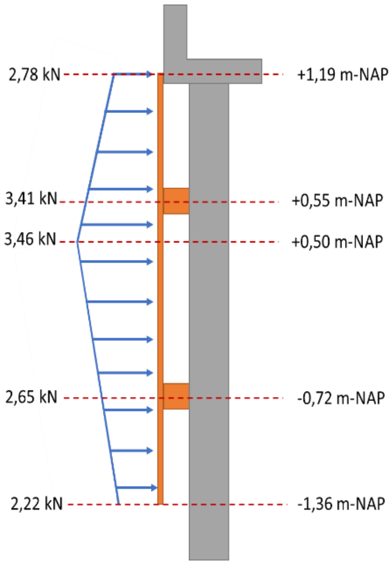


Figure 39: 3D impression of concrete mole

7.3.1 Determining the dimensions of the wooden breakwater

For the design of the wooden breakwater the first step to take was to determine what force it will have to transfer to the rest of the construction. This load was determined with the results found in appendix 23, which were then recalculated to find the load on one of the planks. The most important values of this calculation can be found in figure 40. With the loads and dimensions of the figures below a model was built in MatrixFrame. In this model the following safety factors were used:

- Permanent load factor: 1,15
- Variable load factor: 1,5



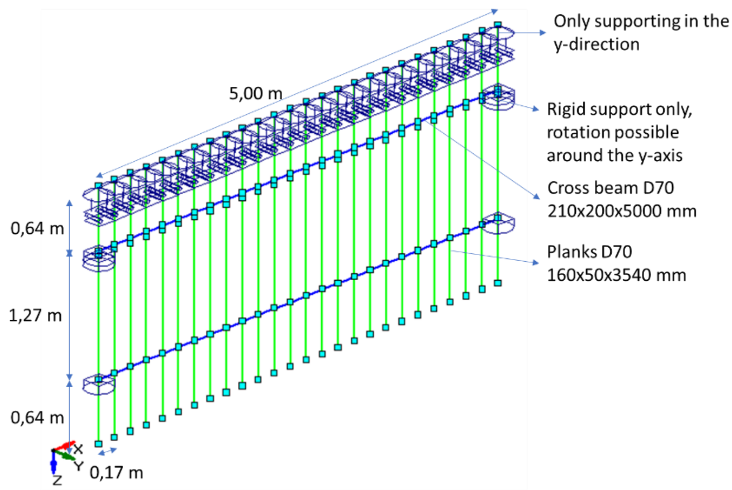


Figure 40: Model in MatrixFrame and wave load

The force diagrams and the different checks can be found in Appendix 23.

7.3.2 Determining the dimensions of the concrete breakwater and promenade boulevard

Also for the determination of the wave loads on the concrete breakwater appendix 16 was used. This time the load was converted to the force that acts on the half meter of the construction. Together with a load of 10 kN per half meter for the traffic the following concrete model was made in MatrixFrame.

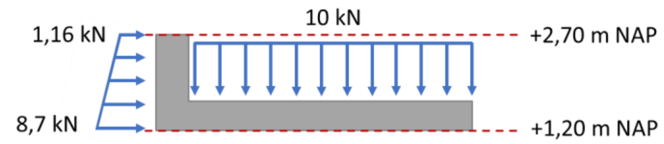


Figure 41: Different loads that acts on the concrete wave breaker and boulevard

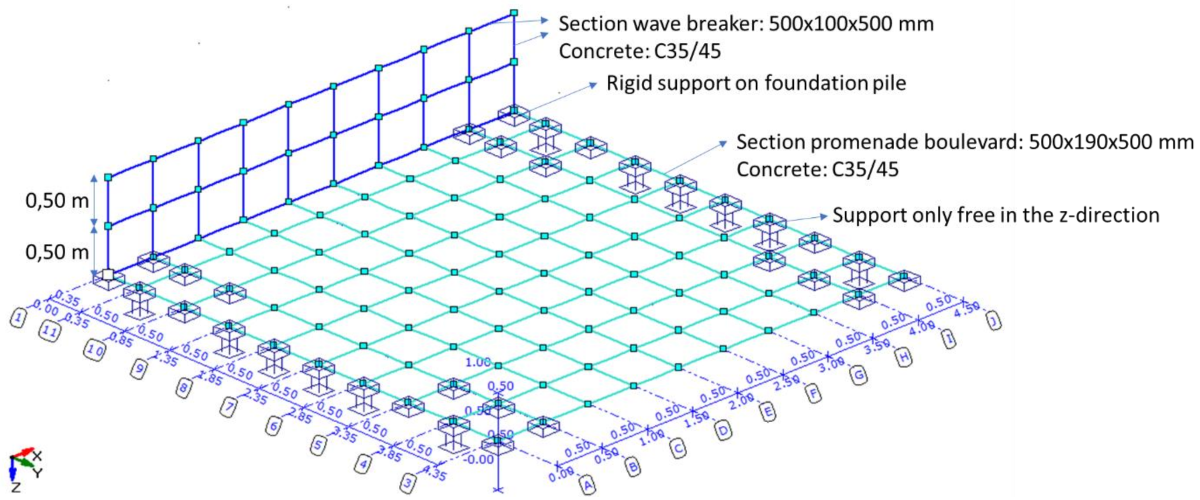


Figure 42: Model of the concrete breakwater and the promenade boulevard

For the calculations with this model still the same load factors were used as for the calculation of the wooden variant. The force diagrams produced by Matrix frame and the different checks for the concrete structure with and without reinforcement can be found in appendix 24.

Finally the following dimensions were found for the concrete slab :

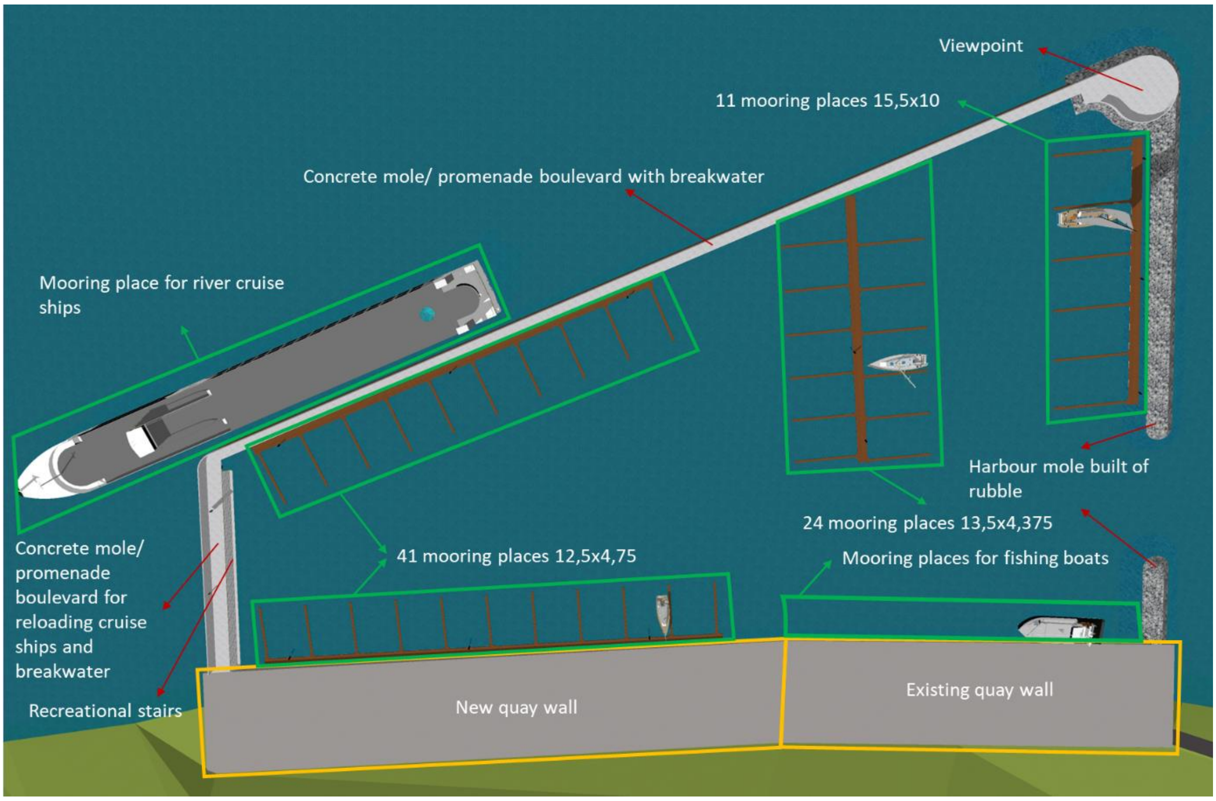
- Concrete strength class: C35/45
- Height of the concrete slab: 190 mm
- Width of the concrete slab: 4500 mm
- Length of the concrete slab: 5000 mm
- Steel strength class: B500
- Reinforcement steel diameter: 16 mm
- Reinforcement in x direction: 4 bars/m
- Reinforcement in y direction: 4 bars/m

And for the concrete wall:

- Concrete strength class: C35/45
- Height of the concrete wall: 1000 mm
- Width of the concrete wall: 100 mm
- Length of the concrete wall: 5000 mm
- Steel strength class: B500
- Reinforcement in x direction: 2 bars/m

7.4 Final design

After calculating different designs for the long harbour mole, the following design of the total harbour was made. A short description of the different design choices will be presented under the figure.

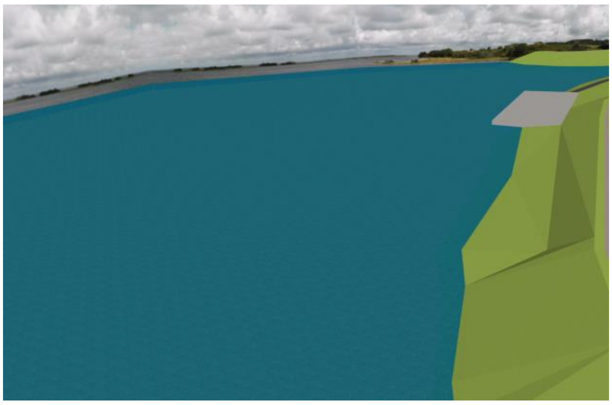


- Harbour entrance : The entrance of the harbour has been oriented thus that no traveling waves coming from the Grevelingen lake could enter the harbour.
- Harbour mole built of rubble : The harbour moles next to the harbour entrance are made of rubble. This choice was made in order to make the water current in and out of the original harbour remain in the harbour fairway. In this way the channel will keep its required depth.
- In the middle of the harbour an area has been kept free for boats to turn.
- Mooring places for fishing boats are located along the quay wall next to the harbour entrance. This way boats can easily unload their goods and transport them outside the harbour.
- Site of the mooring place for river cruise ships : this site was chosen since it lies next to the navigation channel/fairway which makes it easy to reach. Also by opting for this site only a small part of the mole has to be designed to carry the load of a small delivery van. By doing this the rest of the mole can be designed lighter and smaller.
- New artificial beach: apart from the harbour there is also the idea to create a new artificial beach (see the figure below, this beach is situated on the left side of the harbour). By doing this the visitors of the harbour or the guests from the holiday parks do not have to walk around the whole harbour to reach a beach part.

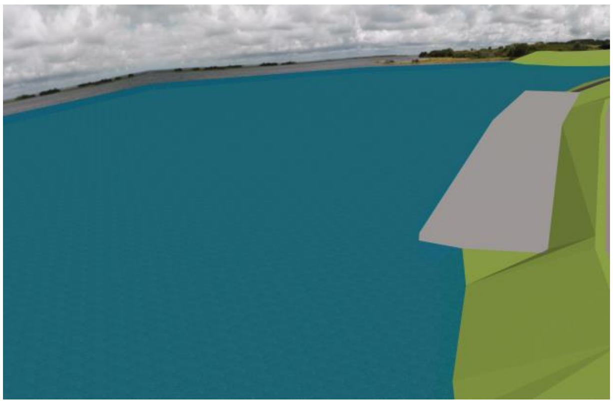
- New quay wall: This extra area can be used as parking space for the ship owner. Or as a place to build a small restaurant which would also be ideal for people sitting on the new beach.

7.5 Construction method

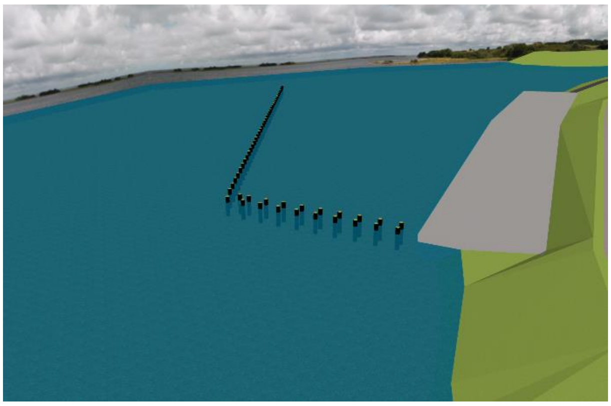
- **Phase 1: Preliminary work**
  In this phase all the preliminary aspects for starting the construction of the new harbour will be executed. Such as notifying the inhabitants of Brouwershaven and the different users of the harbour and the harbour master. This will be done by organising information evenings and notifying guests by the use of banners. Just before the construction starts a part of the building site will be used to set up a construction shack and a container for storing various equipment elements. Next to the construction site an area has to be reserved for storing the various (different building materials.



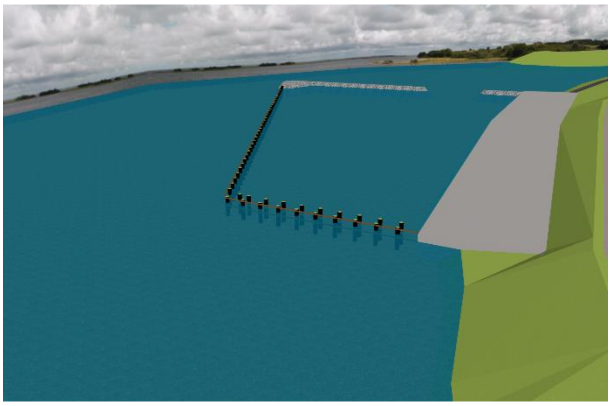
- **Phase 2: Construction new quay wall**
  For the construction of this new quay wall firstly a sheet pile wall will be driven into the ground at the contours where the quay wall has to come. The idea is to integrate the sheet pile wall as a mean component of the quay wall. For the construction of this sheet pile wall a pontoon with a crane and a vibratory hammer with variable moment will be used. After the sheet pile wall is constructed the space behind it will be filled with soil up to a certain level. The next step is to place the ground anchors to secure the stability of the sheet pile wall. After which a capstone will be placed on top of the sheet pile wall, filling the space behind it with soil and a top layer of reinforced concrete.



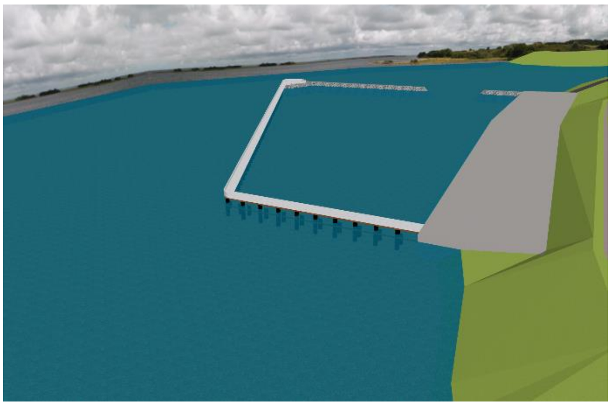
- **Phase 3: Driving the piles for the harbour mole construction**
  With the same machinery as in the previous phase steel tubular piles will be driven into the ground.



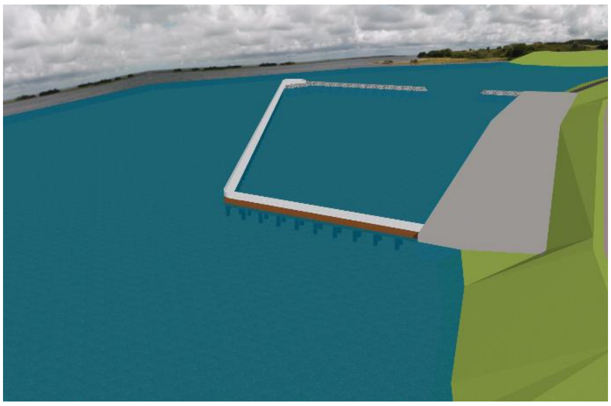
- **Phase 4: Constructing the harbour mole with rubble**
  After placing the tubular pile wall the rest and the other harbour mole will be constructed using stone rubble. For the part extending the tubular pile wall a small bulk ship will be used with a crane to dump the rubble on the right spot. For the short mole extending from the quay wall a 6by6 truck will be used to transport the rubble to the area where a crane will dump it on the right spot.



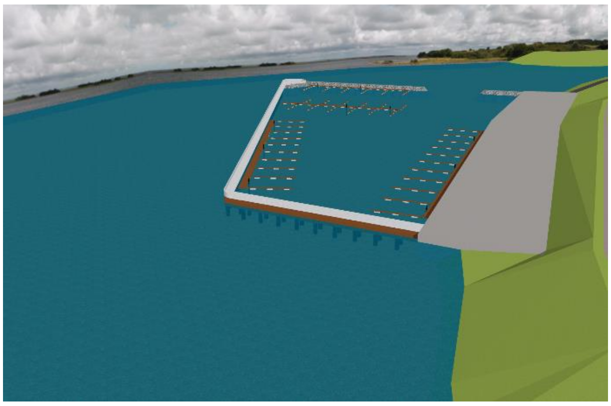
- **Phase 5: Constructing the concrete L-shape**
  While the rubble mole is constructed, the work on the concrete boulevard/ wave breaker can start. This work will be done in segments starting with the placement of the concrete formwork. After which the reinforcement will be placed and the concrete poured.



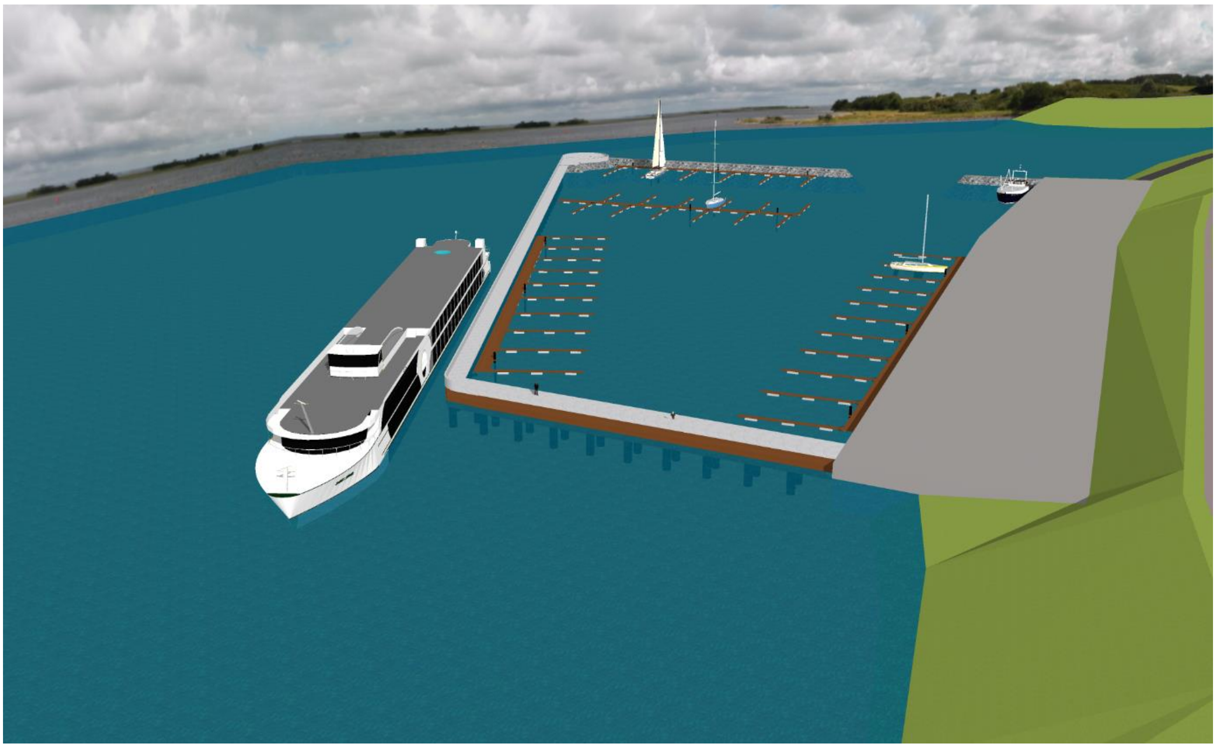
- **Phase 6: Placing the wooden wave breakers**
  In this phase the wooden wave breakers will be fixed to the tubular piles. This action can be started from the moment the first segment of concrete dried.



- **Phase 7: Constructing the floating scaffoldings**
  During the driving of the steel tubular piles of the mole, some extra piles will be driven to connect the floating scaffoldings to. These floating scaffoldings will be prefabricated and transported to the construction site, where they will be placed into the water and connected to the piles.



- **Phase 8: Removing equipment/ unused material**
  The last step before the harbour can be put into use, is to clean up the sites construction shack was and the area were the materials were stored.



7.6 Cost estimation

A total overview of the estimation of the construction costs can be found in appendix 25. After all it is estimated that the total construction cost of the harbour would be around 7 million EUR.

8 Cost-benefit analysis

Since the cost benefit analysis can be a research on its own, it was decided to only briefly describe the benefits of the various solutions.

- **Solution 0: Nothing changes**
  By not changing anything to the harbour, the revenues from ships staying over for the night or from the ships who have a permanent spot will decrease. This can directly be linked to low tide in the Grevelingen lake. Now the mean water depth in the guard lock/ harbour is around 2.3 m, at low tide the water depth will reduce to 2.05 m. Which makes it unsafe for the ships with a draught of 2 m to enter the harbour. This means that half of the time the harbour will not be accessible to larger vessels, which makes the harbour uninviting for these vessels. Since the price to moor in the harbour is directly related to the length of the ship, this also means a reduction of revenues.
- **Solution 1: Demolish the concrete sills in the guard lock and dredging of the harbour**
  In this option costs will be made to prevent the scenario described above. By realising this option the revenues from the ships will remain unchanged.
- **Solution 2: Building a new harbour in front of the guard lock**
  In this option a large investment has to be done, but it will also create new kinds of income for the harbour. By constructing a new harbour in front of the guard lock, the water depth in the harbour can be deeper so even larger ships as well as the fishing boats can moor in the harbour. Just outside the harbour a spot will be reserved for mooring a river cruise ship that can bring tourists who can visit the old city of the harbour and tour around the largest salt water lake in Europe. Also with the new artificial beach next to the harbour sailors could stop at the harbour to enjoy a day at the beach. The extra space can be placed in good use since there apparently already is a waiting list for ships who want a permanent spot in the harbour.

9 Conclusion

- Rijkswaterstaat has decided to bring back a reduced tide into the Grevelingen lake in the near future. The actions raised questions by the Gemeente Schouwen-Duiveland who is still the owner of the harbour of Brouwershaven. The Gemeente Schouwen-Duiveland wanted to know whether the stability of the construction in the harbour could be guaranteed when changing the boundary conditions. This was also the question I was asked to investigate.
- After calculating and estimating the new boundary conditions, the different harbour constructions in the new harbour were checked. The decision to check only these constructions was made because this part of the harbour was designed and built after the Grevelingen was closed by means of the Brouwersdam and the Grevelingendam. After checking the different scaffoldings and quay walls it could be concluded that there was no major threat for instability in the future. The only problem is that for the water levels that occur only once in a 100 and a 1000 years the quay walls and scaffoldings will be flooded. A bigger problem is the water depth in the harbour and guard lock, which are now at a depth of 2,30 m. This already is a minimum water depth to safely let ships with a draught of 2 m enter and leave the harbour. This safety cannot be guaranteed during low tide were the water level drops to 2,1 m. This would mean that the harbour wouldn't be accessible for half a day, which would make the harbour less interesting for ships to stay.

  To resolve this problem a brainstorm session and a multi criteria analysis was done. Which led to the following two solutions which were worked out in further detail.

  - Demolition of the sills in the guard lock and the dredging of the harbour.
  - Constructing a new harbour in front of the guard lock
- Both these solutions were worked out in further detail, after which an estimation of the construction cost was made.

  For the first solution an investigation has to be done to determine what concrete class and how the reinforcement was positioned in the guard lock. This will be needed to be able to check if the construction will still be stable after demolishing the sills on the bottom slab. After the stability has been approved the demolition can be started, after which the harbour has to be dredged as well. Then a bulkhead will have to be made to place in the notches of the guard lock. This bulkhead can then be used in case the water level in the Grevelingen lake exceeds its limit. Finally it was estimated that the realisation of this solution would cost 300.000 EUR.

  For the second solution a whole lot more has to be designed. But because of the strict time schedule it was decided to make a design of one of the harbour structures only. In this report 2 different designs for the harbour mole were made. Hereby the construction was checked for the following loads:

  - The load of the waves
  - The load of a small delivery van
  - The load provoked by a river cruise ship head-on collision.

  Eventually it was estimated that the realisation of this project would cost 7.000.000 EUR.

This last solution is much more expensive than the previous one, but if it were redesigned smaller and more optimised it could present opportunities for the harbour to expand its capacity and make Brouwershaven even more attractive for new functions.

10 Recommendations

- Further investigation if the construction of an tidal power plant into the Brouwersdam is profitable.
- Optimising the constructions and the layout of the new harbour in front of the guard lock.
- Designing a construction to moor the river cruise ship.

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Appendix 1: Rijksoverheid Kabinetsbesluit (Dutch state government bill)

Rijksoverheid

Terugbrengen van getij herstelt waterkwaliteit Grevelingen en Volkerak-Zoommeer

Rijksoverheid.nl | *Nieuwsbericht* | *10-10-2014*

Op de Grevelingen komt er weer eb en vloed. Het Volkerak-Zoommeer wordt weer zout en krijgt ook weer eb en vloed. Hierdoor herstelt de waterkwaliteit van beide, momenteel stilstaande en afgesloten, wateren. De ministerraad heeft hiermee ingestemd op voorstel van minister Schultz van Haegen van Infrastructuur en Milieu. De verbeterde waterkwaliteit is goed voor de natuur, recreatie en toerisme, landbouw, schelpdierteelt en de kwaliteit van de leefomgeving. Voorafgaand aan het zout maken van het Volkerak-Zoommeer, worden maatregelen getroffen om de zoetwatervoorziening van gebieden rondom het meer veilig te stellen.

In de ontwerp-Rijksstructuurvisie Grevelingen en Volkerak-Zoommeer kiest het kabinet voor een bijzondere manier van bekostiging van de maatregelen. Rijk en regio nodigen marktpartijen en gebruikers van het gebied uit om samen met hen de kosten te dekken voor het gebied. Terugbrengen van zout en getij is een duurzame oplossing, met positieve effecten voor waterkwaliteit en natuur. Ook ontstaan er nieuwe mogelijkheden voor schelpdierteelt, recreatie, landbouw en de stedelijke ontwikkeling van Bergen op Zoom. De regio wil dan ook een actieve rol spelen in de bekostiging en uitvoering van de plannen van het kabinet. De totale kosten van die uitvoering worden geraamd op ongeveer 350 miljoen euro, te financieren door Rijk, regio, marktpartijen en gebruikers. Voor ongeveer 100 miljoen euro zijn al afspraken gemaakt of in voorbereiding.

Sinds de afsluiting van de Grevelingen en het Volkerak-Zoommeer, is de waterkwaliteit in beide watergebieden achteruit gegaan. Het zoute water van de Grevelingen is in de diepere delen inmiddels regelmatig vrijwel zuurstofloos, met als gevolg schade aan bodemleven, kreeften en vissen en verlies van aantrekkingskracht voor duikers. Het zoete Volkerak-Zoommeer kampt met vertroebeling, te hoge concentraties voedingsstoffen en regelmatig terugkerende overlast van blauwalgen. Die overlast is de laatste jaren afgenomen met de komst van een exotische mosselsoort (quaggamossel), alleen is dit waarschijnlijk niet een blijvende en betrouwbare oplossing voor de slechte waterkwaliteit.

De ontwerp-Rijksstructuurvisie Grevelingen en Volkerak-Zoommeer, het bijbehorende milieueffectrapport (MER) en de maatschappelijke kosten-batenanalyse (MKBA) zijn van 21 oktober tot en met 1 december 2014 te raadplegen bij overheden in de regio en via www.zwdelta.nl/rgv en www.platformparticipatie.nl. Tot en met 1 december 2014 is er gelegenheid een zienswijze in te dienen. Het kabinet betrekt deze zienswijzen bij het definitief vaststellen van de rijksstructuurvisie.

Appendix 2: Concept claims, boundary conditions and wishes in case of sluice caisson construction Brouwersdam

Concept 161016

Topeisen 50 cm getij
GREVELINGENMEER

- Het peil op het Grevelingenmeer mag:
  - 90% van de tijd niet hoger zijn dan NAP +0,05 m;
  - 99% van de tijd niet hoger zijn dan NAP + 0,10 m;

- In de 1% dat het peil van NAP + 0,10 m wordt overschreden gelden de volgende bovengrenswaarden:
  - 1 x per 10 jaar max. NAP + 0,3 m;
  - 1 x per 100 jaar max. NAP + 0,5 m;
  - 1 x per 1.000 jaar max. NAP + 0,7 m.

- Het peil op het Grevelingenmeer mag (gemeten bij meetpaal BOM1):
  - 90% van de tijd niet lager zijn dan NAP -0,45 m;
  - 100% van de tijd niet lager zijn dan NAP -0,50 m.

- Het middenpeil waaromheen de getijslag plaatsvindt dient (gemeten bij meetpaal BOM1) gemiddeld NAP -0,20 m te zijn, met een toegestane afwijking van + of – 2,5 centimeter. Uitzondering hierop zijn extreme omstandigheden om calamiteiten buiten de invloedsfeer van de Opdrachtnemer. Zie voor definitie extreme omstandigheden en calamiteiten document definitietabel, bijlage XX.

- Indien zeespiegelstijging hiertoe aanleiding geeft moet het midden peil op het Grevelingenmeer kunnen worden verhoogd en het verschil tussen hoog- en laagwater zoals hierboven vermeld kunnen worden gehandhaafd, zonder ingrijpende aanpassingen aan de kunstwerken waarmee de waterstanden worden gereguleerd.

Waterkwaliteit:

- Er dient twee maal per etmaal via een doorlaatmiddel in de Brouwersdam een getijbeweging op te treden op het gehele Grevelingenmeer. Uitzondering hierop zijn extreme omstandigheden en calamiteiten buiten de invloedsfeer van de Opdrachtnemer. Zie voor definitie extreme omstandigheden en calamiteiten document definitietabel, bijlage XX.

- Het verschil tussen hoog- en laag water (de getijdenslag) dient per getij gemiddeld 50 cm per dag te zijn

Locatie en breedte doorlaatmiddel

- Het doorlaatmiddel in de Brouwersdam dient gerealiseerd te worden (binnen een zoekgebied van 800 meter) ten noorden van "Port Zélande"/ Kabelaarsbank zoals weergegeven op bijgevoegde kaart conform bijlage XX.
- Het doorlaatmiddel in de Brouwersdam dient in totaal maximaal 400 meter lang te zijn.

Morfologie:

- In het gehele meer geldt een maximale waterstroming van 0,25 m/s met uitzondering van het gebied binnen de veiligheidsmarkering.
- Het oppervlakte van de Bollen van de Ooster in de voordelta mogen niet afnemen als gevolg van de aanleg en het gebruik van het doorlaatmiddel.

Natuur:

- Het aantal hectare landoppervlak van de eilanden in het Grevelingenmeer en in de buitendijkse gebieden dient bij middenpeil gelijk te zijn aan de huidige situatie
- Voor elk nieuw kunstwerk (en toegevoegde functies) geldt: Vismortaliteit voor de aal dient maximaal 0,7 % te bedragen per passage.
- Voor elk nieuw kunstwerk (en toegevoegde functies) geldt: Vismortaliteit voor alle vissoorten, met uitzondering van de aal, dient maximaal 1,0 % te bedragen per passage.
- Voor elk nieuw kunstwerk (en toegevoegde functies) geldt: mortaliteit zeezoogdieren dient maximaal 0,01% te bedragen per passage.

Waterkering

- De waterkeringen en waterkerende kunstwerken dienen te allen tijde te voldoen aan de vigerende Waterwet en het Wettelijk Toetsinstrumentarium (WTI) 2017.
- De deuren van het Doorlaatmiddel dienen op basis van hun eigen gewicht te kunnen worden gesloten (bij niet functioneren aandrijvingsmechanisme).

- Tijdelijke waterkeringen die tijdens de bouw de waterkerende functie van een primaire waterkering overnemen worden ook aangemerkt als primaire waterkering in de zin van de Waterwet en dienen te allen tijde als zodanig te functioneren;

- Voor elk nieuw kunstwerk geldt een functioneel gebruik van minimaal 100 jaar en dient te worden ontworpen volgens de geldende richtlijnen;

- Elk nieuw kunstwerk moet zo worden uitgevoerd dat vandalisme geen effect heeft op de beschikbaarheid en betrouwbaarheid van de functies van het kunstwerk;

Wegverkeer

- Het huidige scheepvaart- en wegverkeer dient tijdens de uitvoering van werkzaamheden niet gestremd te worden. Hinder is in overleg met en na toestemming van de (vaar)wegbeheerder toegestaan.

- Het huidige wegverkeer op de locatie(s) waar (een) nieuw(e) kunstwerk(en) word(t)(en) ingezet om de getijbeweging op het Grevelingenmeer te bewerkstelligen moet na voltooiing van de bouw hiervan beschikken over het wegprofiel dat voldoet aan de eisen van het wegtracé waarin het kunstwerk is ingebouwd.

Leefbaarheid/Milieu/duurzaamheid

- "Het werk dient bij te dragen aan een duurzame leefomgeving door:
  - Het toepassen van de Omgevingswijzer en kansen uit de Omgevingswijzer te implementeren in het project;
  - RWS doelstellingen Energie en klimaat
  - 20% energievermindering te bereiken ten opzichte van 2009;
  - Energieneutraal of energieleverend te opereren;

- "Het werk dient bij te dragen aan een circulaire economie door:
  - Onderzoek uit te voeren naar kansen voor inzet van circulaire materialen;
  - De doelstelling te bereiken 20% betere milieuprestatie van materialen in 2020 ten opzichte van 2010."

- Het Werk dient als geheel te worden in gepast in het (delta)landschap door gebruikmaking van de volgende documenten:
  - Kader ruimtelijke kwaliteit en vormgeving (Rijkswaterstaat);
  - Landschapsplannen wegen N59 en N57 (Rijkswaterstaat);
  - Landschapsstudie Brouwersdam 2020 (Rijkswaterstaat);
  - Provinciale beleidsnota's."

Gebruiksfuncties

- Aanwezige (recreatieve) voorzieningen (strandjes Grevelingenmeer, strand buitenzijde Brouwersdam, steigers, jachthavens, etc.) dienen schadevrij, veilig, toegankelijk, functioneel en bereikbaar te blijven na uitvoering van de maatregelen. Als dat niet mogelijk blijkt dienen mitigerende maatregelen te worden getroffen.

- De bevaarbaarheid van vaarwegen en vaargeulen dient gewaarborgd te blijven voor beroepsvaart en recreatievaart. Aanpassingen aan vaarwegen en vaargeulen dienen te voldoen aan Richtlijn Vaarwegen 2011 en het Binnenvaartpolitiereglement.

- De veiligheid van recreanten dient te allen tijde geborgd te zijn. Specifieke aandacht dient te worden verleend aan de stroomsnelheden bij de doorlaat in de Brouwersdam (zowel binnenzijde als buitenzijde). Het gebied waarin stroomsnelheden groter zijn dan 0,25 m/s dient middels een fysieke afscherming te worden afgeschermd voor recreanten.

Appendix 3: Python programme to determine reduced tide with sluice caisson

```
from pylab import *
from numpy import *
import os;
import matplotlib.pyplot as plt
close ('all')
print 'Made by Fons De Vlieger'
print 'Aquavia'
print '13/07/2017\n'

close ('all')
os.chdir('C:\Users\AThou\Desktop\Python')            #Verplaatsen werkdirectory

#Parameters
Zeestijging = 0
uitzondering = 0
waterstand =Zeestijging + genfromtxt('BRBU2016waterstand3.txt',deletechars=None,usecols=[2])   #dit is het getij buiten Brouwersdam
where_are_NaNs = isnan(waterstand)
waterstand[where_are_NaNs] = 6.70

Tijd = linspace(0,366,size(waterstand))
plot(Tijd,waterstand)

nrhoog = 0
nrlaag = 0
h = 0
l = 0
t = 0

for j in range(2):
    hoogwater = zeros(nrhoog)
    hoogtijd = zeros(nrhoog)
    getij = zeros(nrhoog+nrlaag)
    tijdgetij = zeros(nrhoog+nrlaag)
    laagwater = zeros(nrlaag)
    laagtijd = zeros(nrlaag)

    for i in range(size(waterstand)):
        if (waterstand[i-4] < waterstand [i-2]) and (waterstand[i-3] < waterstand [i-2]) and (waterstand[i-1] <= waterstand [i-2]) and (waterstand[i] <= waterstand [i-2]) and waterstand[i-2] > 50:
            nrhoog = nrhoog + 1
            if j==1:
                hoogwater[h] = waterstand [i-2]
                getij[t] = waterstand [i-2]
                hoogtijd[h] = Tijd[i-2]
                tijdgetij[t] = Tijd[i-2]
                h = h+1
                t = t+1

        elif (waterstand[i-4] > waterstand [i-2]) and (waterstand[i-3] > waterstand [i-2]) and (waterstand[i-1] >= waterstand [i-2]) and (waterstand[i] >= waterstand [i-2]) and waterstand[i-2] < 23:
            nrlaag = nrlaag + 1
            if j==1:
                laagwater[l] = waterstand [i-2]
                getij[t] = waterstand [i-2]
                laagtijd[l] = Tijd[i-2]
                tijdgetij[t] = Tijd[i-2]
                l = l+1
                t =t+1
gemwater = (mean(hoogwater)+mean(laagwater))/2.0
print gemwater

plot(hoogtijd,hoogwater,'r')
plot(laagtijd,laagwater)
plot([0,366],[gemwater,gemwater],'y',linewidth=2)
legend(['Waterstand Noordzee gemeten om de 10 min.','Hoogwaters','Laagwaters','gemiddeld waterpeil Noordzee'], ncol=3, fancybox=True, shadow=True)
xlim(0,366)
xlabel('Dagen van het jaar 2016')
ylabel('Waterstand in cm-NAP')

aantal = 0
a = 0

for j in range(2):
    getijverschil = zeros(aantal)
    for i in range (size(getij)):
        if (getij[i-1] < 23 and getij[i] > 50) or (getij[i-1] > 50 and getij[i] < 23):
            aantal = aantal + 1
            if j==1:
                getijverschil[a] = abs(getij[i-1] - getij[i])
                a = a+1
print 'Gemiddelde waterstand:', mean(waterstand),(mean(hoogwater)+mean(laagwater))/2.0
print 'Gemiddelde getij:', mean(getijverschil)

#Simulatie getij Noordzee met meer datapunten
#Parameters
figure()
Ampl = mean(getijverschil)/200.0     #amplitude getij
d = 8          #dit is de diepte van de drempel>
b = 108.0        #dit is het kombergend oppervlak van het haven Brouwershaven, in km2>
Bs = 8*15          #dit is de doorlatende breedte van de kering>
Qr = 0           #dit is de afvoer van het gemaal>
dt = 10           #dit is de tijdstap in minuten >
NofTides = 5  #dit is het aantal getijden dat je wilt doorrekenen>
Tt = 12.5        #dit is de getijperiode in uren>

#Omzetten gegevens in andere eenheid
g = 9.81
Tt = Tt *3600   #Getijperiode in seconden
b = b * 1000000 #Oppervlakte in m2
dt = dt * 60

#Reeksen met waterpeil
H1 = (waterstand)/100.0
h2 = zeros(size(H1))
h3 = zeros(size(H1))

#Resetten opties
umin = 0
umax = 0
omega = 2 * pi/Tt
nr = NofTides * Tt /dt
T = 0
check = 1
h2[0] = H1[0]
h3[0] = d - mean(waterstand)/100.0 - 0.20

for i in range(size(H1)-1):
    H1[i] = H1[i] + d                                     #Waterdiepte tot drempel
    a = h2[i] * Bs                                                 #Oppervlakte doorstroomopening
    q = 0
    qp = 0
    #if (H1[i]-h3[i]) > 1.5:
        #check = 1

    if H1[i] > h3[i] and h3[i] < d + 0.05 + uitzondering and check == 1:
        if h3[i] > (2.0/3.0)*H1[i]:
            h2[i+1] = h3[i]
            if (H1[i] == h2[i+1]):
                q = 0
            else:
                q = a * sqrt(2.0 * g * abs(H1[i]-h2[i+1]))*(H1[i]-h2[i+1])/abs(H1[i]-h2[i+1])
        else:
            h2[i+1] = 2.0 * H1[i] / 3.0
            if H1[i] <= 0:
                q = 0
            else:
                q = a * sqrt(2/3.0*g*H1[i])
    elif H1[i] < h3[i] and h3[i] > d - 0.45 - uitzondering:
        #check = 0
        if H1[i] > 2/3.0*h3[i]:
            h2[i+1] = H1[i]
            if h3[i] == h2[i+1]:
```

```
                q = 0
            else:
                q = -a * sqrt(2.0*g*(h3[i]-h2[i+1]))
        else:
            h2[i+1] = 2/3.0 * h3[i]
            if h3[i] <= 0:
                q = 0
            else:
                q = -a * sqrt(2/3.0*g*h3[i])
    else:
        h2[i+1] = H1[i]

    if a == 0:
        u = 0
    else:
        u = q/a
    if T > (NofTides - 1) * Tt:
        if u > umax:
            umax = u
            Hflood = h2[i+1]
        if u < umin:
            umin = u
            Hebb = h2[i+1]
    if i<(size(H1)-1):
        h3[i+1] = h3[i] + q/b * dt + Qr/b * dt
        T = T+dt

#H1[nr-1] = h2[nr-1] = h3[nr-1] = -0.2+d+0.23

plot (linspace(0,366,size(H1)),H1-8,'y',linewidth=3)
plot (linspace(0,366,size(H1)),h2-8,'k--',linewidth=1)
plot (linspace(0,366,size(H1)),h3-8,'r',linewidth=2)
plot ([0,366],[0.05,0.05],'b--',linewidth=3)
plot ([0,366],[- 0.45,- 0.45],'b--',linewidth=3)
legend(['Waterpeil Noordzee','Waterpeil in doorlaatconstructie','Waterpeil in Greveleningenmeer','toegelaten peil'], ncol=3, fancybox=True, shadow=True)

xlabel('Aantal dagen')
xlim(0,10)
ylabel('Waterstand in m-NAP')
ylim(-2,4)
title('Getij in het Grevelingenmeer na constructie doorlaat')
grid ('on')


# getij in Brouwershaven

figure()

db = 2.3          #dit is de diepte van de drempel>
b = 0.04        #dit is het kombergend oppervlak van het haven Brouwershaven, in km2>
Bs = 8.5           #dit is de doorlatende breedte van de kering>
Qr = 0           #dit is de afvoer van het gemaal>

#Omzetten gegevens in andere eenheid
b = b * 1000000 #Oppervlakte in m2

#Reeksen met waterpeil
h4 = zeros(size(H1))
h5 = zeros(size(H1))

#Resetten opties
umin = 0
umax = 0
omega = 2 * pi/Tt
nr = NofTides * Tt /dt
T = 0
check = 0
h3 = h3 - 8
h4[0] = h3[0] + db
h5[0] = h3[0] + db

for i in range(size(h3)-1):
    h3[i] = h3[i] + db                                          #Waterdiepte tot drempel
    a = h4[i] * Bs                                                          #Oppervlakte doorstroomopening
    q = 0
    qp = 0

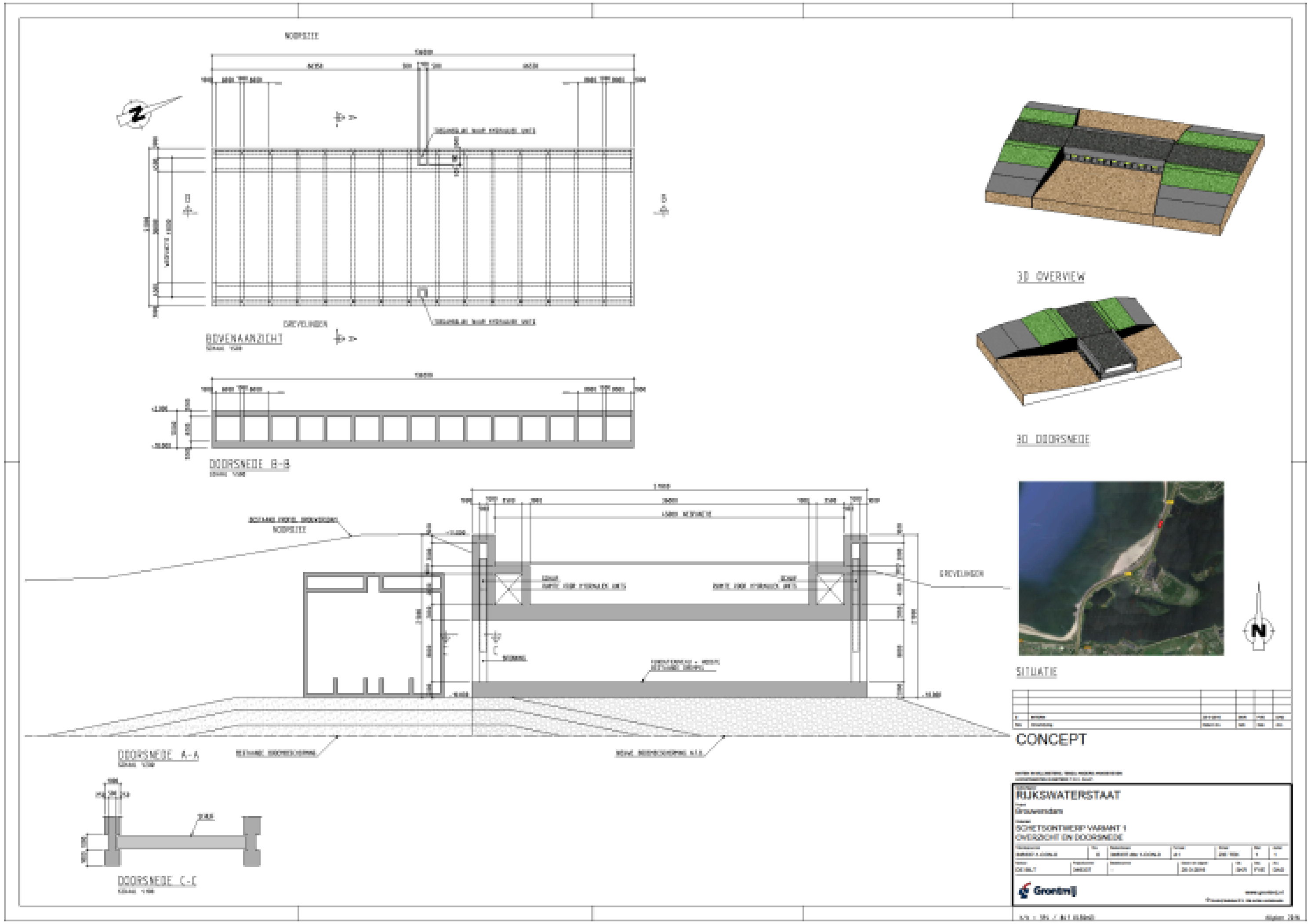
    if h3[i] > h5[i]:
        if h5[i] > (2.0/3.0)*h3[i]:
            h4[i+1] = h5[i]
            if (h3[i] == h4[i+1]):
                q = 0
            else:
                q = a * sqrt(2.0 * g * abs(h3[i]-h4[i+1]))*(h3[i]-h4[i+1])/abs(h3[i]-h4[i+1])
        else:
            h4[i+1] = 2.0 * h3[i] / 3.0
            if h3[i] <= 0:
                q = 0
            else:
                q = a * sqrt(2/3.0*g*h3[i])
    elif h3[i] < h5[i]:
        if h3[i] > 2/3.0*h5[i]:
            h4[i+1] = h3[i]
            if h5[i] == h4[i+1]:
                q = 0
            else:
                q = -a * sqrt(2.0*g*(h5[i]-h4[i+1]))
        else:
            h4[i+1] = 2/3.0 * h5[i]
            if h5[i] <= 0:
                q = 0
            else:
                q = -a * sqrt(2/3.0*g*h5[i])
    else:
        h4[i+1] = h3[i]

    if a == 0:
        u = 0
    else:
        u = q/a
    if T > (NofTides - 1) * Tt:
        if u > umax:
            umax = u
            Hflood = h4[i+1]
        if u < umin:
            umin = u
            Hebb = h4[i+1]
    if i<nr-1:
        h5[i+1] = h5[i] + q/b * dt + Qr/b * dt
        T = T+dt

#H1[nr-1] = h2[nr-1] = h3[nr-1] = -0.2+d+0.23

plot (linspace(0,366,size(H1)),h3 - db,'g',linewidth=2)
plot (linspace(0,366,size(H1)),h4 - db,'b',linewidth=1)
plot (linspace(0,366,size(H1)),h5 - db,'r--',linewidth=2)
#plot ([0,NofTides],[d-mean(waterstand)/100.0 + 0.05,d-mean(waterstand)/100.0 + 0.05],'b--',linewidth=3)
legend(['waterpeil Grevelingenmeer','Waterpeil in keersluis','Waterpeil in haven Brouwershaven','gemiddeld waterpeil Grevelingen'], ncol=3, fancybox=True, shadow=True)
xlabel('Aantal dagen')
xlim(0,2.5)
ylabel('Waterstand in m-NAP')
ylim(-0.8,0.5)
title('Getij in haven Brouwershaven na constructie doorlaat')
grid ('on')
show()
```

Appendix 4: Concept design Sluice caisson Brouwersdam

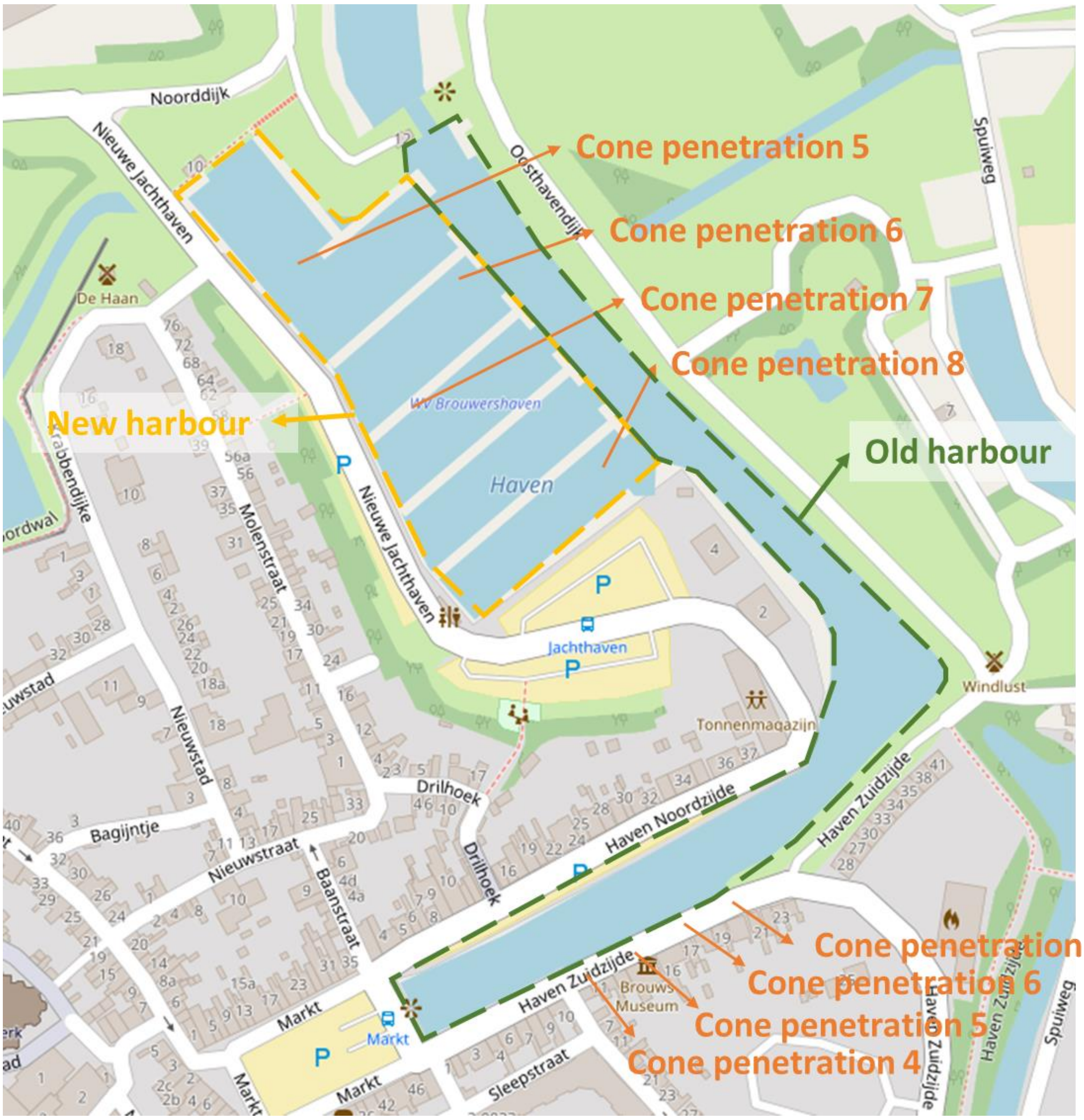


Appendix 5: Python programme to determine reduced tide with tidal power plant

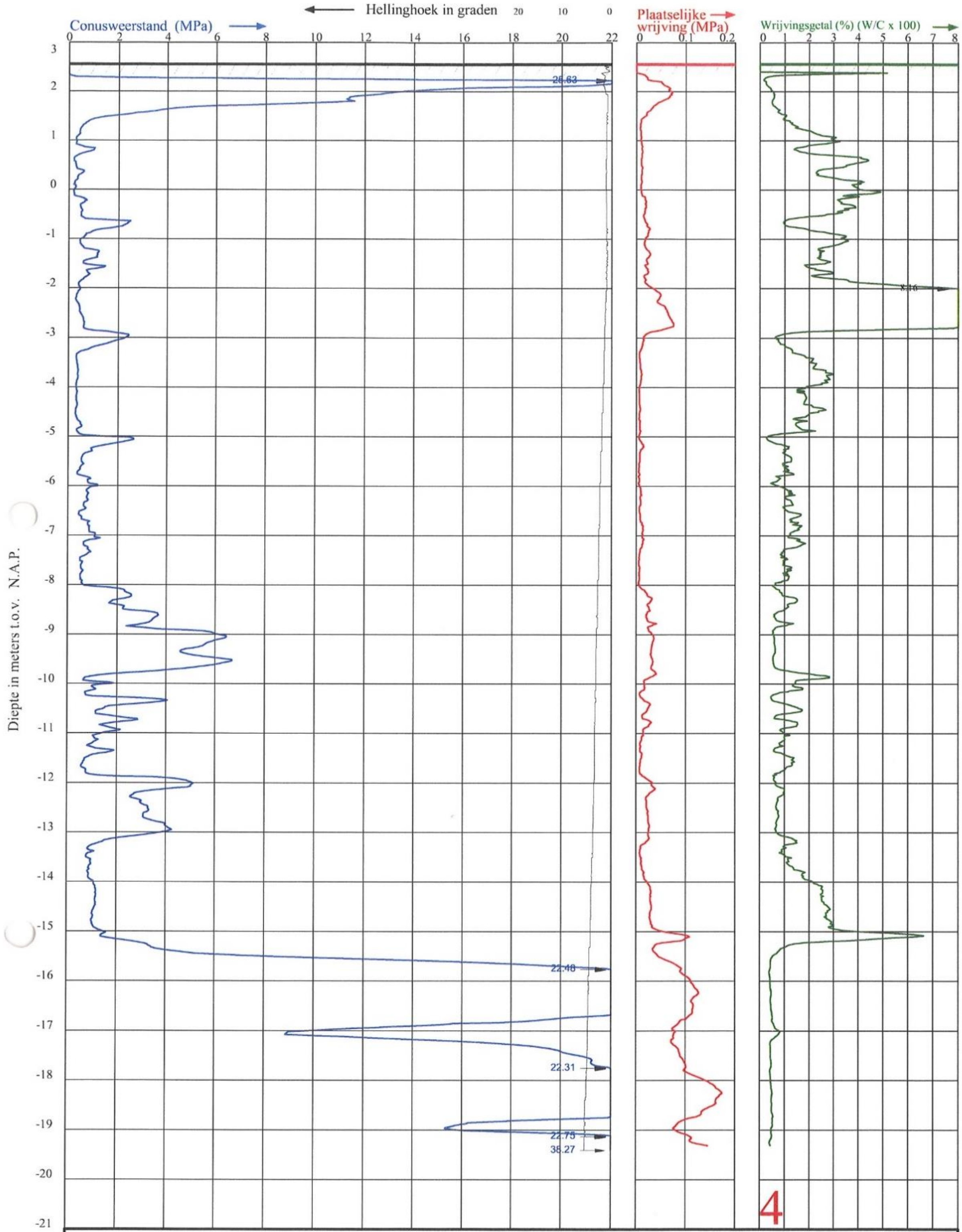
```python
from pylab import *
from numpy import *
import os;
import matplotlib.pyplot as plt
close ('all')
print ('Made by Fons De Vlieger')
print ('Aquavia')
print ('13/07/2017\n')

close ('all')
os.chdir('C:\Users\AThou\Desktop\Python')            #Verplaatsen werkdirectory

#Parameters
Zeestijging = 0          #in cm
uitzondering = 0          #in m
waterstand =Zeestijging + genfromtxt('BRBU2016waterstand3.txt',deletechars=None,usecols=[2])   #dit is het getij buiten Brouwersdam
where_are_NaNs = isnan(waterstand)
waterstand[where_are_NaNs] = 29.46

Tijd = linspace(0,366,size(waterstand))
plot(Tijd,waterstand)

nrhoog = 0
nrlaag = 0
h = 0
l = 0
t = 0

for j in range(2):
    hoogwater = zeros(nrhoog)
    hoogtijd = zeros(nrhoog)
    getij = zeros(nrhoog+nrlaag)
    tijdgetij = zeros(nrhoog+nrlaag)
    laagwater = zeros(nrlaag)
    laagtijd = zeros(nrlaag)

    for i in range(size(waterstand)):
        if (waterstand[i-4] < waterstand [i-2]) and (waterstand[i-3] < waterstand [i-2]) and (waterstand[i-1] <= waterstand [i-2]) and (waterstand[i] <= waterstand [i-2]) and waterstand[i-2] > 50:
            nrhoog = nrhoog + 1
            if j==1:
                hoogwater[h] = waterstand [i-2]
                getij[t] = waterstand [i-2]
                hoogtijd[h] = Tijd[i-2]
                tijdgetij[t] = Tijd[i-2]
                h = h+1
                t = t+1

        elif (waterstand[i-4] > waterstand [i-2]) and (waterstand[i-3] > waterstand [i-2]) and (waterstand[i-1] >= waterstand [i-2]) and (waterstand[i] >= waterstand [i-2]) and waterstand[i-2] < 23:
            nrlaag = nrlaag + 1
            if j==1:
                laagwater[l] = waterstand [i-2]
                getij[t] = waterstand [i-2]
                laagtijd[l] = Tijd[i-2]
                tijdgetij[t] = Tijd[i-2]
                l = l+1
                t =t+1
gemwater = (mean(hoogwater)+mean(laagwater))/2.0
print (gemwater)

plot(hoogtijd,hoogwater,'r')
plot(laagtijd,laagwater)
plot([0,366],[gemwater,gemwater],'y',linewidth=2)
legend(['Waterstand Noordzee gemeten om de 10 min.','Hoogwaters','Laagwaters','gemiddeld waterpeil Noordzee'], ncol=3, fancybox=True, shadow=True)
xlim(0,366)
xlabel('Dagen van het jaar 2016')
ylabel('Waterstand in cm-NAP')

aantal = 0
a = 0

for j in range(2):
    getijverschil = zeros(aantal)
    for i in range (size(getij)):
        if (getij[i-1] < 23 and getij[i] > 50) or (getij[i-1] > 50 and getij[i] < 23):
            aantal = aantal + 1
            if j==1:
                getijverschil[a] = abs(getij[i-1] - getij[i])
                a = a+1
print ('Gemiddelde waterstand:', mean(waterstand),(mean(hoogwater)+mean(laagwater))/2.0)
print ('Gemiddelde getij:', mean(getijverschil))

#Simulatie getij Noordzee met meer datapunten
#Parameters
figure()
Ampl = mean(getijverschil)/200.0     #amplitude getij
d = 8 + gemwater/100.0          #dit is de diepte van de drempel>
b = 108.0        #dit is het kombergend oppervlak van het haven Brouwershaven, in km2>
Bs = 8*20          #dit is de doorlatende breedte van de kering>
Qr = 0           #dit is de afvoer van het gemaal>
dt = 0.05            #dit is de tijdstap in minuten >
NofTides = 5  #dit is het aantal getijden dat je wilt doorrekenen>
Tt = 12.5        #dit is de getijperiode in uren>

#Omzetten gegevens in andere eenheid
g = 9.81
Tt = Tt *3600   #Getijperiode in seconden
b = b * 1000000 #Oppervlakte in m2
dt = dt * 60

#Reeksen met waterpeil
H1 = zeros(NofTides * Tt/dt)
h2 = zeros(NofTides * Tt/dt)
h3 = zeros(NofTides * Tt/dt)

#Resetten opties
umin = 0
umax = 0
omega = 2 * pi/Tt
nr = NofTides * Tt /dt
T = 0
check = 0
H1[0] = d +Ampl
h2[0] = H1[0]
h3[0] = d - mean(waterstand)/100.0 - 0.20

for i in range(size(H1)-1):
    H1[i] = Ampl * cos(omega * T) + d                                  #Waterdiepte tot drempel
    a = h2[i] * Bs                                                     #Oppervlakte doorstroomopening
    q = 0
    qp = 0
    if (H1[i]-h3[i]) > 1.5:
        check = 1

    if H1[i] > h3[i] and h3[i] < d-gemwater/100.0 + 0.05 + uitzondering and check == 1:
        if h3[i] > (2.0/3.0)*H1[i]:
            h2[i+1] = h3[i]
            if (H1[i] == h2[i+1]):
                q = 0
            else:
                q = a * sqrt(2.0 * g * abs(H1[i]-h2[i+1]))*(H1[i]-h2[i+1])/abs(H1[i]-h2[i+1])
        else:
            h2[i+1] = 2.0 * H1[i] / 3.0
            if H1[i] <= 0:
                q = 0
            else:
                q = a * sqrt(2/3.0*g*H1[i])
    elif H1[i] < h3[i] and h3[i] > d-gemwater/100.0 - 0.45 - uitzondering:
        check = 0
        if H1[i] > 2/3.0*h3[i]:
            h2[i+1] = H1[i]
            if h3[i] == h2[i+1]:
                q = 0
            else:
                q = -a * sqrt(2.0*g*(h3[i]-h2[i+1]))
```

```
        else:
            h2[i+1] = 2/3.0 * h3[i]
            if h3[i] <= 0:
                q = 0
            else:
                q = -a * sqrt(2/3.0*g*h3[i])
    else:
        h2[i+1] = H1[i]

    if a == 0:
        u = 0
    else:
        u = q/a
    if T < (NofTides - 1) * Tt:
        if u > umax:
            umax = u
            Hflood = h2[i+1]
        if u < umin:
            umin = u
            Hebb = h2[i+1]
    if i<nr-1:
        h3[i+1] = h3[i] + q/b * dt + Qr/b * dt
        T = T+dt

#H1[nr-1] = h2[nr-1] = h3[nr-1] = -0.2+d+0.23

plot (linspace(0,NofTides,nr),H1-8,'y',linewidth=3)
plot (linspace(0,NofTides,nr),h2-8,'k--',linewidth=1)
plot (linspace(0,NofTides,nr),h3-8,'r',linewidth=2)
plot ([0,NofTides],[0.05,0.05],'b--',linewidth=3)
plot ([0,NofTides],[mean(h3)-8,mean(h3)-8],'g--',linewidth=3)
plot ([0,NofTides],[- 0.45,- 0.45],'b--',linewidth=3)
legend(['Waterpeil Noordzee','Waterpeil in doorlaatconstructie','Waterpeil in Greveleningenmeer','toegelaten peil','Gemiddeld waterpeil haven'], ncol=3, fancybox=True, shadow=True)

xlabel('Aantal doorgerekende getijden')
ylabel('Waterstand in m-NAP')
ylim(-2,2)
title('Getij in het Grevelingenmeer na 100 jaar constructie getijdecentrale')
grid ('on')

print ('Gemiddeld waterniveau Grevelingenmeer:',mean(h3)-8)
# getij in Brouwershaven

figure()

db = 2.3          #dit is de diepte van de drempel>
b = 0.04        #dit is het kombergend oppervlak van het haven Brouwershaven, in km2>
Bs = 8.5          #dit is de doorlatende breedte van de kering>
Qr = 0           #dit is de afvoer van het gemaal>

#Omzetten gegevens in andere eenheid
b = b * 1000000 #Oppervlakte in m2

#Reeksen met waterpeil
h4 = zeros(NofTides * Tt/dt)
h5 = zeros(NofTides * Tt/dt)

#Resetten opties
umin = 0
umax = 0
omega = 2 * pi/Tt
nr = NofTides * Tt /dt
T = 0
check = 0
h3 = h3 - 8
h4[0] = h3[0] + db
h5[0] = h3[0] + db

for i in range(size(h3)-1):
    h3[i] = h3[i] + db                                                  #Waterdiepte tot drempel
    a = h4[i] * Bs                                                                  #Oppervlakte doorstroomopening
    q = 0
    qp = 0

    if h3[i] > h5[i]:
        if h5[i] > (2.0/3.0)*h3[i]:
            h4[i+1] = h5[i]
            if (h3[i] == h4[i+1]):
                q = 0
            else:
                q = a * sqrt(2.0 * g * abs(h3[i]-h4[i+1]))*(h3[i]-h4[i+1])/abs(h3[i]-h4[i+1])
        else:
            h4[i+1] = 2.0 * h3[i] / 3.0
            if h3[i] <= 0:
                q = 0
            else:
                q = a * sqrt(2/3.0*g*h3[i])
    elif h3[i] < h5[i]:
        if h3[i] > 2/3.0*h5[i]:
            h4[i+1] = h3[i]
            if h5[i] == h4[i+1]:
                q = 0
            else:
                q = -a * sqrt(2.0*g*(h5[i]-h4[i+1]))
        else:
            h4[i+1] = 2/3.0 * h5[i]
            if h5[i] <= 0:
                q = 0
            else:
                q = -a * sqrt(2/3.0*g*h5[i])
    else:
        h4[i+1] = h3[i]

    if a == 0:
        u = 0
    else:
        u = q/a
    if T > (NofTides - 1) * Tt:
        if u > umax:
            umax = u
            Hflood = h4[i+1]
        if u < umin:
            umin = u
            Hebb = h4[i+1]
    if i<nr-1:
        h5[i+1] = h5[i] + q/b * dt + Qr/b * dt
        T = T+dt
h3[i+1] = h3[i]
#H1[nr-1] = h2[nr-1] = h3[nr-1] = -0.2+d+0.23

plot (linspace(0,NofTides,nr),h3 - db,'g',linewidth=2)
plot (linspace(0,NofTides,nr),h4 - db,'b',linewidth=1)
plot (linspace(0,NofTides,nr),h5 - db,'r--',linewidth=2)
#plot ([0,NofTides],[d-mean(waterstand)/100.0 + 0.05,d-mean(waterstand)/100.0 + 0.05],'b--',linewidth=3)
legend(['waterpeil Grevelingenmeer','Waterpeil in keersluis','Waterpeil in haven Brouwershaven','gemiddeld waterpeil Grevelingen'], ncol=3, fancybox=True, shadow=True)
xlabel('Aantal doorgerekende getijden')
ylabel('Waterstand in m-NAP')
ylim(-0.5,0.5)
title('Getij in haven Brouwershaven na constructie getijdecentrale')
grid ('on')
show()
```

Appendix 6: Cone penetration test “ancient and new harbour”

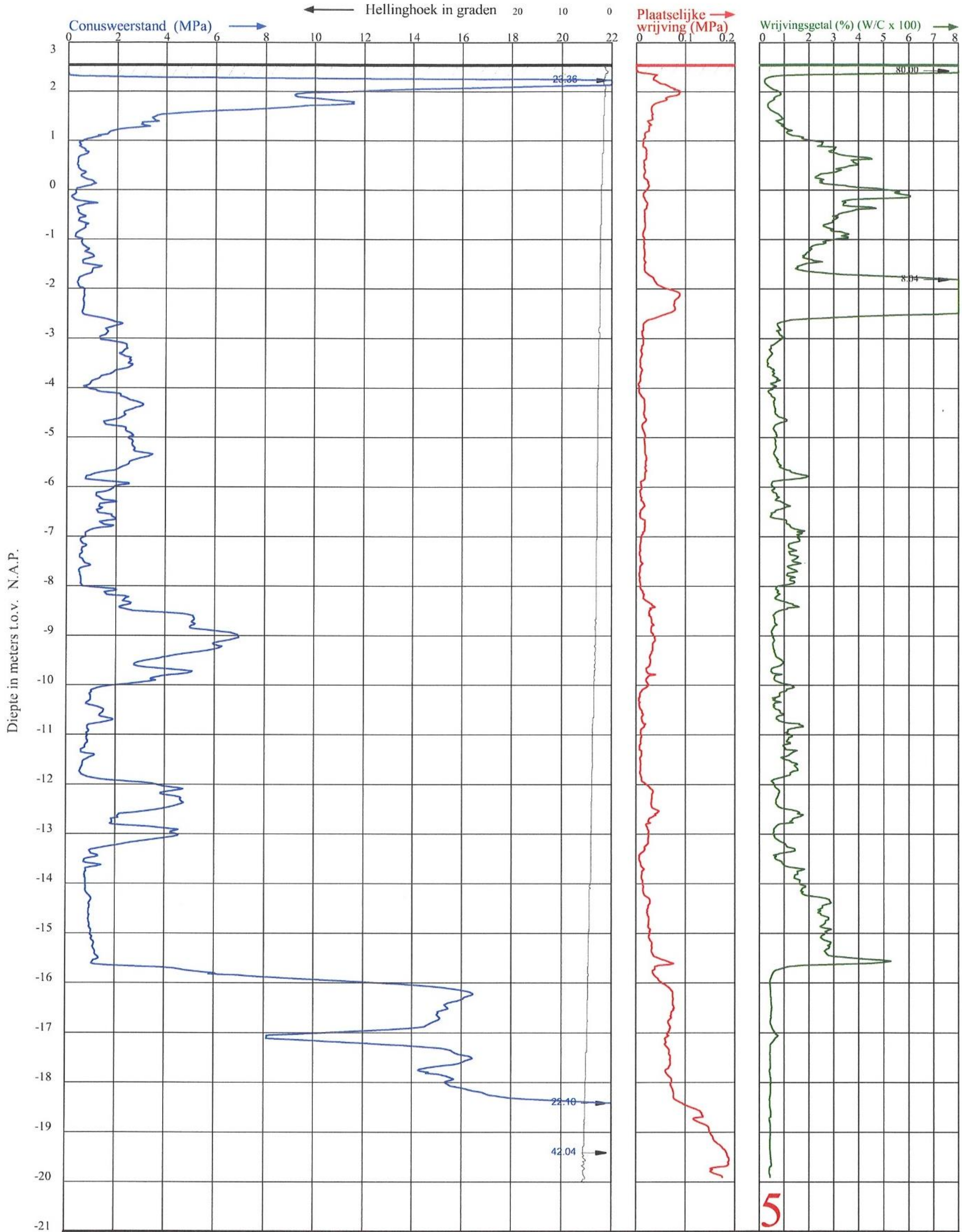


Cone penetration tests ancient harbour [4]



VAN DER STRAATEN AANNEMINGSMAATSCHAPPIJ B.V. Postbus 5 4417 ZG Hansweert
Afdeling Geotechniek Telefoon (0031) 113-382510 E-mail : info@vd-straaten.nl Internet : www.vd-straaten.nl

PLAATS :	BROUWERSHAVEN	HOOGTE MAAIVELD :	**2.58** m1 t.o.v. **N.A.P.**	CONUS TYPE :	SUB-15
LOCATIE :	HAVEN BROUWERSDAM	GRONDWATERSTAND:	m1- MAAIVELD	CONUS NR. :	150901
OPDRACHTGEVER :	GEM SCHOUWEN DUIVELAND	DATUM :	7-6-2016	SONDERING VOLGENS :	- NEN-EN-ISO 22476-1
PROJECTNUMMER:	**160259**	TIJD :	16:49		- TOEPASSINGSKLASSE 3
ID SONDERING :	**4**	X-COÖRDINAAT (RD):	**53252.413**	Y-COÖRDINAAT (RD) :	**416280.388**



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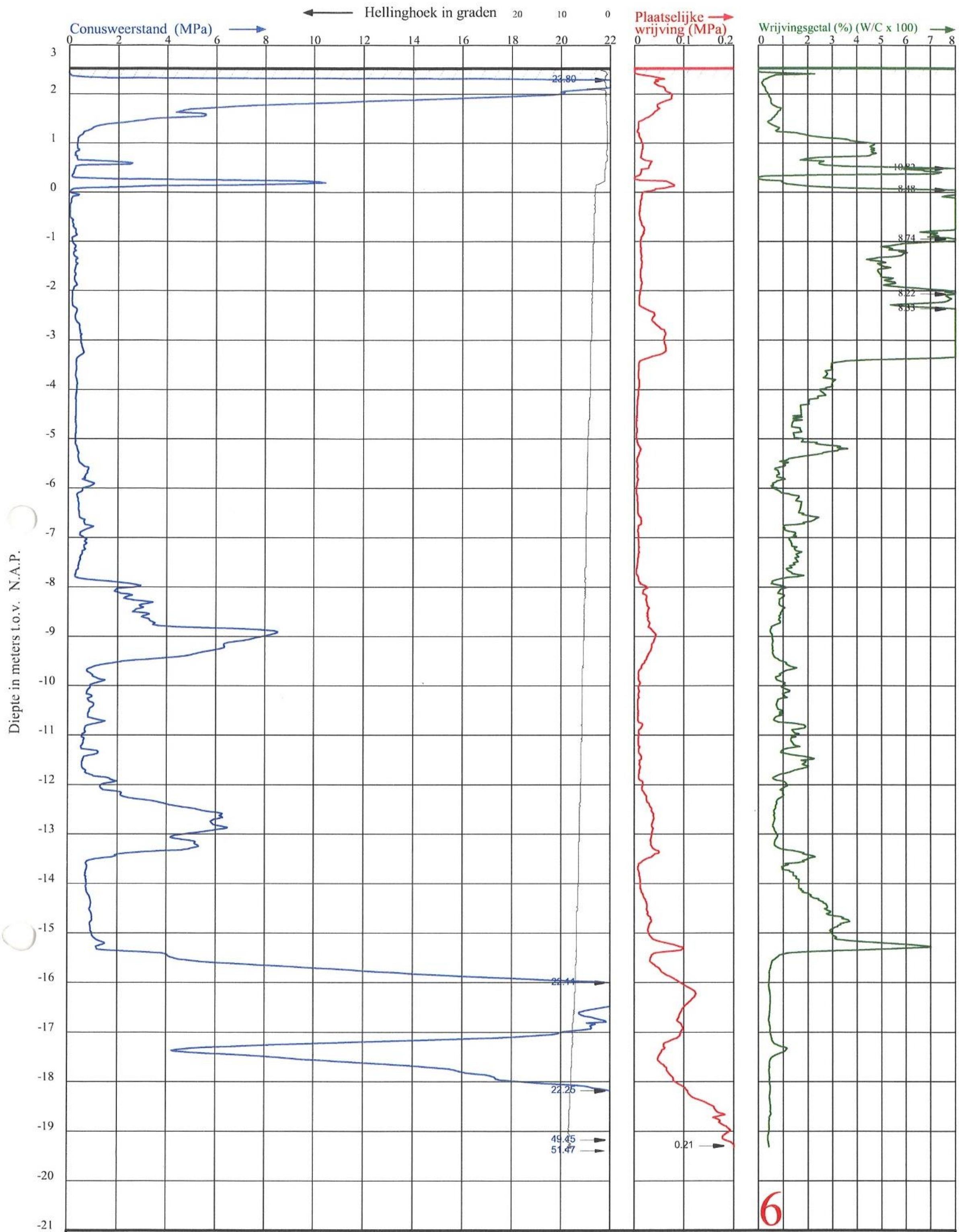
PLAATS :	BROUWERSHAVEN	HOOGTE MAAIVELD :	**2.57** ml t.o.v. **N.A.P.**	CONUS TYPE :	SUB-15
LOCATIE :	HAVEN BROUWERSDAM	GRONDWATERSTAND:	ml - MAAIVELD	CONUS NR. :	150901
OPDRACHTGEVER :	GEM SCHOUWEN DUIVELAND	DATUM :	7-6-2016	SONDERING VOLGENS :	- NEN-EN-ISO 22476-1 - TOEPASSINGSKLASSE 3
PROJECTNUMMER :	**160259**	TIJD :	15:48		
ID SONDERING :	**5**	X-COÖRDINAAT (RD):	**53274.278**	Y-COÖRDINAAT (RD) :	**416290.664**



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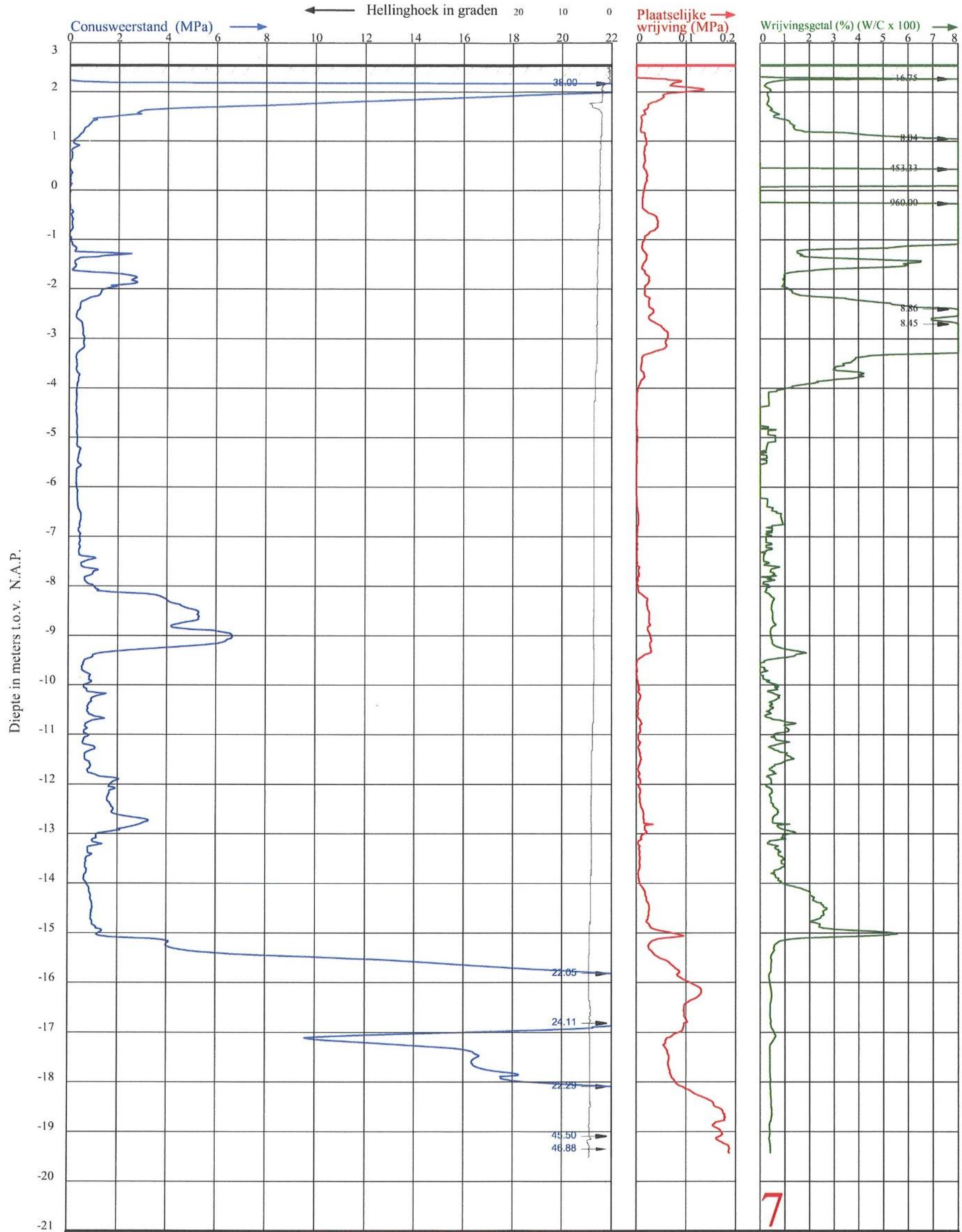
Afdeling Geotechniek Telefoon (0031) 113-382510 E-mail : info@vd-straaten.nl Internet : www.vd-straaten.nl

PLAATS :	BROUWERSHAVEN	HOOGTE MAAIVELD :	2.55 ml t.o.v. N.A.P.	CONUS TYPE :	SUB-15
LOCATIE :	HAVEN BROUWERSDAM	GRONDWATERSTAND	ml - MAAIVELD	CONUS NR. :	150901
OPDRACHTGEVER :	GEM SCHOUWEN DUIVELAND	DATUM :	7-6-2016	SONDERING VOLGENS :	- NEN-EN-ISO 22476-1
PROJECTNUMMER :	160259	TIJD :	14:54		- TOEPASSINGSKLASSE 3
ID SONDERING :	6	X-COÖRDINAAT (RD):	53296.349	Y-COÖRDINAAT (RD) :	416301.021



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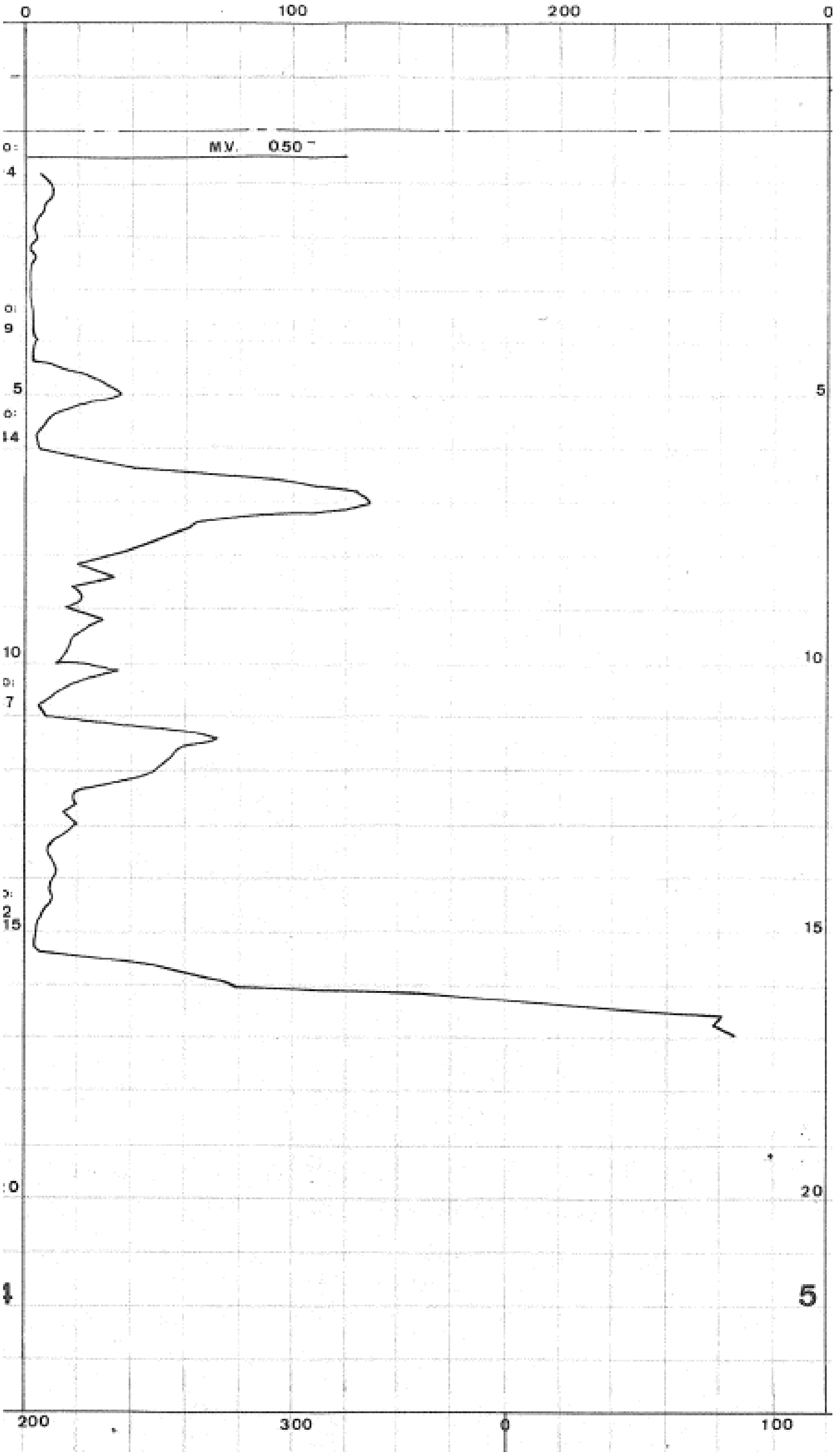
Afdeling Geotechniek Telefoon (0031) 113-382510 E-mail : info@vd-straaten.nl Internet : www.vd-straaten.nl

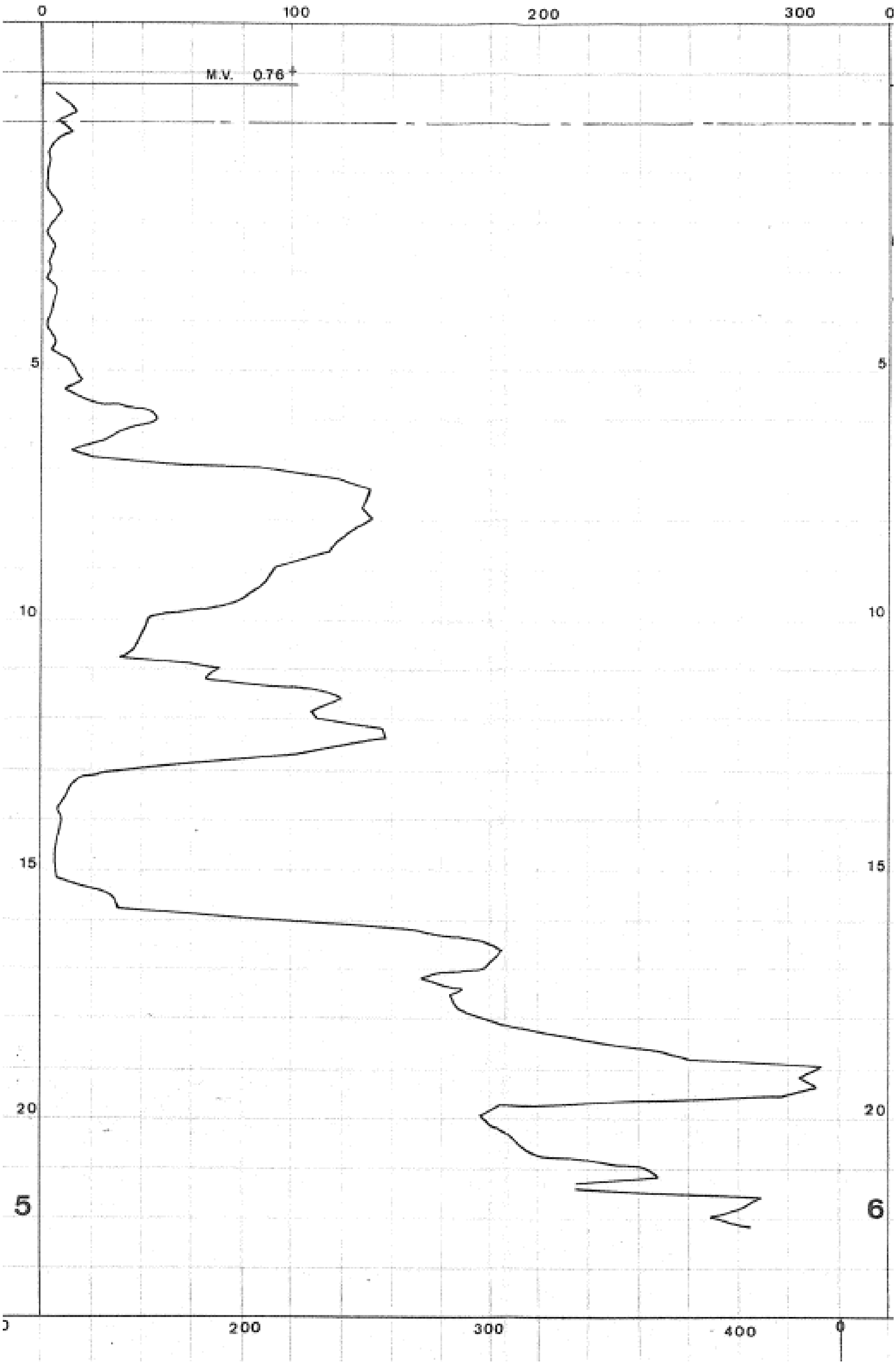
PLAATS	: BROUWERSHAVEN	HOOGTE MAAIVELD :	2.57 ml t.o.v. N.A.P.	CONUS TYPE :	SUB-15
LOCATIE	: HAVEN BROUWERSDAM	GRONDWATERSTAND	ml - MAAIVELD	CONUS NR. :	150901
OPDRACHTGEVER :	GEM SCHOUWEN DUIVELAND	DATUM :	7-6-2016	SONDERING VOLGENS :	- NEN-EN-ISO 22476-1
PROJECTNUMMER :	**160259**	TIJD :	14:03		- TOEPASSINGSKLASSE 3
ID SONDERING :	**7**	X-COÖRDINAAT (RD):	**53320.430**	Y-COÖRDINAAT (RD) :	**416310.462**

Cone penetration tests new harbour [5]

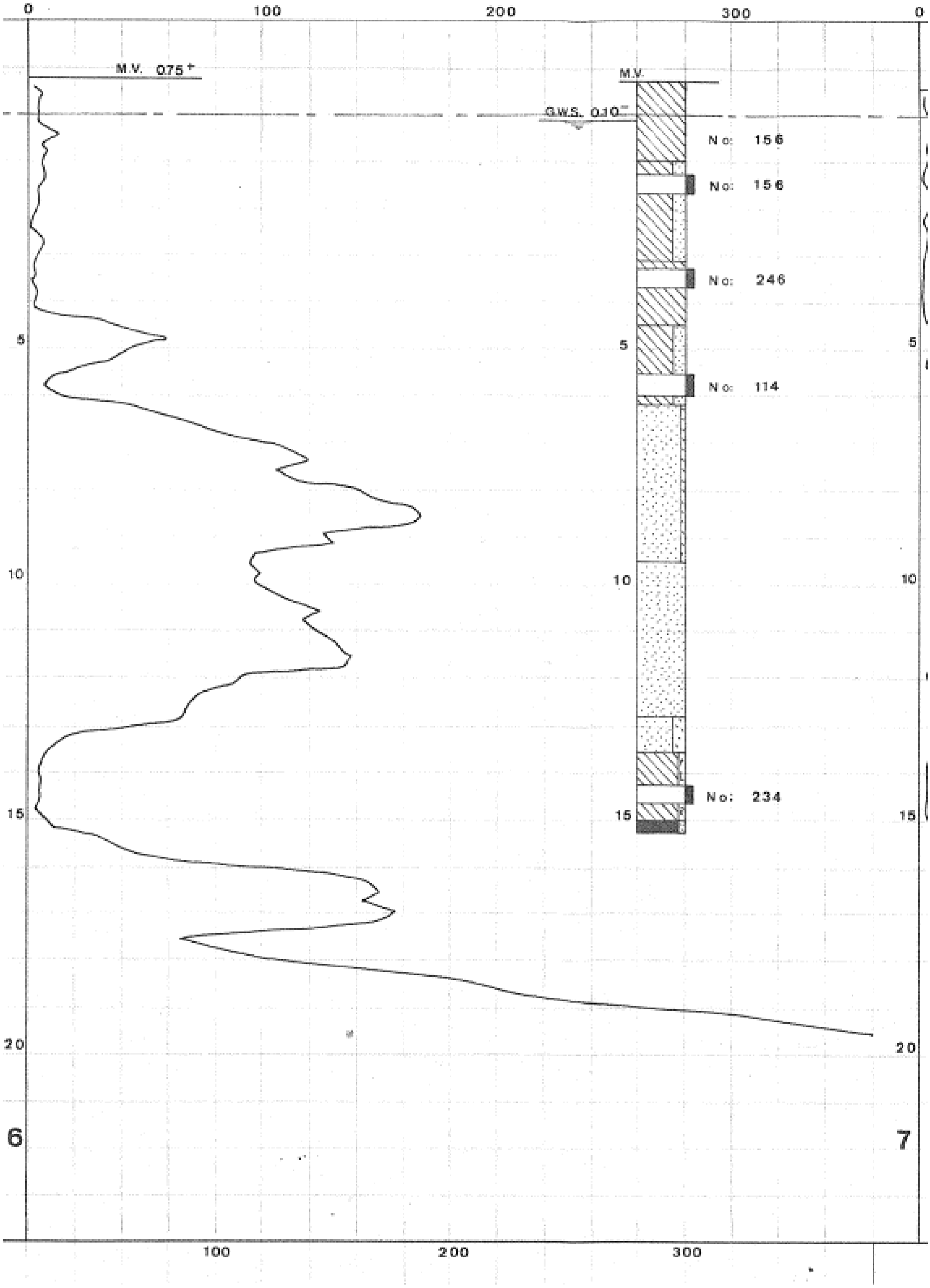
Cone penetration test 5



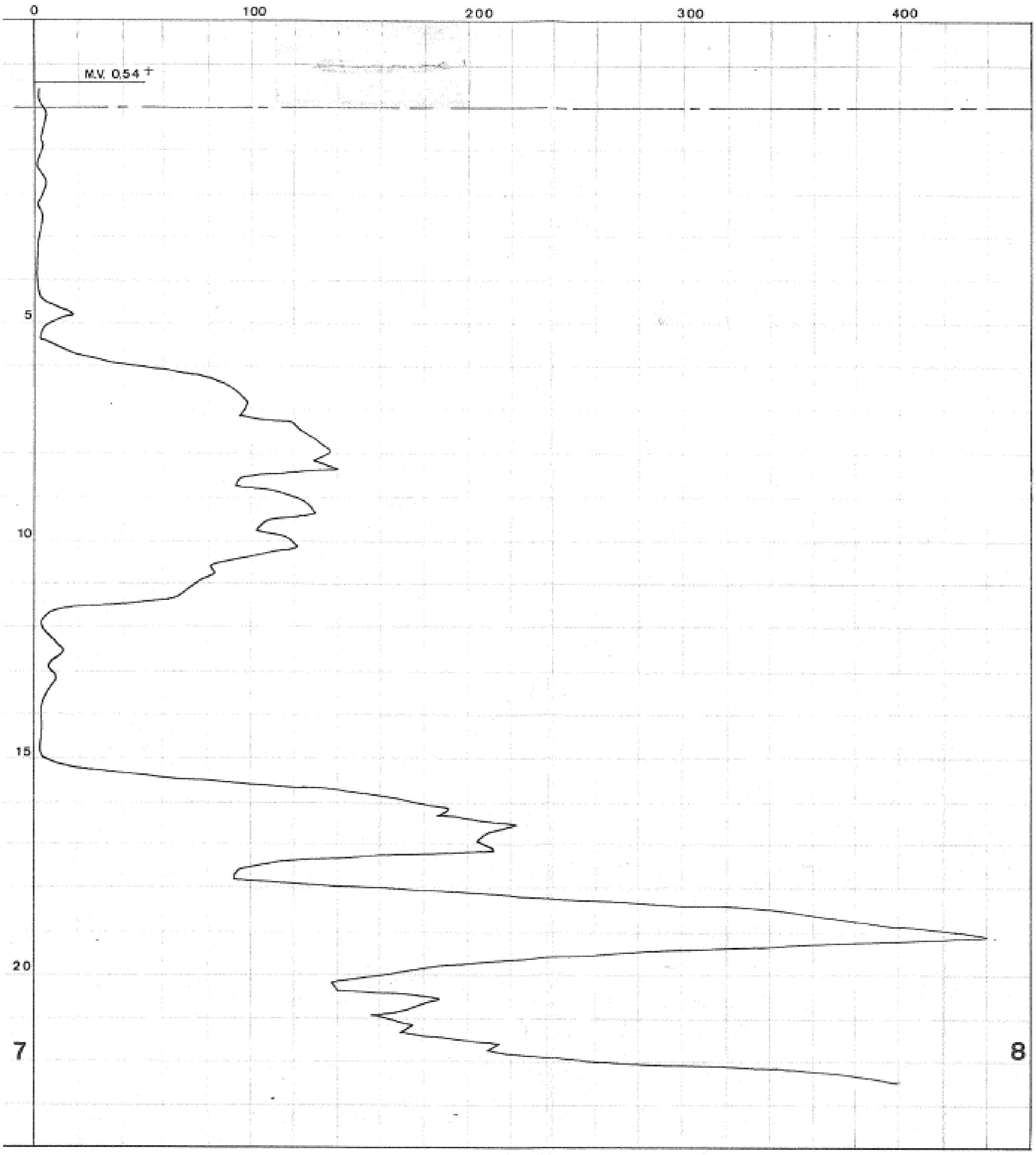
Cone penetration test 6



Cone penetration test 7



Cone penetration test 8



Appendix 7: Depth of the bottom of the harbour

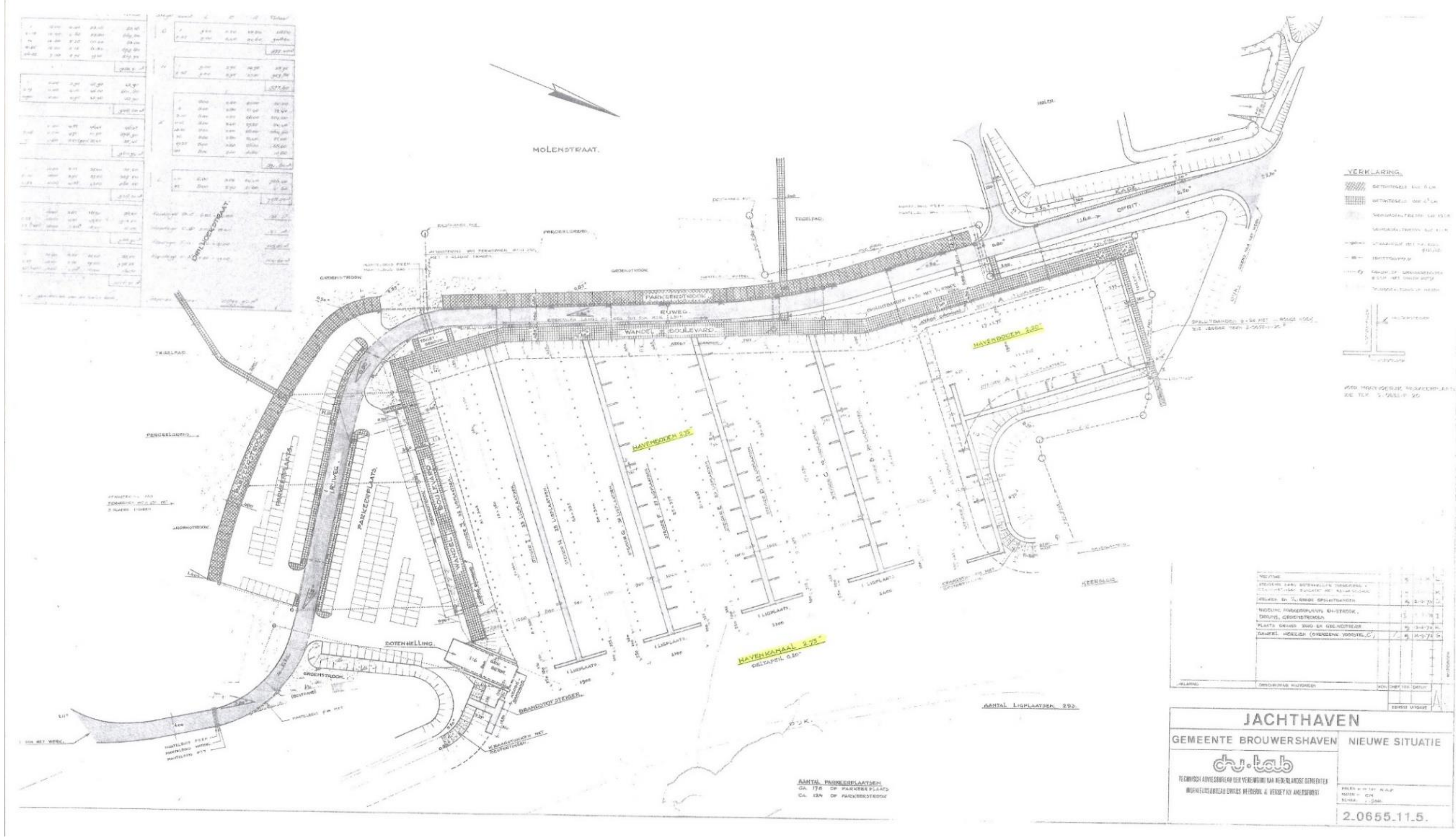


Figure 43: Technical drawing of the new harbour

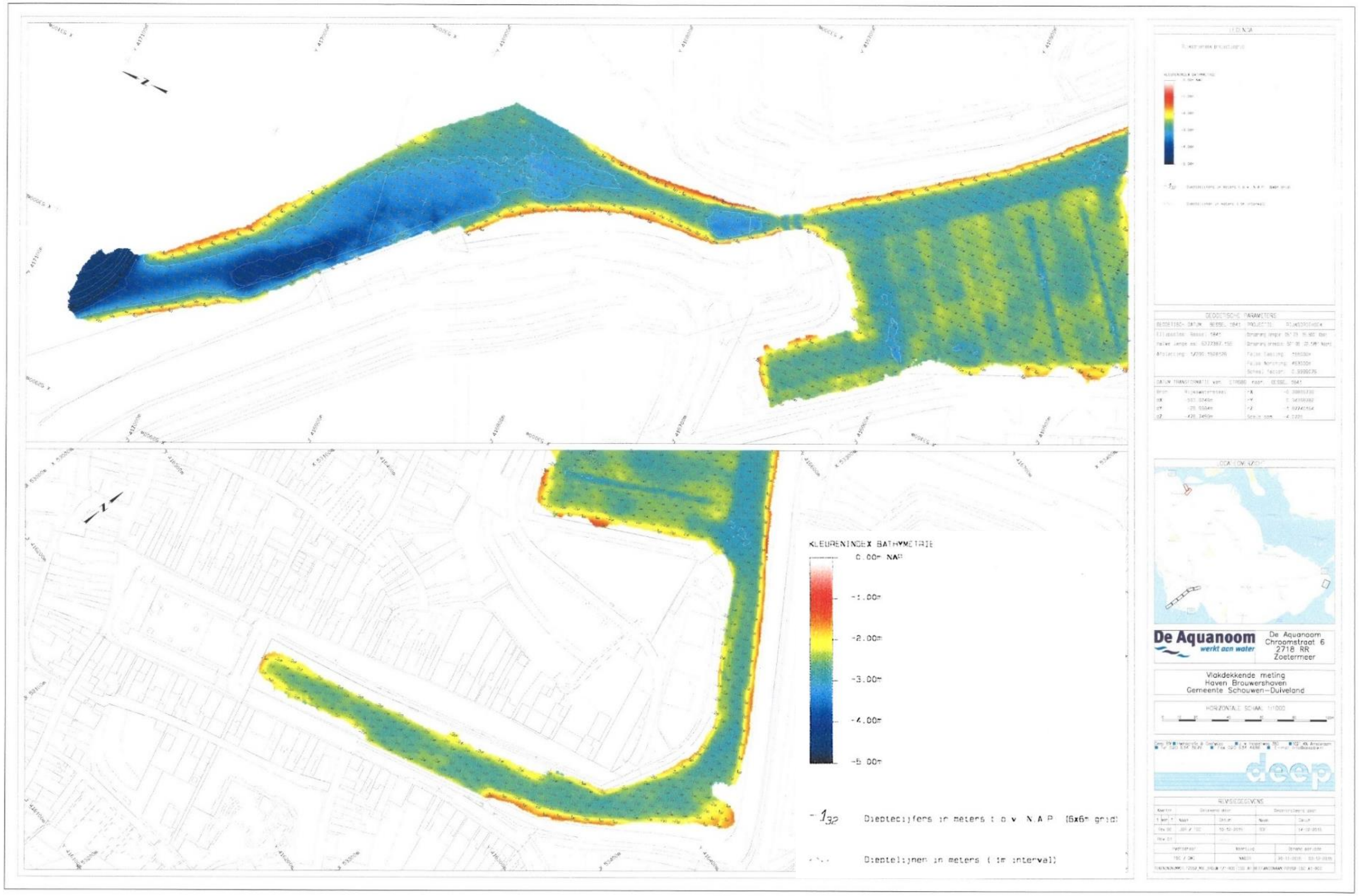


Figure 44: Depth of the harbour

Appendix 8: Effect of high and low tide on the harbour and his constructions

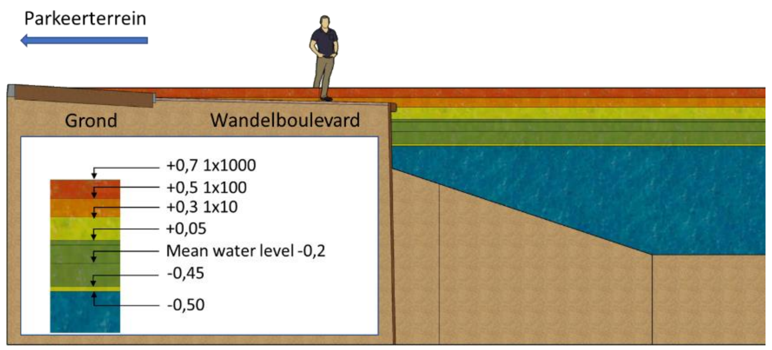
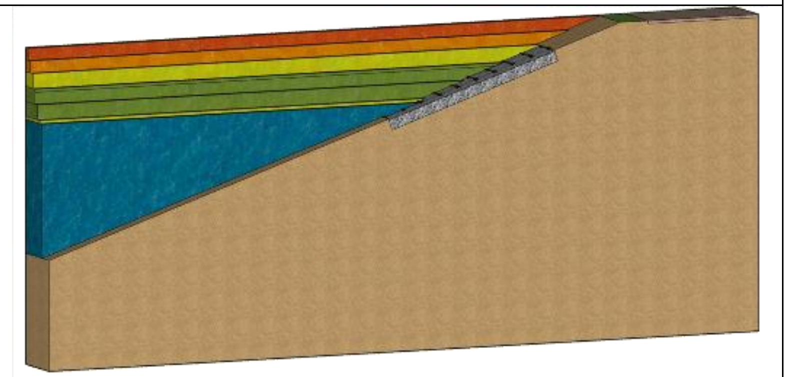
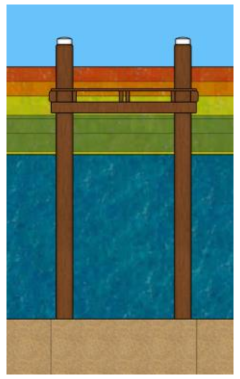
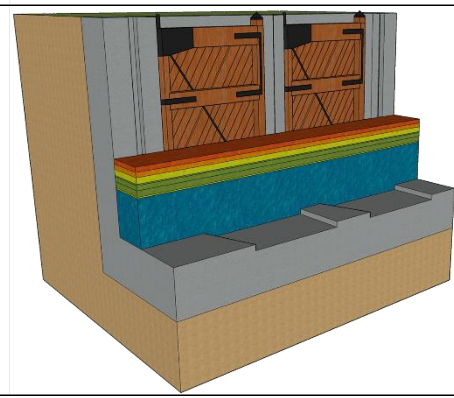
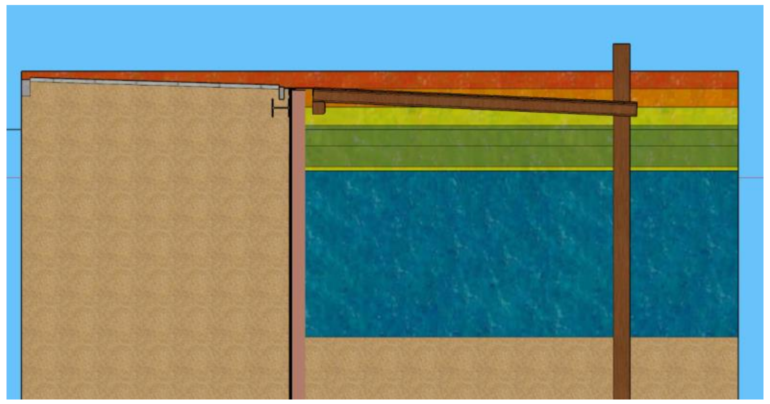
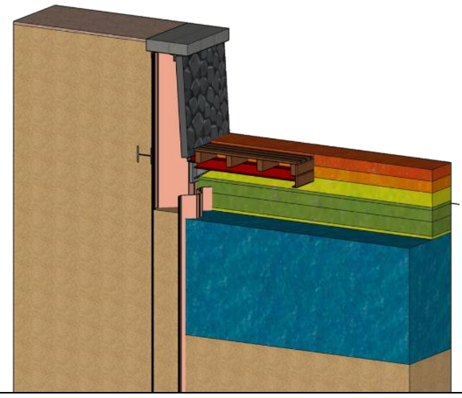
Construction	Visualisation	Construction	Water level issue
Wooden pile wall (new harbour)		Acclivity (new harbour)	
Scaffoldings (new harbour)		Guard lock (old harbour)	
Steel pile wall with side scaffolding (new harbour)		New quay wall at the southern side of the harbour (old harbour) 	

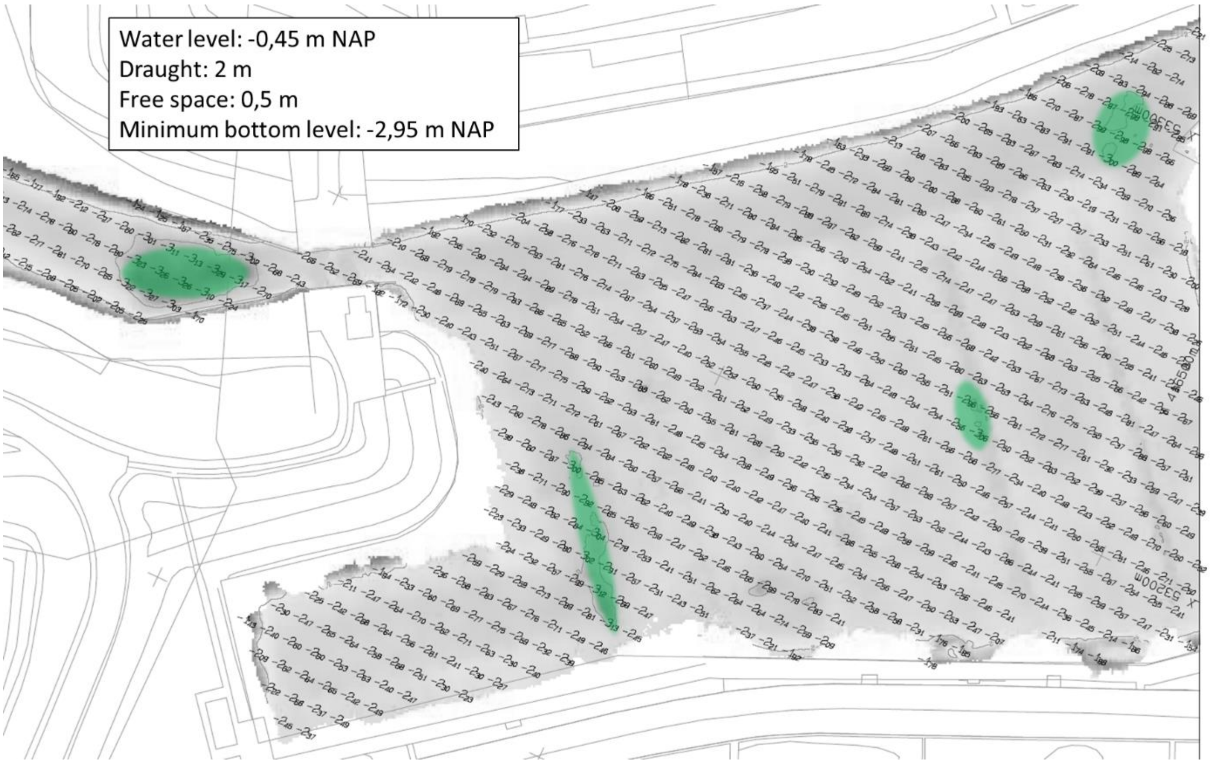
Figure 45: repaired sidewalk next to old quay wall

Through virtually bringing the various constructions in relation to the future water levels it was easy to see that the constructions are not influenced by the ½ m tide (green area). However, the water levels occurring once every 100 years (orange area) and once every 1000 years (red area) seemed to present some issues. These issues create the biggest problems for the new harbour. In case of the most extreme water levels in this harbour the scaffoldings as well as the quays get under water for 30 cm maximally. This can be seen in the figures in the above presented table. The old harbour is practically not incommoded by the extreme water levels as this harbour is familiar with a larger tide dating back from the days when the Grevelingen lake was not yet closed. The scaffoldings in this harbour are built higher as well, through which they get under the water level for 5 cm maximally only. The only problem in this harbour is the condition the old quay walls are in. We can already see now that during heavy rainfall sand and soil washes out from behind the quay wall. This creates holes in the walking path next to the quay wall. Which can lead to dangerous situations for the walking path users. With the coming tide, which will also influence the ground-water level behind the pile wall, this process can be reinforced, thus also creating a potential danger for the quay wall's stability.

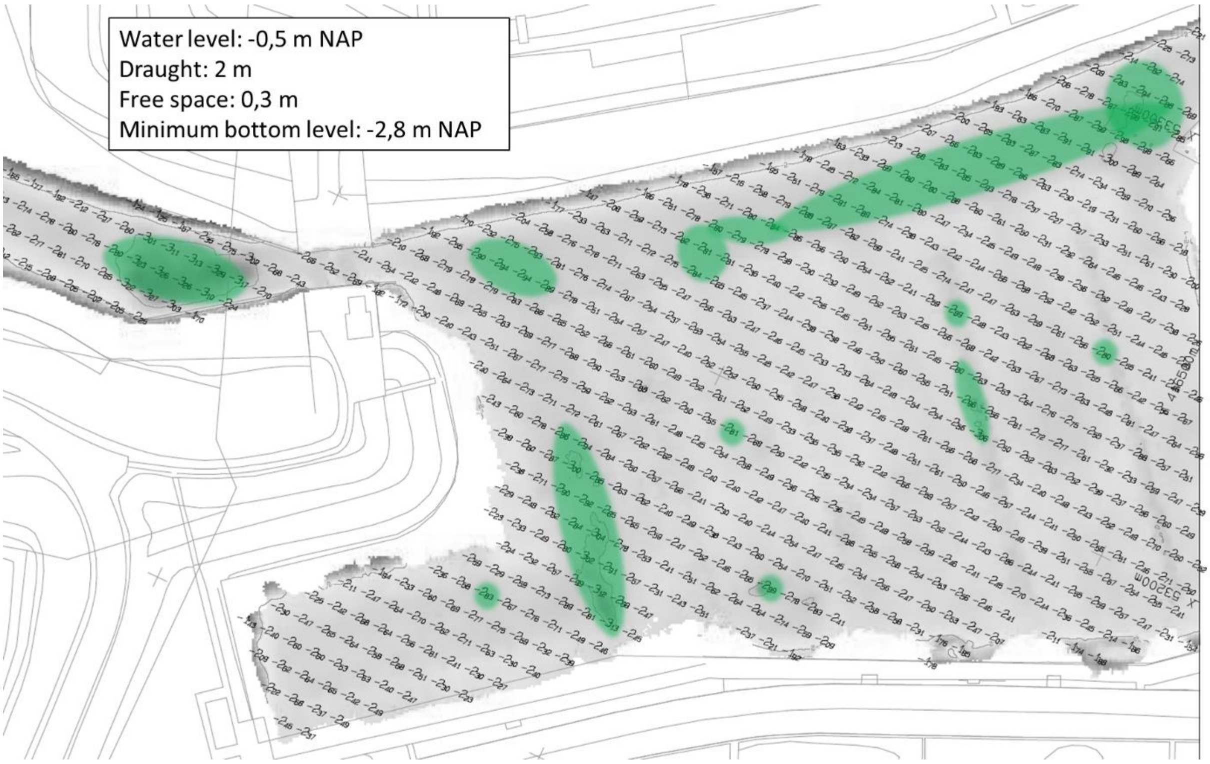
Low tide as well can have consequences for the accessibility of the harbour. The limiting factor for the harbour is determined by the guard lock, the sill being situated at a depth of -2,5 m NAP. In the current situation there is an available water depth of 2,3 m. The harbour is now used by ships having a maximal draught of 2 m, leaving 30 cm of free room under the ship when passing the lock. The following empirical rule shows us that this is low already when compared to the free room used as a basis for designing the access areas and the lock itself.

$$D_{water} = Draught + 0{,}5\ m = 2m + 0{,}5m = 2{,}5\ m$$

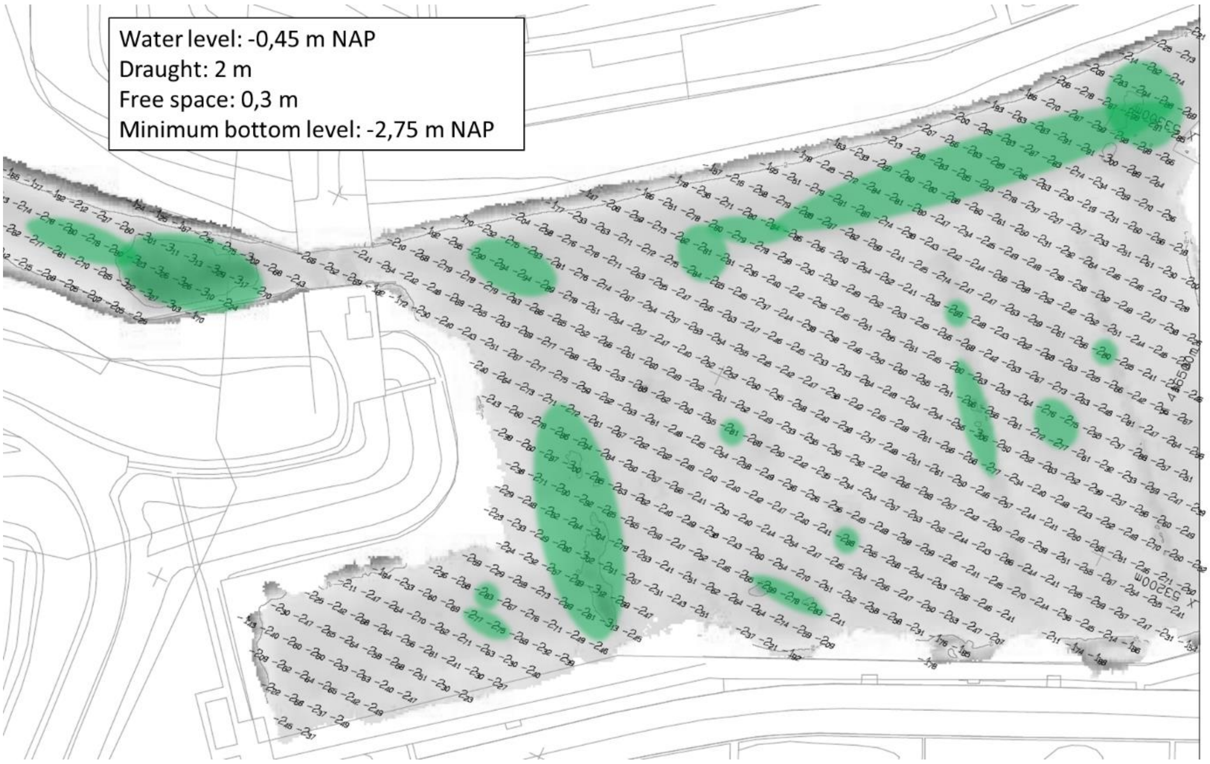
In the following figures we can see to what extent the harbour remains navigable during (extreme) low tide taking into account the necessary water depth according to the abovementioned formula and the depth provided in the guard lock at present.



In the image above it can be clearly seen that, implementing the empirical rule explained earlier, there are almost no areas in the new harbour having sufficient water depth during low tide.



At the water level of -0,5m NAP, which is allowed to occur 10% of the time, there even is no free room available between the boat's bottom and the guard lock's floor. This also applies for the harbour itself as the practical depth is equal to the guard lock's floor. If we now look at the harbour's bottom in its current condition we primarily see that the harbour would still be navigable in between many parts of the ancient harbour canal and the scaffoldings situated most to the north. In some places here the bottom has attained a depth of -3 m NAP already. This can be due to bottom erosion caused by the ships' propellers.



The final situation we take into account is the one when low tide reaches -0,45 m NAP. Apparently especially the former zones which sufficed already extend a bit.

As the fairway towards the harbour has an design depth of -3,5 m NAP no impediment will occur for the harbour's accessibility.

Appendix 9: Stability check of quay walls

In order to determine the wooden and steel pile walls' stability we used the D-Sheet Piling programme. The following models were established using this programme.

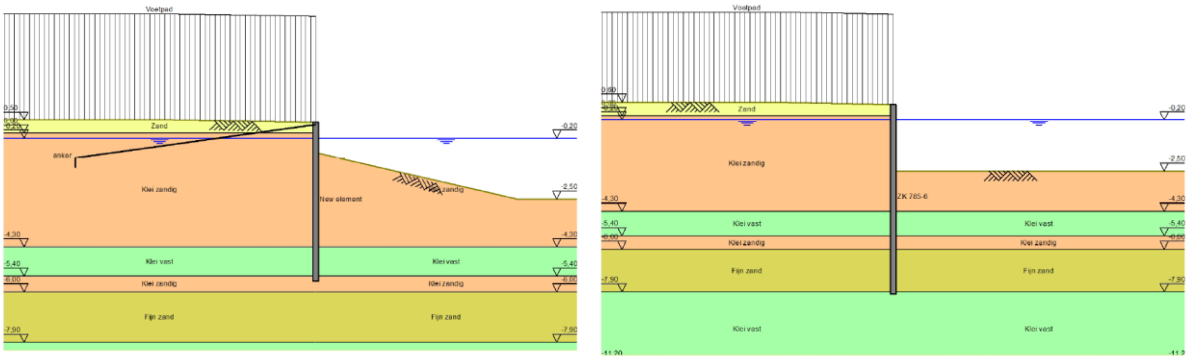


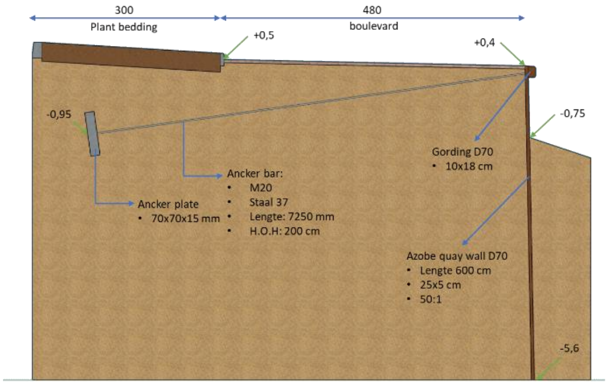
Figure 46: D-sheet model of quay walls in the new harbour

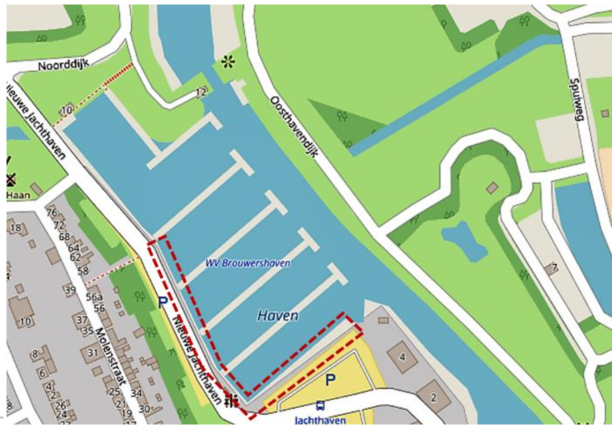
The dimensions and the technical data to establish these models are based on the engineering drawings found in the Schouwen-Duiveland archives. For the soil structure we used the new harbour's soil composition as described in chapter 3.4. As soil specifications lack, we chose to use the cone penetration tests' specifications of the new harbour. Which delivers the following soil composition and specifications :

Layer name	Level [m]	Unit weight Unsat [kN/m³]	Unit weight Sat. [kN/m³]	Cohesion [kN/m²]	Friction angle phi [degree]	Delta friction angle [degree]
Sand	0,50	19,00	21,00	0,00	35,00	16,60
Sandy clay	0,00	18,00	18,00	5,00	22,50	15,00
Solid clay	-4,30	19,00	19,00	13,00	17,50	11,70
Sandy clay	-5,40	18,00	18,00	5,00	22,50	15,00
Fine sand	-6,00	18,00	20,00	0,00	30,00	20,00
Solid clay	-7,90	19,00	19,00	13,00	17,50	11,70
Fine sand	-11,20	18,00	20,00	0,00	30,00	20,00
Solid clay	-12,20	19,00	19,00	13,00	17,50	11,70
Fine sand	-15,50	18,00	20,00	0,00	30,00	20,00

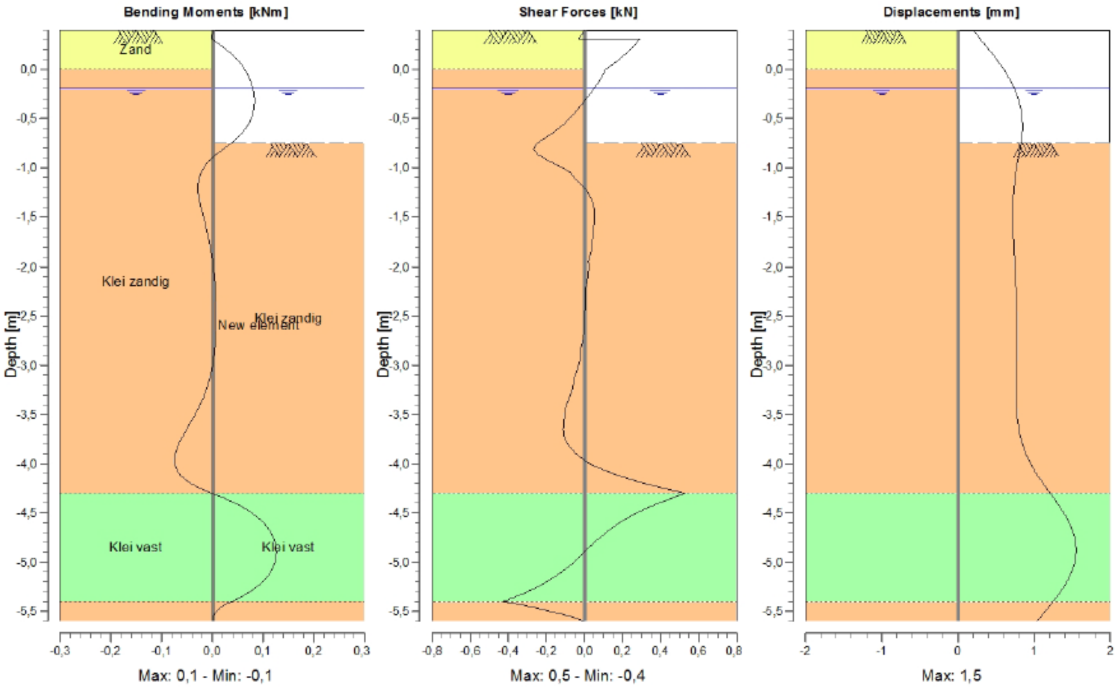
As along the quay wall a promenade boulevard is situated as well, an evenly discharged load of 5 kN/m2 was attributed to this segment just as was the case in the project of the new quay wall at the southern side of the harbour.

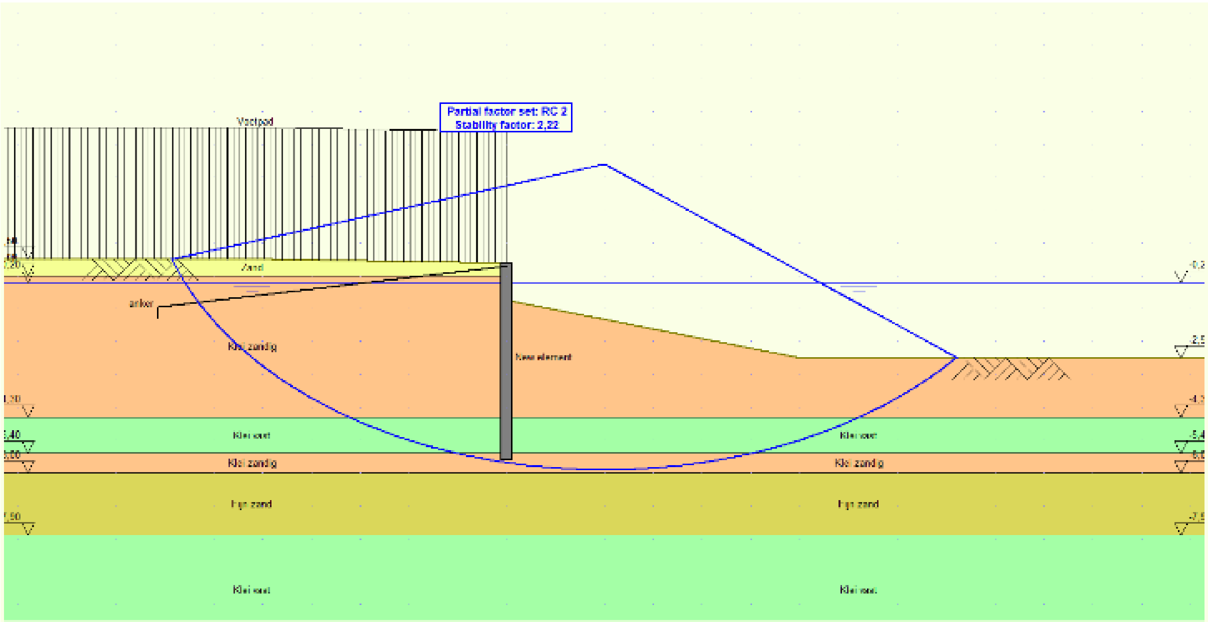
Wooden pile wall



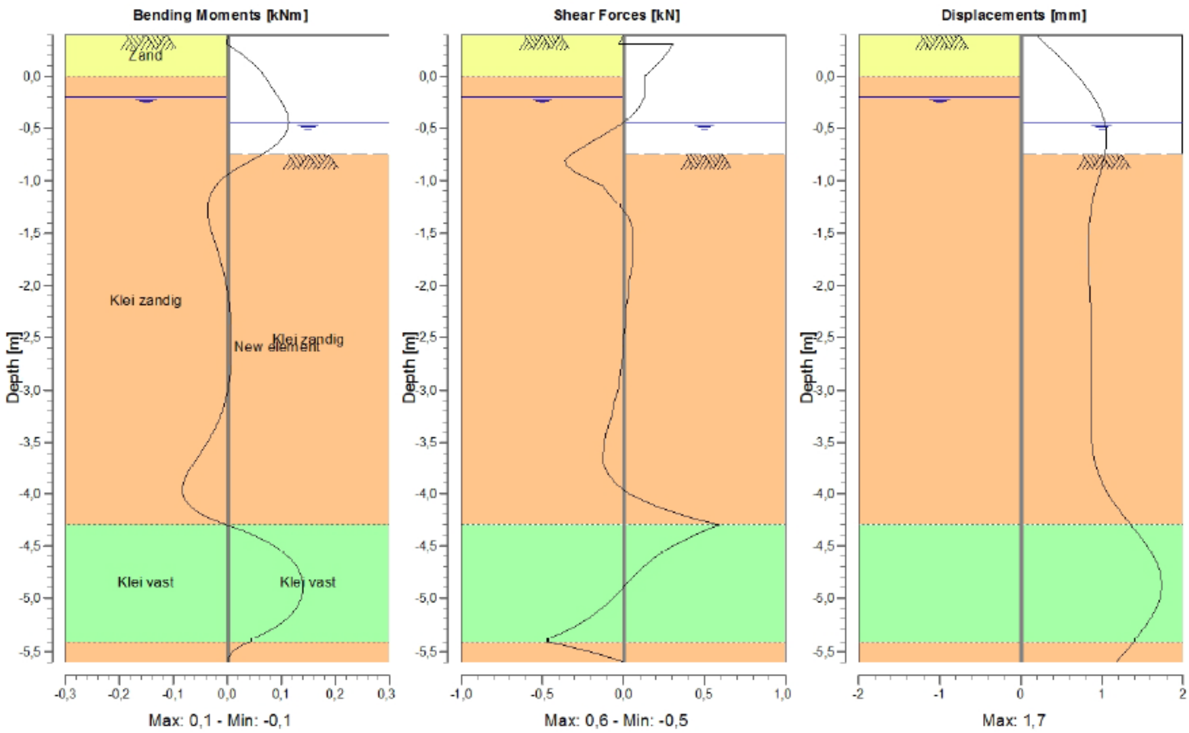


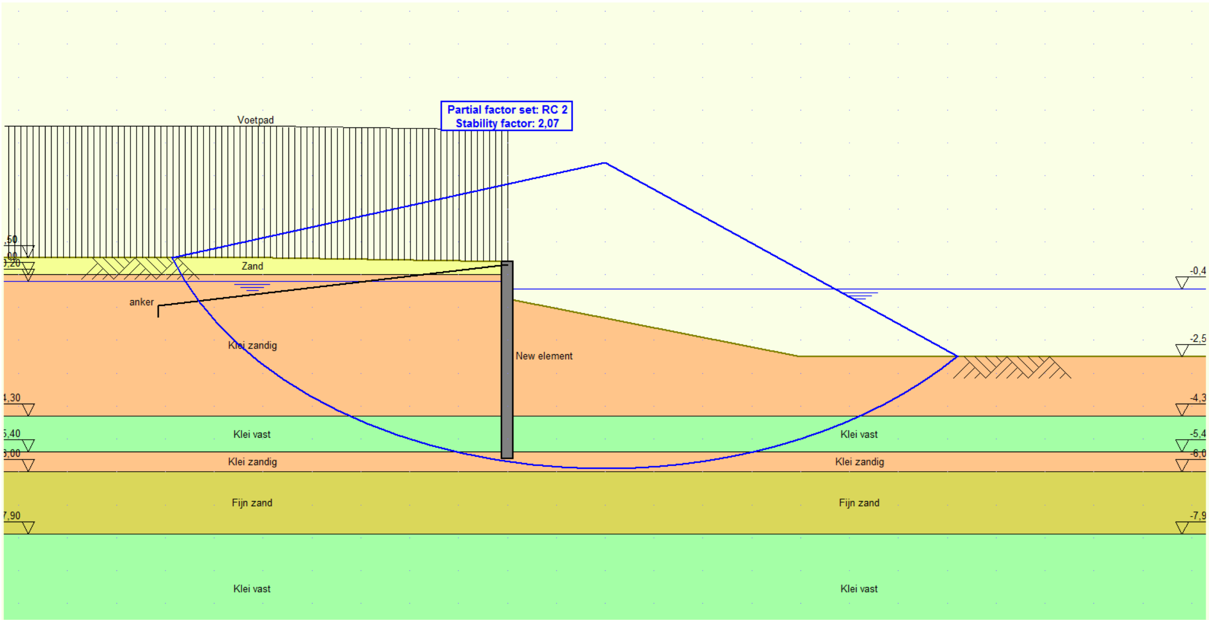
Current situation



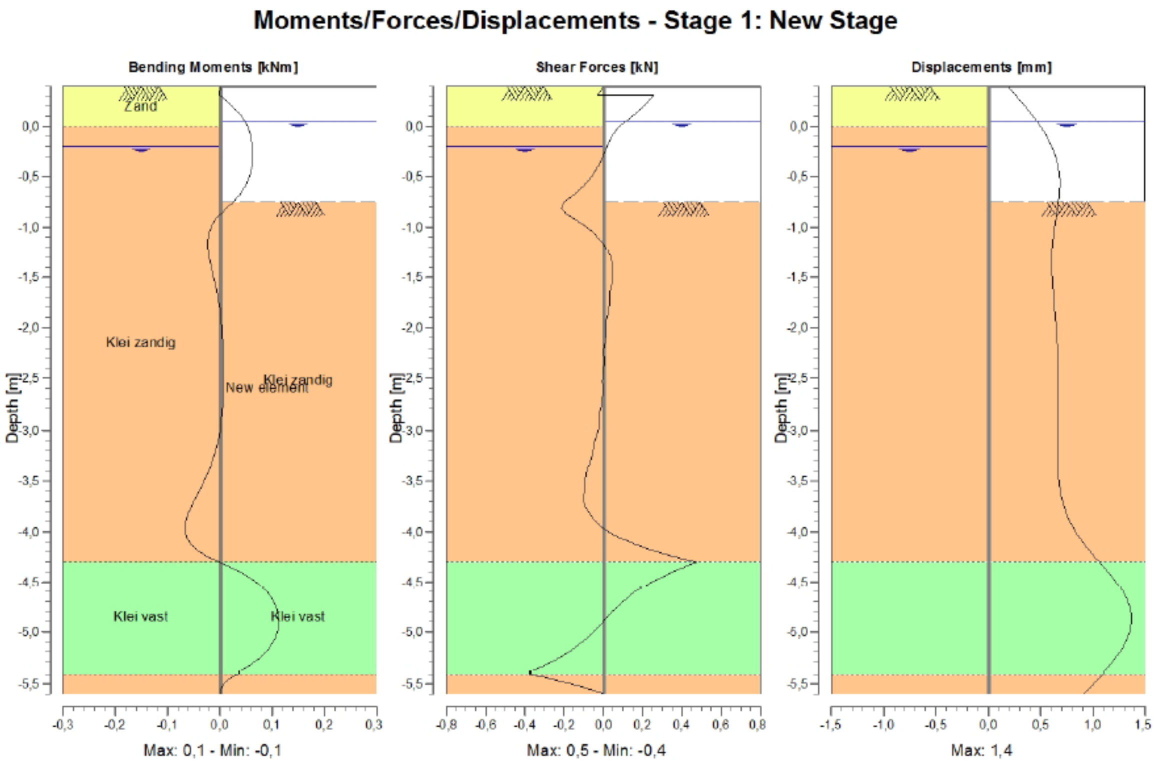


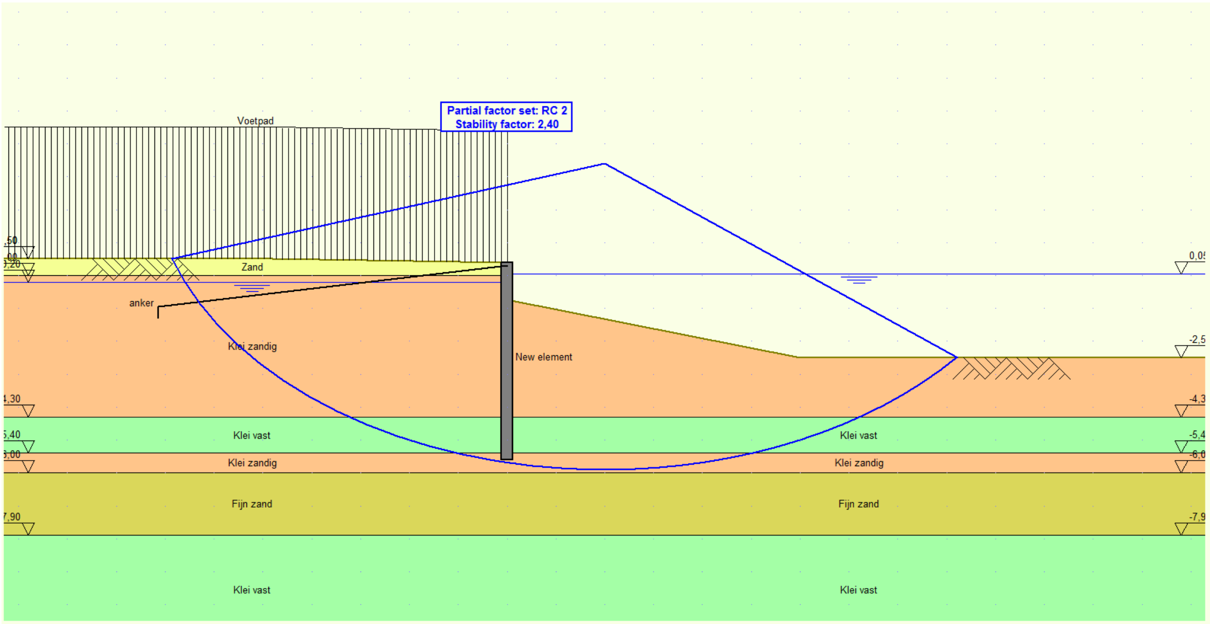
Water level of -0,45 m NAP during low tide



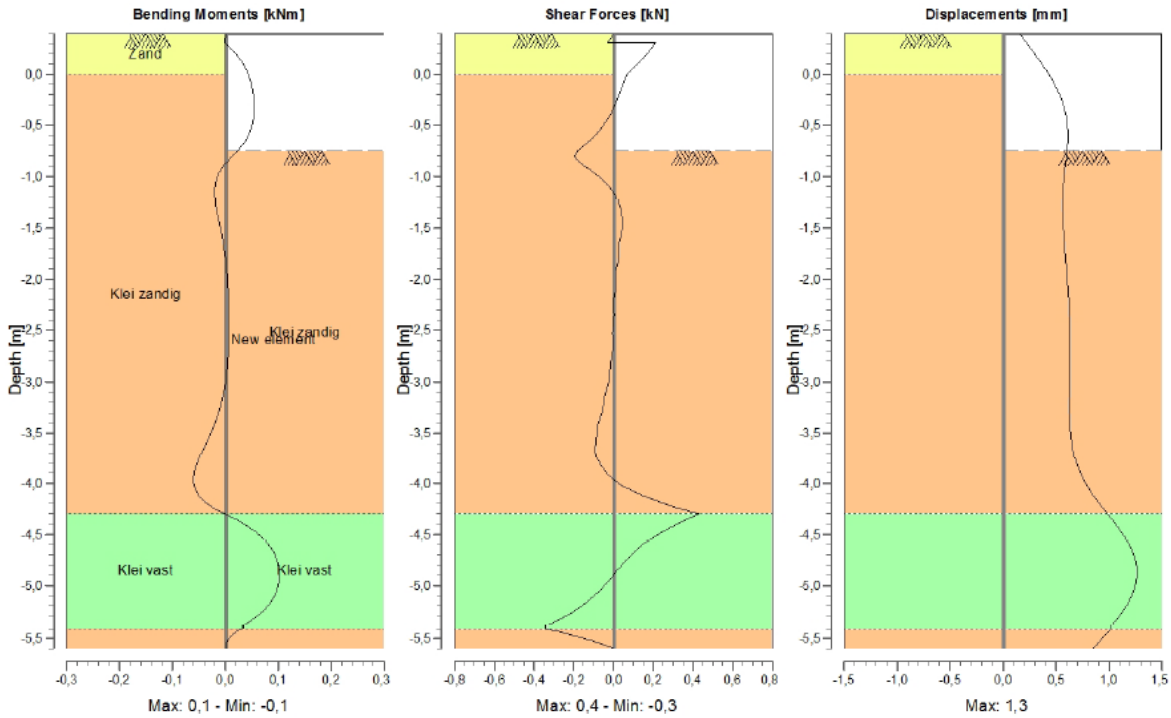


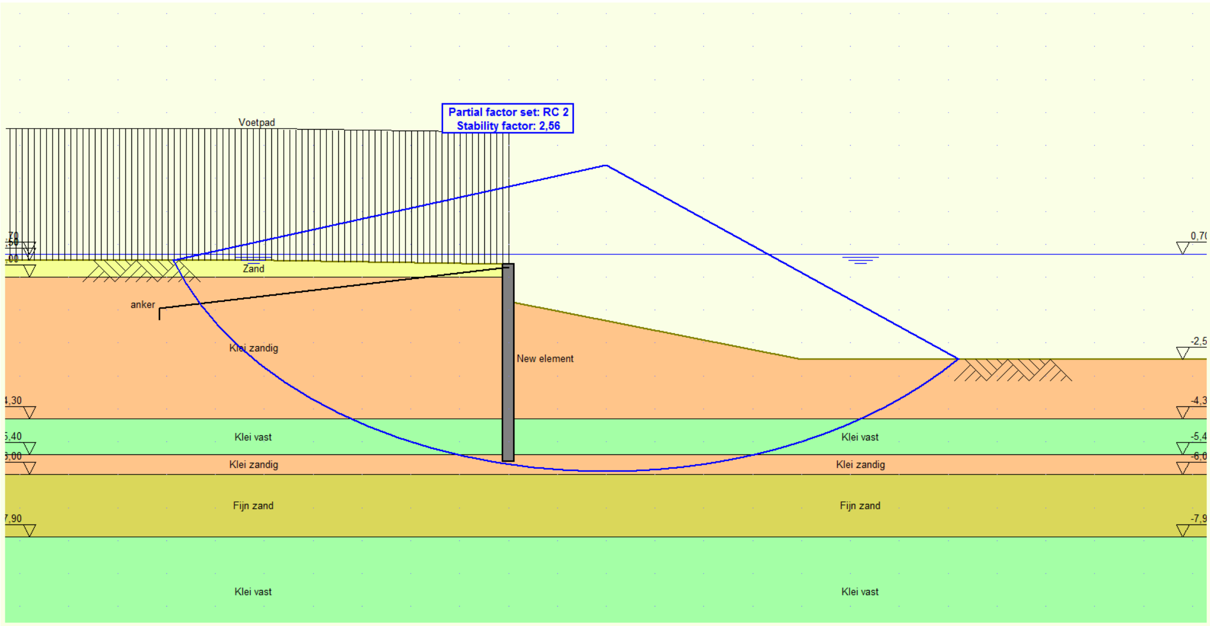
Water level of 0,05 m NAP during high tide





Water level of 0,70 m NAP during extreme high tide





Checking calculations

The abovementioned situations show that a low tide situation produces the largest load for the pile wall, thus presenting the highest risk of creating a slip plane. Therefore the checking calculations will also be made for this situation.

Collecting the occurring moment of force

Through the moment of force in the wooden pile wall pushing and pulling forces occur in the extremist sides of the board. The following formula enables us to check whether these do or do not exceed the characteristic values of Azobé wood :

$$\frac{N}{A} \pm \frac{M}{W} \leq f_{t,0,k} \; of \; f_{c,0,k}$$

The characteristic tensile and compressive strengths in the longitudinal direction of the wood fibres are :

- $f_{t,0,k} = 42$ N/mm2
- $f_{c,0,k} = 34$ N/mm2

In this calculation we disregard the dead weight of the pile wall, leaving us the following formula :

$$\frac{M}{W} = \frac{0{,}14\,kNm}{\frac{b \cdot h^2}{6}} = \frac{0{,}14\,kNm}{\frac{0{,}25 \cdot 0{,}05^2}{6}} = 1344\frac{kN}{m^2} = 1{,}344\frac{N}{mm^2}$$

This value is lower than the characteristic values, through which this condition has been met.

Collecting the sliding force

The collection of the sliding forces by the pile wall can be checked using the following formula :

$$\sigma_{v,d} \leq f_{vk}\frac{k_{mod}}{\gamma_M}$$

As the pile wall is situated in a very wet environment its environmental class is 3/4. Here the load will be there permanently, creating a modifying factor of $k_{mod} = 0{,}5$. The material properties have been based on sawn timber, which has a value of 1,3 [-].

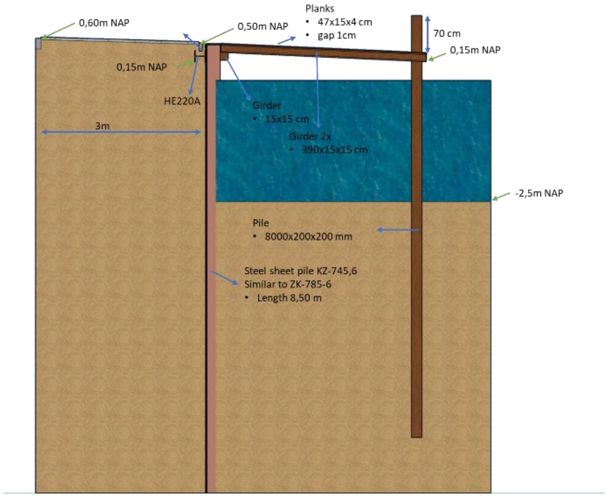
$$\frac{V}{b \cdot 27} = \frac{0{,}59}{25 \cdot 27} = 0{,}087 \frac{N}{mm^2} \leq 4 \frac{N}{mm^2} \cdot \frac{0{,}5}{1{,}3} = 1{,}54 \frac{N}{mm^2}$$

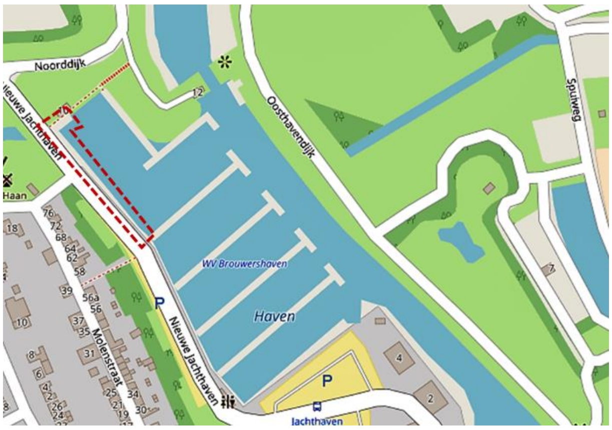
This condition has been met as well.

Stability of the complete quay wall

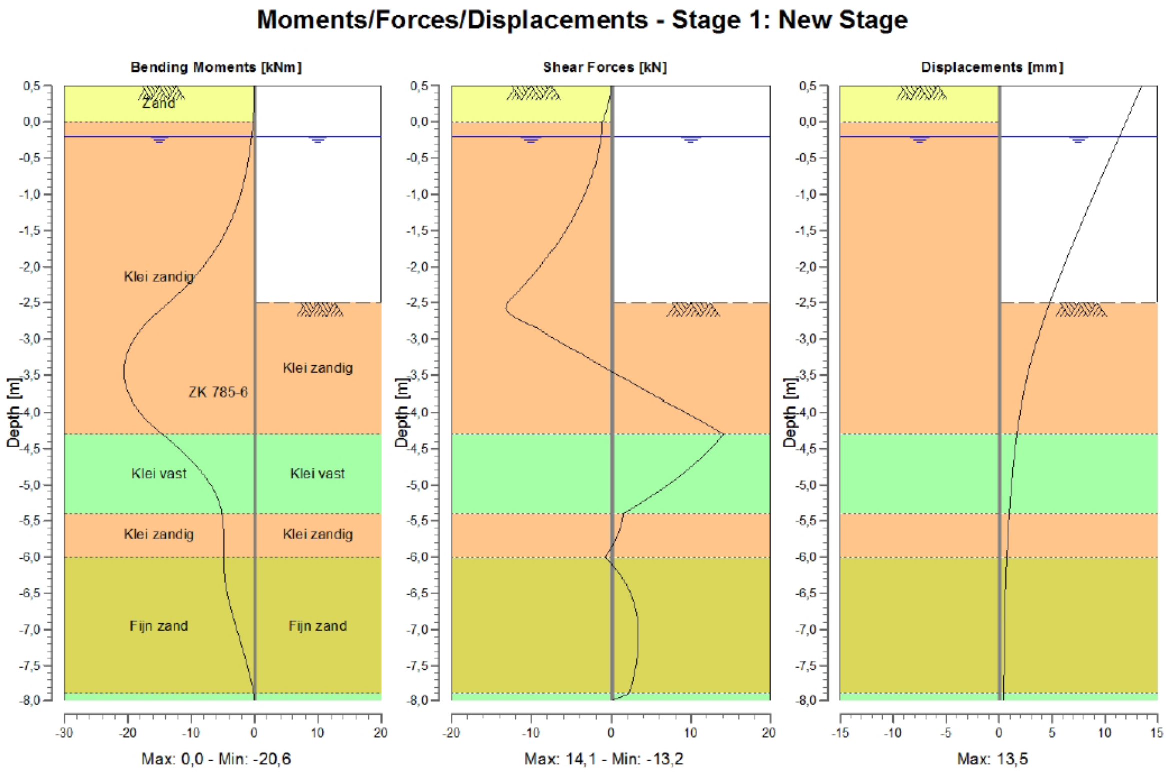
In order to check the stability of the complete quay wall we used the D-sheet pile programme. This programme has been established as such that it also looks for the slip planes having the lowest security factor. For permanent constructions the factor should be more than 1,3 [-]. During low tide this safety factor is 2,07, which is more than sufficient.

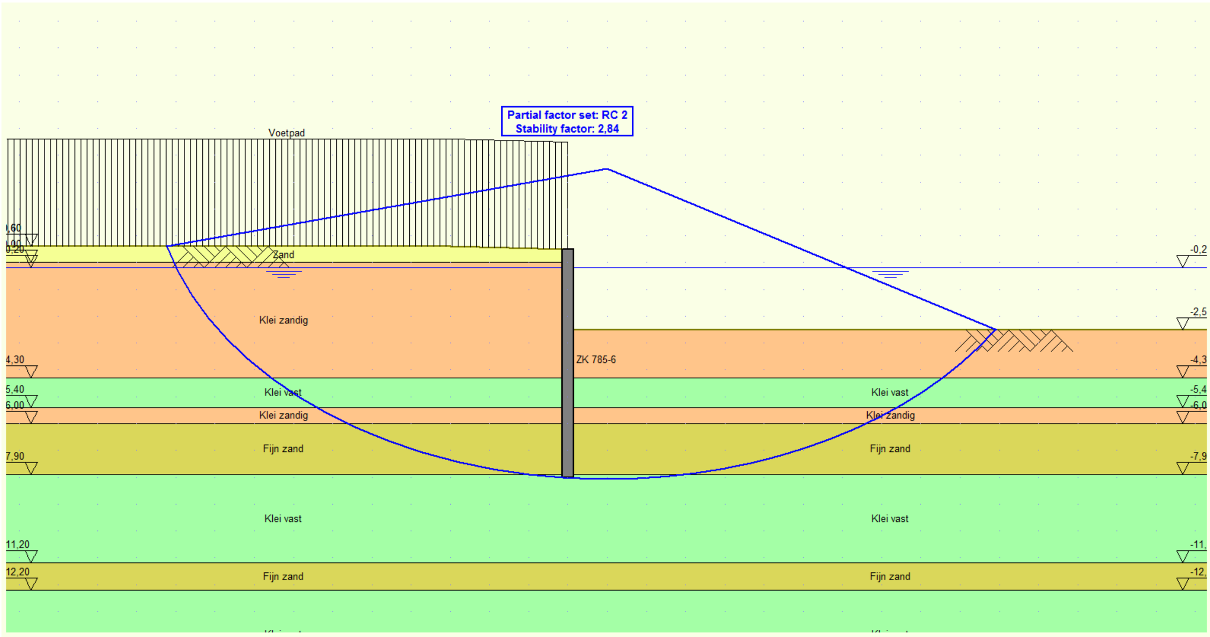
Steel sheet pile wall



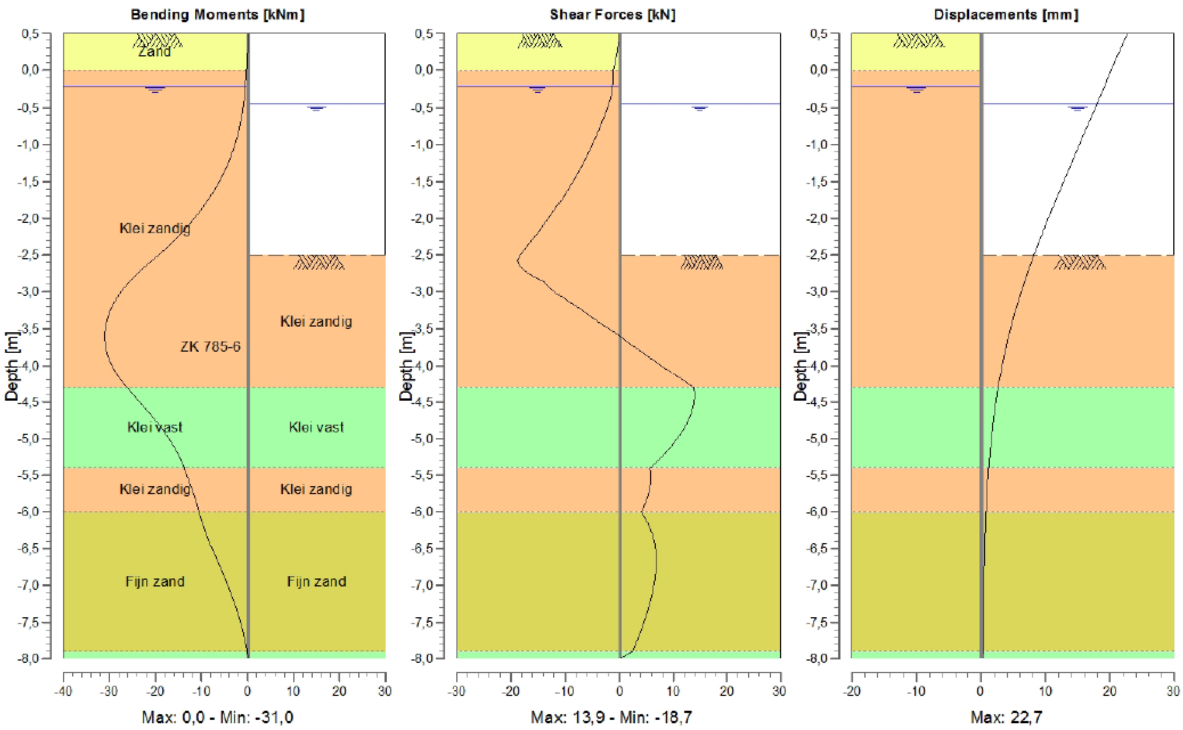


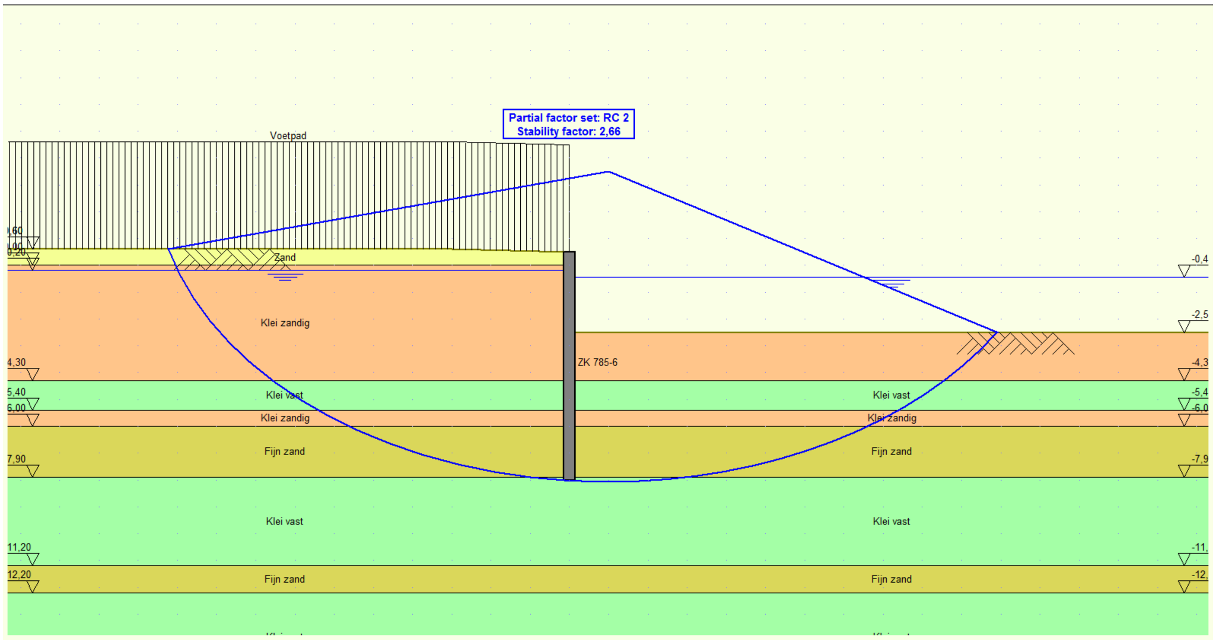
Current situation



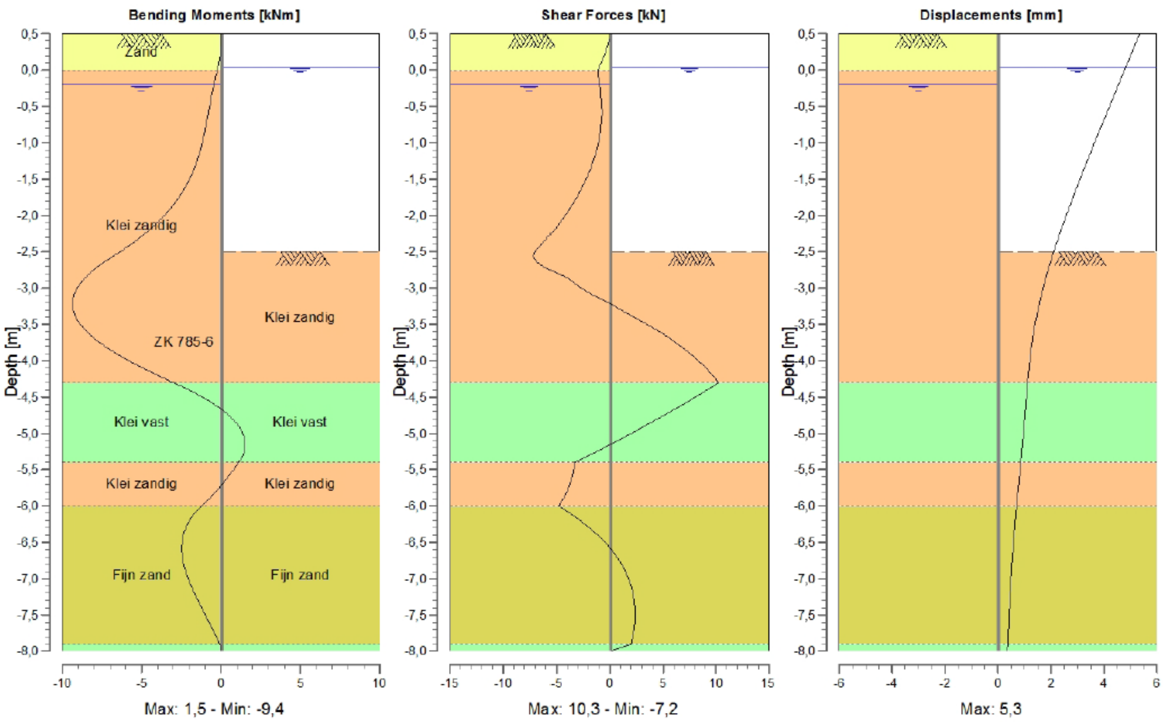


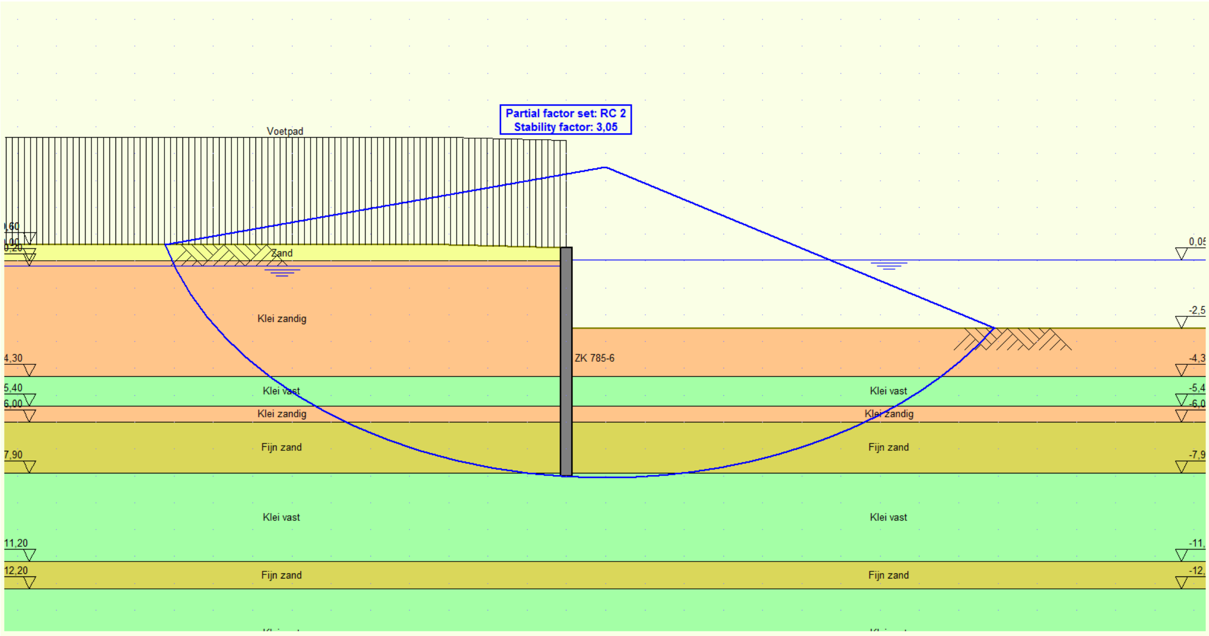
Water level of -0,45 m NAP during low tide



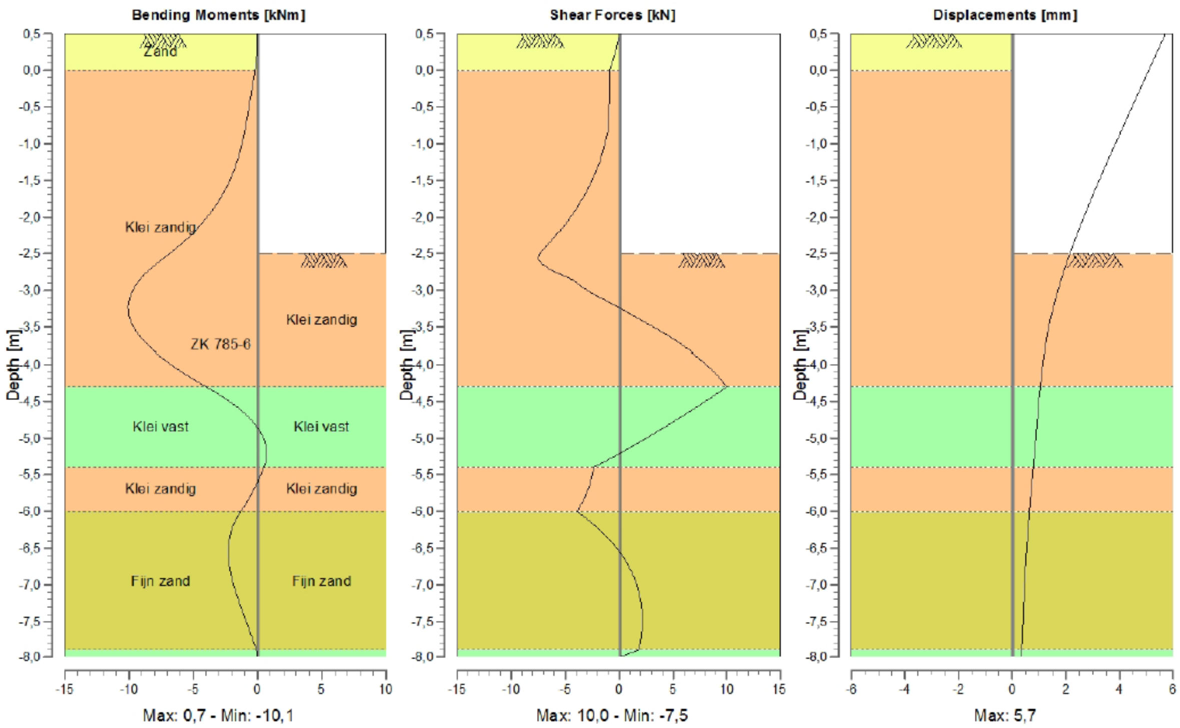


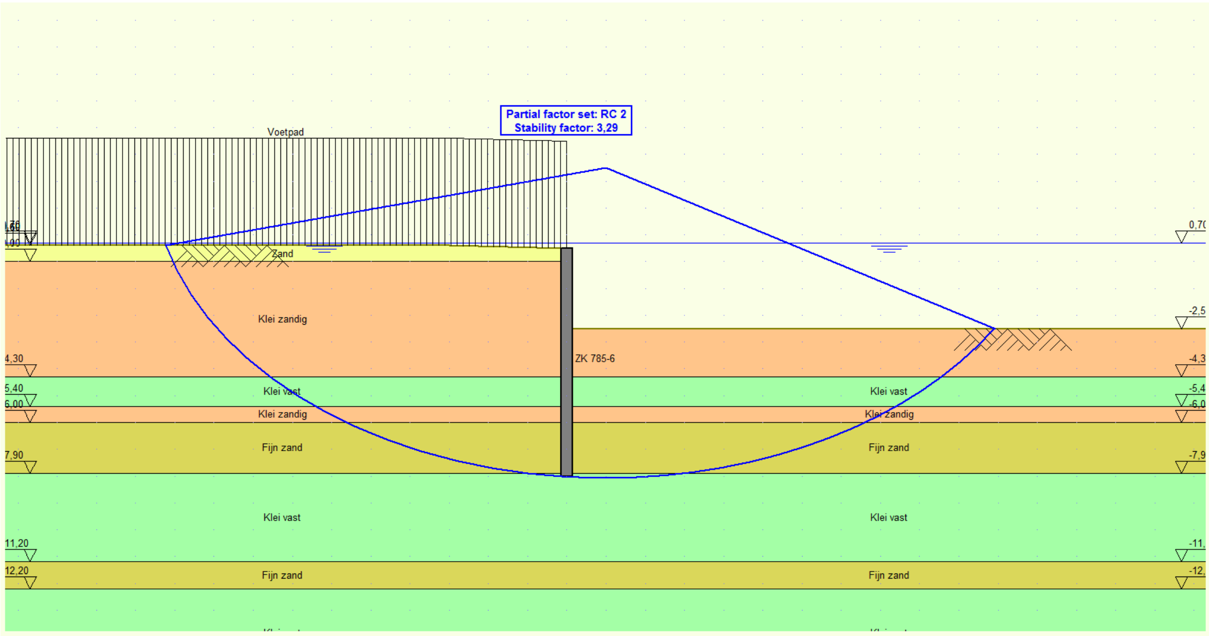
Water level of 0,05 m NAP during high tide





Water level of 0,70 m NAP during extreme high tide





Calculations' check

For this construction the same checking calculations were made as was the case in the abovementioned construction. Here the following specifications were used for the steel pile wall.

Lightweight sections — **ZK 785-5 to ZK 785-9 and ZK 675-5 to ZK 675-9**

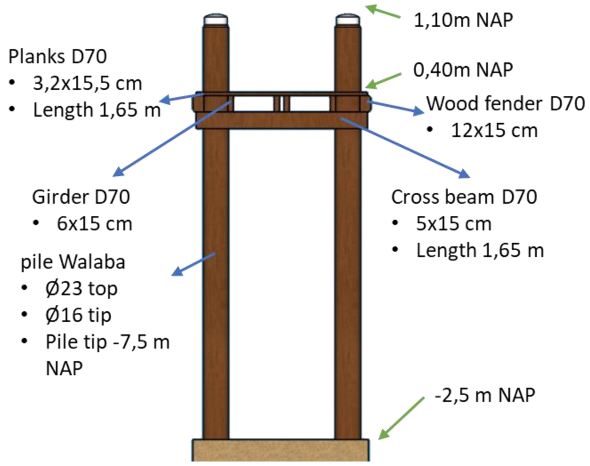
	Section modulus	Weight		Second moment of inertia	Section width	Wall height	Back thick-ness	Web thick-ness
	W_y cm³/m	kg/m²	kg/m	I_y cm⁴/m	b	h	t	s
	Wall	Wall	Single pile	Wall	mm	mm	mm	mm
ZK 785-5	**605**	**53.4**	41.9	**8395**	785	276	5.0	
ZK 785-6	**724**	**64.2**	50.4	**10053**	785	277	6.0	

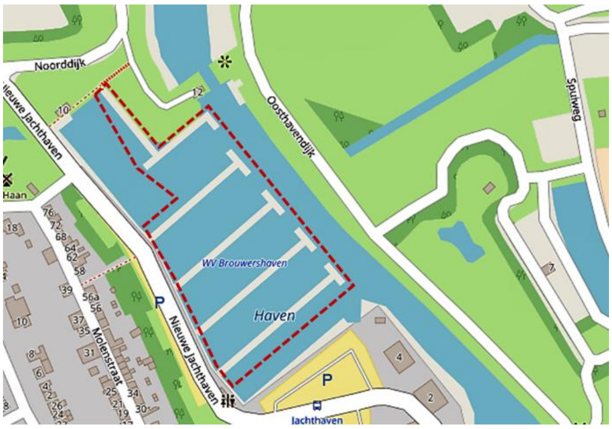
Calculations in a low tide situation produced the following results :

- The occurring moment of force can be collected by the pile wall : $\sigma_x = 42817{,}68\ kN/m^2 \leq f_y = 235000\ kN/m^2$
- The security factor concerning the creation of slip plane is sufficiently high so that we can be sure that these will not occur.

Appendix 10: Stability check of scaffoldings and side scaffoldings

The scaffoldings





In the following table calculations have been made to determine what force the piles will have to convey to the subsoil.

Materials	**Specific weight**								
Walaba	1125,00	kg/m3	http://innovita-advies.nl/wp-content/uploads/2014/06/walaba.pdf						
Azobé	1060,00	kg/m3	http://www.houtinfo.nl/node/312						
	number	**volume**		**weight**		**mass**		**total mass**	
piles	2,00	0,26	m3	288,94	kg	2834,52	N	5669,03	N
Cross beams	2,00	0,01	m3	15,74	kg	154,42	N	308,84	N
Wood fenders	2,00	0,05	m3	57,24	kg	561,52	N	1123,05	N
Girders	4,00	0,03	m3	28,62	kg	280,76	N	1123,05	N
Boards	20,00	0,01	m3	8,68	kg	85,10	N	1702,04	N
								9926,01	N
							per pile	4963,01	N
Variable load			**Area**						
Pedestrians		kN/m2	4,95	m2				24750,00	N
							per pile	12375,00	N
							Total	24,52	kN

In order to determine the bearing force of the scaffolding pile we used the following formulas :

$$F_{r;max} = F_{r;max;tip} + F_{r;max;shaft} - F_{s;nk;rep} - W$$

Taking into account :

- $F_{r;max;tip} = A_{tip} \cdot p_{r;max;tip}$
  - $p_{r;max;tip} = \frac{1}{2}\alpha_p \beta s \left(\frac{q_{c;I;mean}+q_{c;II;mean}}{2}\right) + q_{c;III;mean}$
- $F_{r;max;shaft} = O_{p;mean} \int_0^{\Delta L} p_{r;max;shaft} dz$

- $p_{r;max;shaft} = \alpha_s q_c$

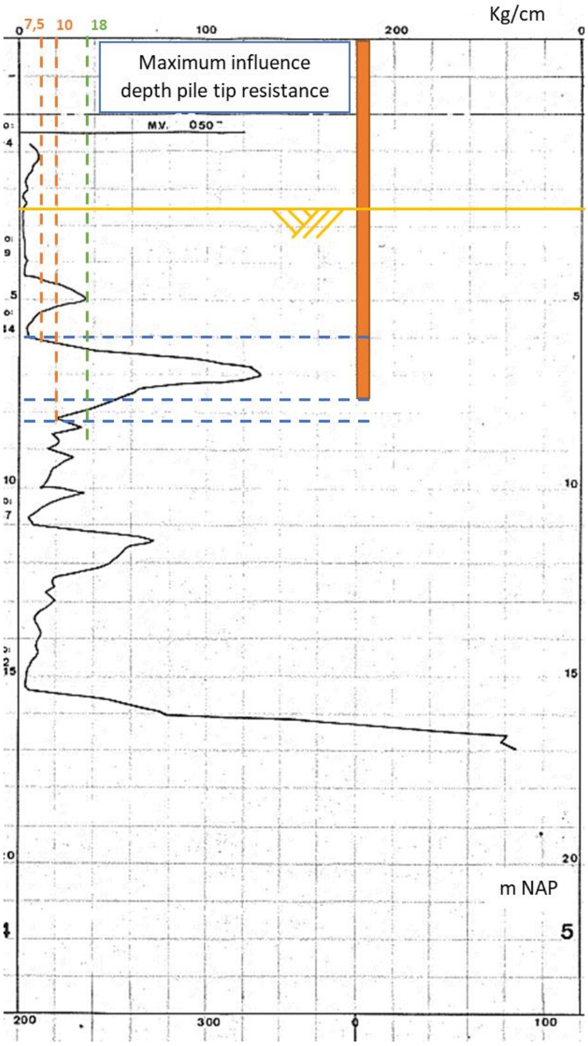
- $F_{s;nk} = O_s h K_0 \sigma_v' \tan\delta$

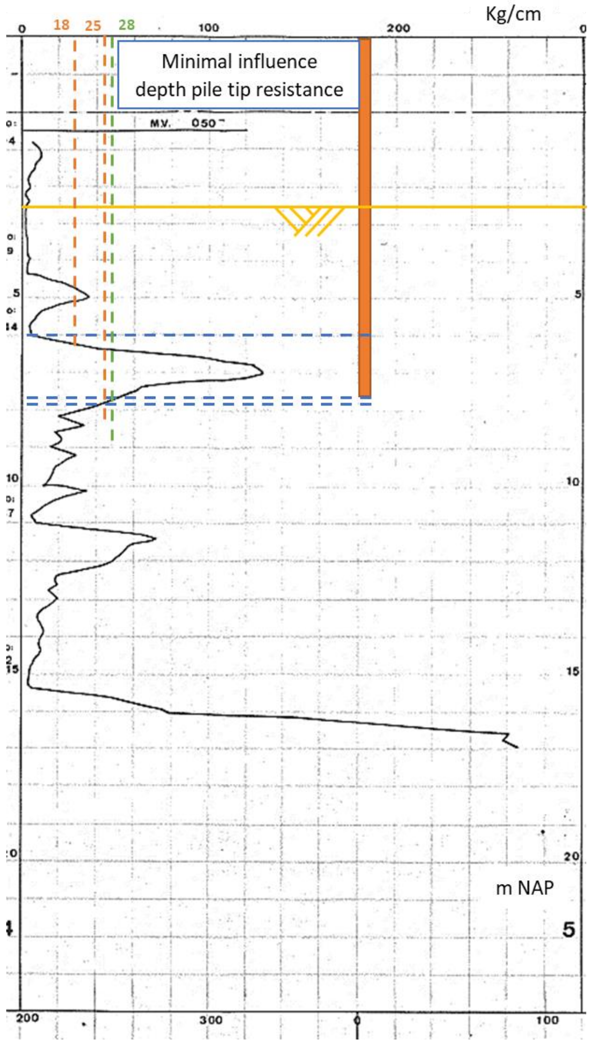
These values were determined as presented in the following tables :

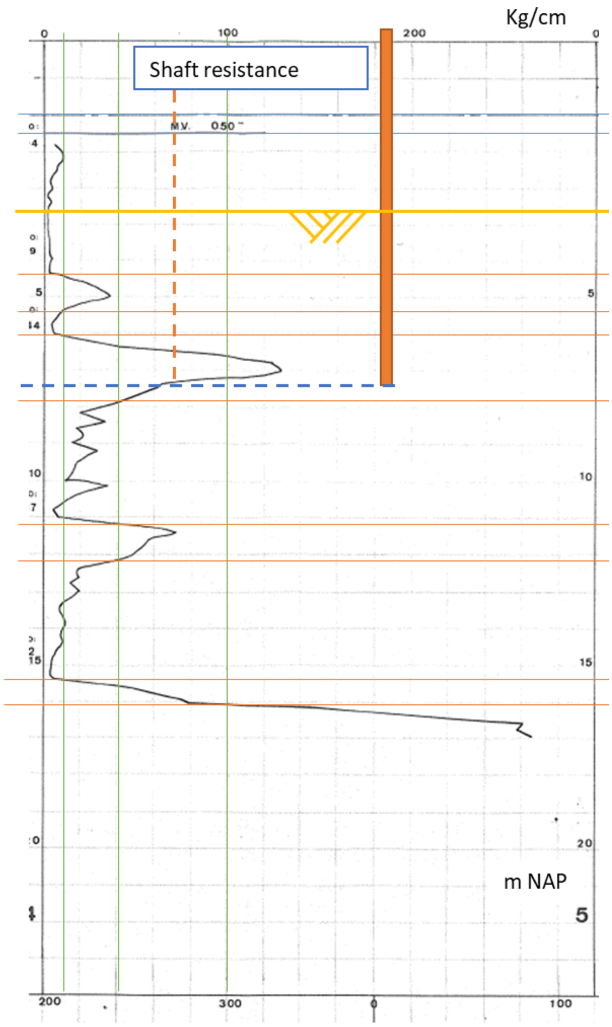
General values			
Pile class factor	αp	1	-
Factor for the shape of the pile's foot	β	1	-
Shape influence factor of the cross-section	s	1	-
Minimal pile diameter	ø	160	mm
Maximal pile diameter	ø	220	mm
Average pile diameter	ø	190	mm
factor of shaft friction	αs	0,0012	-
Pile length	L	8,6	m
Volumetric weight of reinforced concrete	γb	25	kN/m3
Material factor of the pile	γm,g	1,1	-
Volumetric weight of water	γw	10,05329	kN/m3

Maximum end resistance			
Minimum depth influence	d	133	mm
Maximum depth influence	d	760	mm
Influencing distance of the piles	dIII	1520	mm
Min influence depth			
	qc,I,gem, min	2,75	Mpa
	qc,II, gem	2,45	MPa
	qc,III,gem	2,30825	MPa
Maximum tip load	pr,max,punt	2,454125	MPa
Max influence depth			
	qc,I,gem, max	1,765	Mpa
	qc,II, gem	0,980665	MPa
	qc,III,gem	0,944539	MPa
Maximum tip load	pr,max,punt	1,158686	MPa
Maximum tip force			
Maximum tip force	Fr,max,punt	131,4083	kN

Maximum shaft resistance			
Cone resistance of sand	qc	6,86	Mpa
Maximum shaft resistance	pr,max,schacht	0,008232	Mpa
Maximum shaft force	Fr,max,schacht	14,74110671	kN
Functional length	ΔL	1,5	m

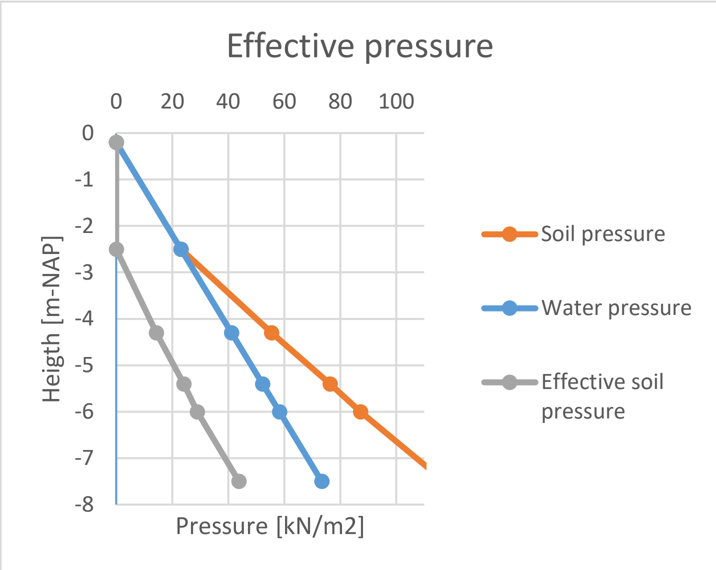

7,5
10
18
Kg/cm
Maximum influence
depth pile tip resistance
m NAP


18
25
28
Kg/cm
Minimal influence
depth pile tip resistance
m NAP



Negative shaft friction (Normal situation)			
Volumetric weight of soft clay	γks	18	kN/m3
Volumetric weight of solid clay	γkv	19	kN/m3
Volumetric weight of loose sand	γz	20	kN/m3
Volumetric weight of water	γw	10,053288	kN/m3
Pressure			
Soil pressure	-0,2	0	kN/m2
Water pressure		0	kN/m2
Effective soil pressure		0	kN/m2
Soil pressure	-2,5	23,1225624	kN/m2
Water pressure		23,1225624	kN/m2
Effective soil pressure		0	kN/m2
Soil pressure	-4,3	55,5225624	kN/m2
Water pressure		41,2184808	kN/m2
Effective soil pressure		14,3040816	kN/m2
Soil pressure	-5,4	76,4225624	kN/m2
Water pressure		52,2770976	kN/m2
Effective soil pressure		24,1454648	kN/m2

Soil pressure	-6	87,2225624	kN/m2
Effective soil pressure		58,3090704	kN/m2
Soil pressure		28,913492	kN/m2
Water pressure	-7,5	117,2225624	kN/m2
Effective soil pressure		73,3890024	kN/m2
Effective soil pressure		43,83356	kN/m2

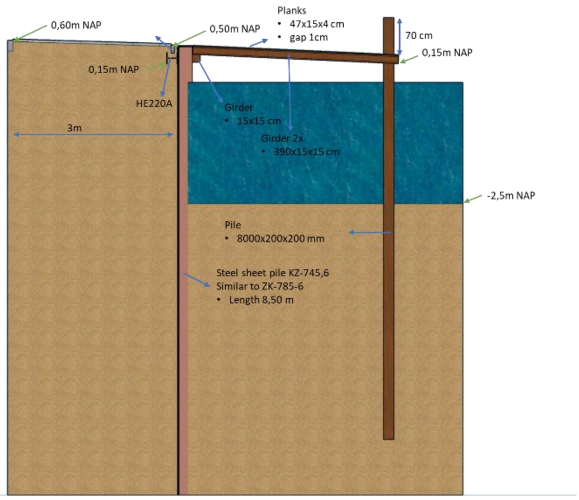


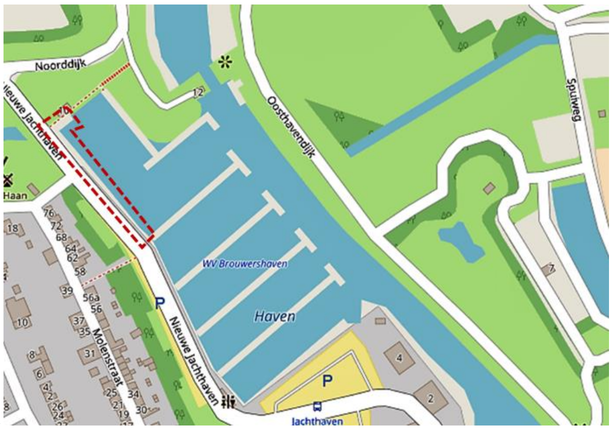
Angle of friction between pile and soil			
Sand	δz	20	degrees
Sandy clay	δks	11,7	°
Solid clay	δkh	11,7	°
Neutral coefficient of earth pressure			
Sand	K0	0,657979857	-
Soft clay	K0	0,797212705	-
Solid clay	K0	0,797212705	-
Average effective vertical pressure			
Height			
-3,4	m NAP	7,1520408	kN/m2
-4,85	m NAP	19,2247732	kN/m2
-5,7	m NAP	26,5294784	kN/m2
Force of negative shaft friction			
Negative shaft friction	Fs,nk	0,935033058	kN

Maximum bearing force			
	Fr,max	146,1494	kN
	Fr, tot	107,5662	kN
Maximum bearing force			
Total load		24,51811	kN

The table above shows that the scaffold pile's bearing force is abundantly sufficient to permit to convey the weight of the scaffolding and that of potential users to the soil.

Side scaffoldings





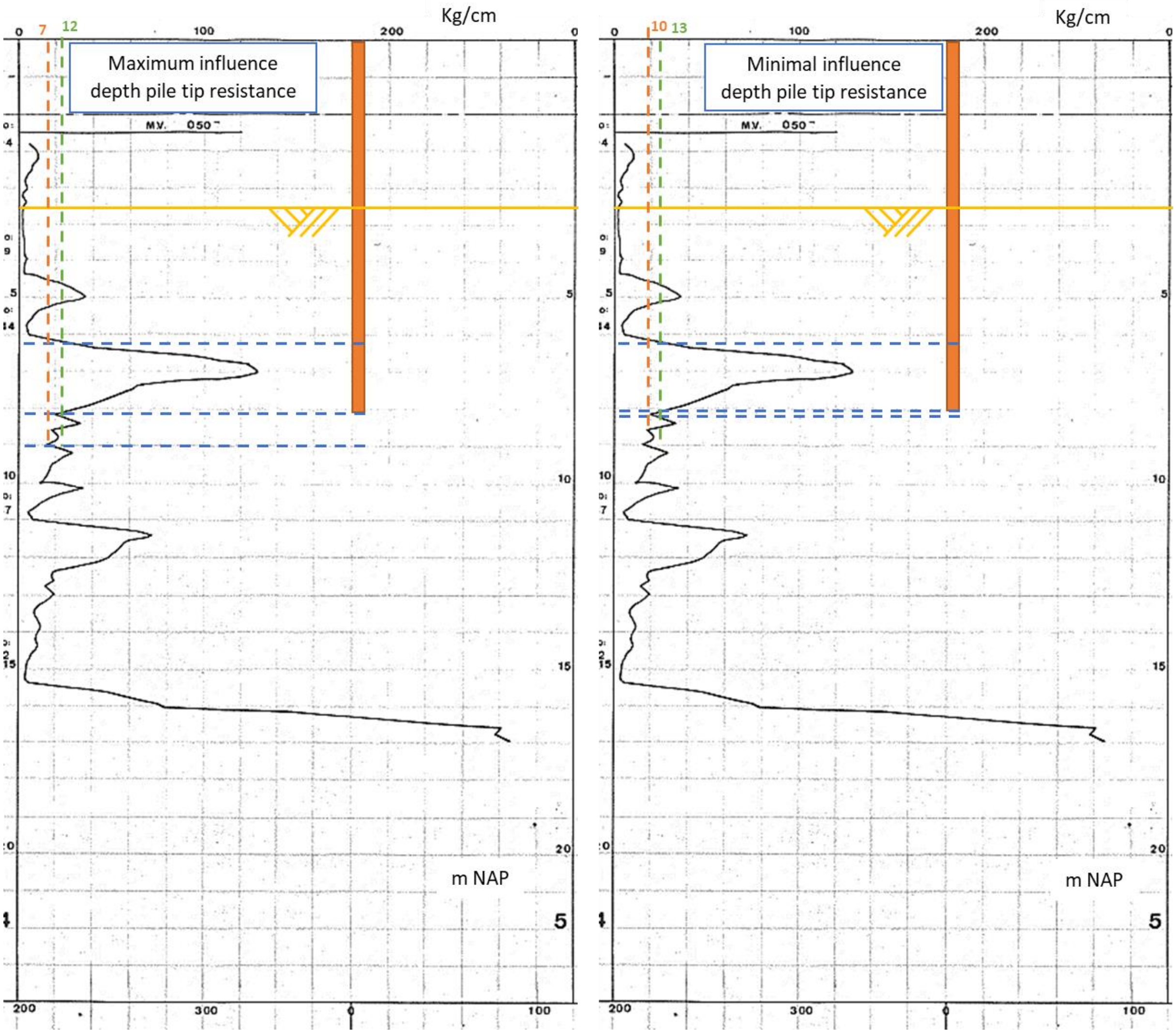
In order to determine its bearing force we apply the same method as we did for the scaffolding construction.

Materials	Specific weight		
Azobe	1060,00	kg/m3	http://www.houtinfo.nl/node/312

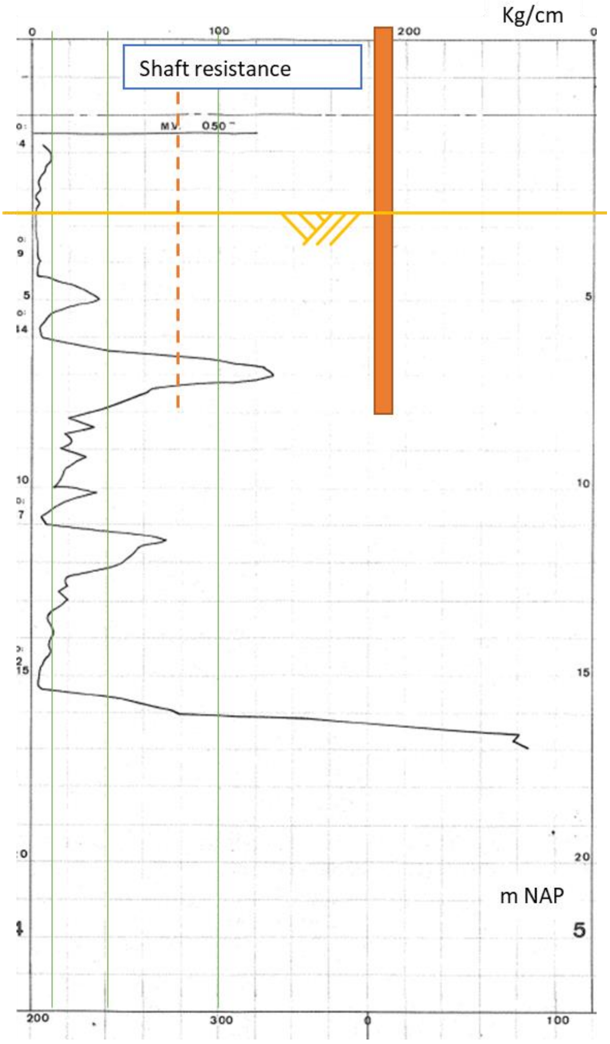
	number	volume		weight		mass		total mass	
piles	1,00	0,32	m3	339,20	kg	3327,55	N	3327,55	N
Girders	2,00	0,09	m3	93,02	kg	912,48	N	1824,95	N
Boards	26,00	0,00	m3	2,99	kg	29,32	N	762,43	N
								5914,93	N
							per pile	2957,47	N
Variable load			**Area**						
Pedestrians	5,00	kN/m2	1,83	m2				9165,00	N
							per pile	4582,50	N
							total	10,42	kN

General values			
Pile class factor	αp	1	-
Factor for the shape of the pile's foot	β	1	-
Factor influence shape of the cross-section	s	1	-
Pile diameter	ø	200	mm
equivalent pile diameter	ø	225,6758	mm
factor of shaft friction	αs	0,0012	-
Pile length	L	8,6	m
Volumetric weight of reinforced concrete	γb	25	kN/m3
Material factor of the pile	γm,g	1,1	-
Volumetric weight of water	γw	10,05329	kN/m3

Maximum tip resistance			
Minimum influence depth	d	157,9731	mm
Maximum influene depth	d	902,7033	mm
Influencing distance of the pile	dIII	1805,407	mm
Min influence depth			
	qc,I,gem, min	1,274865	Mpa
	qc,II, gem	0,980665	MPa
	qc,III,gem	0,980665	MPa
Maximum tip load	pr,max,punt	1,054215	MPa
Max influence depth			
	qc,I,gem, max	1,176798	Mpa
	qc,II, gem	0,686466	MPa
	qc,III,gem	0,686466	MPa
Maximum tip load	pr,max,punt	0,809049	MPa
Maximum tip force			
Maximum tip force	Fr,max,punt	129,4478	kN

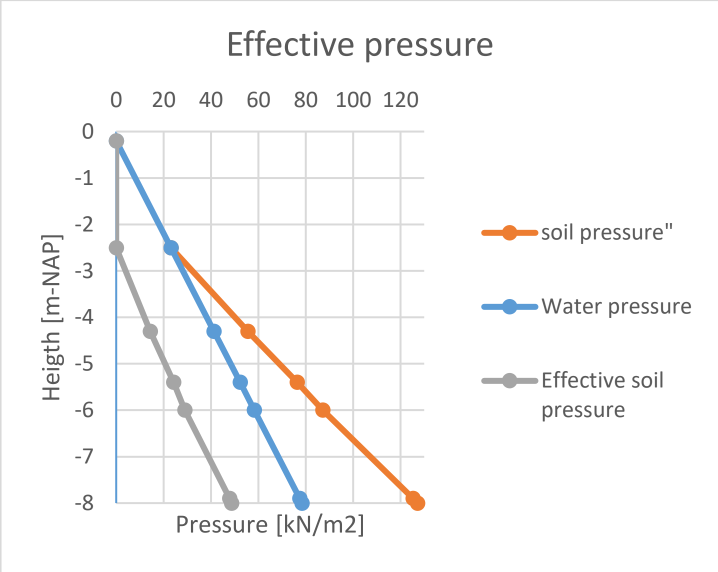


Maximum shaft resistance			
Cone resistance sand	qc	8,825985	Mpa
Maximum shaft resistance	pr,max,schacht	0,010591182	Mpa
Maximum shaft force	Fr,max,schacht	28,53401961	kN
Functional length	ΔL	1,9	m



Negative shaft friction (Normal situation)			
Volumetric weight of soft clay	γks	18	kN/m3
Volumetric weight of solid clay	γkv	19	kN/m3
Volumetric weight of loose sand	γz	20	kN/m3
Volumetric weight of water	γw	10,053288	kN/m3
Pressures			
Soil pressure	-0,2	0	kN/m2
Water pressure		0	kN/m2
Effective soil pressure		0	kN/m2
Soil pressure	-2,5	23,1225624	kN/m2
Water pressure		23,1225624	kN/m2
Effective soil pressure		0	kN/m2
Soil pressure	-4,3	55,5225624	kN/m2
Water pressure		41,2184808	kN/m2
Effective soil pressure		14,3040816	kN/m2
Soil pressure	-5,4	76,4225624	kN/m2
Water pressure		52,2770976	kN/m2
Effective soil pressure		24,1454648	kN/m2

Soil pressure	-6	87,2225624	kN/m2
Water pressure		58,3090704	kN/m2
Effective soil pressure		28,913492	kN/m2
Soil pressure	-7,9	125,2225624	kN/m2
Water pressure		77,4103176	kN/m2
Effective soil pressure		47,8122448	kN/m2
Soil pressure	-8	127,1225624	kN/m2
Water pressure		78,4156464	kN/m2
Effective soil pressure		48,706916	kN/m2

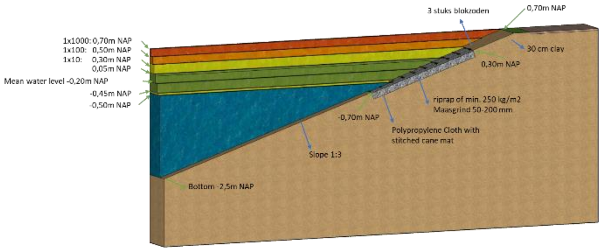


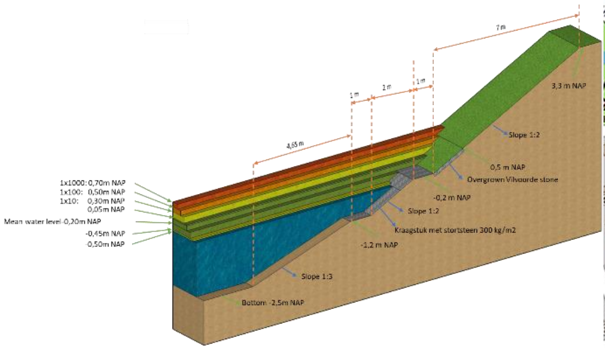
Angle of friction between pile and soil			
Sand	δz	20	°
Soft clay	δks	11,7	°
Solid clay	δkh	11,7	°
Neutral coefficient of earth pressure			
Sand	K0	0,657979857	-
Soft clay	K0	0,797212705	-
Solid clay	K0	0,797212705	-
Average effective vertical pressure			
Height			
-3,4	m NAP	7,1520408	kN/m2
-4,85	m NAP	19,2247732	kN/m2
-5,7	m NAP	26,5294784	kN/m2
-7,95	m NAP	48,2595804	kN/m3
Force of negative shaft friction			
Negative shaft friction	Fs,nk	1,446615418	kN

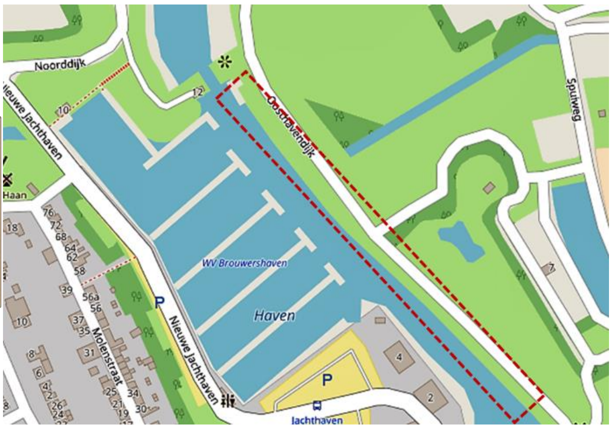
Maximum bearing force			
wooden pile	Fr,max	157,9818	kN
	Fr, tot	115,952	kN
Total load			
Total load		24,51811	kN

For this scaffolding construction as well the pile's bearing force is abundantly sufficient to permit to convey the occurring loads to the subsoil.

Appendix 11: Slope protection







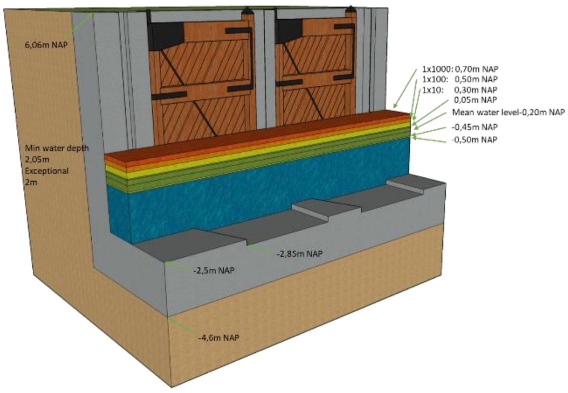
As these dike bodies were constructed based on the water level occurring before the Grevelingen lake was closed and before the slope revetment for the occurring waves was built, in this check only the impact of the new water levels on the slope revetment is taken into account. The above presented figures show that the normal tide is still situated within the area provided with slope revetment. The only test here would be to check whether the waves created by the passing ship traffic do or do not exceed this limit.

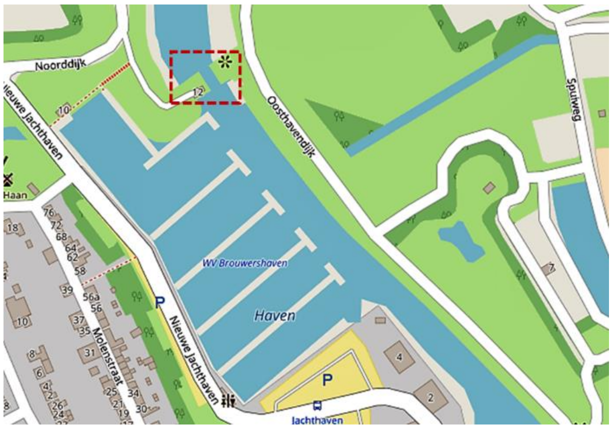
The maximum speed for ships navigating through the harbour is 5 km/hour (1,39 m/s). Which then creates a bow wave as high as follows :

$$h = \frac{v_s^2}{2g} = \frac{\left(1{,}39\,\frac{m}{s}\right)^2}{2g} 0{,}0983\ m = 9{,}83\ cm$$

When these waves plus high tide occur there is no danger for the dike body as both of these situate themselves within the range of the slope revetment. However, extreme water levels do exceed the slope revetment. Hence these could possibly harm the dike body. But as this situation will never last long and as it is fairly rare the damage will remain reduced to only lightly harming the grass cover.

Appendix 12: Guard lock

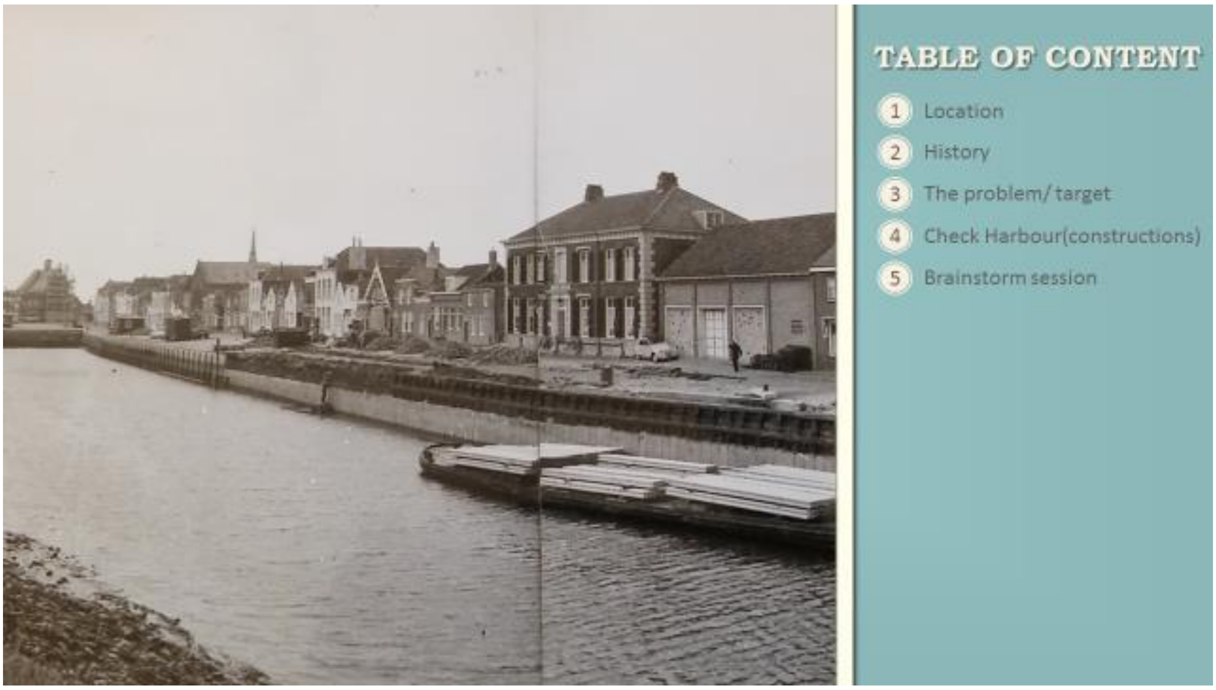




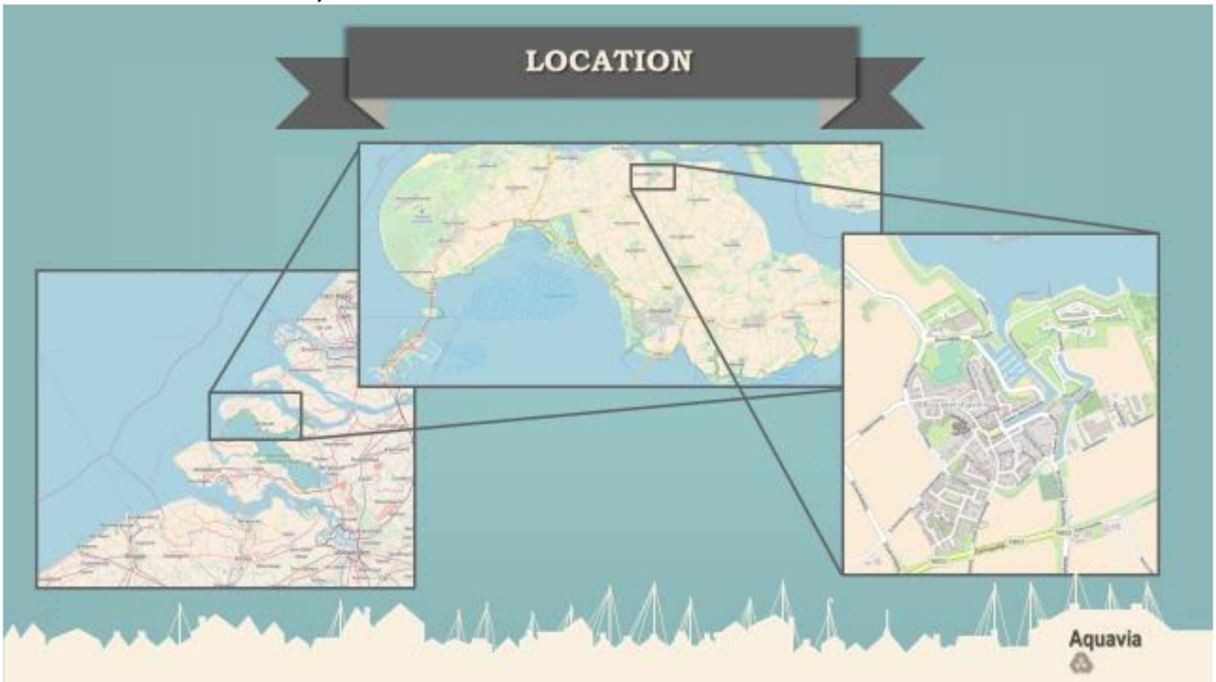
The guard lock was established and built just after the flood disaster of 1953. This lock blocked storm-surge levels until the construction of the Brouwersdam was finished in 1971. This historical fact enables us to conclude that the guard lock's stability is not endangered by the introduction of the new tide. The only limiting factor of this construction is that the guard lock's floor has been fixed at -2,5 m NAP, which can create a problem for the navigable depth during low tide.

Appendix 13: The Brainstorm session presentation

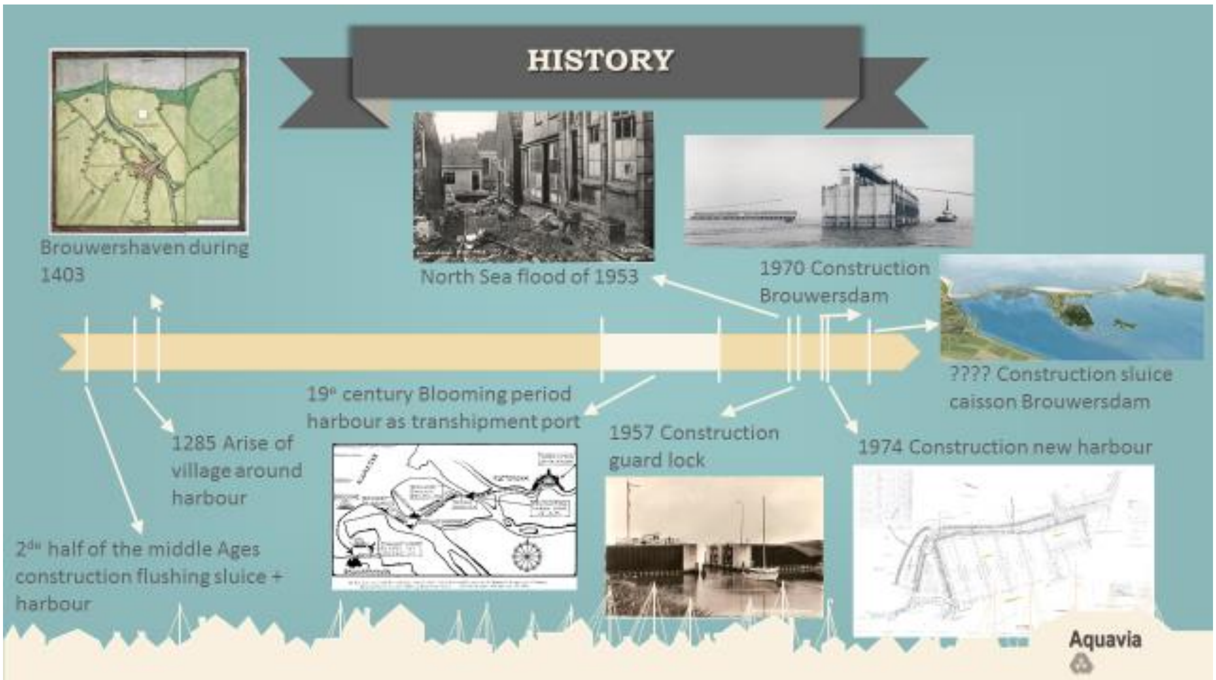
Good afternoon and be welcome to the brainstorm session of my research survey in Brouwershaven. Before we start the brainstorm session itself we will show you a short presentation to discuss the problem we want to solve.



During this presentation we will first deal with the site where our research survey took place. Then we will shortly present some historical facts about Brouwershaven. After which the problem to be solved and the goal of the research survey will be dealt with. As a fourth element we will treat the problems that emerged for the harbour and the harbour constructions. Eventually we will have the brainstorm session itself.

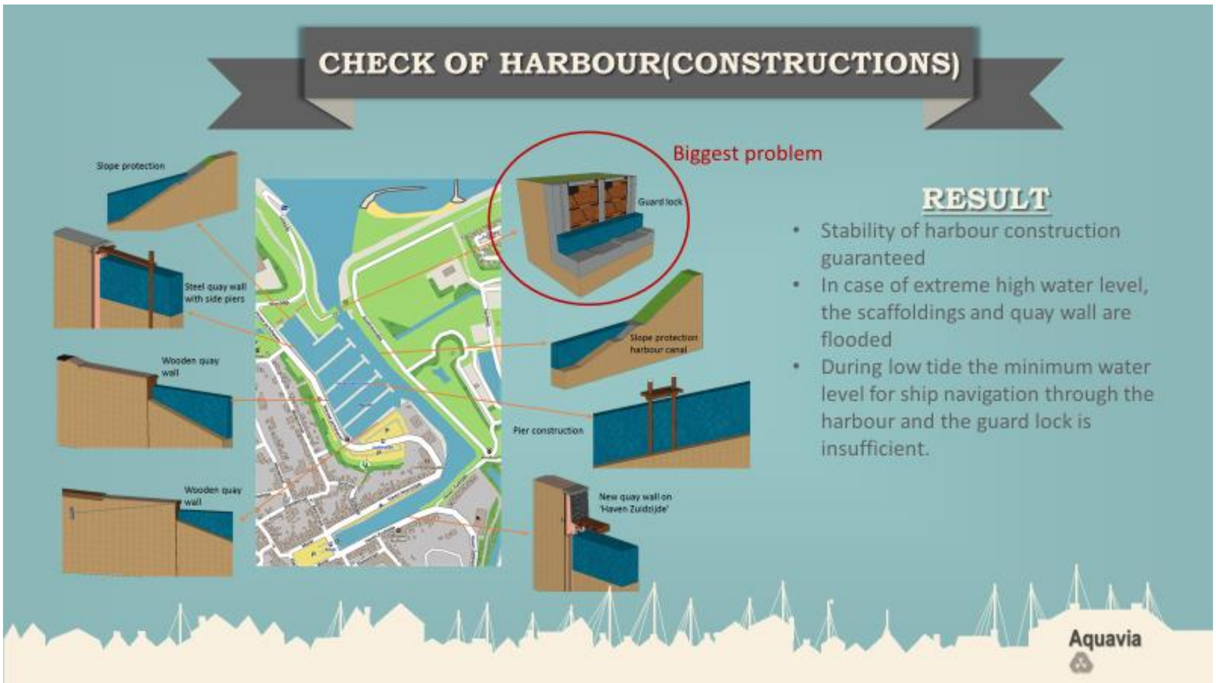


The specific site subject to our research survey is situated in the Dutch province of Zeeland, more precisely in Brouwershaven, which is situated on the island of Schouwen-Duiveland.

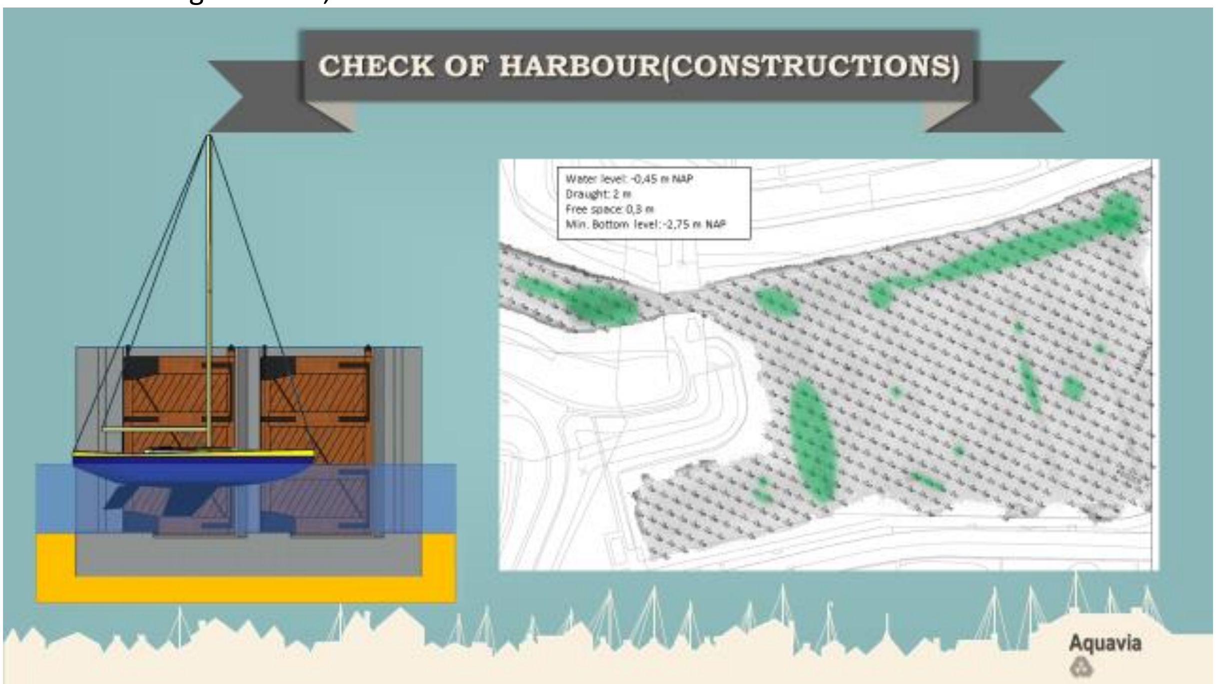


Brouwershaven developed during the second half of the Middle Ages. It is in those days that the decision was taken to build a harbour with a scouring sluice here. It does not take long before the wish was expressed to build a village around the harbour. This is when the harbour develops into a busy trading harbour. This trend is weakening throughout the years because the size of the ships enhances. Which made it impossible for the ships to access the harbour any longer, making them rather choose for larger harbours such as Rotterdam. Through the silting up of the river Maas the harbour flourished up to a maximum during the 19th century as ships were opting for Brouwershaven as a transfer harbour. However, when the 'Nieuwe waterweg' opened in Rotterdam an end was put to this period. In addition to this Brouwershaven also suffered severely from the storm flood disaster in 1953. A direct reaction to this was the construction of the guard lock, which was meant to function as a primary water barrier first and would be degraded to a secondary water barrier after the construction of the Brouwersdam. As Brouwershaven would be deprived completely of cargo shipping traffic through the construction of the Brouwersdam the construction of the new yacht harbour started in 1974 in order to thus be capable of focussing completely on recreative shipping. Because of the bad water quality in the Grevelingen lake Rijkswaterstaat decided to build a sluice caisson or a tidal power plant in Brouwersdam in the future, resulting in a reduced tide in the Grevelingen lake. Which of course immediately leads us to the issue and the goal of this report.

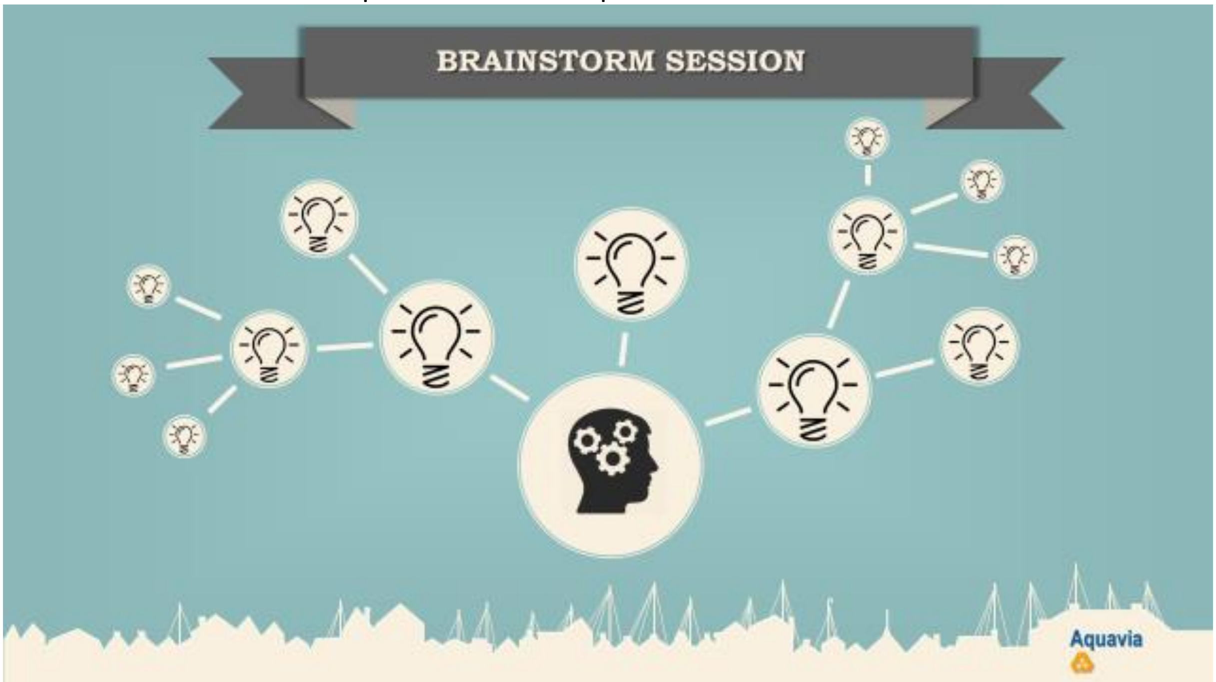
As the harbour has had a fixed water level ever since 1970 the renewed harbour constructions and the new harbour were developed in function of this fixed water level. Which leaves the impact of the reduced tide on the harbour itself and on its constructions as an unanswered question. Which leads us to the following research issue : 'Is it necessary to adapt the constructions in the harbour and/or the harbour of Brouwershaven itself or to protect them from the reduced tide in the Grevelingen lake ?'



The map in this slide shows all of the constructions that were checked. Here we focussed on the constructions in the new harbour as these constructions emerged only after the closure of the Grevelingen lake. During this check it could be established that there is no danger for the stability of the constructions. However, problems may occur taking into account the acting water levels. During extreme high tide for example the water will float over the quay wall causing the promenade pier and the jetty to get under water for 30 cm. During low tide the danger is that the water depth needed for ships in the harbour is no longer attained, neither at the guard lock, nor in the harbour itself.

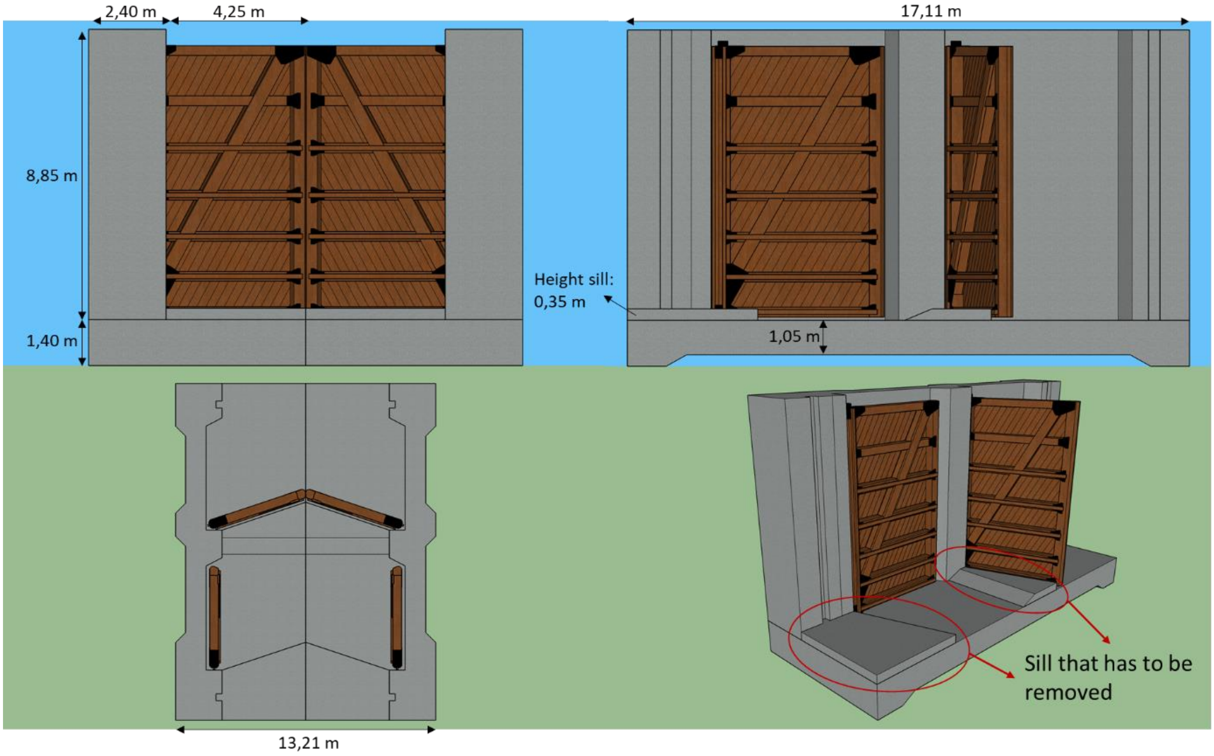


At present at the guard lock ships having a draught of 2 m have a spare room here of 30 cm. According to the design rules for locks this is tight already. Should the water in reduced tide fall to -0.45 m NAP, then this spare room under the ship would decrease to 5 cm, or even to 0 cm at the water level that is allowed to occur during 10% of time. A completely similar problem can be seen in the harbour itself where the depth used lies at -2,5 m NAP (height equal to that of the guard lock's sills). The map shows which areas in the new harbour would still suffice for this water depth needed for ships.

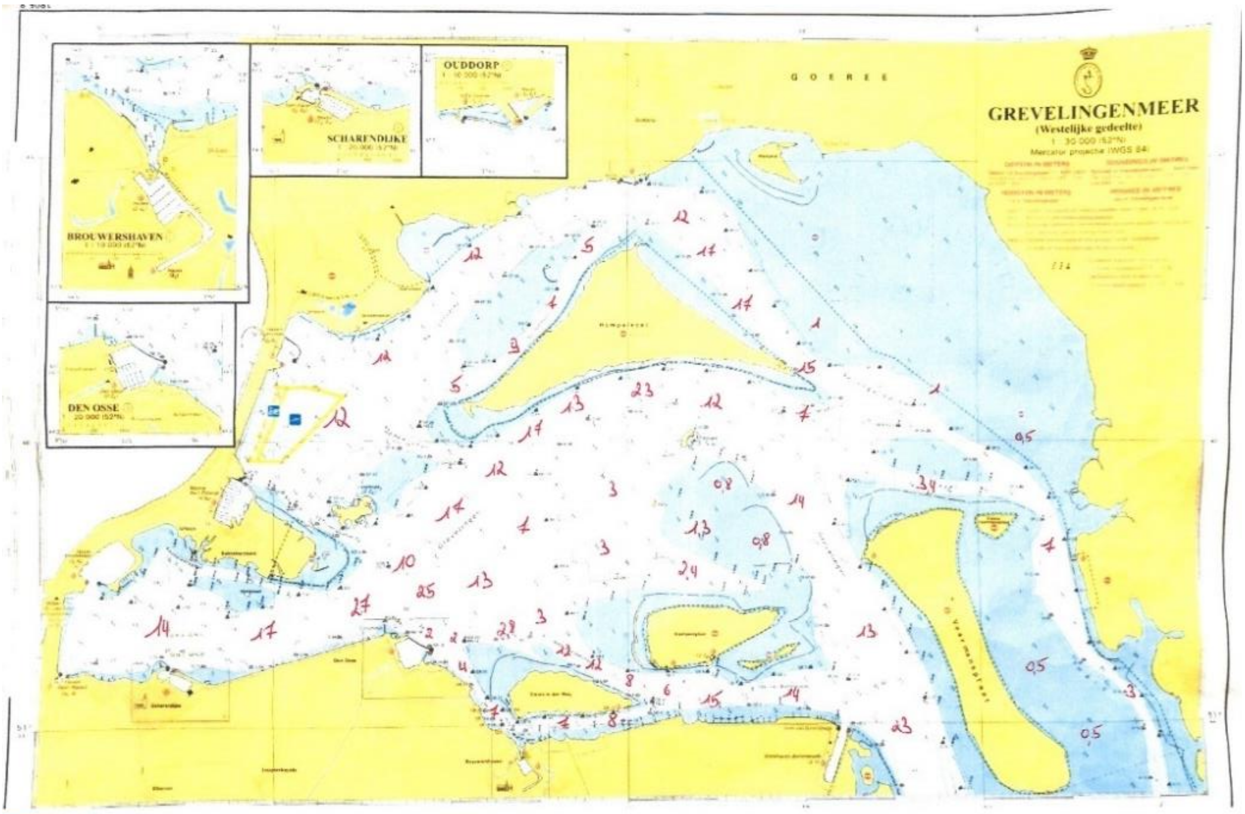


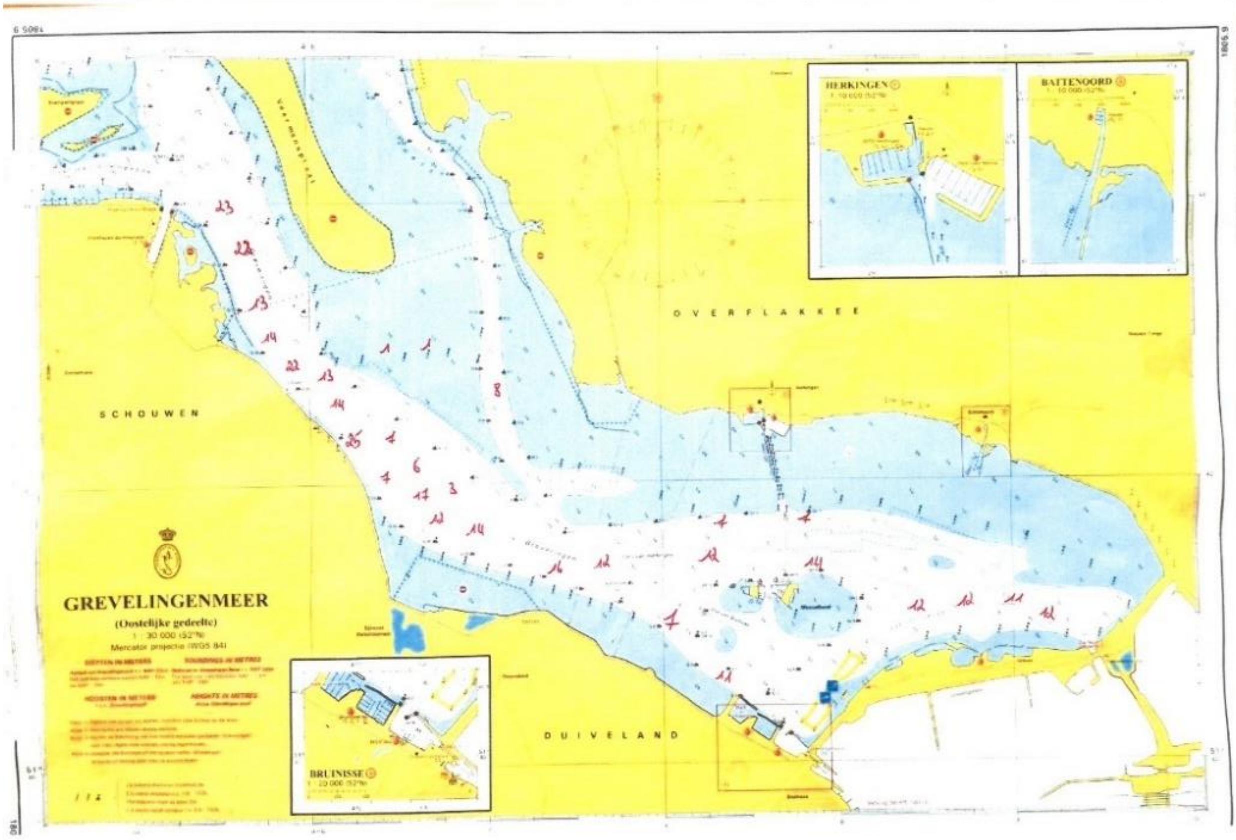
Eventually we arrived at the brain storm session itself here. You can now let your creativity and fantasy take their own course.

Appendix 14: Dimensions of the guard lock



Appendix 15: Shipping maps





Appendix 16: Determination of the wave dimensions

With a fetch length of 6 km and a windspeed of 25 m/s the ‘Groen and Dorrestein’ nomogram gives a specific wave height of 1,7 meter and a wave period of 3,5 seconds.

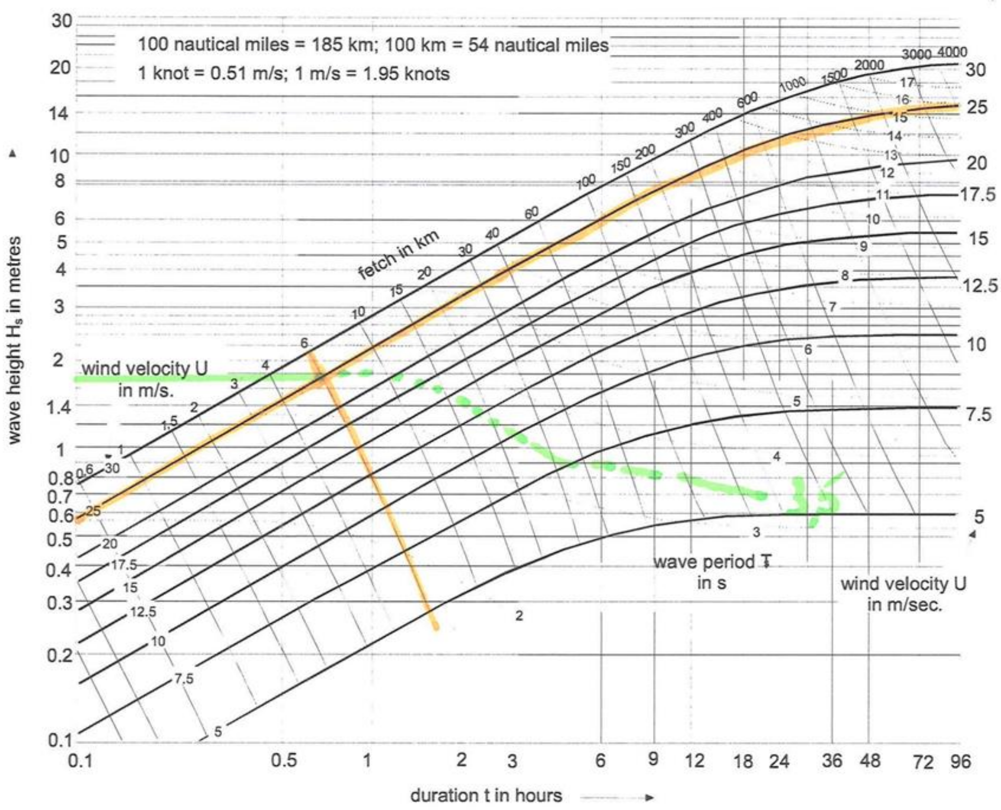


Figure 47: Nomogram of 'Groen and Dorrenstein'

To obtain the length of this wave the formulas of table 14-1 of the Manual Hydraulic structures. First an estimation of the wave length for deep water is made.

$$L = \frac{gT}{2\pi} = \frac{9{,}81 \cdot 3{,}5}{2\pi} = 19{,}12\ m$$

By the use of the formula for the water transition of deep to shallow water, it can be checked if the length is correct.

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

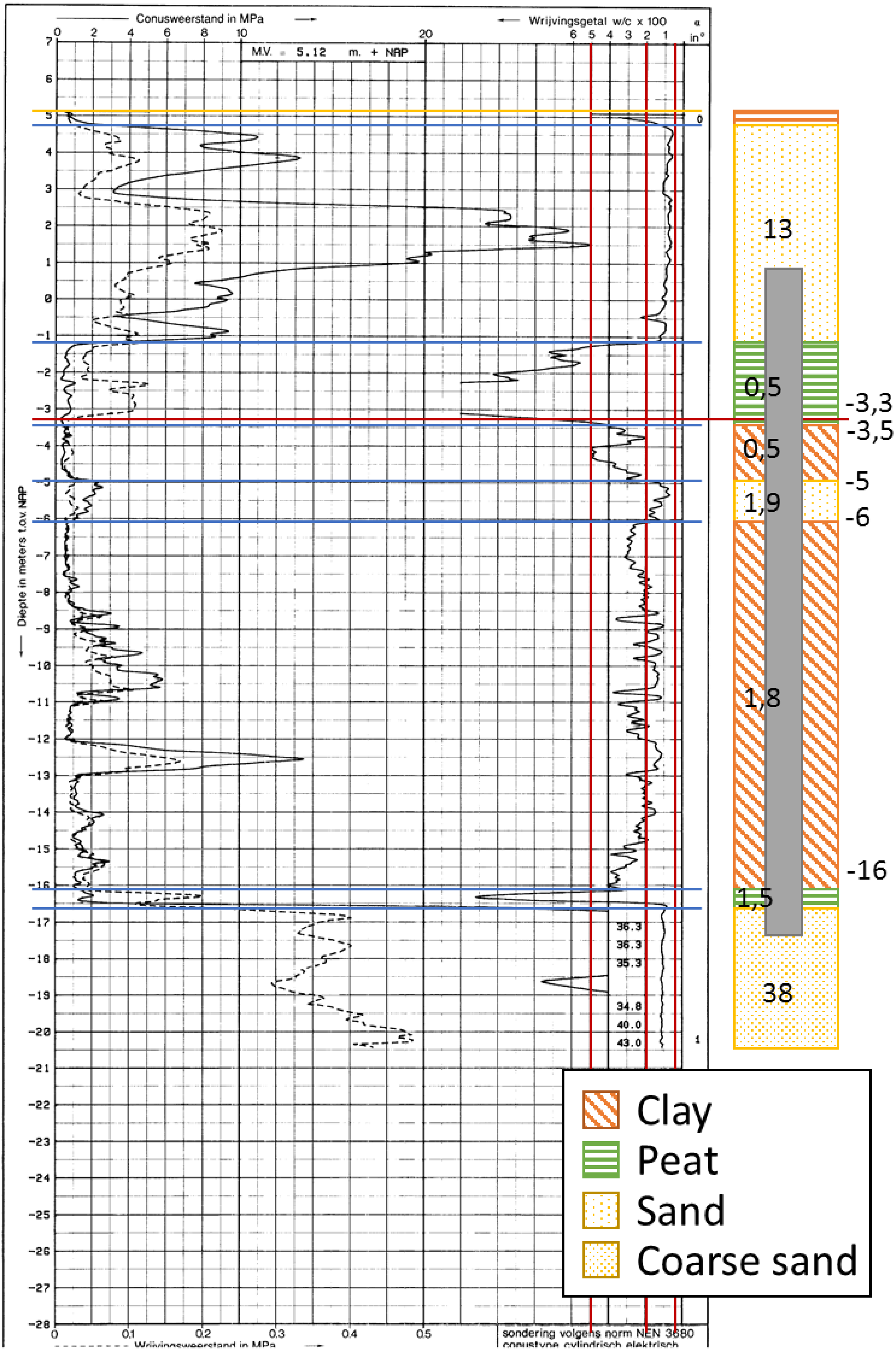
By iterating this formula a length of 12,96 m can be found. At last a check is done in order to see whether this wave is really situated in a transition area. If so the following conditions have to be met:

$$\frac{1}{20} < \frac{h}{L} < \frac{1}{2}$$

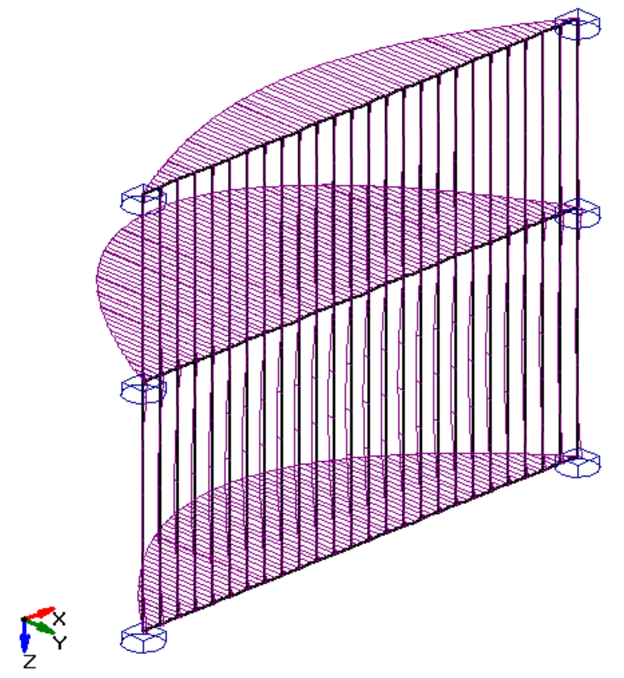
$$0{,}05 < \frac{1{,}7}{12{,}96} = 0{,}13 < 0{,}5$$

Conditions are met.

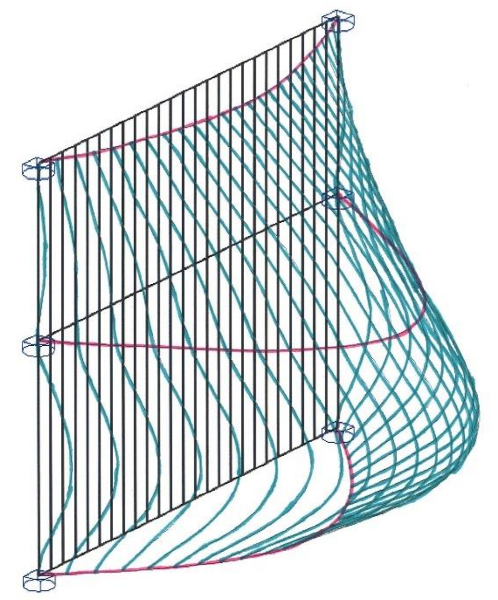
Appendix 17: Cone penetration test near "Loskade"



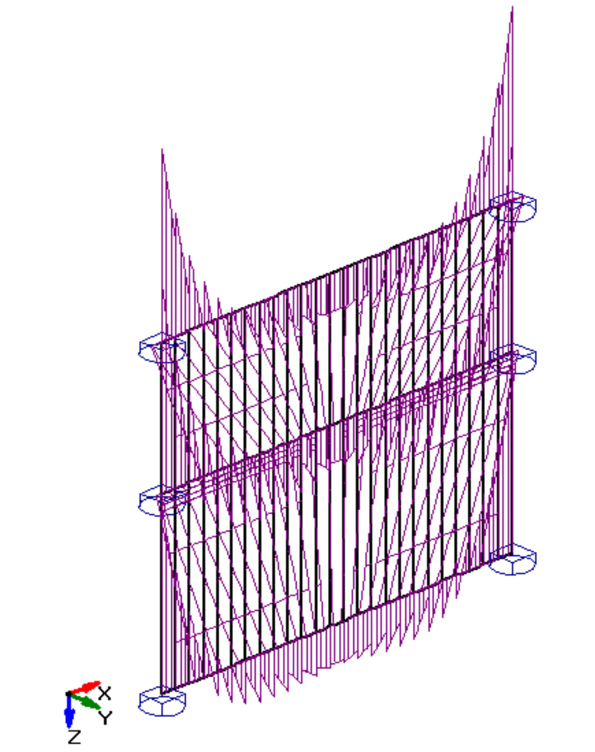
Appendix 18: Calculations for wooden mole design [j]



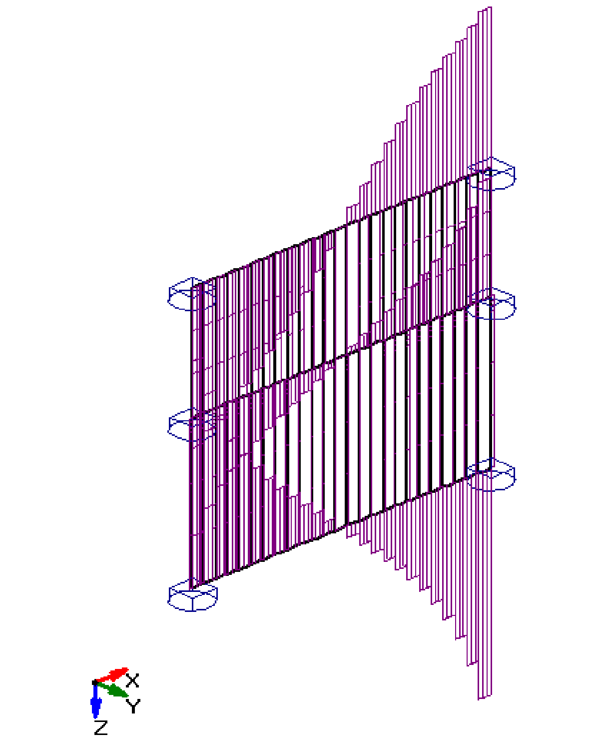
Moment distribution around Z-axis



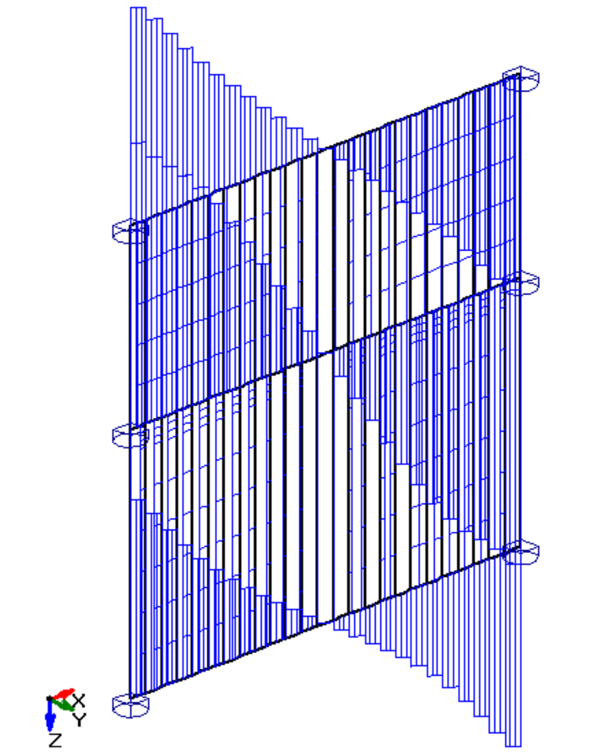
Deformation of the construction



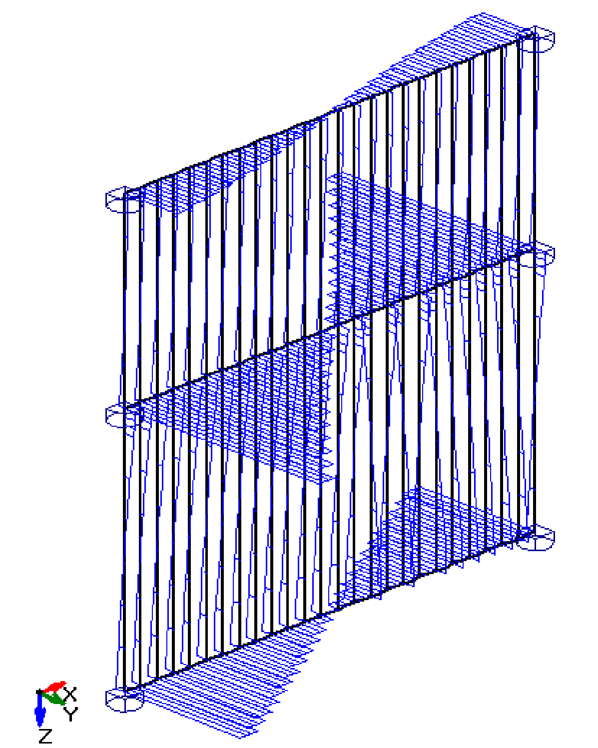
Moment distribution around Y -axis



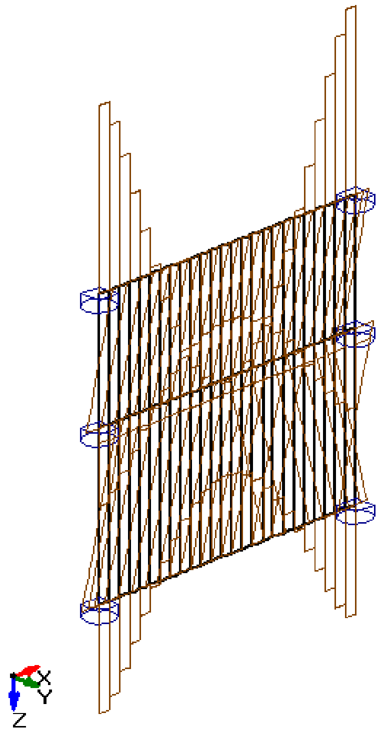
Moment distribution around X-axis



Shear force distribution in Z-direction



Shear force distribution in Y-direction



Normal force distribution

After determining the occurring internal forces the occurring maximum values were determined, which were then applied for the various checks

Normal forces

When checking this the remittance of the pulling force as well as of the pressure force were taken into account. These forces were checked as follows :

- Pressure force

$$\sigma_{c;0;d} = \frac{N_c \cdot \gamma_b}{A} \leq k_c \cdot f_{c;0;d} \; of \; \frac{\sigma_{c;0;d}}{k_c \cdot f_{c;0;d}} \leq 1$$

Taking into account:

- Breakpoint factor: $k_c = \frac{1}{k+\sqrt{k^2+\lambda_{rel}^2}}$, which consists of the following factors :
- $k = 0{,}5 \cdot (1 + \beta_c \cdot (\lambda_{rel} - 0{,}3) + \lambda_{rel}^2)$
- $\beta_c = 0{,}2$
- $\lambda_{rel} = \frac{\lambda}{\pi}\sqrt{\frac{f_{c;0;k}}{E_{0;0{,}05}}}$
- $\lambda = \frac{l_k}{i} = \frac{l_k}{\sqrt{\frac{I}{A}}}$

- Tension

$$\sigma_{t;d} = \frac{N_t \cdot \gamma_b}{A} \leq f_{t;0;d}$$

Shear forces

We check shear forces using the following formula :

$$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$$

In this formula's right side the shear strength of the wood is still multiplied by a modification factor k_{mod}. This value is determined applying the climate class, which is, in this case, class 3 (high humidity) and a short load duration class.

Flexural strength

In order to check this flexural strength we applied the double bending formula :

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$
$$k_m\frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

Taking into account :

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_y};\ \sigma_{m,z,d} = \frac{M_{z,d}}{W_z};\ f_{m,d} = f_{m,0,k} \cdot \frac{k_{mod}}{\gamma_m}k_h$$

In this formula the k_h factor occurs, which can be considered to be a height effect. This effect takes into account possible defects (tassels) that can exist in the wood. For sawn timber this value can be determined applying the following formula :

$$k_h = \left(\frac{150}{h}\right)^{0.2} \quad where: 1 \leq k_h \leq 1{,}3$$

Check calculations

Specification materials			
Wood Azobe D70			
Bending	fm;k	70	N/mm2
Tension into the longitudinal direction	ft;0;k	42	N/mm2
Tension into the transverse direction	ft;90;k	0,6	N/mm2
Pressure into the longitudinal direction	fc;0;k	34	N/mm2
Pressure into the transverse direction	fc;90;k	13,5	N/mm2
Mean MVE longitudinal	E0,mean	20000	N/mm2
5% MVE longitudinal	E0,05	16800	N/mm2
mean MVE transverse	E90,mean	1330	N/mm2
Mean shear modulus	Gmean	1250	N/mm2
Shear stress	fv;k	5	N/mm2
Modification factor	kmod	0,9	-
Material factor	γM	1,3	-
Initial curvature of the bars	βc	0,2	-
General			

Load factor	γb	1,35	-
Minimum height effect	kh	1	-

Beam

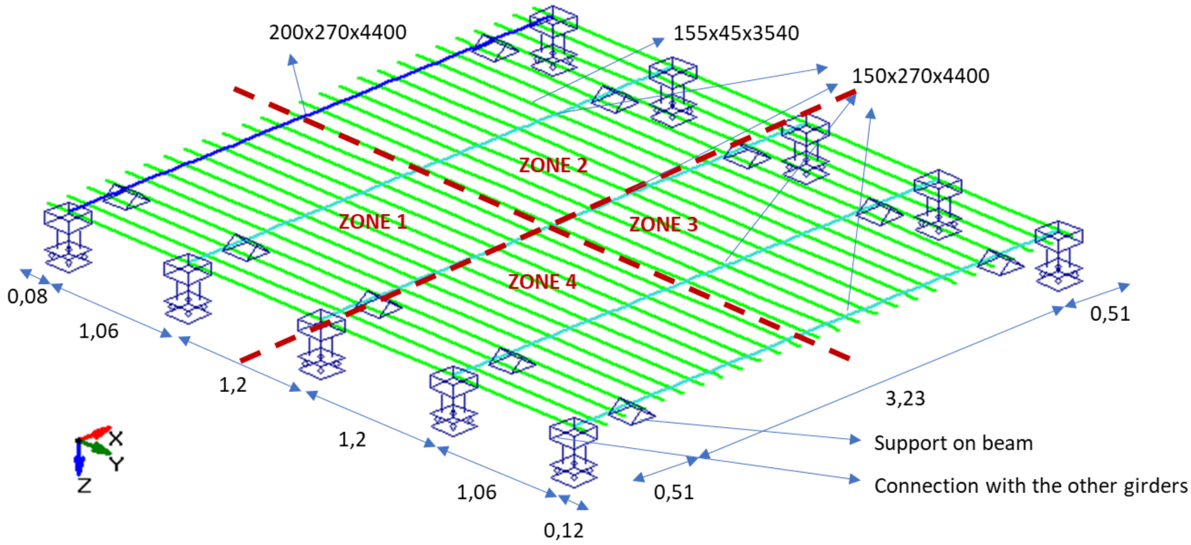
Cross beams			
Dimensions			
Length	l	4400	mm
Height	h	200	mm
Width	b	250	mm
Inertia			
	Iy, Izz	260416667	mm4
	Iz, Iyy	260416667	mm4
Normal force (biggest tension force)			
Normal force (tension)	Nd	1,60	kN
Area	A	50000,00	mm2
Tension stress	σt	0,04	N/mm2
Design stress tension	ft;0;d	0,65	mm
σt		<	ft;0;d
0,04		<	0,65
Normal force (biggest compression force)			
Normal force (compression)	Nd	1,17	kN
Area	A	50000,00	mm2
Compression stress	σc;0;d	0,03	N/mm2
Design stress	fc;0;d	26,15	mm
Radius of inertia	i	57,74	mm
Buckle length	lk	4400	mm
Slenderness	λ	76,21	-
	λrel	1,09	-
	k	1,17	-
Buckle factor (≤1)	kc	0,62	-
0,001943513		<	1,00
Deflection			
Deflection S1			
Deflection	u	27,30	mm
Shear force			
$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$			
Maximum shear force	Vd	83,48	kN

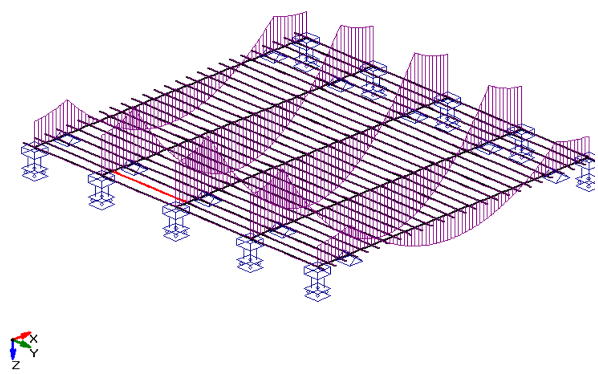
Planks of the breakwater			
Dimensions			
Length	l	3540	mm
Height	h	80	mm
Width	b	200	mm
Inertia			
	Iy, Ixx	53333333	mm4
	Ix, Iyy	8533333	mm4
Normal force (biggest tension force)			
Normal force (tension)	Nd	0,27	kN
Area	A	16000,00	mm2
Tension stress	σt	0,02	N/mm2
Design stress tension	ft;0;d	32,31	mm
σt		<	ft;0;d
0,02		<	32,31
Normal force (biggest compression force)			
Normal force (compression)	Nd	0,28	kN
Area	A	16000,00	mm2
Compression stress	σc;0;d	0,02	N/mm2
Design stress	fc;0;d	26,15	mm
Radius of inertia	i	23,09	mm
Buckle length	lk	1598,06	mm
Slenderness	λ	69,20	-
	λrel	0,99	-
	k	1,06	-
Buckle factor (≤1)	kc	0,70	-
0,001297667		<	1,00
Deflection			
Deflection S6			
Deflection	u	28,00	mm
Shear force			
$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$			
Maximum shear force	Vd	4,82	kN

Width	b	250	mm
Height	h	200	mm
Design shear stress	σv,d	2,50	N/m m2
Resistance shear stress	fv,k	5	N/m m2
2,50	≤		3,46
Bending strength			
Max moment around Y-axis	My	0,36	kNm
Max moment around Z-axis	Mz	85,44	kNm
Moment of resistance around Y-axis	Wy	1666667	mm3
Moment of resistance around Z-axis	Wz	2083333	mm3
Bending stress around Y-axis	σm,y,d	0,22	N/m m2
Bending stress around Z-axis	σm,z,d	41,01	N/m m2
Redistribution stress factor	km	0,7	-
Height effect around Y-axis	khy	0,90	-
Height effect around Z-axis	khz	0,94	-
Bending resistance om Y-axis	fm,y,d	48,46	N/m m2
Bending resistance om Z-axis	fm,z,d	48,46	N/m m2
0,60	<		1
0,85	<		1

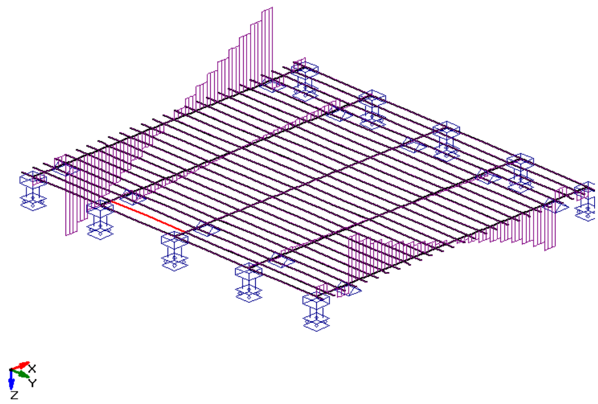
Width	b	200	mm
Height	h	80	mm
Design shear stress	σv,d	0,45	N/m m2
Resistance shear stress	fv,k	5	N/m m2
0,45	≤		3,46
Bending strength			
Max moment around Y-axis	My	1,49	kNm
Max moment around x-axis	Mx	9,80	kNm
Moment of resistance around Y-axis	Wy	533333	mm3
Moment of resistance around x-axis	Wx	213333	mm3
Bending stress around Y-axis	σm,y,d	2,78	N/m m2
Bending stress around x-axis	σm,x,d	45,94	N/m m3
Redistribution stress factor	km	0,70	-
Height effect around Y-axis	khy	1,13	-
Height effect around x-axis	khx	0,94	-
Bending resistance om Y-axis	fm,y,d	54,95	N/m m2
Bending resistance om x-axis	fm,x,d	48,46	N/m m2
0,71	<		1
0,00	<		1

Appendix 19: Determining the dimensions of the wooden promenade boulevard [j]

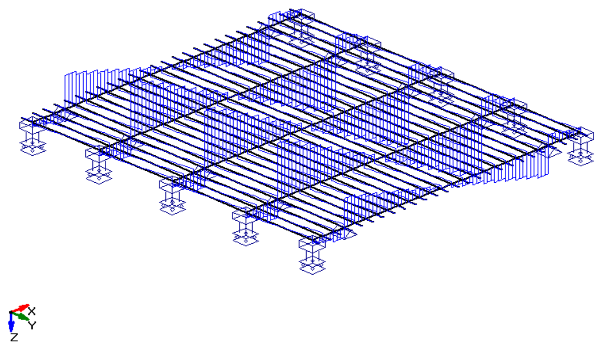




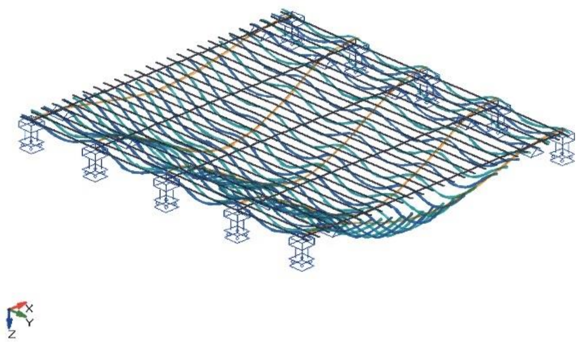
Moment distribution around Y-as



Moment distribution around X-as



Shearforce distribution in X-direction



Deformation of the construction

To check if the construction can bear the loads, the same checks are performed as was done for the wave breaker. For the girders an extra check for the kip stability has been performed. This check is done by checking if the maximum kip stability is bigger than the critical kip moment. This was done with the following formula :

$$M_{crit} = \frac{\pi}{l_{eff}} \sqrt{\frac{EI_z GI_t}{1 - \frac{I_z}{I_y}}}$$

Since the cross section is rectangular this formula can be reduced to:

$$M_{crit} = \frac{\pi}{6l_{eff}} \sqrt{EG} \cdot hb^3 \sqrt{\frac{1 - 0.63\frac{b}{h}}{1 - \frac{b^2}{h^2}}}$$

Girders			
Dimensions			
Length	l	4400	mm
Height	h	270	mm
Width	b	150	mm
Inertia			
	Iy, Ixx	246037500	mm4
	Ix, Iyy	75937500	mm4
Deflection			
Deflection S1			
Deflection z-direction	uz	7,5	mm
Shear force			
$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$			
Max Shear force	Vd	90,65	kN
Width	w	150	mm
Height	h	270	mm
Design shear stress	σv,d	3,36	N/mm2
Resistance shear stress	fv,k	5	N/mm2
3,36	≤		3,46
Bending stress			
Max moment around Y-axis	My	11,23	kNm
Max moment around X-axis	Mx	0,18	kNm
Moment of resistance around Y-axis	Wy	1822500	mm3
Moment of resistance around X-axis	Wx	1012500	mm3
Bending stress around Y-axis	σm,y,d	6,16	N/mm2

Girders

Planks			
Dimensions			
Length	l	3540	mm
Height	h	45	mm
Width	b	155	mm
Inertia			
	Iy, Ixx	13964531	mm4
	Ix, Iyy	1177031	mm4
Deflection			
Deflection S6			
Deflection z-direction	uz	9,75	mm
Shear force			
$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$			
Max Shear force	Vd	5,10	kN
Width	b	155	mm
Height	h	45	mm
Design shear stress	σv,d	1,10	N/mm2
Resistance shear stress	fv,k	5	N/mm2
1,10	≤		3,46
Bending stress			
Max moment around Y-axis	My	1,04	kNm
Max moment around X-axis	Mx	0,00	kNm
Moment of resistance around Y-axis	Wy	180188	mm3
Moment of resistance around X-axis	Wx	52313	mm3
Bending stress around Y-axis	σm,y,d	5,77	N/mm2

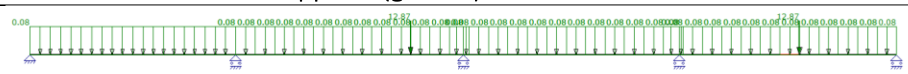
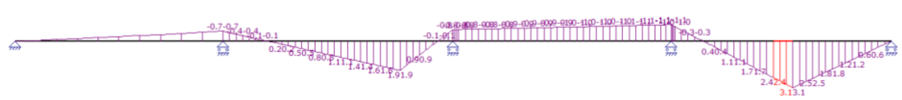
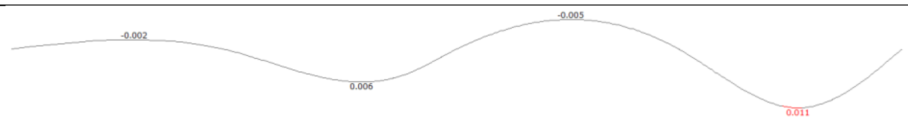
Bending stress around X-axis	σm, x,d	0,18	N/m m2
Redistribution stress factor	km	0,7	-
Height effect around Y-axis	khy	1,00	-
Height effect around X-axis	khx	0,89	-
Bending resistance om Y-axis	fm,y ,d	48,46	N/m m2
Bending resistance om X-axis	fm,x ,d	48,46	N/m m2
0,13	<		1
0,09	<		1
Kip stability			
Effective length	lef	1,625	m
critical kip moment	Mcri t	3,7E+1 1	Nm m
		367087	kNm

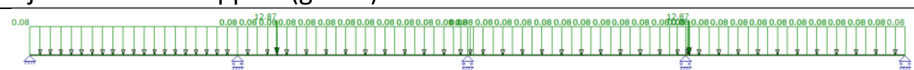
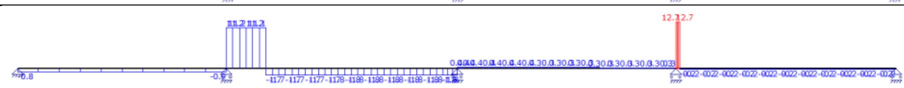
Bending stress around X-axis	σm, x,d	0,00	N/m m3
Redistribution stress factor	km	0,70	-
Height effect around Y-axis	khy	1,27	-
Height effect around X-axis	khx	0,99	-
Bending resistance om Y-axis	fm,y ,d	61,66	N/m m2
Bending resistance om X-axis	fm,x ,d	48,46	N/m m2
0,09	<		1
0,07	<		1

As a second check the different components of the pier were exposed to the point loads of the tires of a small delivery van. This load corresponds to a point load:

$$F_{Delivery\ van} = 3{,}5ton \cdot 9{,}81\frac{m}{s^2} = 34{,}34\ kN$$

Which means a load of 8,58 kN per tire. The loads of these tires were positioned at different locations on the planks, to see at which situation the maximum internal forces would occur. So there are two different situations were maximums occur. In the situation where the tire situates itself in the middle of two girders, the maximum moment and deflection of the plank occurs. The largest shear force can be found in the situation were the tire is situated just next to the support (girder) of the plank. The different force distributions for these situations can be found in the following figures.

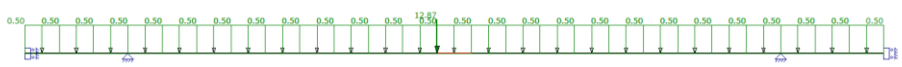
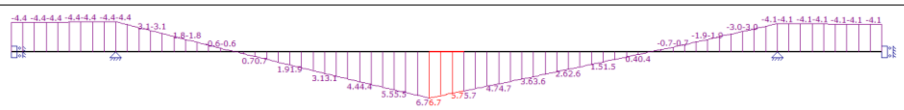
One of the tires located in the middle of two supports (girders)	
Load	
Moment distribution	
Deflection	

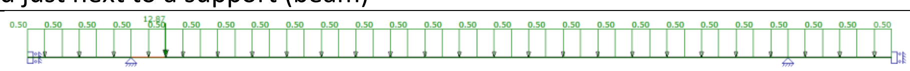
One of the tires located just next to a support (girder)	
Load	
Shear force distribution	

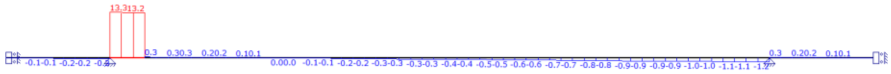
The checks that are displayed underneath it was clear that the planks need a thickness of 5 cm.

Planks			
Dimensions			
Length	l	3540	mm
Height	h	50	mm
Width	b	155	mm
Inertia			
	Iy, Ixx	1614583	mm4
Deflection			
Deflection S6			
Deflection z-direction	uz	10,60	mm
Shearforce			
$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$			
Max Shear force	Vd	12,73	kN
Width	b	155	mm
Height	h	50	mm
Design shear stress	σv,d	2,46	N/mm2
Resistance shear stress	fv,k	5	N/mm2
2,46	≤		3,46
Bending stress			
Max moment around Y-axis	My	3,08	kNm
Moment of resistance around Y-axis	Wy	64583	mm3
Bending stress around Y-axis	σm,y,d	47,69	N/mm2
Height effect around Y-axis	khy	1,25	-
Bending resistance om Y-axis	fm,y,d	48,46	N/mm2
0,98	<		1

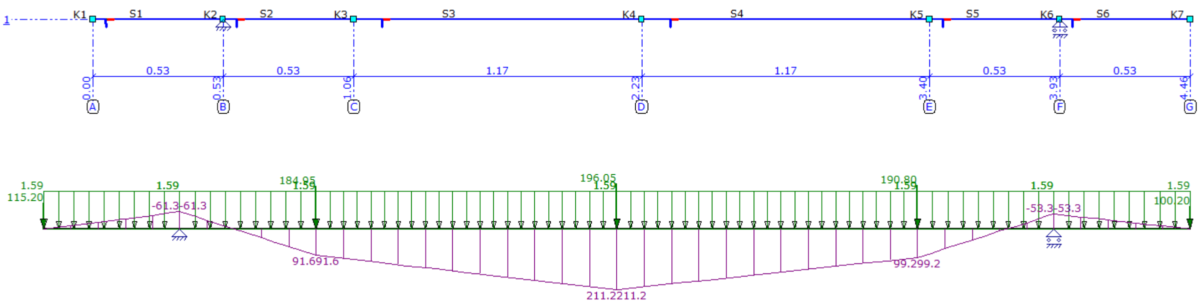
If now the same method is used for the checks of the girder. The same conclusions can be made as with the planks. However, the deflection of the component is minimal.

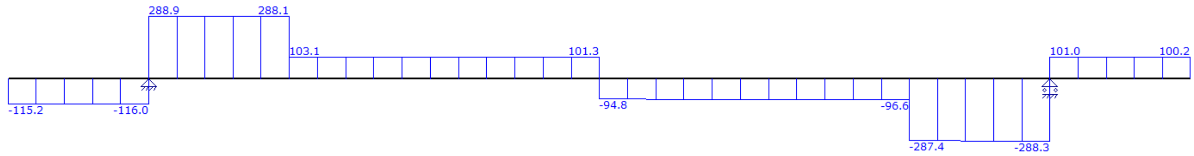
One of the tires located in the middle of the girders	
Load	
Moment Distribution	

One of the tires located just next to a support (beam)	
Load	

Shear force distribution			
Girder			
Dimensions			
Length	l	4400	mm
Height	h	270	mm
Width	b	150	mm
Inertia			
	Iy, Ixx	246037500	mm4
Deflection			
Deflection S6			
Deflection z-direction	uz	1,00	mm
Shear force			
$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$			
Max Shear force	Vd	13,30	kN
Width	b	50	mm
Height	h	155	mm
Design shear stress	σv,d	2,57	N/mm2
Resistance shear stress	fv,k	5	N/mm2
2,57	≤		3,46
Bending stress			
Max moment around Y-axis	My	6,73	kNm
Moment of resistance around Y-axis	Wy	1822500	mm3
Bending stress around Y-axis	σm,y,d	3,69	N/mm2
Height effect around Y-axis	khy	0,89	-
Bending resistance om Y-axis	fm,y,d	43,09	N/mm2
0,09	<		1

The last wooden component which the dimension was determined of is the beam that transfers the load from the girder to the foundation pile. For this calculation the forces on the supports out of the 3D model was used as point loads on the beam. Which gives:





Beam			
Dimensions			
Length	l	4460	mm
Height	h	400	mm
Width	b	320	mm
Inertia			
	Iy, Ixx	1706666667	mm4
Deflection			
Deflection S6			
Deflection z-direction	uz	7,60	mm
Shear force			
$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$			
Max Shear force	Vd	288,90	kN
Width	b	320	mm
Height	h	400	mm
Design shear stress	σv,d	3,39	N/mm2
Resistance shear stress	fv,k	5	N/mm2
3,39	≤		3,461538
Bending stress			
Max moment around Y-axis	My	211,20	kNm
Moment of resistance around Y-axis	Wy	8533333	mm3
Bending stress around Y-axis	σm,y,d	24,75	N/mm2
Height effect around Y-axis	khy	0,82	-
Bending resistance om Y-axis	fm,y,d	39,83	N/mm2
0,62	<		1

Appendix 20: Determining the dimensions of the pile foundation [f]

The total bearing capacity of the pile foundation can be determined by the following formula:

$$F_{r;max} = F_{r;max;tip} + F_{r;max;shaft} - F_{s;nk;rep} - W$$

This formula exists of the following terms from left to right: Maximum tip resistance, max shaft resistance, negative shaft resistance and the dead weight of the pile. A short explanation of the different terms can be find below:

- Maximum tip resistance

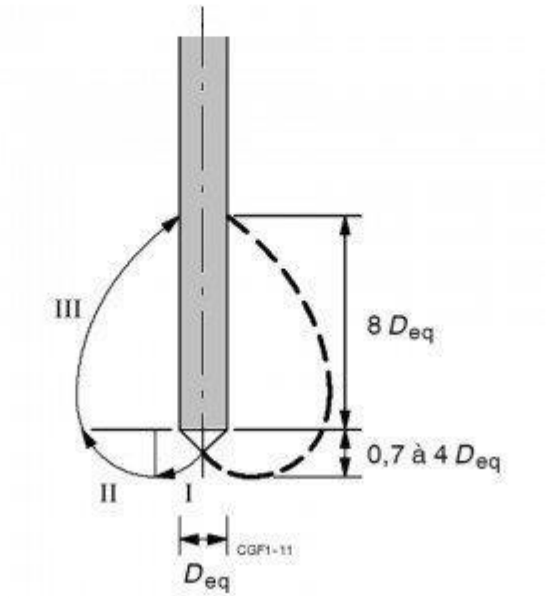


Figure 48: Influence of the tip resistance area [23]

The maximum tip resistance can be found by multiplying the maximum pressure the tip and ground can bear at the area of the tip.

$$F_{r;max;tip} = A_{tip} \cdot p_{r;max;tip}$$

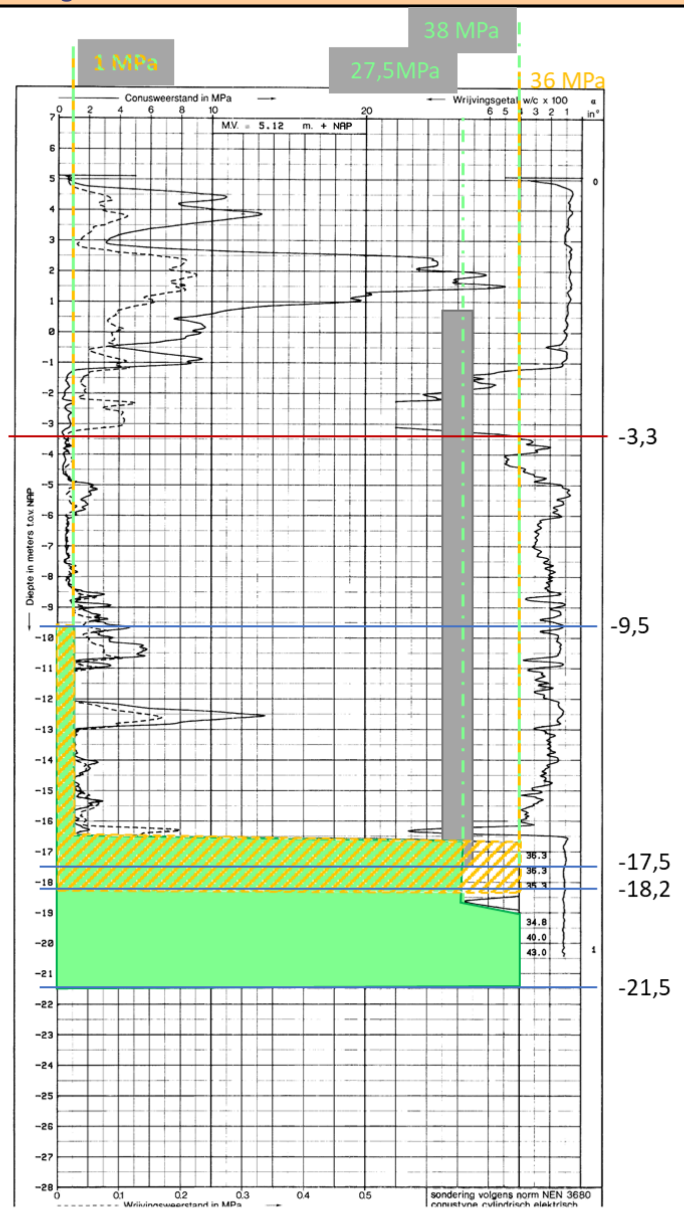
The maximum pressure the soil around the tip can bear can be determined with the method of Koppejan. This method makes use of the slip planes around the pile tip.
Using the following formula the resistance can be determined:

$$p_{r;max;tip} = \frac{1}{2}\alpha_p \beta s \left(\frac{q_{c;I;mean} + q_{c;II;mean}}{2} + q_{c;III;mean}\right)$$

As can be seen in the formula as well as in figure 48 the slip plane is split up in 3 parts. With the help of the diagram resulting from a cone penetration test, the different mean values of the slip resistance can be estimated. Next to these values some reduction factors are also present in the formula. One reduction factor is α_p which stands for the pile class factor. As the surface of the steel tube is smooth this factor is equal to 1,0. Next to this factor there is also β en s which both can be related by the shape or cross section of the foot of the pile. For a tube pile both these factors are equal to 1,0.

This gives the following results:

General values of the steel tube piles			
Pile class factor	αp	1	-
Reduction factor shape of the foot of the pile	β	1	-
Reduction factor shape of the cross-section of the foot of the pile	s	1	-
Mean pile diameter	Ø	1000	mm
Steel thickness	t	10	mm
Shaft friction factor	αs	0,0075	-
Length of the pile that is under water		17	m
Length of the pile that is above water		3,11	m
Pile length	L	20,11	m
Specific weight of steel S253	γb	77,0085	kN/m3
Material pile factor	γm,g	1,1	-
Specific weight of water	γw	10,05525	kN/m3



Maximum tip resistance			
Minimum depth influence	d	700	mm
Maximum depth influence	d	4000	mm
Pile distance influence	dIII	8000	mm
Minimum depth influence			
	qc,I,gem, min	36	Mpa
	qc,II, gem	35	MPa
	qc,III,gem	5,41	MPa
Maximum tip resistance	pr,max,punt	20,46	MPa
Maximum depth influence			
	qc,I,gem, max	38	Mpa
	qc,II, gem	37,07	MPa
	qc,III,gem	4,31	MPa
Maximum tip resistance	pr,max,punt	20,92	MPa
Maximum tip force			
Maximum tip force	Fr,max,punt	7033,32	kN

- Maximum shaft resistance
  The resistance which the shaft exerts on the ground can be determined with the help of the following formula:

$$F_{r;max;shaft} = O_{p;mean} \int_0^{\Delta L} p_{r;max;shaft} dz$$

$$p_{r;max;shaft} = \alpha_s q_c$$

For this calculation the mean conus resistance of the layers which has positive shaft friction was used.

Maximum shaft resistance			
Conus resistance of sand	qc1	38	Mpa
Layer thickness	ΔL1	0,8	m
Conus resistance of clay	qc2	1,8	Mpa
Layer thickness	ΔL2	10	m
Conus resistance of sand	qc3	1,9	Mpa
Layer thickness	ΔL3	1	m
Conus resistance of clay	qc4	0,5	Mpa
Layer thickness	ΔL4	1,5	m

Maximum shaft resistance	pr,max,shaft	0,285	Mpa
Maximum shaft force	Fr,max,shaft	716,283125	kN

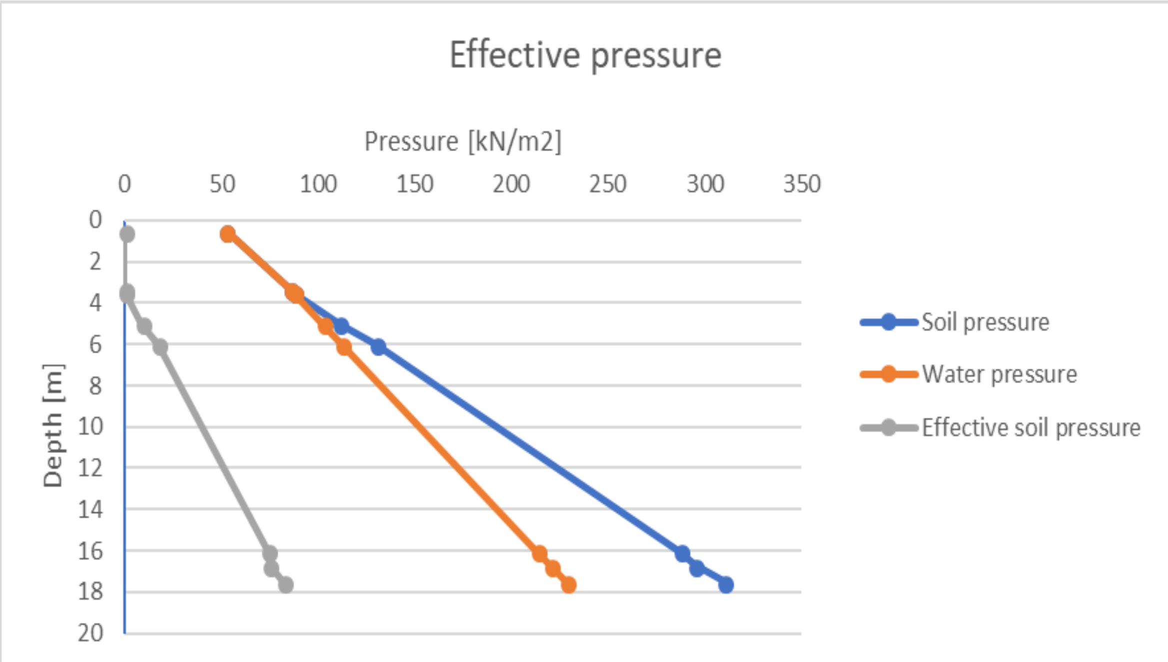
- Negative shaft friction
  Negative shaft friction originates because the curtain layer starts to consolidated after some amount of time. The effect of this is that these layers start to pull on the pile which means that the friction coefficient starts to work in the other direction. This effect is mostly caused by clay and peat. The range of this negative effect can be calculated with the following formula:

$$F_{s;nk} = O_s h K_0 \sigma_v' \tan \delta$$

  In this formula the following components are present:
    - Circumference of the pile O_s
    - Height of the layer h
    - Ground pressure coefficient K_0
    - Mean effective vertical soil pressure σ_v'
    - Friction angle between the ground and the pile δ

Negative shaft friction			
Specific weight of peat	γv	12	kN/m3
Specific weight of sand	γz	21	kN/m3
Specific weight of clay	γk	14	kN/m3
Specific weight of water	γw	10,05525	kN/m3
Pressures			
Soil pressure	0,5	0	kN/m2
Water pressure		0	kN/m2
Effective soil pressure		0	kN/m2
Soil pressure	3,3	28,1547	kN/m2
Water pressure		28,1547	kN/m2
Effective soil pressure		0	kN/m2
Soil pressure	3,5	30,5547	kN/m2
Water pressure		30,16575	kN/m2
Effective soil pressure		0,38895	kN/m2
Soil pressure	5	51,5547	kN/m2
Water pressure		45,248625	kN/m2
Effective soil pressure		6,306075	kN/m2
Soil pressure	6	72,5547	kN/m2

Water pressure		55,303875	kN/m2
Effective soil pressure		17,250825	kN/m2
Soil pressure	16	212,5547	kN/m2
Water pressure		155,856375	kN/m2
Effective soil pressure		56,698325	kN/m2
Soil pressure	16,7	220,9547	kN/m2
Water pressure		162,89505	kN/m2
Effective soil pressure		58,05965	kN/m2
Soil pressure	17,5	237,7547	kN/m2
Water pressure		170,93925	kN/m2
Effective soil pressure		66,81545	kN/m2



Angle of friction between pile and soil			
Sand	δz	30	°
Peat	δv	32	°
Clay	δk	22	°
Neutral soil pressure coefficient			
Sand	K0	0,5	-
Peat	K0	0,47	-
Clay	K0	0,63	-
Friction force of negative shaft resistance			

- Dead weight of the pile
  The weight of the pile can be calculated with the next formula:

$$W_{pile,d} = (r_{outer}^2 - r_{inner}^2) \cdot \pi \cdot h \cdot \gamma'_{pile,d} \; with \; \cdot \gamma'_{pile,d} = \frac{\gamma_b}{\gamma_{m,g}} - \gamma_w$$

Self-weight of the pile			
Volume of pile below water level	Vpile,d	0,51317916	m3
Volume of pile above water level	Vpile,d	0,081175613	
Effective specific weight	γ'pile,d	59,95247727	kN/m3
Specific weight	γpile,d	77,0085	kN/m3
Weight of the pile	Wpile,d	37,01757409	kN

In total this results in a bearing force of 7322 kN that the pile can transfer to the ground. From the design of the wave breaker and the mole it turns out that the pile has to bear a load of 795 kN. This is almost a tenth less than the bearing capacity of the pile. Which means that this will not be a problem.

Appendix 21: Dimensions river of cruise ships

Name	Photo	Length [m]	Width [m]	Draught [m]
Salvinia		91,5	10	1,5
Rembrandt van Rijn		110	10,5	1,4
Antonio Bellucci		110	11,4	1,6

Azolla		76	8	1,3
Da Vinci		105	11,4	1,4

Appendix 22: Head-on collision of river cruise ships with the main harbour mole [f]

The mole would be hit the hardest in case of a head-on collision by a river cruise ship. In this situation the construction has to be able to transfer the energy transferred by the collision to the different components of the system. First it is determined what amount of kinetic energy the ship has before it hits the construction.

$$E_{kin} = \frac{1}{2} m_s v_s^2 C_H C_E C_S C_C$$

With:

- C_H hydrodynamic coefficient $= \frac{m_s + m_w}{m_s}$
- C_E eccentricity coefficient
- C_S Softness coefficient
- C_C Configuration coefficient
- m_s Mass of the ship
- v_s Velocity of the ship

For a head-on collision the above formula can be reduced to:

$$E_{kin} = \frac{1}{2} m_s v_s^2 C_H$$

The hydrodynamic coefficient is the ratio between the mass of the ship and the mass of the water that moves along with the ship. This last aspect can be determined with the following formula:

$$m_w = \rho L \frac{1}{4} \pi D^2$$

With:

- ρ Mass density of salt water
- L Length of the ship
- D Draught of the ship

An estimation of the mass of the largest river cruise ship that can enter the Grevelingen lake was made since these data were missing. This estimation was done with the help of the largest ship that now makes use of the wharf.

Specifications for the ship Isabel				Specifications for the ship Antonio Bellucci			
Width	bs	8	[m]	Width	bs	11,4	[m]
Length	ls	31,38	[m]	Length	ls	110	[m]
Draught	ds	1,2	[m]	Draught	ds	1,6	[m]
Weight	ms	175	[tons]	Weight	ms	1165,551	[tons]
Average speed	vs	6,3	[knots]	Average speed	vs	6,3	[knots]
		11,6676	[km/h]			11,6676	[km/h]
		3,241	[m/s2]			3,241	[m/s2]

hydrodynamic coefficient			
Ship's weight	ms	1165551	[kg]
Water weight	mw	226653,1	[kg]
Hydrodynamic coefficient	CH	1,19446	[-]
Kinetic energy			
Kinetic energy	Ek	7311,914	[kN]

The next step is to determine the stiffness of the mole construction. This can be done by running through the following steps:

- Spring stiffness of one pile:

$$k_{1pile} = \frac{3EI}{L_i^3}$$

- Spring stiffness for all piles:

$$k_{all\ piles} = n \cdot k_{1pile}$$

- Polar moment of inertia of pile plan:

$$I_p = \sum k_{pile\ i}(x_i^2 + y_i^2)$$

- Spring stiffness of the mole:

$$\frac{1}{k_{mole}} = \frac{1}{k_{all\ piles}} + \frac{e^2}{I_p}$$

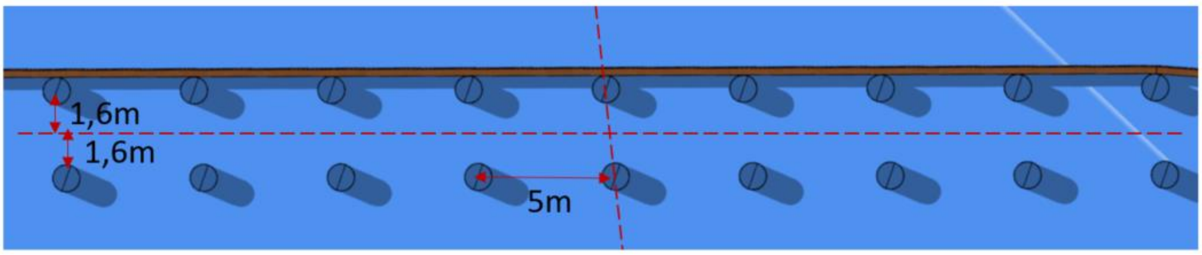
- Stiffness of the mole together with fenders:

$$\frac{1}{k_{total}} = \frac{1}{k_{mole}} + \frac{1}{k_{fender}}$$

Finally the maximum impact force can be found with:

$$F_{st} = \sqrt{2kE_{kin,max}}$$

Pile stiffness			
Fictitious pile length	li	14940	[mm]
Moment of inertia	I	3,81E+09	[mm4]
Modulus of elasticity	E	210000	[N/mm2]
Stiffness	k	1,61E+11	[N/mm]
		160694	[kN/m]
Pile stiffness			
Number of piles		18	[-]
Total stiffness	kall piles	2892492	[kN/m]
polar moment of inertia of the pile plan			
Polar moment	Ip	4,89E+08	kNm

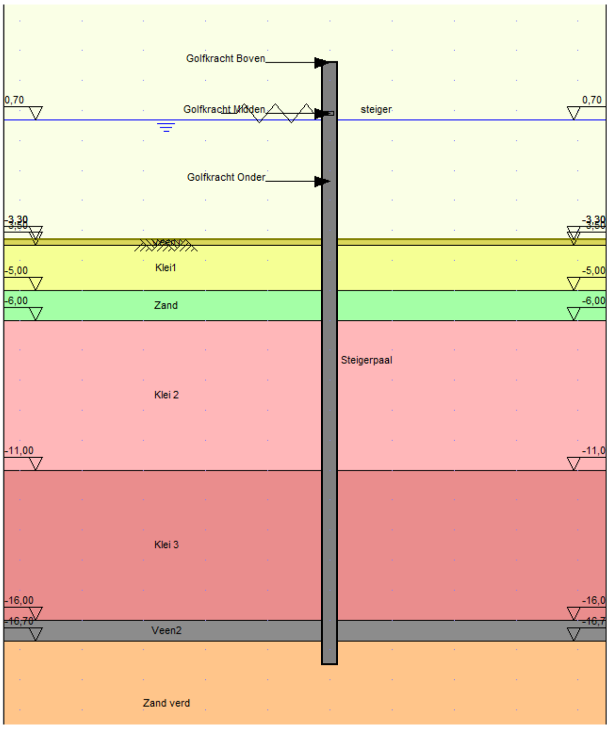


Stiffness of the mole			
		0,000456	
Stiffness of the mole	kmole	2191,345	kN/m
stiffness of the fender	kfender	66	kN/m
		0,015608	
Total stiffness	ktotal	64,0703	kN/m
Maximum impact force	Fst	967,9633	kN

Pile foundation stability check

For the stability check of the pile foundation the D-sheet piling programme created by Deltares was used.

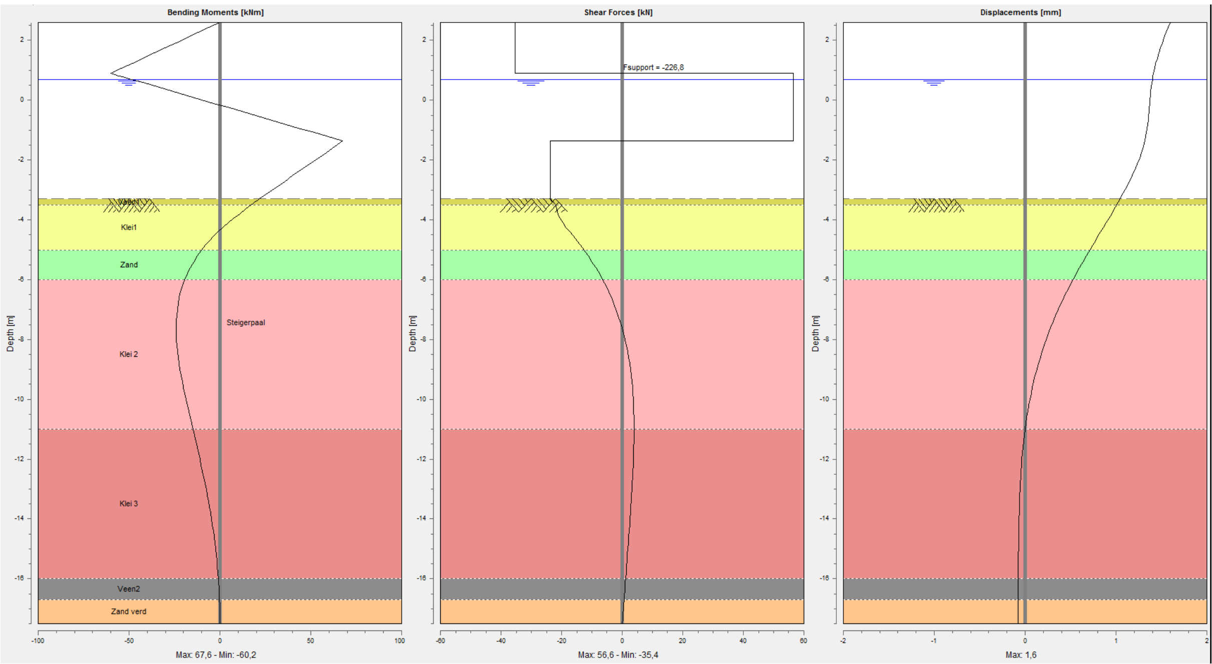
If the forces were put only on one pile without adding the effect of the other piles to it, it can be seen that the pile will have a deflection of 25 cm if only the waves during a storm have impact on the wave breaker. If the impact of a river cruise ship is added to this it can be seen that the pile becomes unstable.



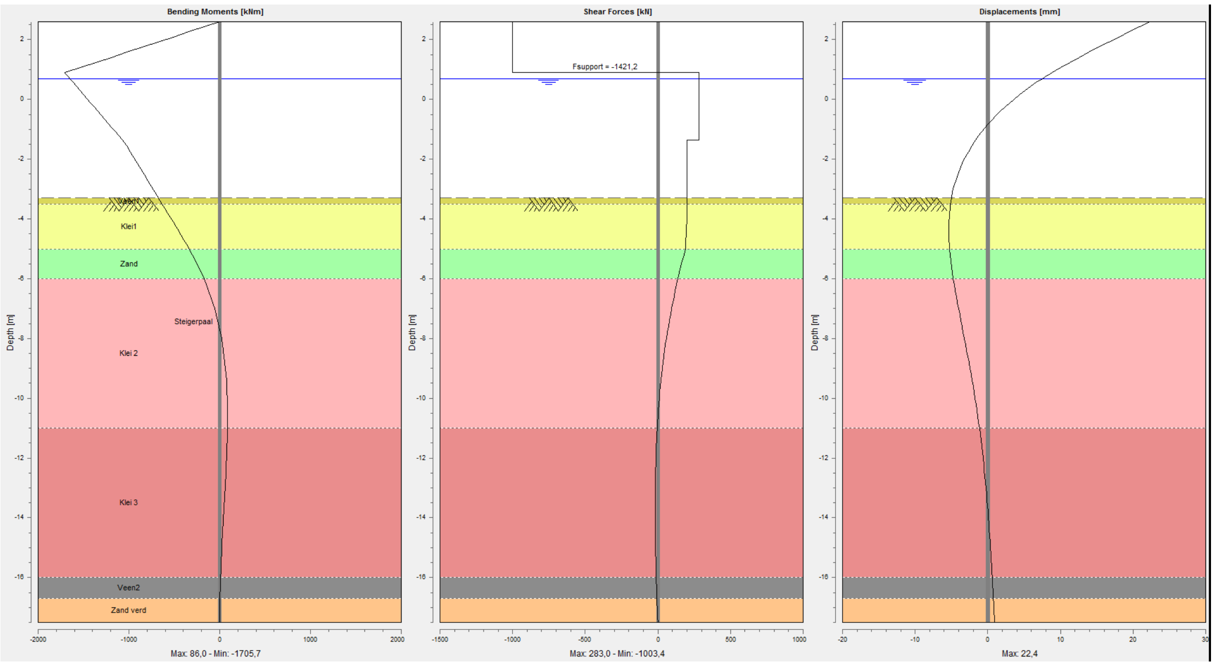
Now the same check was performed but this time with the spring stiffness of the other pile added to it. A schematic overview of this situation is displayed in the figure to the right. The specifications of the steel tube pile are as follows:

Pile top level	2,60 m NAP
Bottom level of the pile	-17,5 m NAP
Stiffness EI	8,0056*10^5 kNm2
Diameter	1 m
Characteristic moment	116329 kNm

Internal forces and deflection from wave loading during a storm.



Internal forces and deflection from wave loading during a storm and head-on collisions of a river cruise ship.



Internal bearing forces check

- Bending moment
  Check:

$$\frac{M_{ED}}{M_{c,Rd}} \leq 1{,}0$$

$$M_{c,Rd} = \frac{W_{eff,min} \cdot f_y}{\gamma_{M0}}$$

Bending moment			
Resistance moment of inertia	Wy	0,008	m3
Yield stress	fy	235	N/mm2
Material factor	jM0	1,3	-
	Mc,Rd	1791049696	Nmm
		1791,05	kNm
	Med	1705,7	kNm
0,95	<		1

- Shear stress
  Check:

$$\frac{\tau_{Ed}}{\frac{f_y}{\sqrt{3}\gamma_{M0}}} \leq 1,0$$

with:

$$\tau_{Ed} = \frac{V_{Ed} \cdot S}{I \cdot t}$$

This changes for a tube to:

$$\tau_{max} = \frac{V_{ed}}{\frac{1}{2} \cdot \pi \cdot d \cdot t - \frac{1}{4} \cdot \pi \cdot t^2}$$

Shear			
	Ved	1003,4	kN
	τED	64199,42508	kN/m2
0,473177302	<		1

- Combination of bending and shear stress
  Determining the yield strength reduction factor:

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

with:

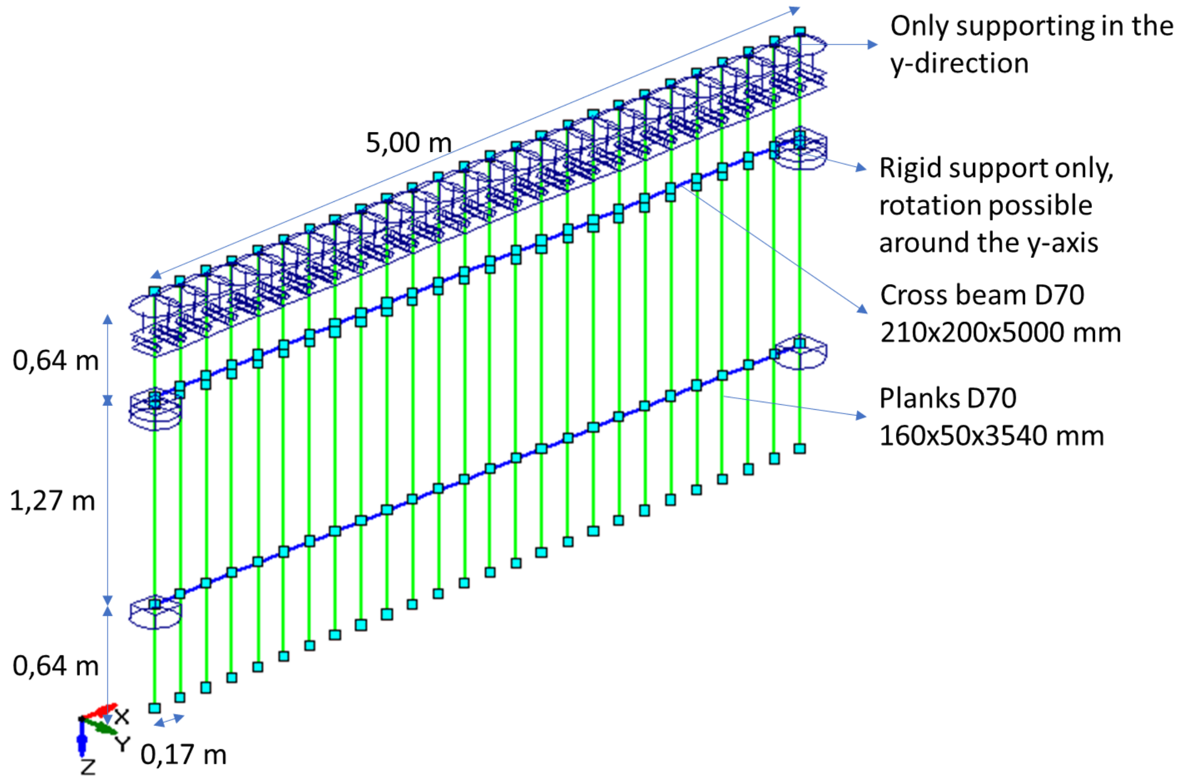
$$V_{pl,Rd} = \frac{A_v \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}}$$

New yield stress $f_{ynew} = (1 - \rho) f_y$
Recalculation of $M_{c,Rd}$

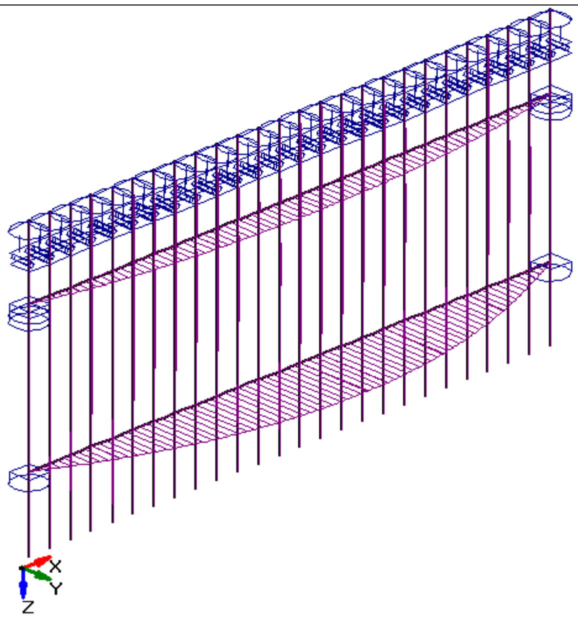
Shear + Bending moment			
	Vpl,Rd	4219804,221	N
	ρ	0,27502988	
	fynew	170,3679781	N/mm2
	Mc,rd	1298457512	Nmm
		1298,46	kNm
1,31	<		1

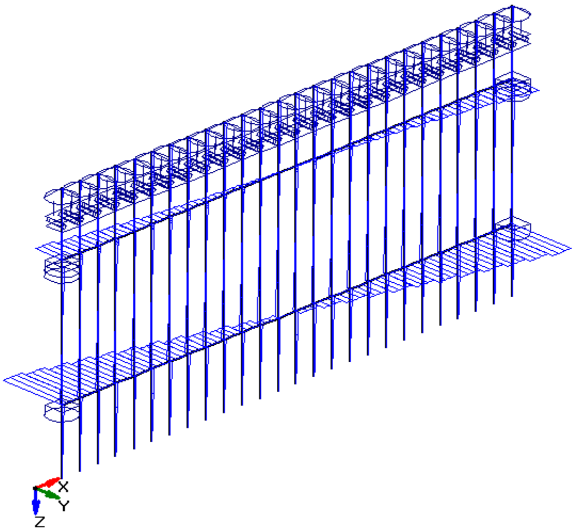
From this last check it can be concluded that the mole can’t bear the bending moment and shear force of the collision of a river cruise ship at once.

Appendix 23: Designing the wooden breakwater for the concrete design of the mole [j]

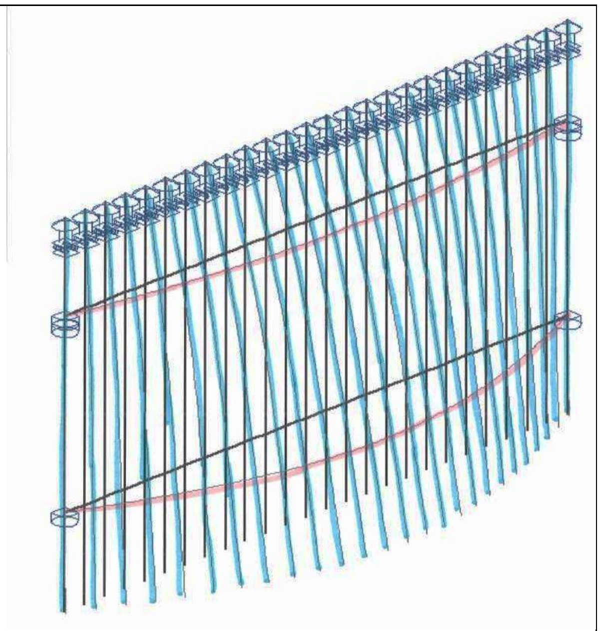


The following moment and force diagrams were obtained by the use of Matrix frame. These will be used to check whether the construction can bear the loads

 Moment distribution around the Z-axis	Moment distribution around the Y-axis



Shear force distribution in Y-direction



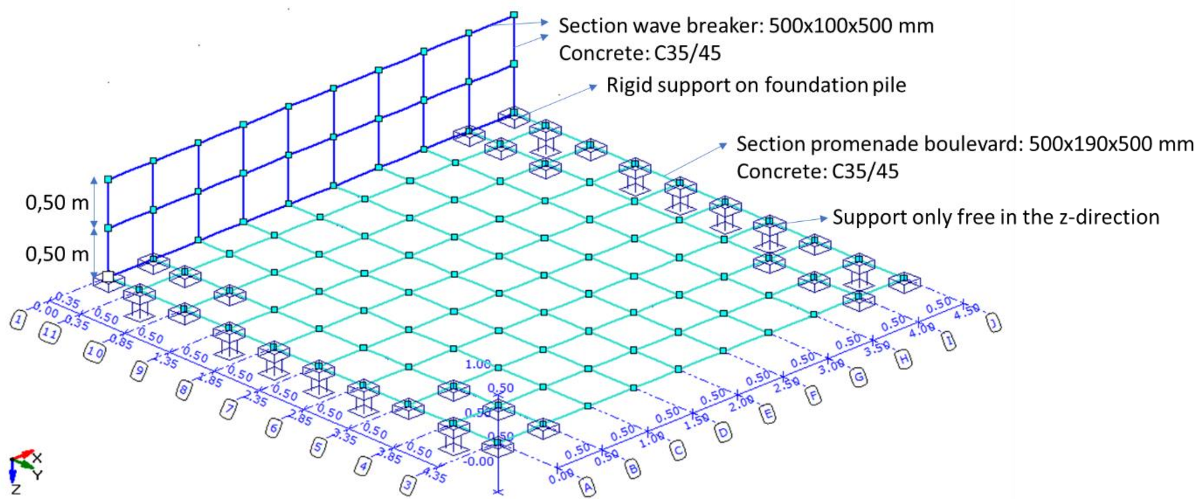
Deflection of the construction

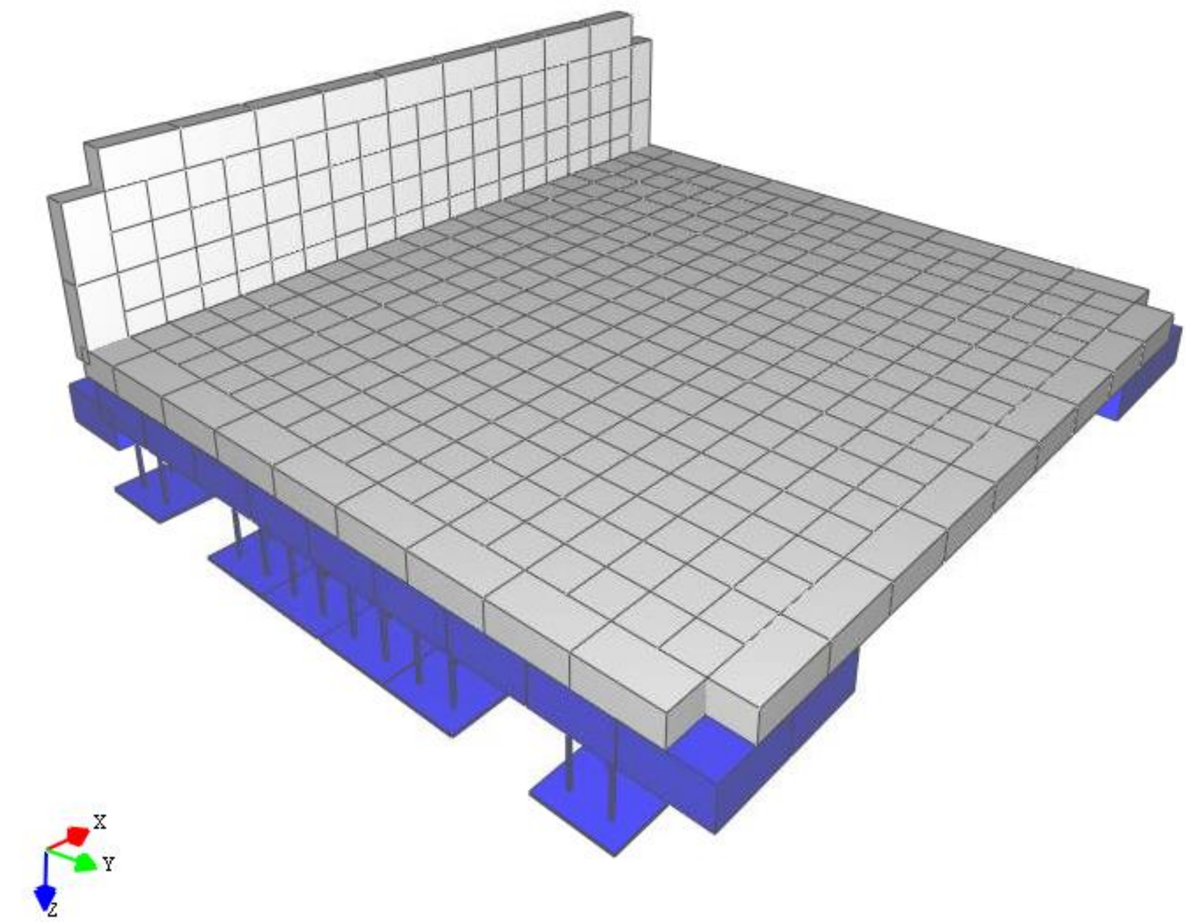
Specifications of materials			
Wood Azobé D70			
Bending	fm;k	70	N/mm2
Tension into the longitudinal direction	ft;0;k	42	N/mm2
Tension into the cross direction	ft;90;k	0,6	N/mm2
Pressure into the longitudinal direction	fc;0;k	34	N/mm2
Pressure into the cross direction	fc;90;k	13,5	N/mm2
Mean MVE longitudinal	E0,mean	20000	N/mm2
5% MVE longitudinal	E0,05	16800	N/mm2
mean MVE cross	E90,mean	1330	N/mm2
Mean shear modulus	Gmean	1250	N/mm2
Shear stress	fv;k	5	N/mm2
Modification factor	kmod	0,9	-
Material factor	γM	1,3	-
Initial curvature of the bars	βc	0,2	-
In general			
Load factor	γb	1,35	-
Minimum height effect	kh	1	-

Cross beam			
Dimensions			
Length	l	500	mm
Height	h	200	mm
Width	b	210	mm
Inertia			
	Iy, Izz	154350000	mm4
	Iz, Iyy	154350000	mm4
Normal force			
Normal force (compression)	Nd	2,12	kN
Area	A	42000	mm2
Compression stress	σc;0;d	0,07	N/mm2
Design stress	fc;0;d	26,15	mm
Radius of inertia	i	57,74	mm
Buckle length	lk	500	mm
Slenderness	λ	8,66	-
	λrel	0,12	-
	k	0,49	-
buckle factor (≤1)	kc	1,04	-
0,003		<	1,00
Deflection			
Deflection			

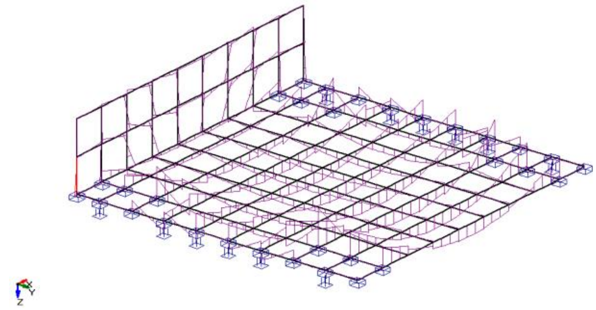
Shear force			
$$\sigma_{v,d} = \frac{3}{2}\frac{V_d}{b \cdot h} \leq f_{v,k}\frac{k_{mod}}{\gamma_M}$$			
Maximum shear force	Vd	62,36	kN
Width	b	210	mm
Height	h	200	mm
Design shear stress	σv,d	2,23	N/mm2
Resistance shear stress	fv,k	5	N/mm2
2,23	≤		3,461538462
Bending strength			
Maximum moment around Y-axis	My	0,00	kNm
Maximum moment around Z-axis	Mz	67,95	kNm
Moment of resistance around Y-axis	Wy	1400000	mm3
Moment of resistance around Z-axis	Wz	1470000	mm3
Bending stress around Y-axis	σm,y,d	0,00	N/mm2
Bending stress around Z-axis	σm,z,d	46,22	N/mm2
Redistribution stress factor	km	0,7	-
Height effect around Y-axis	khy	0,93	-
Height effect around Z-axis	khz	0,94	-
Bending resistance around Y-axis	fm,y,d	48,46	N/mm2
Bending resistance around Z-axis	fm,z,d	48,46	N/mm2
0,67	<		1
0,95	<		1

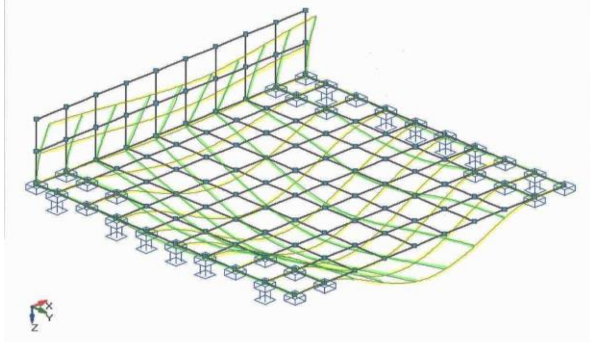
Appendix 24: Designing the concrete boulevard and wave breaker[k]





Still the same load factors as in the wooden variant are used for the calculations done for this model. With the help of Matrix Frame the following distributions were obtained.


Moment distribution around the y-axis


Deflection of the construction

The following checks were performed with the values obtained from MatrixFrame.

General values			
Concrete class		C35/45	
Characteristic compression stress	fck	35	N/mm2
Mean concrete tension stress	fctm	3,2	N/mm2
Material factor of concrete	γm	1,5	-
Design compression stress	fcd	23,33333	N/mm2
Minimal reinforcement	ρmin	0,19	%
Maximal reinforcement	ρmax	2,15	%
Steel class		B500	
Characteristic tension stress	ft;k	500	N/mm2
Characteristic yield stress	fy;k	435	N/mm2
Material factor of steel	γm	1,15	-
Design yield stress	fyd	434,7826	N/mm2

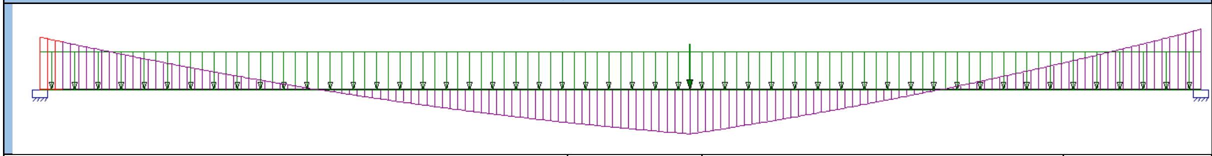
Concrete floor slab			
Dimensions			
Width	b	4500	mm
Thickness	d	190	mm
Length	l	5000	mm
Check without reinforcement			
Bending stress of concrete			
Bending stress of concrete	fctm,fl	4,512	N/mm2
	fctm,fl	>	fctm
	4,512	>	3,2
Bending stress of concrete	fctm,fl	4,512	N/mm2
Check section S193			
Width	b	500	mm
Thickness	d	190	mm
Length	l	500	mm
Bearing the design moment			
Resistance moment	Wc	3008333	mm3
Break moment	Mcr	13573600	Nmm

		13,57	kNm
Design moment	Med	30,7	kNm
Med		<	Mcr
30,7		<	13,57
Unity Check	U.C.	2,26	-
Determine the amount of reinforcement in x-direction			
2nd degree equation			
	D	294,57	
	ρ	9,91	%
	ρ	0,41	%
Reinforcement			
Reinforcement area	As	386,8991774	mm2
Diameter of the reinforcement bars	ø	16	mm
Section area of one reinforcement bar	Aw	201,0619298	mm2
Number of bars	n	2	-
Determine the number of reinforcement bars in y-directions			
Design moment	Med	17	kNm
2nd degree equation			
	D	318,06	
	ρ	10,10	%
	ρ	0,22	%
Reinforcement			
Reinforcement area	As	210,3002869	mm2
Diameter of the reinforcement bar	ø	16	mm
Area of one reinforcement bar	Aw	201,0619298	mm2
Number of bars	n	2	-

2nd degree equation					
-0,52*(fyd/fcd)^2	ρ^2	+(fyd/fcd)	ρ	-Mrd/(bd^2fcd)	=0
Floor slab in x-direction					
-180,55		18,63		-7,29E-02	
Floor slab in y-direction					
-180,55		18,63		-4,04E-02	

These checks show that 4 reinforcement bars per meter are needed with a diameter of 16 mm to bear the loads. This means that there will be a centre-to-centre distance of 0,25 m between the bars. By this configuration the construction will have a deflection of 1,1 mm.

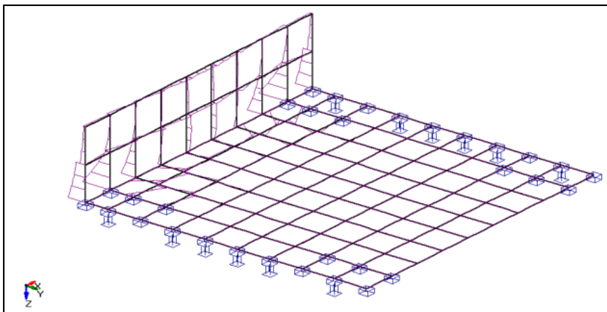
Check of the reinforcement with point load of the vans' tires in x-direction			



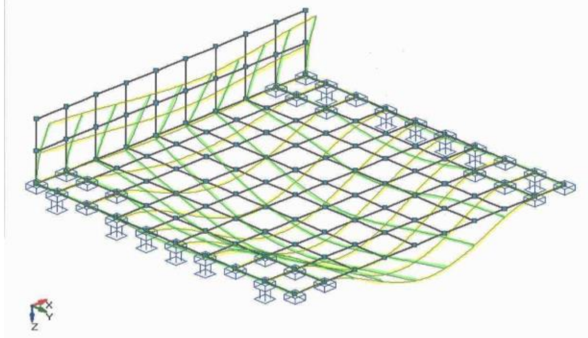
Design moment	Med	19	kNm
2nd degree equation			
	D	314,63	
	ρ	10,07	%
	ρ	0,25	%
Reinforcement area	As	235,6645256	mm2
Diameter of the reinforcements	ø	16	mm
Section area of one reinforcement bar	Aw	201,0619298	mm2
Number of reinforcement bars	n	2	-

2nd degree equation					
-0,52*(fyd/fcd)^2	ρ^2	+(fyd/fcd)	ρ	-Mrd/(bd^2fcd)	=0
Floor slab in x direction with the load of a small van					
-180,55	0	18,63	0	-4,51E-02	0

Also 4 reinforcement bars with a diameter of 16 mm per meter seems enough to bear the point load the tires of a small delivery van of 3,5 tons exert on the concrete construction.



Moment distribution around the x-axis



Deflection of the construction

Break water			
Dimensions			
Width	b	1500	mm
Thickness	d	100	mm
Length	l	5000	mm
Check without reinforcement			
Bending stress of concrete			
Bending stress of concrete	fctm,fl	4,8	N/mm2
	fctm,fl	>	fctm
	4,8	>	3,2
Bending stress of concrete	fctm,fl	4,8	N/mm2
Checking section S193			

Width	b	500	mm
Thickness	d	100	mm
Length	l	500	mm
Check of the bearing of the design moment			
Resistance moment	Wc	833333	mm3
Break moment	Mcr	4000000	Nmm
		4,00	kNm
Design moment	Med	4,5	kNm
Med	<		Mcr
4,5	<		4,00
Unity Check	U.C.	1,13	-
Determining the number of reinforcement bars needed in z- direction			
2^{nd} degree equation			
	D	319,35	
	ρ	10,11	%
	ρ	0,21	%
Reinforcement			
Reinforcement area	As	105,6636133	mm2
Diameter of the reinforcement bars	ø	16	mm
Section area of one reinforcement bar	Aw	201,0619298	mm2
Number of reinforcement bars	n	1	-

Wave breaker in z-direction					
-180,55		18,63		-3,86E-02	
-0,52*(fyd/fcd)^2	ρ^2	+(fyd/fcd)	ρ	-Mrd/(bd^2fcd)	=0

Appendix 25: Cost estimate of harbour expansion

Weken	30						
			Number		Price per piec	Amount in EUR	Subtotal
Preparation/ Work guidance							
	Setting up a contract document (Basic agreement, question specification and annexes)						
		Process planner	120	hours	62	€ 7 440,00	
	Setting up a quality plan						
		Process planner	120	hours	62	€ 7 440,00	
	Communication with stakeholders						
		Communication and coordination before the start and during the execution of the project					
		Process planner	80	hours	62	€ 4 960,00	
		Attending/ organising info evenings					
		Process planner	40	hours	62	€ 2 480,00	
	Communication with client						
		Progress meeting (12 hours per 4 weeks)					
		Process planner	84	hours	62	€ 5 208,00	
		progress report (8 hours every week)					
		Process planner	240	hours	62	€ 14 880,00	
		Capturing communication with client (4 hours per week)					
		Process planner	120	hours	62	€ 7 440,00	
		Starting up the project					
		Process planner	16	hours	62	€ 992,00	
	Attending system, process and product tests (each 4 sessions)						
		Process planner	96	hours	62	€ 5 952,00	
	Taking care of permits, exemptions, decisions and permissions						
		Process planner	80	hours	62	€ 4 960,00	
	Design guidance						
		Extending the quay wall					
		Process planner	64	hours	62	€ 3 968,00	
		Harbour mole + breakwater					
		Process planner	64	hours	62	€ 3 968,00	
		mole built of stone rubble					
		Process planner	64	hours	62	€ 3 968,00	
		Floating scaffoldings					
		Process planner	64	hours	62	€ 3 968,00	
	Purchasing orders, execution guidance, remaining tasks process planner (2 days per week)						
		Process planner	480	hours	62	€ 29 760,00	
						€ 107 384,00	**€ 107 384,00**
Design							
	tender costs						
	setting up EMVI-plan						
		Advisory office	12000	hours	1	€ 12 000,00	
	Design costs tender phase						
		Engineering office	13000	hours	1	€ 13 000,00	
	Cone penetration tests for the purpose of calculating alternatives						
		Cone penetration tests	1	hours	1200	€ 1 200,00	
	Research						
	Additional cone penetration tests for the purpose of the definitive design after the tender						
		Cone penetration tests	1	hours	4000	€ 4 000,00	
	Designing quay wall extension						
		Setting up the definitive design					
		Engineering office	2500	hours	1	€ 2 500,00	
		Setting up the job description and the implementation plan					
		Process planner	48	hours	62	€ 2 976,00	
		Preparing working visit					
		Preparing working visit	1750	hours	1	€ 1 750,00	
						€ 37 426,00	**€ 37 426,00**
Extending the quay wall							
	Working area						
	Establishing working area						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
		Construction fence	1200	hours	0,1	€ 120,00	
	Cleaning up the working area						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
	Preparatory ground work						
		1000l mobile crane	8	hours	60	€ 480,00	
		Tractor + trailer 8m3	16	hours	55	€ 880,00	
	Project execution						
	Installation of the sheet pile wall						
		Sheet pile wall	1535	tons	835	€ 1 281 399,29	
		KH 125 crane	54,8	hours	62,5	€ 3 425,00	
		Carpenter for outside jobs	120	hours	47,5	€ 5 700,00	
		Vibrating machine	15	days	500	€ 7 500,00	
	Refilling of soil after installing the sheet pile wall						
		1000l mobile crane	197	hours	60	€ 11 811,43	
		Sand	8613	m3			
		Self-propelling vibrating roller	3	days	165	€ 495,00	
		Navvy	197	hours	44,5	€ 8 760,14	
	Installion of the grout injection anchors						
		Grout injection anchors (estimation)	129	Pieces	1,175	€ 151,67	
		Tractor + watertrailer	163	hours	65	€ 10 597,98	
		Welder	245	hours	62,5	€ 15 285,55	
		Welding of the anchor block	258	hours	62,5	€ 16 134,75	
		Mobile crane	129	hours	60	€ 7 744,68	

	Installation of the steel girder						
		UNP 200	4346043	kg	0,59	€ 2 564 165,37	
		Mobile crane	424	hours	60	€ 25 444,71	
		Welder	848	hours	62,5	€ 53 004,02	
	Refilling of sand after installing the sheet pile wall						
		1000l mobile crane	230	hours	60	€ 13 780,00	
		Sand	4306	m3			
		Self-propelling vibrating roller	29	days	165	€ 4 736,88	
	Installation of the sheet pile cap						
		Installation of the sheet pile cap					
		Carpenter for outside jobs	208	hours	47,5	€ 9 880,00	
		Mini crane	104	hours	50	€ 5 200,00	
		Natural stone for the sheet pile caps	130	m	222	€ 28 860,00	
		Attachment material for the sheet pile caps	130	m	25	€ 3 250,00	
		Stamped concrete	11,7	m3	62	€ 725,40	
		Navvy	7	hours	44,5	€ 296,67	
	Installation of the concrete floor slab						
		Wood for formwork	310	m	5	€ 1 550,00	
		Carpenter for outside jobs	49,6	hours	47,5	€ 2 356,00	
		concrete reinforcement	6000	kg	1,25	€ 7 500,00	
		Concrete pump	1	piece	205	€ 205,00	
		Concrete C28/35	650	m3	76	€ 49 400,00	
		Carpenter for outside jobs	390	hours	47,5	€ 18 525,00	
		Costs for pumping	650	m3	7	€ 4 550,00	
						€ 132 298,07	**€ 132 298,07**
Floating scaffoldings 1							
	Installation of the anchoring piles for the scaffoldings						
		Pile made of synthetic material	5	piece	175	€ 875,00	
		1500 caterpillar crane	10	hours	75	€ 750,00	
		Carpenter for outside jobs	20	hours	47,5	€ 950,00	
		Vibrating machine	1,25	days	200	€ 250,00	
	Installation of the floating scaffoldings						
		1500 caterpillar crane	8	hours	75	€ 600,00	
		Building site crane	8	hours	65	€ 520,00	
		Carpenter for outside jobs	16	hours	47,5	€ 760,00	
		Floating scaffoldings 1 (inter boat marinas)	1	hours	101918	€ 101 917,79	
	Finishing						
		Process planner	80	hours	62	€ 4 960,00	
		Inspection job	1	(once)	1750	€ 1 750,00	
						€ 113 332,79	**€ 113 332,79**
Floating scaffoldings 2							
	Installation of the anchoring piles for the scaffoldings						
		Pile made of synthetic material	5	pieces	175	€ 875,00	
		1500 caterpillar crane	10	hours	75	€ 750,00	
		Carpenter for outside jobs	20	hours	47,5	€ 950,00	
		Vibrating machine	1,25	days	200	€ 250,00	
	Installation of the floating scaffoldings						
		1500 caterpillar crane	8	hours	75	€ 600,00	
		Building site crane	8	hours	65	€ 520,00	
		Carpenter for outside jobs	16	hours	47,5	€ 760,00	
		Floating scaffoldings 2 (inter boat marinas)	1	hours	91361	€ 91 361,16	
	Finishing						
		Process planner	80	hours	62	€ 4 960,00	
		Inspection job	1	(once)	1750	€ 1 750,00	
						€ 102 776,16	**€ 102 776,16**
Floating scaffoldings 3							
	Installation of the anchoring piles for the scaffoldings						
		Pile made of synthetic material	3	pieces	175	€ 525,00	
		1500 caterpillar crane	6	hours	75	€ 450,00	
		Carpenter for outside jobs	12	hours	47,5	€ 570,00	
		Vibrating machine	0,75	days	200	€ 150,00	
	Installation of the floating scaffoldings						
		1500 caterpillar crane	4	hours	75	€ 300,00	
		Building site crane	4	hours	65	€ 260,00	
		Carpenter for outside jobs	8	hours	47,5	€ 380,00	
		Floating scaffoldings 3 (inter boat marinas)	1	hours	71444	€ 71 444,13	
	Finishing						
		Process planner	80	hours	62	€ 4 960,00	
		Inspection job	1	(once)	1750	€ 1 750,00	
						€ 80 789,13	**€ 80 789,13**
Floating scaffoldings 4							
	Installation of the anchoring piles for the scaffoldings						
		Pile made of synthetic material	3	pieces	175	€ 525,00	
		1500 caterpillar crane	6	hours	75	€ 450,00	
		Carpenter for outside jobs	12	hours	47,5	€ 570,00	
		Vibrating machine	0,75	days	200	€ 150,00	
	Installation of the floating scaffoldings						
		1500 caterpillar crane	4	hours	75	€ 300,00	
		Building site crane	4	hours	65	€ 260,00	
		Carpenter for outside jobs	8	hours	47,5	€ 380,00	
		Floating scaffoldings 3 (inter boat marinas)	1	hours	56282	€ 56 282,07	
	Finishing						
		Process planner	80	hours	62	€ 4 960,00	
		Inspection job	1	(once)	1750	€ 1 750,00	
						€ 65 627,07	**€ 65 627,07**

Harbour mole							
	Installation of the tubular piles						
		Tubular piles	59	pieces	175	€ 10 325,00	
		1500 caterpillar crane	118	hours	75	€ 8 850,00	
		Carpenter for outside jobs	236	hours	47,5	€ 11 210,00	
		Vibrating machine	14,75	days	200	€ 2 950,00	
		Pontoon	118	hours	185	€ 21 830,00	
	Installation of L-walls						
		prefab concrete L-walls	120	pieces	180	€ 21 600,00	
		1000l mobile crane	120	hours	60	€ 7 200,00	
		Carpenter for outside jobs	240	hours	47,5	€ 11 400,00	
	Construction of wave breakers						
		Azobé	56	m3	1500	€ 83 255,04	
		Carpenter for outside jobs	222		47,5	€ 10 545,64	
	Installation of wave breakers						
		flatbed trailer	48	hours		€ 3 960,00	
		1000l mobile crane	48	hours	60	€ 2 880,00	
		Carpenter for outside jobs	96	hours	47,5	€ 4 560,00	
	Dumping of stone rubble						
		Stone rubble	7425	tons		€ 950 454,68	
		1000l mobile crane	40	hours	60	€ 2 400,00	
						€ 1 153 420,36	**€ 1 153 420,36**
Miscellaneous							
	CAR-insurance				3300	€ 3 300,00	
	PI-insurance (3,00 ‰)				6000	€ 6 000,00	
	Bank guaranty 5%				100000	€ 100 000,00	
	Camera surveillance		60	weeks	150	€ 9 000,00	
	Installation of temporary depositories						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
		Construction fences	1200	hours	0,1	€ 120,00	
		Loading costs for the road plates	20	hours	4	€ 80,00	
		Road plates	80	hours	0,48	€ 38,40	
	Upkeep of temporary depositories						
		Construction fences	7500	days	0,1	€ 750,00	
		Road plates	3000	days	0,48	€ 1 440,00	
		Navvy	16	hours	44,5	€ 712,00	
		1000l mobile crane	16	hours	60	€ 960,00	
		6x6 truck + crane	16	hours	67,5	€ 1 080,00	
	disassembly of the temporary depositories						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
		Construction fences	1200	hours	0,1	€ 120,00	
		Loading costs for the road plates	20	hours	4	€ 80,00	
		Road plates	80	hours	0,48	€ 38,40	
	Miscellaneous equipment						
		Large power unit	20	days	100	€ 2 000,00	
		Small power unit	40	days	30	€ 1 200,00	
		Construction shack	30	weeks	54	€ 1 620,00	
		dixi-toilet unit	45	weeks	13	€ 585,00	
		direction shack	150	days	15	€ 2 250,00	
						€ 133 165,80	**€ 133 165,80**
Cost aspects							
	Vibrating machine						
		supply and despatch of the vibration machine	2	times	1000	€ 2 000,00	
		supply and despatch of the vibration machine for tubular piles	1	(once)	2500	€ 2 500,00	
	Sheet pile wall						
		supply and despatch of the sheet pile walls	1	(once)	3000	€ 3 000,00	
		Installation for sheet pile wall examination	1	(once)	150	€ 150,00	
	mobile/ caterpillar crane						
		supply and despatch of the cranes	4	times	200	€ 800,00	
	back planes, beetlehead frame						
		Welder	40	hours	62,5	€ 2 500,00	
		flatbed trailer	6	hours		€ 495,00	
		profile steel	3000	kg	0,6	€ 1 800,00	
	Traffic measures and communication signs						
		Traffic measures	7500	times	1	€ 7 500,00	
	Costs for execution						
		Executor	1200	hours	61,25	€ 73 500,00	
						€ 94 245,00	**€ 94 245,00**
						subtotal	€ 6 251 797,99
						8%	€ 500 143,84
						5%	€ 312 589,90
						Contracted price	**€ 7 064 531,72**

Appendix 26: Cost estimate demolition sills in guard lock and dredging of the harbour

			Number		Price per piece	Amount in EUR	Subtotal
Preparation/ Work guidance							
	Setting up a quality plan						
		Process planner	24	hours	62	€ 1 488,00	
	Communicatie met belanghebbenden						
		Communication and coordination before the start and during the execution of the project					
		Process planner	20	hours	62	€ 1 240,00	
		Attending/ organising info evenings					
		Process planner	8	hours	62	€ 496,00	
	Communication with client						
		Progress meeting (12 hours per 4 weeks)					
		Process planner	0	hours	62	€ -	
		progress report (8 hours every week)					
		Process planner	24	hours	62	€ 1 488,00	
		Capturing communication with client (4 hours per week)					
		Process planner	12	hours	62	€ 744,00	
		Starting up the project					
		Process planner	16	hours	62	€ 992,00	
	Taking care of permits, exemptions, decisions and permissions						
		Process planner	24	hours	62	€ 1 488,00	
	Purchasing orders, execution guidance, remaining tasks process planner (2 days per week)						
		Process planner	48	hours	62	€ 2 976,00	
						€ 10 912,00	**€ 10 912,00**
Design							
	Research						
	Additional research to investigate what and how the concrete reinforcement bars are placed in the structure of the guard lock						
		Research	1	hours	12000	€ 12 000,00	
						€ 12 000,00	**€ 12 000,00**
Preparatory work							
	Working area						
	Establishing working area						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
		Construction fence	120	hours	1	€ 120,00	
	Cleaning up the working area						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
	Project execution						
	Supply of working material						
		Work vessel	8	hours	200	€ 1 600,00	
		Supply and despatching of the vibrating machine	2	pieces	412,5	€ 825,00	
	Installation of the sheet pile wall (30m)						
		Hiring costs for the sheet pile walls	25	tons	140	€ 346,92	
		KH 125 crane	16	hours	62,5	€ 1 000,00	
		Carpenter for outside jobs	16	hours	47,5	€ 760,00	
		Work vessel	32	hours	67,5	€ 4 320,00	
		Hiring costs for the vibrating machine	2	days	500	€ 1 000,00	
	Applying clay for sealing						
		Clay supply	150	m3	15	€ 2 250,00	
		Work vessel	8	hours	200	€ 3 200,00	
	Instalation of the bulkhead						
		Construction of the bulkhead				€ 70 500,00	
		800l mobile crane	8	hours	58	€ 464,00	
		Carpenter for outside jobs	16	hours	47,5	€ 760,00	

	Hiring of pump and accessories						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Carpenter for outside jobs	16	hours	44,5	€ 712,00	
		100 m3 pump for dirty water	3	weeks	78,9	€ 236,84	
		Gasoline	2000	liters	62,5		
	Removal of the clay						
		Work vessel	16	hours	165	€ 2 640,00	
		8x4 truck	60	hours	66	€ 3 960,00	
		Costs for dumping clean soil	540	ton	5	€ 2 700,00	
	Removal of the bulkheads						
		800l mobile crane	8	hours	58	€ 464,00	
		Carpenter for outside jobs	16	hours	47,5	€ 760,00	
	Removal of the sheet pile wall (30m)						
		KH 125 crane	16	hours	62,5	€ 1 000,00	
		Carpenter for outside jobs	16	hours	47,5	€ 760,00	
		Hiring costs for the vibrating machine	2	days	500	€ 1 000,00	
	Despatch of equipment						
		Work vessel	8	hours	165	€ 1 320,00	
		Supply and despatching of the vibrating machine	2	pieces	412,5	€ 825,00	
						€ 28 235,76	**€ 28 235,76**
Demolition of the sills in the guard lock							
	Cleaning up the working area						
		Carpenter for outside jobs	20	hours	47,5	€ 950,00	
	Slopen drempels + beschermen wapening						
		Carpenter for outside jobs	80	hours	47,5	€ 3 800,00	
		Building site crane	40	hours	65	€ 2 600,00	
		6x6 truck	16	hours	68	€ 1 080,00	
	Finishing						
		Process planner	80	hours	62	€ 4 960,00	
		Inspection	1	(once)	1750	€ 1 750,00	
						€ 15 140,00	**€ 15 140,00**
Dredging							
		Dredging	5143	m3	3,5	€ 18 000,50	
						€ 18 000,50	**€ 18 000,50**
Miscellaneous							
	CAR-insurance					€ 3 300,00	
	PI-insurance (3,00 ‰)					€ 6 000,00	
	Bank guaranty 5%					€ 133,39	
	Camera surveillance		3	weeks	150	€ 450,00	
	Installation of temporary depositories						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
		Construction fences	1200	hours	0,1	€ 120,00	
		Loading costs for the road plates	20	hours	4	€ 80,00	
		Road plates	80	hours	0,48	€ 38,40	
	Upkeep of temporary depositories						
		Construction fences	7500	days	0,1	€ 750,00	
		Road plates	3000	days	0,48	€ 1 440,00	
		Navvy	16	hours	44,5	€ 712,00	
		1000l mobile crane	16	hours	60	€ 960,00	
		6x6 truck + crane	16	hours	67,5	€ 1 080,00	
	disassembly of the temporary depositories						
		6x6 truck + crane	8	hours	67,5	€ 540,00	
		Navvy	8	hours	44,5	€ 356,00	
		Construction fences	1200	hours	0,1	€ 120,00	
		Loading costs for the road plates	20	hours	4	€ 80,00	
		Road plates	80	hours	0,48	€ 38,40	

	Miscellaneous equipment						
		Large power unit	21	days	100	€ 2 100,00	
		Small power unit	21	days	30	€ 630,00	
		Construction shack	3	weeks	54	€ 162,00	
		dixi-toilet unit	3	weeks	13	€ 39,00	
		direction shack	21	days	15	€ 315,00	
						€ 20 340,19	**€ 20 340,19**
Cost aspects							
	Vibrating machine						
		supply and despatch of the vibration machine	2	times	1000	€ 2 000,00	
		supply and despatch of the vibration machine for tubular piles	1	(once)	2500	€ 2 500,00	
	Sheet pile wall						
		supply and despatch of the sheet pile walls	1	(once)	3000	€ 3 000,00	
		Installation for sheet pile wall examination	1	(once)	150	€ 150,00	
	mobile/ caterpillar crane						
		supply and despatch of the cranes	4	times	200	€ 800,00	
	back planes, beetlehead frame						
		Welder	40	hours	62,5	€ 2 500,00	
		flatbed trailer	6	hours		€ 495,00	
		profile steel	3000	kg	0,6	€ 1 800,00	
	Costs for execution						
		Executor	96	hours	61,25	€ 5 880,00	
						€ 19 125,00	**€ 19 125,00**
						subtotal	€ 272 209,22
						8%	€ 21 776,74
						5%	€ 13 610,46
						Contracted pric	**€ 307 596,41**