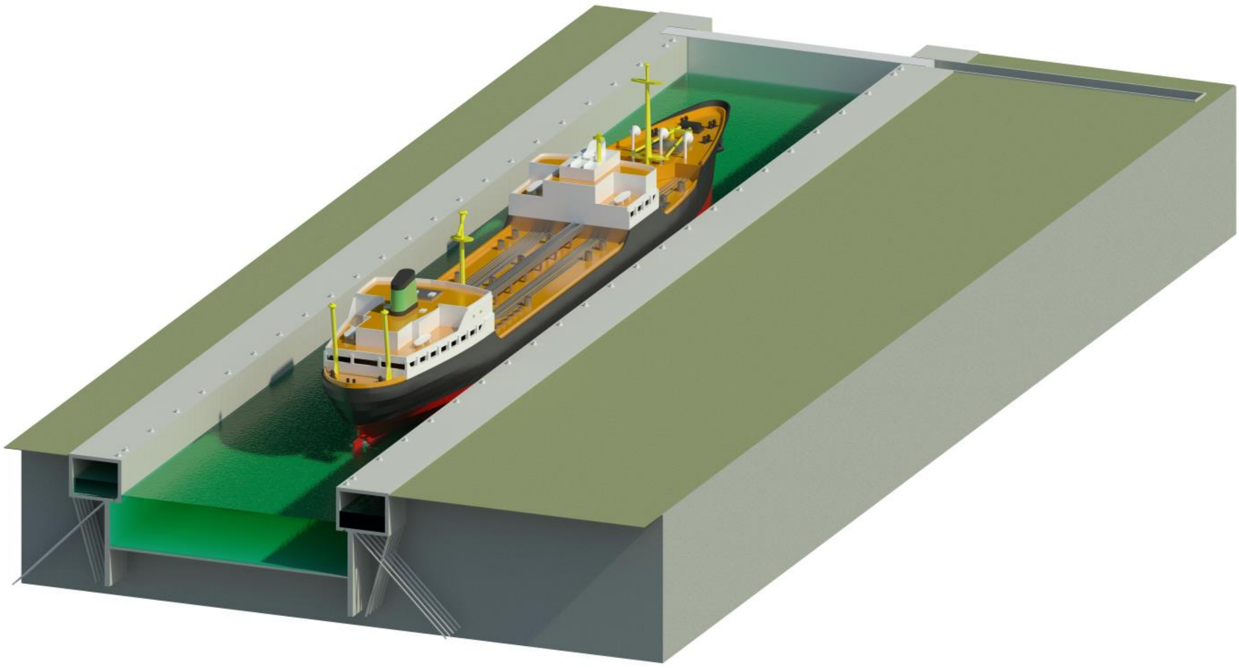
# Closure of the New Waterway

An adaptive and innovative design of a lock complex in Rotterdam

S.G. Dorrepaal

Master of Science Thesis



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# Closure of the New Waterway

## An adaptive and innovative design for a navigation lock in Rotterdam

By

Siebe Dorrepaal

in partial fulfilment of the requirements for the degree of

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in Civil Engineering

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# Preface

This master thesis 'Closure of the New Waterway' in front of you composes a measure which has large consequences regarding economy, ecology and safety for the Netherlands and their inhabitants. Too large to be computed by one person, you could say. But when one wants to have the qualities of a civil engineer, one should not be limited. One should have a broad perspective of the world and put these perspectives in the right place. However, one should not forget their own position in the world and not allow themselves to think that they are able to control this same world in all its manners. In fact, in my humble opinion, the smartest person in the world is the one that does know what he does not know.

In the light of the above, I tried to be this engineer during my thesis work. Whether I accomplished that, is not up to me. I would however like to thank the persons that help me try to achieve my goals during my study and graduation. First of all I would like to thank my thesis committee (Bas Jonkman, Wilfred Molenaar, René Braam, Koen van Gelder and Pepijn van der Ven), for saving their time for me to improve my work with a lot of enthusiasm. Special thanks to Ad van der Toorn, who helped me starting up my thesis and put a lot of time in supervising me before his retirement. Furthermore I would like to thank Antea group, Koen van Gelder and my direct colleagues, for providing a nice work environment and providing help where I needed it. I would like to thank Pepijn van der Ven and Deltares, for the time they took to supervise me, even without being asked to. My fellow students I would like to thank as well, for putting me in a challenging environment. Last but not least I would like to thank my family and my beloved girlfriend for the support they gave me.

Siebe Dorrepaal

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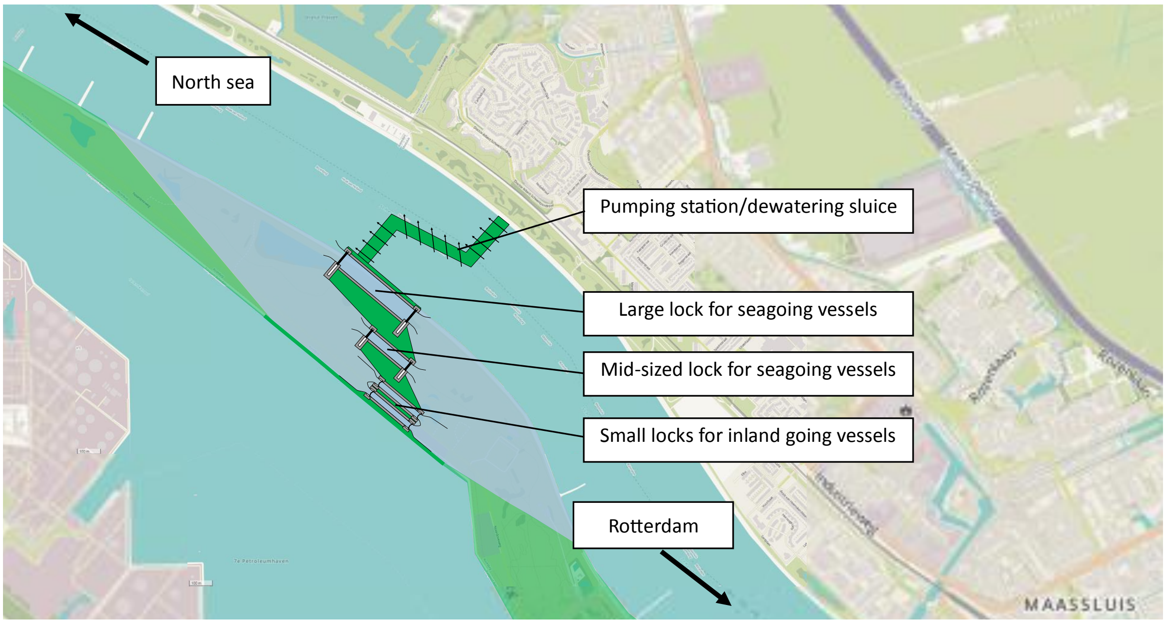
# Abstract

The Netherlands are protected against extreme high waters from the sea by an ingenious system of barriers. One of the most famous ones is the Maeslant barrier in the New Waterway near the port of Rotterdam. This barrier however, is subject to discussion, as it might not function as intended, because of the following reasons. It has a very low reliability regarding closing. Furthermore, because of the sea level rise, it is expected that the barrier should close much more frequently, causing more nuisance for navigation. Therefore a different solution is put forward by Spaargaren c.s. Namely to close off the New Waterway permanently by means of a barrier, consisting out of a pumping station, dewatering sluice and a lock complex. The latter one is further elaborated on by making a feasibility study for it. First of all, the best location is chosen for the barrier, which in this report is the location near Rozenburg, just east of the Maeslant barrier.

Because of the high demands regarding both navigation and flood safety in combination with the uncertainty in these demands for the future, there is a need for a sophisticated design for the lock complex. This is done by first considering the boundary conditions for different (initial and adaptive) scenarios for both sets of requirements (navigation and flood safety). For navigation a larger vessel is taken into account in the adaptive boundary conditions. For flood safety, the adaptive boundary conditions results in a higher water level of about 1 meter to be retained, mostly due to a larger sea level rise. It turns out that the boundary conditions regarding navigation have a way larger impact in the design choices of the lock complex than the boundary conditions regarding flood safety.

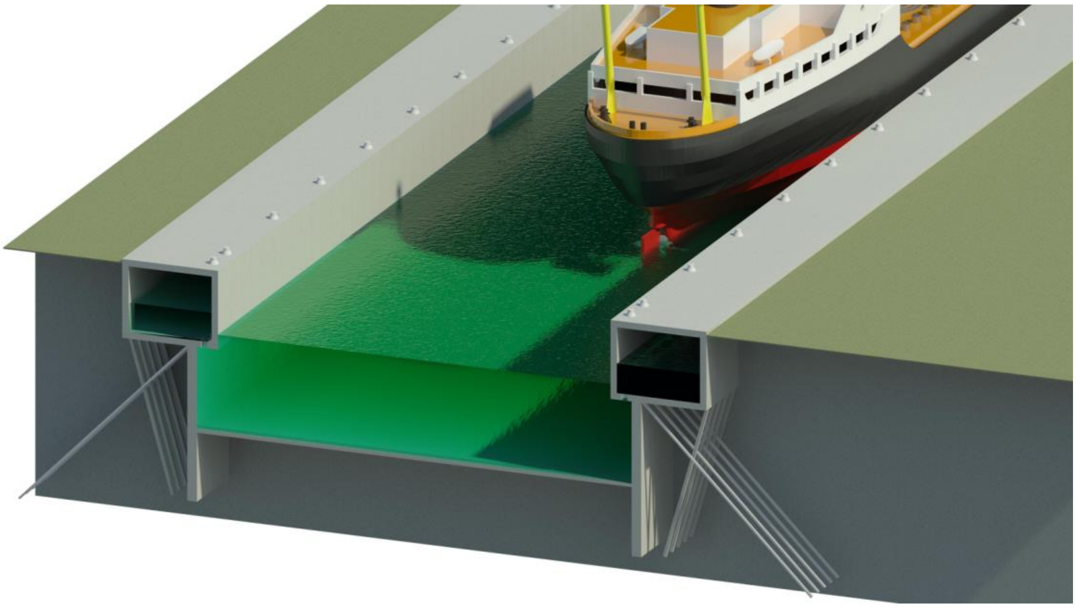
The above boundary conditions are used to create an overall solution for the barrier. 4 locks are needed (in case of an initial design): one large lock for large seagoing vessels, one mid-sized lock for smaller seagoing vessels and two for inland going vessels. When looking at the adaptive requirements, another mid-sized lock should be added.




*Overall solution lock complex with the four proposed locks*

For the large lock, different principal solutions are developed that also take the adaptive boundary conditions into account. First of all, an adaptive design of a lock head is considered by making the head modular. This makes it possible to place a larger gate in the head later on. It turns out that this is probably not cost-efficient.

The second proposed solution is to use a retaining wall in combination with a relieving floor for the chamber walls of the large lock for seagoing vessels. The relieving floor will decrease the horizontal soil loads on the soil retaining wall beneath it, which is structurally beneficial. This relieving floor will be combined with a longitudinal filling system. A longitudinal culvert over the length of the lock is placed on top of the relieving floor. Filling and emptying will be done using openings in the floor of the superstructure. The load on vessels during levelling of the lock are determined for this system, which is the main requirement that determines the filling time. It turns out that this option results in much faster levelling times compared to other levelling systems. Levelling could be done a factor 3 faster than filling through the head.



*Relieving floor with longitudinal levelling system*

It is concluded that the longitudinal culvert in combination with the relieving floor can compete with conventional filling systems. Furthermore, it fits in an adaptive design approach, because even with a wider lock, faster levelling times are possible. However, there are still some considerations to be made regarding this solution. A thorough cost-benefit analysis should be made. Besides of this, the hydraulic computations should be validated by means of a scale model or computational fluid dynamics.

# Content

# Abbreviations

| | |
|---|---|
| CEMT | Conférence Européenne des Ministres de Transport: classification of inland vessels. |
| Ctc | Centre To Centre distance between two piles |
| DWT | Dead Weight Tonnage |
| KNMI | Koninklijk Nederlands Meteorologisch Instituut, Dutch weather institute |
| LCC | Life Cycle Costs |
| MCA | Multi Criteria Analysis |
| MV-pile | Müller Verpress-pile, a tension pile with a grout layer |
| NAP | Nieuw Amsterdams Peil, Dutch reference height |
| NPC | Net Present Costs |
| RoRo | Roll on, Roll of, used to name vessels that transport cars |
| SLS | Serviceability Limit State |
| TNO | Toegepast Natuurwetenschappelijk Onderzoek |
| ULS | Ultimate Limit State |

# Nomenclature

| | | |
|---|---|---|
| $A$ | = | Cross sectional area [$m^2$] |
| $a_1$ | = | Height of the salt layer [m] |
| $a_2$ | = | Height of the fresh layer above the salt layer [m] |
| $A_s$ | = | Cross sectional area of the reinforcement steel [$m^2$] |
| $b$ | = | Beam of the vessel [m] |
| $c'_1$ | = | Coefficient for front velocity (0,42 for a fresh lock, 0,46 for a salt lock) [-] |
| $C_b$ | = | Block coefficient of the vessel [-] |
| $C_E$ | = | Coefficient for the damping exponent [-] |
| $C_f$ | = | Friction coefficient [-] |
| $c_l$ | = | Wave speed without vessel [m/s] |
| $c_v$ | = | Wave speed with vessel [m/s] |
| $C_{wk}$ | = | Capacity per lock per week [#/week] |
| $d$ | = | Water depth [m] |
| $dt$ | = | Time step [s] |
| $d_v$ | = | Draught of the vessel [m] |
| $E$ | = | Dimensionless eccentricity [-] |
| $e$ | = | Eccentricity of the vessel [m] |
| $E_1$ | = | The active force working on the vertical through the anchor [kN] |
| $E_A$ | = | The force working on the active slip wedge through the vertical of the retaining wall. [kN] |
| $E_c$ | = | Concrete stiffness uncracked state [$N/mm^2$] |
| $E_{ccracked}$ | = | Concrete stiffness cracked state [$N/mm^2$] |
| $EI$ | = | Bending stiffness [$Nm^2$] |
| $E_s$ | = | Steel stiffness [$N/ mm^2$] |
| $F_{cc}$ | = | Compression force in the concrete compression zone [kN] |
| $f_{cd}$ | = | Design value pressure strength concrete [$N/mm^2$] |
| $f_{ck}$ | = | Characteristic value pressure strength concrete [$N/mm^2$] |
| $f_{ctd}$ | = | Design value tensile strength concrete [$N/mm^2$] |
| $f_{ctm}$ | = | Characteristic value tensile strength concrete [$N/mm^2$] |
| $F_{kr}$ | = | The maximum anchor force that can be retained by the anchor [kN] |
| $F_m$ | = | Transversal load due to momentum differences [‰] |
| $F_P$ | = | Averaged load due to translatory waves for the purpose of damping [‰] |
| $F_{SG}$ | = | Load due to translatory waves [‰] |

| | | |
|---|---|---|
| $F_{st}$ | = | Tension force in the reinforcement steel [kN] |
| $F_T$ | = | Smoothed translatory wave load without damping [‰] |
| $f_{yd}$ | = | Design strength steel [N/mm$^2$] |
| $F_z$ | = | Unsmoothed translatory wave load without damping [‰] |
| h | = | Water level [m+NAP] |
| H | = | Wave height [m] |
| $I_{lr}$ | = | Transport of momentum due to density differences [kgm/s$^2$] |
| $I_{wk}$ | = | Intensity of vessels per week [#/week] |
| J | = | Counter, amount of time steps [#] |
| k | = | Spring stiffness [kN/m] |
| $k_0$ | = | Neutral earth pressure coefficient [-] |
| $k_a$ | = | Active earth pressure coefficient [-] |
| $k_p$ | = | Passive earth pressure coefficient [-] |
| $L_{eff}$ | = | Effective length of the grout body [m] |
| $L_l$ | = | Length of the lock [m] |
| $l_v$ | = | Length of the vessel [m] |
| M | = | Bending moment [kNm] |
| NBJ | = | Amount of time step points between left wall and vessel [-] |
| NHJ | = | Amount of time step points between left and right side of the vessel [-] |
| NSJ | = | Amount of time step points between right side of the vessel and right wall [-] |
| P(x) | = | Probability of occurence [-] |
| $P_{r,max}$ | = | Bearing capacity of a foundation pile [kN] |
| Q | = | Discharge [m$^3$/s] |
| $Q_1$ | = | The force working on the sliding plane, working under an angle $\phi$ [kN] |
| $q_{c,I}$ | = | Cone resistance according to Koppejan [MPa] |
| $R_e$ | = | Reynolds number [-] |
| $R_f$ | = | Transversal load due to friction [‰] |
| S | = | Wet surface of the hull [m$^2$] |
| $T_k$ | = | Natural period of the lock [s] |
| u | = | Velocity [m/s] |
| w | = | Width of the lock [m] |
| $x_v$ | = | Distance between the vessel and chamber wall [m] |
| $\beta$ | = | Mixing coefficient [-] |
| $\gamma_m$ | = | Material safety factor [-] |
| $\delta$ | = | wall friction angle [°] |
| $\Delta H$ | = | Head, water level difference between lock and approach harbour [m] |
| $\Delta h_{lr}$ | = | Water level difference between the left and right side of the vessel [m] |
| $\epsilon$ | = | Relative density [-] |
| $\varepsilon$ | = | Strain [-] |
| K | = | Curvature [m$^{-1}$] |
| $\mu$ | = | Coefficient of contraction [-] |
| $\nu$ | = | Kinematic viscosity [m$^2$/s] |
| $\rho$ | = | Density [kg/m$^3$] |
| $\rho_{As}$ | = | Reinforcement ratio of the cross section [%] |
| $\tau_{max}$ | = | Maximum shear stress between the grout body and the soil [N/ mm$^2$] |
| $\phi$ | = | Angle of repose [°] |

# List of figures

# List of tables

# 1

# Introduction

## 1.1 Problem description

The Netherlands are protected against flooding by an ingenious system of barriers and other hydraulic structures. To maintain this safety for the future, the Delta program has been created and is updated each year in which the strategy for the future is stated. The currently preferred strategy for the Rhine mouth and Drecht cities is to maintain the current situation, making use of the Maeslant barrier and strengthening of the existing dikes.

*Figure 1 Maeslant barrier in closed position (Rijkswaterstaat)*

Several Dutch experts however, have doubts about this strategy. Different reasons are stated for this:

- The closure reliability of the Maeslant barrier is very low. It has a probability of failure of 1/100 for closing (Rijkswaterstaat, 2008).
- The Maeslant barrier is not suitable to cope with the problems regarding fresh water supply. In case of long droughts this supply could be in danger, because of the salt intrusion into the New Waterway.
- The barrier is not resistant against a negative head. When it is closed and a high water discharge in the rivers occurs, this will probably cause problems.
- It is expected that the storm surge barrier will have to be closed more often in the future, due to sea level rise. This causes a lot of disruptions for navigation purposes.

Besides, nature is unpredictable and when a situation worse than foreseen may occur, it is probably too late and thus impossible to implement any additional measures. Another solution could be to close off the New Waterway completely with a barrier consisting of a dewatering sluice and a navigation lock (also called: plan sluices); a solution suggested by ir. Spaargaren c.s, see Figure 2 (Rijkswaterstaat, 2015). The reliability of the barrier would be increased very much, because it is permanently closed. The fresh water availability will be increased as well, because the salt water is not able to intrude the New Waterway anymore.

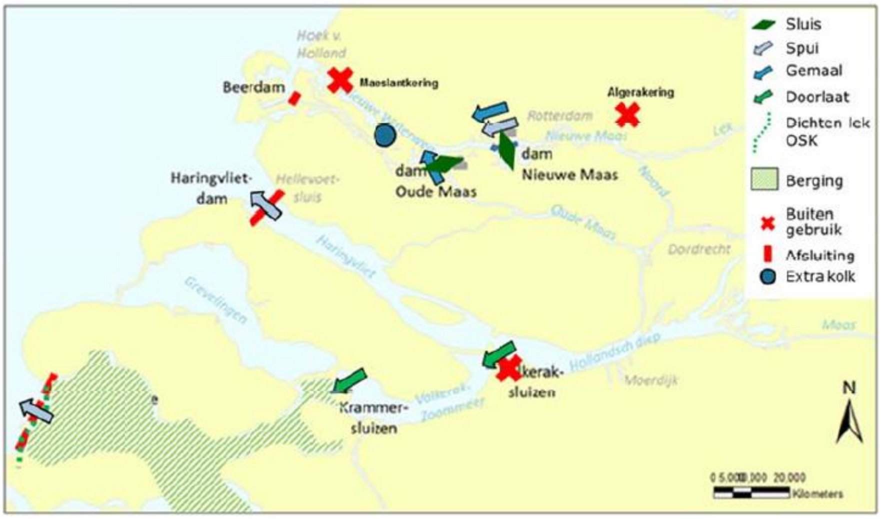



*Figure 2 proposed measures Plan Spaargaren (van Waveren, et al., 2015)*

The plan sluices is not without controversy however. It causes delays and limitations for navigation and it is not quite obvious how navigation will develop. Besides, characteristics of nature and socio-economic developments are still hard to predict. A structure like a navigation lock is mostly designed for a period of 100 years. The predictive scenarios for navigation and extreme water levels for 2100 are however very diverse. Because of this, there is a need for an optimized design, which is robust, but flexible and easily adaptable when the boundary conditions will change.

## 1.2 Objectives

The technical feasibility of the barrier will be researched in this thesis by making a preliminary design of the barrier. At the time of writing there are researches being done or already completed about the ‘plan sluices’. These are mostly studies aimed at demonstrating the usefulness and necessity of the plan at the higher system levels. This will not be done in this research project, but these studies are not unrelated. The principles for the sluices made in the earlier studies, should be included, so that they can be built upon further. On the other hand, existing principles may be revised based on new insights. A study on the technical feasibility of the locks itself has not been performed before, as far as known. This research will therefore start from the beginning of the design process of the structure.

The barrier will consist of several main elements, the focus in this thesis is however on the navigation lock complex. The other elements should be taken into account when looking at spatial availability, so assumptions based on expert judgement and reference projects will be made for the dimensions of the other elements.

As this is a graduation thesis and not the standard design of a lock, a number of aspects will play a major role throughout the process. This guarantees the academic depth and innovation that is expected. These aspects fit within the (future) policy of the government. They are primarily:

- Adaptive delta management
  Adaptive delta management means that when creating a design one takes into account the changing requirements in the future. This does not imply that an exaggerated over-dimensioned design will be made, but that one builds a structure that can be easily adjusted when needed. This strategy is currently applied to the dikes in the Netherlands. Because the lock will be an element of a primary water barrier, it is likely that here too this strategy will be used.
- Standardization
  Within Rijkswaterstaat, the desire exists to apply more standardization in the design of navigation locks. Given this rather unique situation (sealock, primary water barrier), it is unclear if this can provide substantial cost savings. Therefore it will not be taken into account further as a main goal.
- Innovation
  Innovation can be taken into account by using new concept ideas, for example lock gates or chamber walls. Also when implementing the adaptive delta management innovative concepts could be developed.

## 1.3 Research question

**Main research question**

Out of the above aspects and problem description the main research question is formulated:

*How can an adaptive navigation lock system be designed as part of a cost-effective solution to permanently close off the New Waterway?*

**Sub questions**

Several sub questions are stated to answer the main research question:

1. *How can an adaptive approach be incorporated in the design process?*
   a. *What are the main requirements for the navigation lock system?*
   b. *Which possible requirement scenarios should be taken into account to make a cost-effective solution?*
2. *Which solution or set of solutions is the most applicable in the design approach?*
3. *What is the technical feasibility of the proposed solution?*

## 1.4 Adaptive design approach

Within the Delta program an adaptive delta management approach is developed (van Rhee, 2012). This means that the bandwidth of future scenarios are taken into account when designing a flood barrier. There is a large uncertainty in the climate and the social-economic development of the Netherlands during the lifetime of a structure. Both of them influence the requirements for the lock complex. The approach is that newly designed dikes and hydraulic structures are made in such a way that it is safe for now, but that they can easily and relatively cheaply be adapted when another situation will develop in the future, see Figure 3. On the other hand, the structure needs to be robust enough. It is not desirable that every year a structure has to be adapted. For a navigation lock such as in the New Waterway, several requirements could be taken into account in this approach. Not only regarding the safety of the people in the hinterland, but other functional requirements like navigation as well.

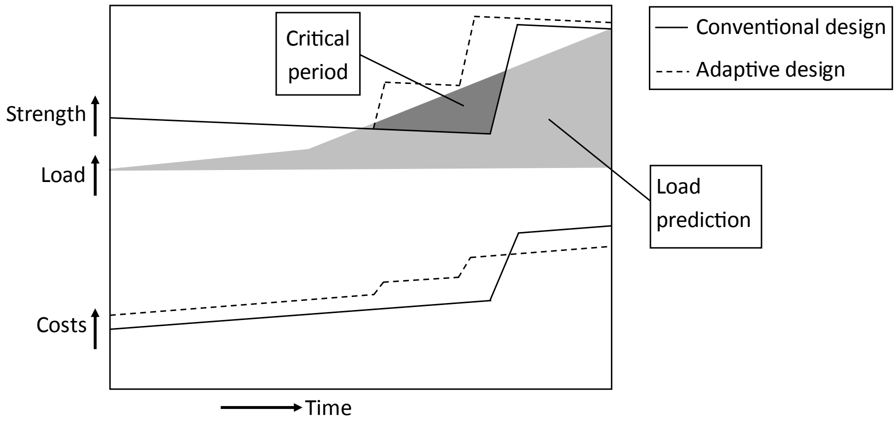



*Figure 3 Strength and costs relation in time*

To make the preliminary design of the lock, various phases have to be passed. Given the available time not everything can be examined. At each phase choices are herein made which aspects will be addressed. The choices depends on the academic interest and to what extent there is profit to be gained for the lock in the form of cost savings, security, maintenance, management et cetera.

The design process is divided into three phases. At the end of each phase, an intermediate product will be given. The design will progress from a global to a detailed product. Thus every subsequent product will be more detailed than the previous one.

### *Phase 1: Analysis*

In this part the environment, the system and subsystem will be researched. The definition of these terms are given in Figure 4. At the end of this phase several alternatives on the subsystem level will be computed.

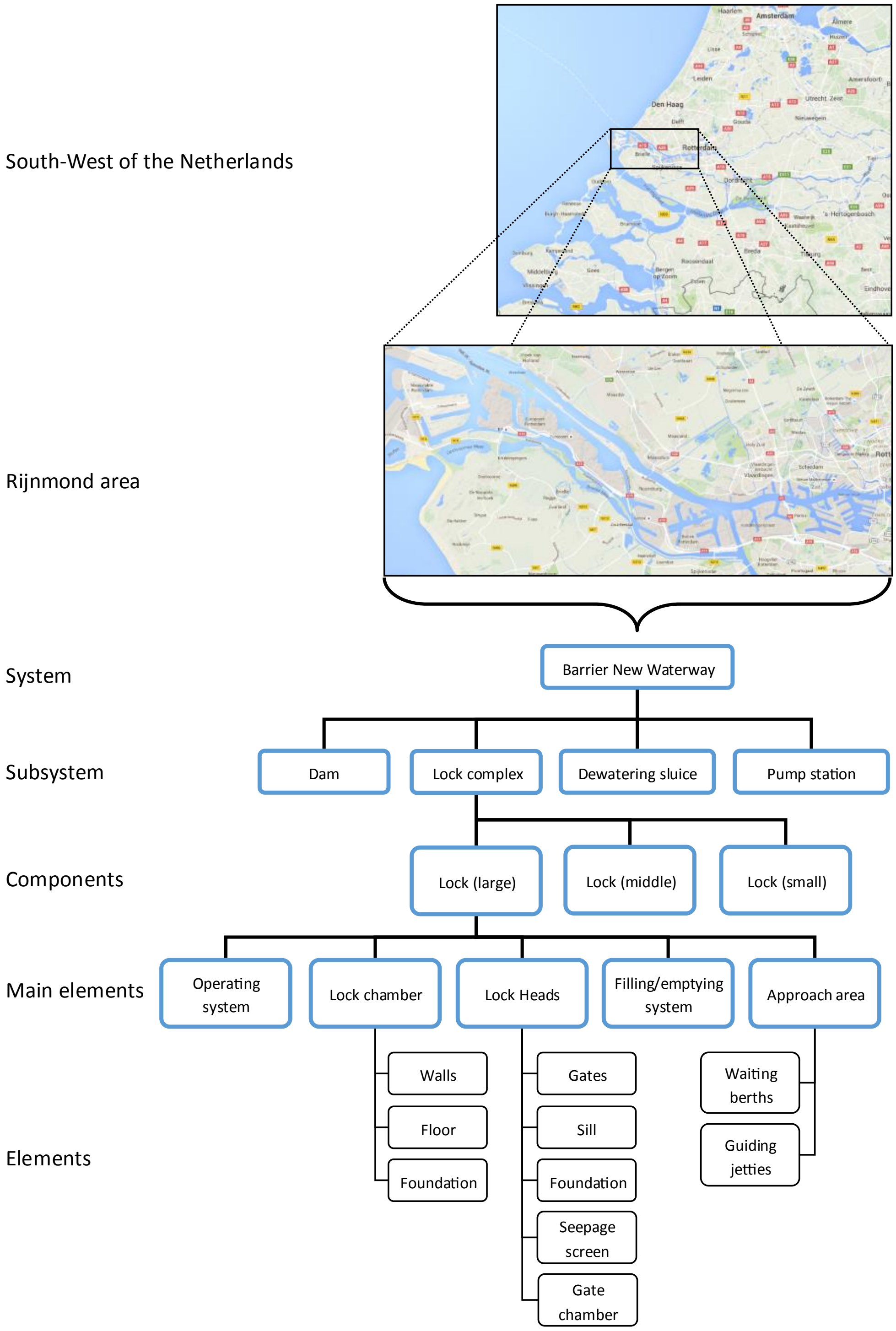



*Figure 4 Definition system parts*

**Analysis higher system parts**

The total study area of the lock will be described: the south west of the Netherlands and in more detail the Rhine mouth area, see Figure 4 for the definition of the system. The lock is part of the larger 'plan sluices' that covers the entire Rhine-Meuse delta. Obviously this overlying plan is also being studied and taken account of. Besides this, the current circumstances and the preferred strategy of the Deltaprogram will be mapped out. Also the vessel movements in the area will be researched to determine the requirements for the lock complex.

**Global functional requirements**

Depending on the analysis of the surrounding area the global functional requirements are set up. These exist mainly out of requirements for navigation and flood safety, see Figure 5. When the location is still unknown, the focus will be first on the navigational functional requirements, as this determines the lock configuration in terms of the number of locks needed and the horizontal dimensions of the lock complex.

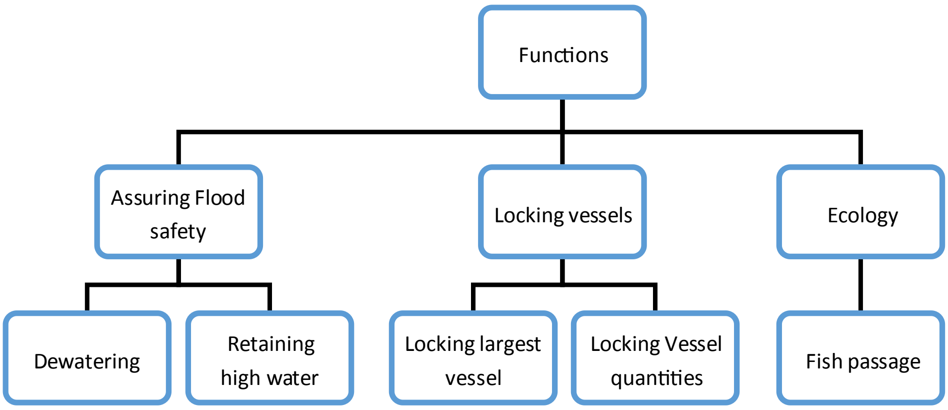



*Figure 5 Function tree lock complex*

**Alternatives on system level: location choice**

When the functional requirements are roughly known, alternatives on system level will be developed. The necessary amount of locks suitable for a certain vessel will be determined and a more extensive research will be done to derive the best location.

### *Phase 2: Overall solution*

**Functional and technical requirements**

Based on the functional requirements and the choice for the configuration of the locks, the technical requirements are determined. In this part it is possible to take the adaptive character into account. Multiple scenarios will be set up. On a later stage, these scenarios will be implemented in the design elements. An estimation of the costs will be made to determine whether it could be feasible to implement the adaptive approach.

**Principal solutions**

Different solutions for different parts of the lock will be developed. It will be taken into account to what extent the solutions are able to meet the technical requirements regarding adaptivity. The pros and cons are weighed.

**Overall solution**

Using the principal solutions, an overall solution is made for the whole lock. In this part it is important that the several solutions will form one complete solution.

### *Phase 3: Design of a (main) element*

**Detailed elaboration element overall solution**

Out of the chosen overall solution, several elements will be chosen to be designed in detail. These are the lock head, chamber walls and the filling/emptying system of the large lock. This choice is based on the academic interest of the elaboration and is explained in this thesis. During this process there will also be reverted to the overall solution, in order to make sure that the different components do not conflict with each other.

**Preliminary design**

Finally a preliminary design for the whole lock will be made.

### *Process*

In the next picture the above phases of the process are visualized. Several times during the process there will be reverted to the system. This will make sure that the several parts always make one whole fitting solution. Besides of that, it can be seen that the aspects, adaptivity and innovation will be taken into account during a large part of the process.

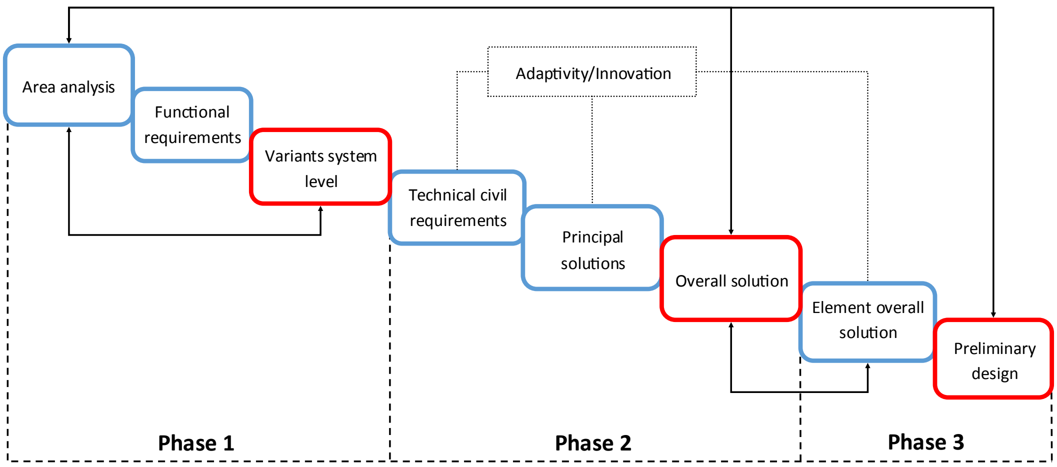



*Figure 6 Visualisation graduation process (blue: process, red: product)*

## 1.5 Thesis outline

An overview of the thesis outline is given in Figure 7. Chapter 1 is the diverging part, where the research questions are stated and an analysis is done on the higher parts of the system. After that, the report is more converging towards the final solution of the lock complex. In chapter 2 the best location of the barrier is chosen. Subsequently, in chapter 3, the boundary conditions according to that location are derived for both the adaptive and the conventional design approach. These are used in chapter 4 to determine the best principal solutions for the lock complex. Which combined makes the overall solution. After this part, two different paths are followed up to the conclusion. One of them is the elaboration of the design of an adaptive lock head: chapter 5. The other path is the design of a relieving floor as chamber wall for a large lock in combination with a longitudinal culvert. In chapter 6 the hydraulic design for the longitudinal culvert is worked out. In chapter 7 the structural design for the relieving floor is elaborated. Together with the other chapters this will lead to the conclusion in chapter 8.

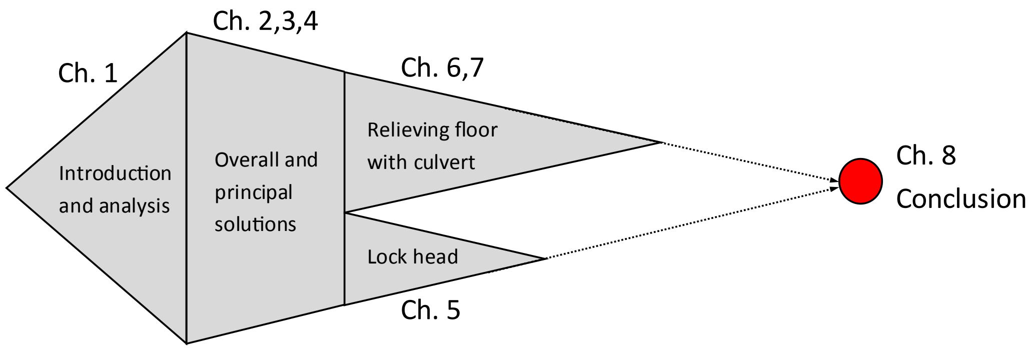



*Figure 7 Thesis outline per chapter, based on: (Jonkman, 2016)*

# 2

# Location of the barrier

The first step in this design process is to determine the best location for the barrier in the New Waterway. First of all, an area description is given. Than the factors that determine the differences between the locations are set. Subsequently, several alternatives are composed. These will be evaluated by means of a multi criteria analysis and the best one will be chosen to be worked out in further detail. The alternatives are made on subsystem level. Thus the location of the lock is taken into account, but also the lock configuration regarding the width, length and number of locks. Besides this, an estimation of the costs and dimensions is made for the other parts of the barrier like the dewatering sluice and the pumping station.

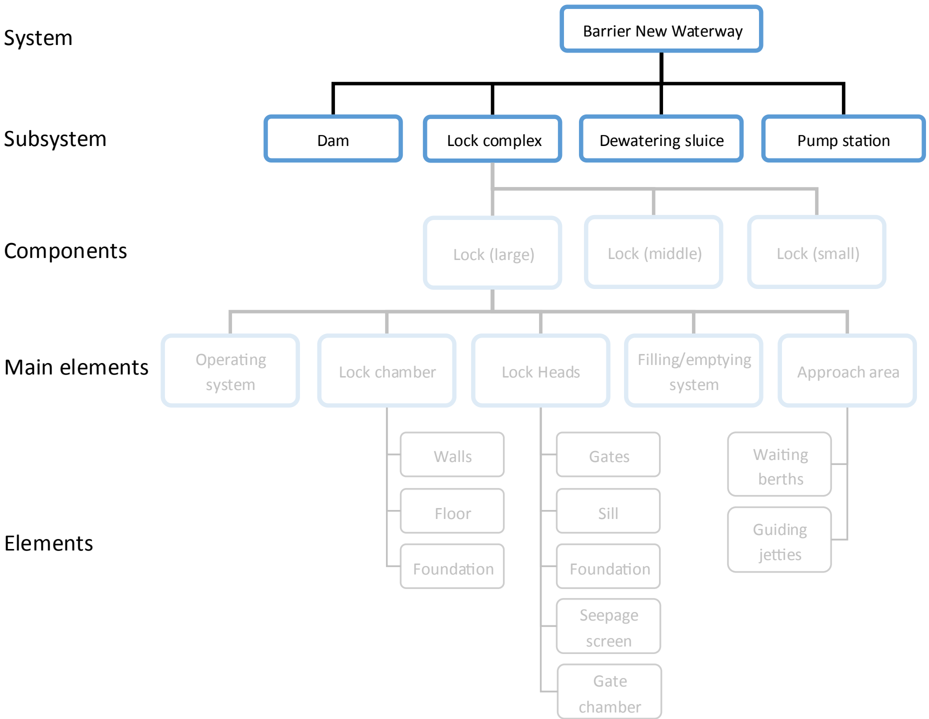



*Figure 8 Design tree: position of the barrier in design process*

When making a design of a hydraulic structure, first of all, the complete system with all its important aspects should be mapped out. In the following paragraphs the south west of the Netherlands are described, as well as parts of the Rhine mouth area. A more thorough investigation of these areas are given in Appendix A. After this is done, the alternatives are composed and evaluated.

### *South west of the Netherlands*

The south west of the Netherlands is described as the water system including the estuaries like the Western Scheldt, see Figure 4. When looking at flood safety, the area is divided into several parts called dike rings. Every dike ring consists of several dike trajectories with a corresponding safety level to ensure enough safety against flooding. One of the most 'famous' dike rings is dike ring number 14, see Figure 74. This dike ring covers most of South Holland, with millions of inhabitants and billions of euros worth of capital.

Both the sea and the river influence the risk level of the area. Therefore an ingenious system of hydraulic structures has been made along the coast to make dewatering from the rivers possible, but also protects the hinterland when high water at sea occurs. One of the best examples of these structures is the Maeslant barrier, whose functioning is subject of discussion in this report. As stated by Spaargaren c.s. the 'plan sluices' might be a more efficient option to protect the Netherlands. This plan provides the idea of closing off the New Waterway permanently.

### *Rhine mouth area*

When zooming more into the Rhine mouth area (Figure 4) more important aspects can be seen other than the ones regarding the safety against flooding. The navigation of vessels is important to take into account. With the presence of the port of Rotterdam, both inland and seagoing vessels visit the area in large amounts. Therefore canals should be wide and deep enough to provide enough space for the vessels to navigate. This demand for navigation can however cause conflicts with the demand to ensure enough safety against flooding for the people in the Netherlands. Therefore a suitable solution should be developed that is adapted to take every requirement into account. Besides of that, the numerous boundary conditions should be taken into account like the existing hydraulic structures in the area.

## 2.1 Method of developing alternatives

Several alternatives are made for the location of the barrier. Important aspects to take into account, when composing these alternatives are discussed below.

These aspects are mainly determined by the functional requirements for navigation and flood safety. For flood safety, they are only evaluated in a qualitative way, because it is very time consuming to get a good quantitative insight in the effects. Therefore this is not done in this phase of the project. For navigation, a rough quantitative estimation of the requirements is made. The performance of the alternatives regarding the requirements need to be compared with the costs of the alternative. The costs are based on the lock configuration, including the dewatering sluice and pumping station, and the length of dikes that need to be strengthened. Out of this the best alternative will be chosen.

The region where the barrier could be build is confined by the fact that the location should be chosen according to the conditions stated in the overlying plan sluices (Rijkswaterstaat, 2015). This means that the location should not be chosen to far landwards, because the benefits regarding flood safety will diminish (longer coast, more unprotected unembanked areas). This limit is chosen to be the Eemhaven, because a location even more landwards will also cause problems regarding spatial availability. On the west side, the region is limited by the existing coastline.


*Figure 9 Boundaries region location choice and important harbour basins*

Numerous locations can be selected in the region to build the complex. To minimize the amount of alternatives to a manageable extent, several considerations are made. First of all the spatial availability is considered. Enough space needs to be available to build the complex without causing too much nuisance for the surrounding area. The required amount of space depends partly on the construction method, but this method is not considered in this stage. Besides of this, when a location in between two harbour basins is considered, only one location is chosen, looking at the environment. Other locations in between the same port areas are assumed to have minor differences.

The assumptions for the spatial requirements for the pumping station, dewatering sluice, the lock configuration and the costs are given below.

### *Pumping station and dewatering sluice*

For the spatial requirements for the pumping station and the dewatering sluice, the IJmuiden lock complex will be used as a reference (Nederlands Gemalen Stichting). This is the largest pumping station in Europe.

| | IJmuiden | | Rotterdam | |
|---|---|---|---|---|
| | Capacity ($m^3/s$) | width (m) | Capacity ($m^3/s$) | width (m) |
| Pumping station | 260 | 50 | 3000 | 580 |
| Dewatering sluice | 700 | 80 | 2000 | 230 |

*Table 1 Dimensions pumping station*

The dimensions of IJmuiden will be scaled to the situation in Rotterdam. A pumping station with a capacity of 3000 $m^3/s$ (van Waveren, et al., 2015) will be considered and a dewatering sluice with a capacity of 2000 $m^3/s$. This results in a width for the pumping station of about 580 meter and for the dewatering sluice a width of 230 meter. This is the total capacity needed for the New Waterway. In case of two different complexes in the New and the Old Meuse, this capacity needs to be divided as well.

## *Lock configuration*

To determine the lock configuration, the passing demand of vessels is needed. In this stage, only a rough estimation is made based on the current situation. To determine the perfect lock configuration, it is recommended to use a traffic simulation for the vessel movements around the lock complex.

Offshore construction vessels often have an unusual shape. Several yards for these vessels are situated east of the proposed locations. They can probably not pass the locks. There are only a few of these vessels. Besides of that, they don't have a determined reason to be so far east in the port of Rotterdam other than the location of the yard of the owner. For these reasons they are not considered any further.

The capacity of the locks will be tested with the following rule of thumb for regular and unregular navigation demands (Glerum, et al., 2000):

$$\text{Unregular offers: } 0{,}55 < I_{wk}/C_{wk} < 0{,}65$$

$$\text{Regular offers: } 0{,}75 < I_{wk}/C_{wk} < 0{,}85$$

Where $I_{wk}$ is the intensity per week and $C_{wk}$ is the capacity of the lock per week. It is assumed that two vessels will be locked at the same time in a lock for inland going vessels and one vessel in a lock for seagoing vessels. A locking cycle is assumed to take 30 minutes for locks for inland going vessels and 45 minutes for a lock for seagoing vessels. The capacity with n being the required number of locks is thus:

$$\text{Locks for inland going vessels: } C_{wk} = 2 * 2/0{,}5 * n * 24 * 7$$

$$\text{Locks for seagoing vessels: } C_{wk} = 2 * 1/0{,}75 * n * 24 * 7$$

**Dimensions lock for seagoing vessels**

For the seagoing vessels it is assumed that one lock is enough to transfer all the seagoing vessels which are too big for the other locks. The governing parameters for the size of the lock is thus the dimensions of the governing vessel. Rules of thumb for the dimensions of the lock are (Glerum, et al., 2000):

- Useful lock length = 1,15*length governing vessel
- Useful lock width = 1,15*width governing vessel

The depth of the lock is not yet quantitatively considered in this stage. The period from 19-01-2016 until 29-04-2016 is considered to determine the governing vessel size for the different port basins. The governing vessels for the different port basins are given below.

| Location | Vessel name | Vessel type | Length (m) | Beam (m) | Draught (m) |
|---|---|---|---|---|---|
| **City centre** | Ovation of the seas | Passenger | **348** | 41 | 8,5 |
| | Aidaprima | Passenger | 300 | **48** | 10,0 |
| **Eemhaven/waalhaven** | Maersk Luz | Container | 300 | 46 | 14,1 |
| **Vondelingenplaat** | Amazon beauty | Oil | 228 | 40 | 12,1 |
| **Botlek** | Mineral Hokkaido | Bulk | **289** | 45 | 18,5 |
| | Arctic | Oil | 273 | **50** | 25,0 |

*Table 2 Governing sea going vessels in the period (19-01-2016 until 29-04-2016)*

**Dimensions lock for inland going vessels**

For the inland going vessels an indicative estimation is made for the amount of locks and their size. It is assumed that this is enough in the current design stage, because the main goal is to compare the different alternatives with each other. Because of the expected amount of locks and the variety of vessels, a more detailed study should be done in the final design stage to make an optimized design.

There is a slight difference in the allowed vessel class between the waterways New Waterway, Old Meuse and New Meuse. The dimensions of the lock for inland going vessels will in all situations however be based on the CEMT class VIb (2*2 push barge). It is expected that this class will be governing for all the waterways in the future. Wider vessels (class VIc) are not widely used, so they can use the lock for seagoing vessels. A standard width for large locks for inland going vessels is 24 meter (Glerum, et al., 2000). This will be used in this approach as well. The length of the lock is determined in such a way that two class Va vessels can be placed in a row in the lock. It is expected that this vessel class is used most common and thus determines the required capacity of the lock. Besides of that, it is the largest class, apart from the push-tow barges. It is thus always possible to place pairs of smaller vessels in the lock as well. The useful lock length for one vessel needs to be 125 meter, so for two vessels the required useful length is 250 m.

### *Costs*

An estimation for the costs of the several elements is given in the table. These costs are mainly based on reference projects and on expert judgment. The given costs are budgetary, so it gives an estimation of the total costs for the structures. The total costs includes the building costs, the governmental procedures, engineering and purchase of land. The costs are however not the total net value of the structures. Besides, the demolishing costs of the old flood defences like the Maeslant barrier and Hartel barrier is not taken into account, because it is not sure when and if they will be demolished. In this phase of a design process, a cost estimation is not always very reliable, because of a lack of detailed information; for instance local soil circumstances, exact dimensions etcetera. Therefore one should be aware that a large band width should be taken into account, in the order of magnitude of 30 %.

| Element | Unit | € mil/unit | Reference |
|---|---|---|---|
| Lock for inland going vessels | lock | 150 | 3rd lock Beatrixlock (270*25 m, € 200 mil) slightly cheaper because of serial construction |
| Lock for seagoing vessels | lock | 600 | New lock IJmuiden (500*70 m, € 820 mil) |
| Dewatering sluice | - | 100 | |
| Pumping station (3000 $m^3$/s) | - | 700 | Estimation Spaargaren c.s. |
| Dam | km | 50 | (Rijkswaterstaat, 2003), scaled to size |
| Strengthening dike | km | 10 | |
| Constructing dike (urban) | km | 100 | |
| Constructing dike (rural) | km | 50 | |
| Dredging land | $m^3$ | 4 (€/$m^3$) | |

*Table 3 Costs elements navigation lock*

## 2.2 Alternatives

Three different alternatives for the closure location are composed, considering the above factors. The first alternative (Rozenburg) is situated just east of the Maeslant barrier. The second alternative lies just east of the junction between the Old and New Meuse and consists of two separate complexes, One in each river, upstream of the junction. The third alternative is similar to the second one, except that the complex in the Old Meuse is situated more upstream, just east of the Beneluxtunnel. The main difference between those locations are the unembanked port areas that are just protected by the barrier. The best location in between two areas are selected based on the spatial availability. As mentioned before, only one location is selected in between two port areas.

NB: To make an estimate of the required amount of locks, the navigation numbers of (van Waveren, et al., 2015) are used. These are the latest available numbers. Compared to earlier research in 2012 however (Ecorys, 2012), large and not logical changes can be seen. The navigation of seagoing vessels in the New Meuse is increased very much for instance. Doubts are therefore stated regarding the reliability of these numbers. A future prediction of the navigation is not yet taken into account in developing the alternatives.

The three alternatives are elaborated in more detail below.

### *Alternative 1 Rozenburg*

The first location is a complex at the west side of the junction of the New Meuse and the Old Meuse. It is also situated west of the Botlek. A connection has to be made between the complex and dike ring 20. This is done via the closure of the Hartel canal. Part of the existing Europoortbarrier and the dike ring 19 is used. A barrier has to be realized through the existing Botlek area to connect the new barrier with the existing dike rings. Because this is a densely built area, the costs for this connection are relatively high. With the formulas for the capacity of the locks, the required number of locks is calculated, see Table 4.

| | Intensity/ year | Required locks |
|---|---|---|
| Seagoing vessels | 33000 | 2,36 |
| Inland going vessels | 57000 | 1,02 |

*Table 4 The calculated required number of locks for alternative one*

The total required amount of locks is 3,38. A total of four locks is assumed for this location; one lock for seagoing vessels and 3 smaller locks for inland going vessels. One lock for seagoing vessels seems to be insufficient for this amount of vessels. Because of the wide variety of vessels however, it is assumed that a large portion of the vessels can use the locks for inland going vessels. This is possible, because these locks have an overcapacity compared to the amount of passing inland vessels.

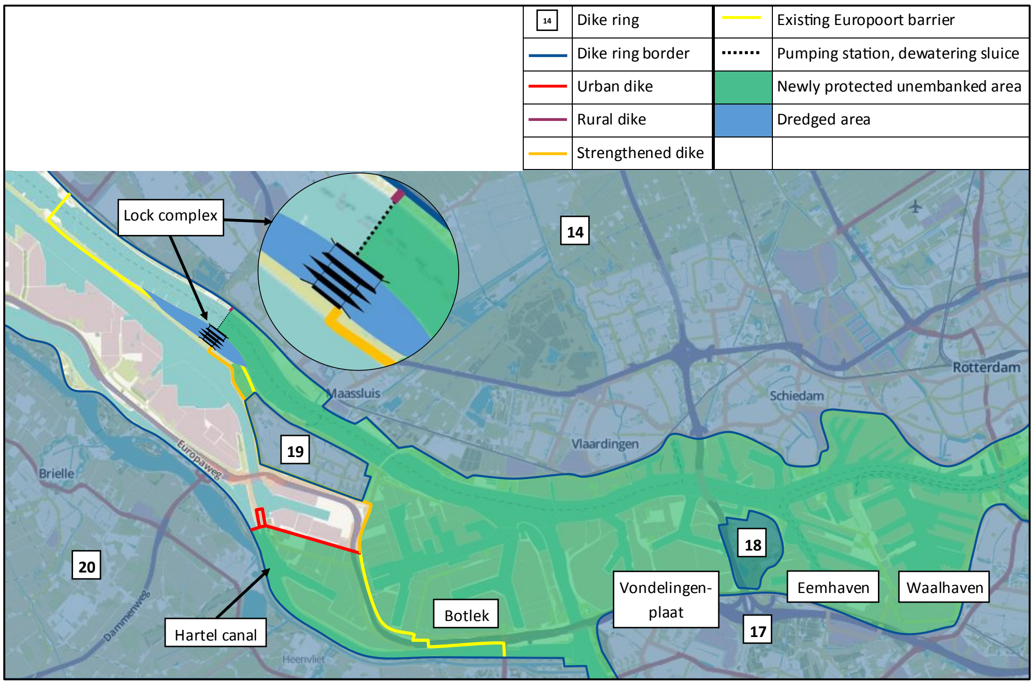


*Figure 10 Alternative one Rozenburg*

The costs for the seagoing vessel lock will be increased with 200 million euros for this location. This is due to the fact that vessels with a relatively large draught visit the port basins in the Botlek area. The depth of a lock can influence the costs for the lock very much.

| | **Description** | **Costs (mil €)** |
|---|---|---|
| Locks for inland going vessels | 3 locks | 450 |
| Lock for seagoing vessels | 400*58 $m^2$ | 800 |
| Dikes (urban) | 2000 m | 200 |
| Dikes (rural) | 50 m | 2,5 |
| Dike strengthening | 2000 m | 20 |
| Dam | 150 m Hartel | 7,5 |
| Dewatering sluice | | 100 |
| Pumping station | | 700 |
| Dredging | 20*250*2500 $m^3$ | 50 |
| **Total** | | **2330** |

*Table 5 Costs alternative 1*

### *Alternative 2 Fusing point New Meuse, Old Meuse*

This alternative is situated exactly at the point where the New Meuse and the Old Meuse come together. Two different complexes will be constructed. One in the New Meuse and one in the Old Meuse. The two complexes will be connected via a new barrier, made along the corner of the Vondelingenplaat. The lock complex in the Old Meuse is connected to the closure in the Hartel canal by the existing Europoortbarrier and through the existing Botlek area. The same as in alternative 1. The required number of locks are calculated using the formulas given before, see Table 6.

| | Intensity/ year | Required locks |
|---|---|---|
| Seagoing vessels New Meuse | 26000 | 1,86 |
| Seagoing vessels Old Meuse | 5000 | 0,36 |
| Inland going vessels New Meuse | 100000 | 1,79 |
| Inland going vessels Old Meuse | 69000 | 1,23 |

*Table 6 The calculated required number of locks for alternative two*

Only one lock for seagoing vessels is considered, namely in the New Meuse. It is not expected that large seagoing vessels enter the Old Meuse. The seagoing vessels that enter the Old Meuse, are considered to be small enough to use the locks for inland going vessels. A total amount of 2 locks in the Old Meuse and 3 locks (one seagoing, two inland navigation) is assumed to be sufficient. For the same reason as in alternative one, only one lock for seagoing vessels is considered (smaller seagoing vessels can use the smaller locks). Furthermore, it is assumed that a portion of the vessels have the option to chance the route to the hinterland without being delayed (the New and Old Meuse congregate again further eastwards). Vessels can thus choice between the two lock complexes.

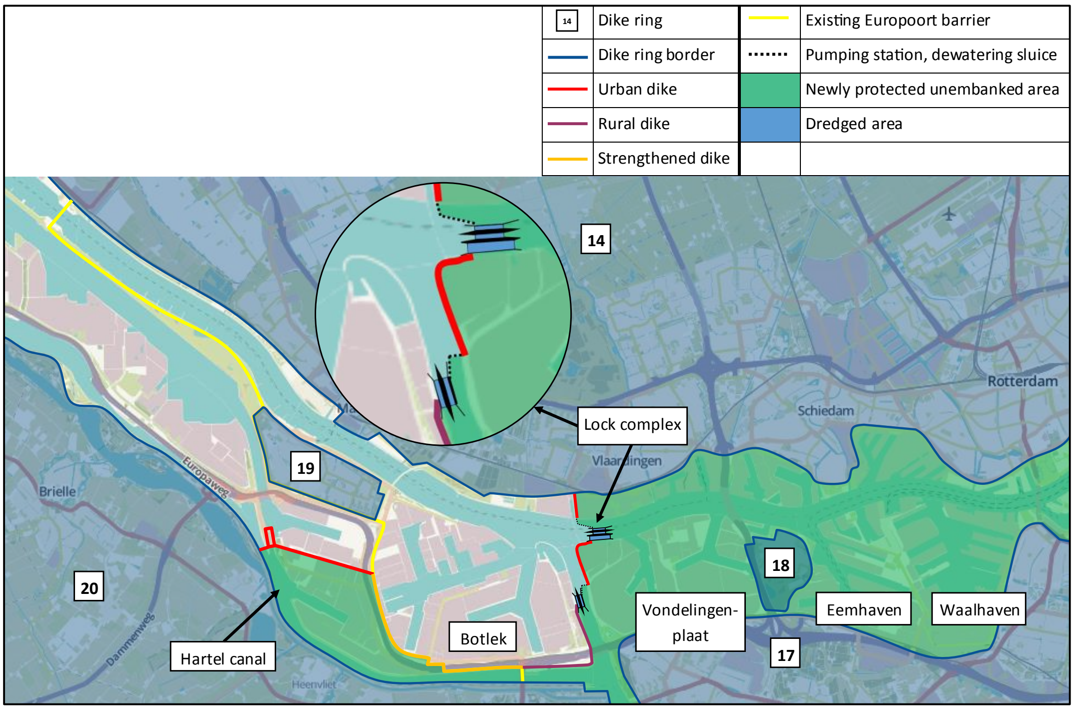



*Figure 11 Alternative two fusing point New Meuse, Old Meuse*

| | Description | Costs (mil €) |
|---|---|---|
| Locks for inland going vessels | Old Meuse: 2, New Meuse: 2 | 600 |
| Lock for seagoing vessels | New Meuse: 400*55 m | 600 |
| Dikes (urban) | 3000 m | 300 |
| Dikes (rural) | 2000 m | 100 |
| Dike strengthening | 4500 m | 45 |
| Dam | 150 m Hartel | 7,5 |
| Dewatering sluice | | 100 |
| Pumping station | | 700 |
| **Total** | | **2452,5** |

*Table 7 Costs alternative 2*

### *Alternative 3 Beneluxtunnel*

The third alternative is comparable to the second one. Two complexes will be built in the New Meuse and the Old Meuse. The difference is the location in the New Meuse. This is next to the Beneluxtunnel. In this way the whole Vondelingenplaat is free accessible from the sea. From the complex a connection will be made with the existing dike ring 17 to dike ring 18. From the lock in the Old Meuse the same connection will be made with the closure of the Hartel canal as in alternative 2. The required amount of locks is calculated according to the formulas given before, see Table 8.

| | Intensity/ year | Required locks |
|---|---|---|
| Seagoing vessels New Meuse | 24000 | 1,72 |
| Seagoing vessels Old Meuse | 5000 | 0,36 |
| Inland going vessels New Meuse | 149000 | 2,66 |
| Inland going vessels Old Meuse | 69000 | 1,23 |

*Table 8 The calculated required number of locks for alternative three*

A total amount of 2 locks for inland going vessels in the Old Meuse and 4 locks (one seagoing, three inland navigation) in the New Meuse are considered. For the same reason as in alternative two it is assumed that this is sufficient. Not all of the seagoing vessels have to use the lock for seagoing vessels, but a large portion of them can use the smaller locks.

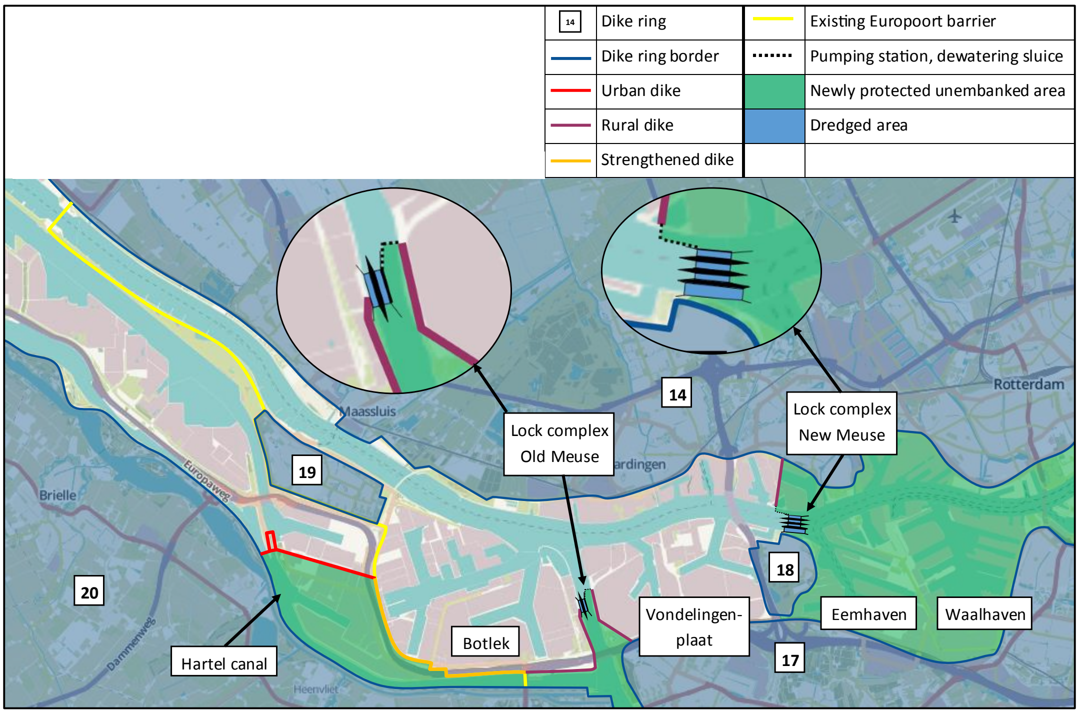



*Figure 12 Alternative three Beneluxtunnel*

| | Description | Costs (mil €) |
|---|---|---|
| Locks for inland going vessels | Old Meuse: 2, New Meuse: 3 | 750 |
| Locks for seagoing vessel | New Meuse: 400*55 m | 600 |
| Dikes (urban) | 2000 m | 200 |
| Dikes (rural) | 3000 m | 150 |
| Dike strengthening | 4500 m | 45 |
| Dam | 150 m Hartel | 8 |
| Dewatering sluice | | 100 |
| Pumping station | | 700 |
| **Total** | | **2553** |

*Table 9 Properties alternative 3*

## 2.3 Multi criteria analysis

To determine the best subsystem alternative out of the three alternatives discussed before, a multi criteria analysis is performed. The most important criteria for this stage are listed. After that, every criteria is weighed, to take the different importance levels of the criteria into account. Subsequently, the alternatives are scored and the best one compared to the costs is chosen.

### Assumptions

Basic assumptions which are used in this stage for the MCA are as follows:

- It is assumed that all the alternatives have the same technical feasibility
- The soil composition is comparable in the region and is therefore neglected in the MCA
- The influences on the ecology system is the same for all the alternatives
- The structures in the neighbourhood like bridges and tunnels are not expected to be demolished soon, thus they will still result in limitations for navigation
- The differences in hydraulic boundary conditions are neglected
- It is assumed that it is possible to implement dike measures in the proposed locations

- The Hartel canal will be closed next to the Rozenburgse lock. It is not proven that this is the best location, but other locations are not taken into account in this study.

### *Criteria*

The most important criteria are listed below. For criteria that are not listed here, it does not mean that they are not important when designing a structure like a navigation lock. The criteria below are the ones that differ due to the different designs and locations and are therefore taken into account.

**Unembanked areas**

The unembanked areas were beforehand only protected by the Maeslantbarrier. By making a lock complex, part of these areas are better protected against flooding. Not every alternative protects the same unembanked areas. This is considered important, because the flood safety levels in the areas are relatively low, while it hosts a high economic value.

**Seagoing/inland Navigation**

Seagoing vessels have to pass the lock complex and will therefore be hindered by the lock. The more vessels that have to pass, the larger the nuisance. This goes for both seagoing and inland vessels. The nuisance for seagoing vessels however, is assumed to be larger, because of their limited manoeuvrability, a higher probability of dangerous situations and a larger passing time.

**Nuisance stakeholders**

During the construction and operation of the complex nuisance for several stakeholders will exist. The vessels are considered separately. The nuisance for neighbouring inhabitants and companies will be considered in this criteria. This nuisance is not seen as an insurmountable problem.

**Spatial integration**

The lock complex has to be integrated in the surrounding area. There should be sufficient space for construction and future expandings. It is also weighed to what extent a complex system is created. In principle, a possible location is chosen for the alternatives, thus the differences for this criteria are probably small.

**User safety**

User safety takes into account to what extent a vessel can pass the lock without dangerous situations. This is mostly related to the configuration of the lock, eg: Small turning circles, narrow passings and vessels crossing each other. In principle every solution can be used, but different safety levels might occur because of the lock configuration.

**Flood safety**

To give a score for flood safety, it is taken into account how much more safety is created by the alternative. This means the length of the dikes to be strengthened are looked at, the protected area, the extended coastline and the complexity of the flood defence system. Every alternative should be feasible as stated in the plan sluices. Therefore this is not the most decisive factor.

**Weighing factors**

The above criteria are given a weighing factor to take the relative importance into account. This is done by comparing them to each other. The most important of two criteria is given the score one, the less important criteria the score zero. The sum of the score, divided by the total amount of points, gives the weighing factor, see Table 10.

| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | Total | Weighing factor |
|---|---|---|---|---|---|---|---|---|---|---|
| Unembanked areas | **1** | x | 0 | 0 | 1 | 1 | 1 | 1 | 4 | 0,19 |
| Seagoing navigation | **2** | 1 | x | 1 | 1 | 1 | 1 | 1 | 6 | 0,29 |
| Inland navigation | **3** | 1 | 0 | x | 1 | 1 | 1 | 1 | 5 | 0,24 |
| Nuisance stakeholders | **4** | 0 | 0 | 0 | x | 1 | 0 | 0 | 1 | 0,05 |
| Spatial integration | **5** | 0 | 0 | 0 | 0 | x | 1 | 0 | 1 | 0,05 |
| Usage safety | **6** | 0 | 0 | 0 | 1 | 0 | x | 0 | 1 | 0,05 |
| Flood safety | **7** | 0 | 0 | 0 | 1 | 1 | 1 | x | 3 | 0,14 |
| Total | | | | | | | | | 21 | 1,00 |

*Table 10 Weighing factors criteria*

### *MCA*

Every alternative is given a score between one and three for all the criteria. The scores for the alternatives are multiplied by the weighing factors. The sum of this gives the total score for the alternative. This score is divided by the total costs in billion euros, which gives the ultimate score to determine the best alternative.

| | | Alternative 1 Rozenburg | | Alternative 2 Fusing point | | Alternative 3 Beneluxtunnel | |
|---|---|---|---|---|---|---|---|
| | Weighing factor | Score | Weighed score | Score | Weighed score | Score | Weighed score |
| Unembanked areas | 0,19 | 3 | 0,57 | 2 | 0,38 | 1 | 0,19 |
| Seagoing navigation | 0,29 | 1 | 0,29 | 2 | 0,57 | 3 | 0,85 |
| Inland navigation | 0,24 | 3 | 0,71 | 2 | 0,47 | 1 | 0,23 |
| Nuisance stakeholders | 0,05 | 2 | 0,10 | 2 | 0,10 | 2 | 0,10 |
| Spatial integration | 0,05 | 3 | 0,14 | 2 | 0,09 | 2 | 0,09 |
| Usage safety | 0,05 | 3 | 0,14 | 1 | 0,05 | 2 | 0,10 |
| Flood safety | 0,14 | 3 | 0,43 | 2 | 0,28 | 2 | 0,28 |
| Total | | | 2,38 | | 1,95 | | 1,85 |
| Costs (billion €) | | | 2,33 | | 2,45 | | 2,55 |
| Score/costs ratio | | | **1,02** | | **0,80** | | **0,73** |

*Table 11 Scores alternatives*

As can be seen in Table 11, the Rozenburg alternative has the highest score/costs ratio. The differences in costs between the alternatives are smaller than the bandwidth of the costs that can be expected in this phase of the design process. When the costs are not taken into account and only the weighed score is considered, the Rozenburg alternative still turns out to be the best location for the barrier. The Rozenburg location is thus chosen to develop in more detail, the other locations are not taken into account anymore.

# 3

# Boundary conditions

With the location for the barrier known, the boundary conditions can be derived. Several types of them are taken into account. These are based on the functional requirements of the lock complex. Only the two main functional requirements are taken into account: making navigation possible and assuring enough safety against flooding. For the latter one, only the function to retain high water is taken into account. The dewatering function will be adopted by the dewatering sluice and pumping station and will therefore not be investigated.

Regarding the ecology, it might be possible that in the future, measures like a fish migration river are desired. These kind of requirements could have a large influence on the total solution of the whole barrier. However, it is not expected that these measures need to be incorporated in the design of the navigation locks at this point. Ecology is therefore also not taken into account, see Figure 13.

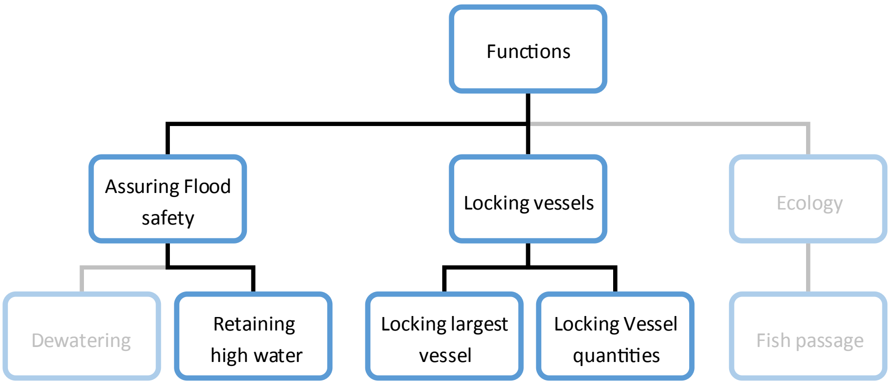



*Figure 13 Investigated functional requirements*

**Adaptive design approach**

By elaborating the boundary conditions, the adaptive design approach can be taken into account. Therefore for every functional requirement, two different kind of requirements are worked out. One for the initial design, using scenarios which are expected the most. The other one will be used for the adaptive design and will be made up out of more extreme scenarios. The requirements for the adaptive design will be more stringent than for the initial design. In reality this is not always the case. Less stringent boundary conditions can develop in the future like a smaller quantity of vessels or less sea level rise. A structure might be adapted in this case to ensure smaller maintenance costs. However, it is expected that a structure will not be adapted when this is the case because, however the structure is over dimensioned, it can still meet the requirements.

First of all the hydraulic boundary conditions in the extreme design situation are derived. This consists of several parts: the water level and the waves on the seaside of the structure and the water level on the inland side of the structure. The water level on the seaside consists of several parts as well: the storm surge seiches and the tide. Furthermore, the water levels for which locking should be available are computed. Subsequently, the demands for the vessel sizes and the vessel quantities are derived.

## 3.1 Hydraulic load seaside

The hydraulic load on the structure consists of several parts. These are the average water level + storm surge, seiches, tide and waves. The hydraulic loads can be static and dynamic. All of the above are static loads, except for the waves. In fact the tide and seiches are also waves, these however, develop slowly and the load will have a static character.
The amplitude of the tide will not chance in the future, only the still water level will probably change due to sea level rise.

**Storm surge**

The storm surge that should be taken into account depends on two aspects. The probability of exceedance of a particular storm and the social-economic situation of the flooded land. The social-economic situation determines what the safety against flooding should be and thus the according probability of exceedance of a storm condition. This storm condition coincides to a certain storm surge level. A future change of storm conditions are not taken into account. The natural variation of storms is very large. Therefore a small development in the occurrence of storms is very hard to determine and for now negligible compared to the natural variation of the storms (KNMI, 2014). It is however not sure how the social-economic situation will change. In the Deltascenarios (Bruggeman, et al., 2013), developments are taken into account with a large growth of population and economy. This would mean that the flood safety level should increase in order to keep the individual and economic risk at the same level. At this moment the governing water level has a probability of exceedance of 1/10.000 for the Maeslant barrier. This requirement may be more stringent in the future due to the above reasons. If for instance the requirement will grow to a return period of 100.000 year, the governing water level can increase with about one meter, see Figure 14.

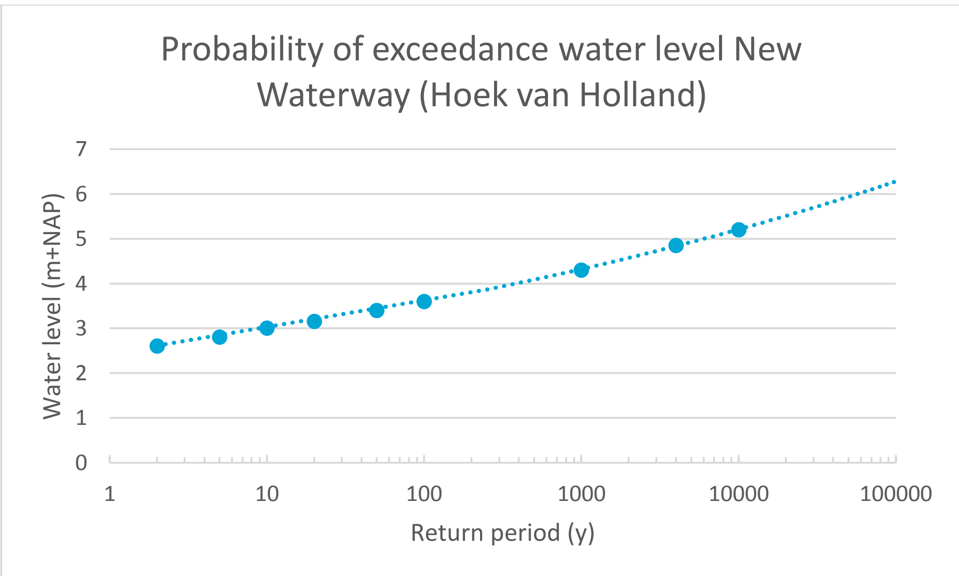



*Figure 14 Probability of exceedance of the water level in the New Waterway at Hoek van Holland (Rijkswaterstaat, 2015) (Chbab, 2012)*

The governing water levels for the year 2100 within the Deltascenario Stoom are calculated for the closure of the New Waterway by (Botterhuis, et al., 2015). For Hoek van Holland this is 5,9 m + NAP, including sea level rise (5,05 m + NAP without sea level rise). A more stringent requirement for the return period is however not yet taken into account (assumption is 1/10.000). To quantify the magnitude of the social economic effects is not easy. It is assumed that this will rise to 1/30.000 in an extreme situation. This means an increase of the governing water level of about 0,5 meter.

**Waves**
Because of the same reasons as for the storm surge, it is not expected that the wave conditions will change. The hydraulic boundary conditions (hr2006) determined for the New Waterway are considered (Rijkswaterstaat, 2007). This gives a significant wave height of 6,95 meter, wave period of 10,9 s and an angle of 318°, determined clockwise from the north direction. These are however boundary conditions for the mouth of the New Waterway, which is situated more westwards. Because of diffraction and refraction in the relatively narrow river, the waves will be reduced before it can reach the considered location. The wave conditions are thus considerably smaller at the structure itself. They are therefore not taken into account in the further design.

**Sea level rise**
The estimated sea level rise for the coming years until 2100 are described in the Deltascenarios. The most extreme scenario is Stoom, with an estimated sea level rise of 85 cm for 2100. The current strategy is to use this scenario when designing new flood defences (Deltaprogramma, 2016). Therefore this value is used in the initial design of the closure of the New Waterway. This is however not considered to be the ‘worst case’ scenario. Thus to take the adaptive approach into account, another scenario is used. This is the scenario made by the second Deltacommittee Veerman. An estimated sea level rise of 1,2 meter is derived in this scenario for the year 2100 (Veerman, 2008).

**Seiches**
A seiche is an oscillatory water level rise in a basin. These waves have a period from a few minutes to a few hours and can have a large influence on the hydraulic loads. Because the New Waterway is closed on three sides, it will act as a basin. Therefore the seiche phenomenon has to be taken into account when designing the lock complex. The influence of seiches in the current situation is investigated by (van Reijmerink, et al., 2014). The current situation however, includes the Maeslant barrier. The new situation with the lock complex will be slightly different. The complex is situated 4 kilometer to the east from the Maeslant barrier. The basin will therefore be increased in length. It is assumed that the influence of seiches will be increased as well. A detailed calculation will not be made. The highest amplitude of the seiche wave normally occurs at the closure or end of the basin. The influence of seiches is given in the net seiche effect. Not only the shape of the basin is taken into account, but also the amount of reflection and damping of the embankment. This differs per water level, because of the amount of overtopping of barriers. In the current situation the net seiche effect (at the Maeslant barrier) is 0,37 m for a water level with a probability of occurrence of 1/4.000 and 0,31 m for a water level with a probability of occurrence of 1/10.000. The reason for this difference is the overtopping of the dikes when a higher water level occurs. The maximum value of 0,37 will be used further. This value is increased with 0,1 meter for the initial design, because of the different location of the newly build complex (with a

larger basin). This results in a net seiche effect of 0,47 meter. This value should be added to the governing storm surge, together with other add-ons for the water level.

The seiches are subject to a high level of uncertainty. Compared to earlier research, changes in the net seiche effect can be seen of about 0,1 m. Apart from that, differences in lay-out of the port basins (like the Maasvlakte 2) influences the net seiche effect. This influence can be both positive or negative (higher or lower effect). A lower effect will not cause problems for the structure or flood safety in the future. A higher effect however can cause a higher load level. Therefore, for the adaptive approach, another 0,1 meter is added to the 0,47 net seiche effect mentioned before. This is based on the order of magnitude of the influence of the Maasvlakte 2.

Resuming, two scenarios will be taken into account:

1. A net seiche effect of 0,47 meter, as in the current situation
2. A net seiche effect of 0,57 meter, taken into account future developments

**Low water levels**

Low water levels can cause problems regarding the availability of the lock complex, depending on the type of filling system and the sill level. In Table 12 the return periods for low water levels at Hoek van Holland are given for the current situation. In all of the scenarios an expected sea level rise can be seen. Therefore it is expected that the return frequency of the low water levels will not increase in the future, but decrease. Besides, it is considered that the storm conditions will probably not change in the future. The development in low water levels are thus only influenced by the sea level rise. This means that at t=0, the possible problems regarding the availability of the lock complex are the largest. Therefore the development of different future scenarios is not executed.

| Return period (y) | Water level (m+NAP) |
|---|---|
| 1 | -1,50 |
| 10 | -1,85 |
| 100 | -2,15 |

*Table 12 Return periods low water levels Hoek van Holland (Rijkswaterstaat, 2015)*

**Seaside hydraulic load scenarios**

All of the above aspects are combined together into two different scenarios. One for the initial design and one for the adaptive design. These scenarios are made for the horizon year of 2100. However, they are not design loads. Aspects like safety factors, add-ons for uncertainty and robustness are not yet taken into account. The governing hydraulic loads are given in Table 13.

| | Initial scenario | Adaptive scenario |
|---|---|---|
| **Dynamic** | | |
| Wave height (m) | 6,95 | 6,95 |
| Wave period (s) | 10,9 | 10,9 |
| **Static** | | |
| Max storm surge (including tide) (m+ NAP) | 5,05 | 5,55 |
| Net seiche effect (m) | 0,47 | 0,57 |
| Sea level rise (m) | 0,85 | 1,2 |
| **Total** | **6,37** | **7,32** |

*Table 13 Hydraulic load scenarios for the horizon year 2100*

## 3.2 Hydraulic load inland side

**High water level**

For the inland side of the lock complex the governing water levels calculated by (Botterhuis, et al., 2015) are used. This is a water level of 2,9 m + NAP, with the usage of a pumping station. A river discharge of 18.000 $m^3/s$ for the Rhine and 4.600 $m^3/s$ for the Meuse. These are currently the most extreme predictions. When this or an higher discharge happens, it is likely that a flood more upstream will occur. This means that the extreme discharge in the downstream delta will not further increase. Therefore other scenarios are not taken into account regarding the water level on the inland side of the lock.

**Low water level**

low water levels can occur due to low discharge levels. The water levels can be controlled in a better way thanks to the newly build lock complex. Problems can arise when a low water level at the seaside occurs for a longer time. Due to the locking losses, the water level at the inland side will change into the same level at the seaside. This causes problems regarding navigation and availability of the navigation lock. The extreme low water levels at seaside occurs however only for a short period of time, because of the changing tide. Therefore it is not expected that this situation will cause problems. With a higher seaside water level, the inland water level can be controlled by dewatering seawater into the New Waterway. This is however not preferred due to the salt intrusion. The same extreme low water levels will be used as considered for the seaside. This is a conservative assumption.

## 3.3 Locking water levels

Regarding the availability of the navigation locks, it is important to know which water levels should be taken into account for which locking still is possible. It is considered that locking should be possible for water levels that occur once in every ten years. These water levels will increase in time, due to the expected sea level rise. For the adaptive scenario, a larger range is assumed. The current low water level will stay the same, while the high water level will increase even more due to seiches and sea level rise, see Table 14.

| | Initial (2010) | Initial (2100) | Adaptive (2100) |
|---|---|---|---|
| Low water (m + NAP) | -1,85 | -1 | -1,85 |
| High water (m + NAP) | 3 | 3,85 | 4,3 |

*Table 14 Extreme water levels for which locking should be possible*

## 3.4 Navigating vessels

The Deltascenarios will be used to determine the amount of vessels for the future. Predictions have been made for both the throughput in tons at the Dutch ports and the inland navigation (van Dorsser, 2015). This throughput will be translated to a passing vessel quantity. A wide variety in predictions can be seen for the year 2100.

*Seagoing vessels*

The average between the predictions for the throughput of the Dutch ports will be used for a first estimate. This will give a throughput of 858 million tons for 2050 and 789 million tons for 2100. The highest average of the two predictions will be used, thus the one for 2050. Compared to the most extreme scenario Stoom for 2100 a difference of 381 million tons can be found. These numbers will be used to predict the quantity of sea-going vessels. The quantity of vessels passing the considered location will not grow with the same factor as the throughput numbers. Several reasons can be named for this. First of all, the growth of throughput will probably occur together with the expansion of the port area. It is expected that new port expansions will take place seaward on the west of the existing port areas, like the Maasvlakte 2. Therefore the navigation of sea-going vessels through the lock will not increase due to this. The only increase might occur due to the fact that the full capacity of the existing port areas will be used or that the efficiency of cargo handling is increased and therefore the capacity. Due to these factors, only a slight increase of the quantity of passing sea-going vessels of 1,1 is assumed. For the most extreme prediction an increase of 1,5. In this case, because of the high demand of transport, it is expected that the trade port area will also be expanded landwards of the lock. This results in more passing vessels of the navigation lock.

| | Reference (2000) | Average (2050) | Maximum (stoom 2100) |
|---|---|---|---|
| Total throughput (million tons) | 424 | 858 | 1238 |
| Throughput growth factor | 1 | 2,02 | 2,92 |
| Seagoing vessel quantity growth factor | 1 | 1,1 | 1,5 |
| Passing vessels New Waterway (n/year) | 33.000 | 36.300 | 49.500 |

*Table 15 Growth sea-going vessels*

The reference level for the amount of passing vessels is 2014, the numbers for this are given in (van Waveren, et al., 2015). For the initial design, the average estimate for 2050 is used (36.300 passing vessels/year). For the adaptive design, the maximum estimate of 49.500 passing vessels/year is used.

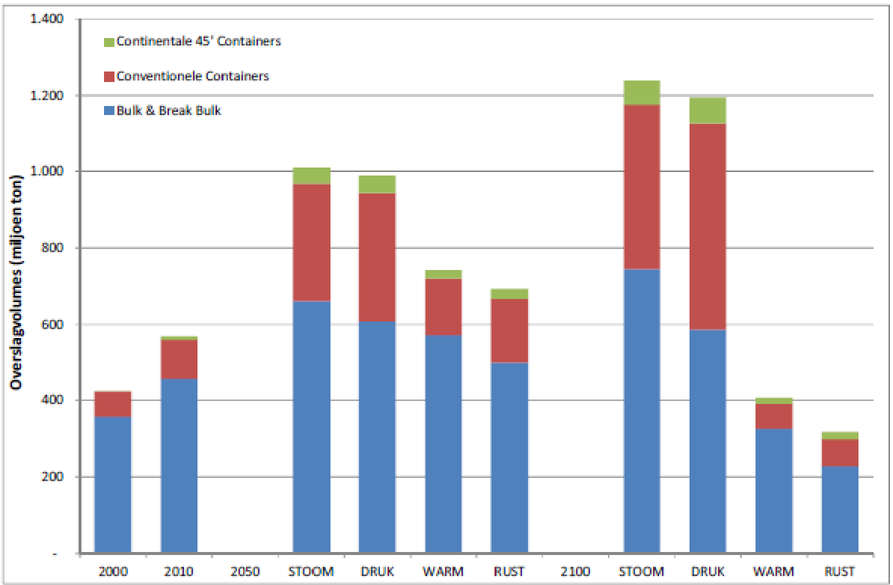



*Figure 15 Scenarios for the throughputvolumes for the dutch ports in million tons (van Dorsser, 2015)*

*Inland vessels*

The same approach as for the Dutch ports will be used for the inland navigation transported volume. For 2050 the average volume is 411 and for 2100 it is 365 million tons. Again the highest of the two averages is the one for 2050. This gives a difference with the highest scenario for 2100 (Druk) of 239 million tons.

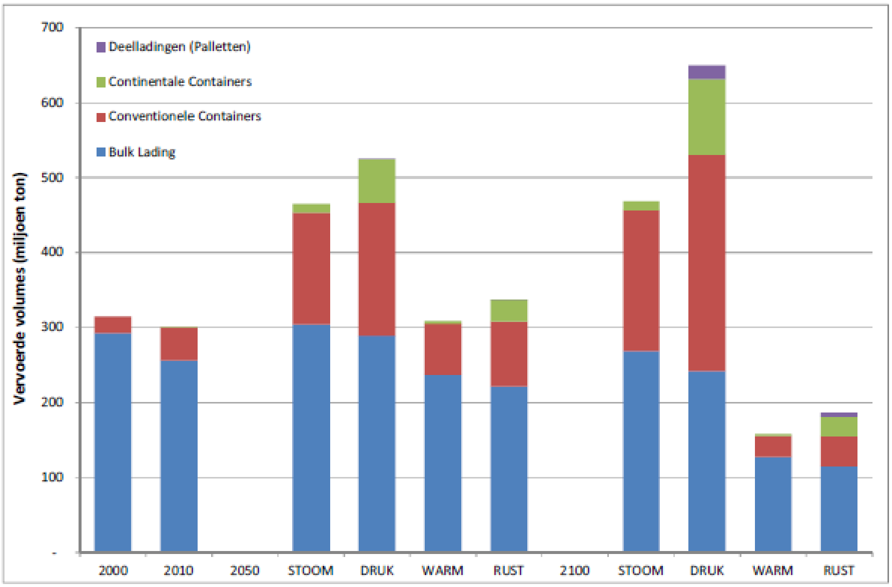



*Figure 16 Scenarios for the transported inland volume in million tons (van Dorsser, 2015)*

A clear relationship between the vessel quantity and the throughput volume is not easy to make. The inland vessel fleet composition can change and thus the transported volume/vessel quantity ratio will be different. It is considered that the maximum dimensions will not change. The reason for this is, that the growth of vessels is restricted by the size of the waterways and structures like bridges and locks. The portion of maximum vessels can be larger however; less smaller vessels, more larger vessels. This scaling-up of the vessels is confirmed by (Quist, et al., 2010). To make an estimate for the increase in vessel quantity, the following approach is used. First of all the composition of the Dutch inland vessel fleet is determined from (Buck Consultants International, 2008). Subsequently, a new fleet composition is made for 2050 and 2100 by keeping the total capacity of the fleet/transported volume ratio constant. The transported volume/vessel quantity ratio is increased slightly with a factor 1,17, which means a scaling-up of the fleet. This approach will eventually give a factor for the increase in quantity of the vessels in the Netherlands.

| | Reference (2000) | Average (2050) | Maximum (druk 2100) |
|---|---|---|---|
| Total shipped volume (million tons) | 315 | 411 | 650 |
| Vessel quantity | 4440 | 4945 | 7850 |
| Vessel quantity growth factor | 1 | 1,11 | 1,77 |
| Passing vessels New Waterway (n/year) | 57.000 | 63.000 | 101.000 |

*Table 16 Vessel quantity growth*

The above numbers are for the whole Dutch fleet. It is however considered that the vessels in the New Waterway will grow in the same manner.
The reference level for the amount of passing vessels is 2014, the numbers for this are given in (van Waveren, et al., 2015). For the initial design, the average estimate for 2050 is used (63.000 passing vessels/year). For the adaptive design, the maximum estimate of 101.000 passing vessels/year is used.

**Maximum vessel size**

*Sea-going vessels*
The size of sea-going vessels is hard to predict. The current trend in the world is that still larger vessels are being built. It is however not sure to what extent this will continue. The current sea port areas behind the lock already have restrictions for the maximum vessel size. Before a larger vessel can visit, the ports have to be adapted. The basins have to be widened or deepened. This is a large investment and it is not likely that this will be cost-effective in the future compared to building new port areas seawards. This is however not the same for cruise vessels. They always have the desire to moor in the city centre for commercial and practical reasons. In other cities, like Amsterdam, there are restrictions as well. Besides of that, Rotterdam is not a location that is very often visited by cruise vessels (32 ships in 2015). For these reasons, the expectation is that the maximum cruise vessel size will not increase as well. Furthermore, cruise ships usually have a high level of manoeuvrability, so when occasionally a large vessel have to pass the lock, smaller restrictions can be allowed.

In the initial design the current governing vessel will be used for the design of the lock for seagoing vessels. Currently, the largest vessels passing the New Waterway have more or less the same dimensions as the Panamax class (l*b*d = 366*49*15,2 m). These dimensions will be used in the initial design.
For the adaptive design, a larger vessel will be taken into account, namely the Malaccamax class, which is the largest ship able to pass the strait of Malacca. This vessel has a length of 400 m, a width of 59 m and a draught of 20,5 m (Maritime connector). The latter one is however decreased to 18 m. It is considered that larger vessels can pass the lock after lightering in the deeper ports.

*Inland vessel size*
The only possible expansion in vessel size is the expansion to the largest CEMT classes VIc and VIIb with three barges next to each other and a width of 34,2 meter. When these larger vessels have to pass, they can use the sea-going lock. This can cause problems regarding the capacity of the lock, so this is only possible when it happens occasionally. The push-tow barges are used for transporting bulk goods. In the current situation, the bulk port areas are mostly reached from the hinterland through the Hartel canal. The demand in the New Waterway therefore depend highly on how and where the Hartel canal is closed. Whatever the situation, a wide lock capable of locking VIc and VIIb vessels is not likely to be built in the Hartel canal. The vessels will thus have to pass the lock complex in the New Waterway. If the amount of these push-tow barges grows in the future, the existing sea-going lock is not sufficient anymore. The lock complex has to be expanded by building a new lock or enlarging one of the existing locks.
Considering the above developments, two different scenarios have been made which are taken into account:

1. The CEMT classes VIc and VIIb push-tow barges can use the larger sea-going lock without hindering the sea-going vessels.
2. The amount of passing CEMT classes VIc and VIIb push-tow barges is too much and a new lock has to be built or one of the existing lock have to be enlarged.

The first scenario is the most likely to occur. Therefore this one will be taken into account in the initial design. The second one will be used as an extreme scenario that can occur later in time.

## 3.5 Adaptive boundary conditions

Summarizing, the requirements given in Table 17 are taken into account to make an adaptive design. In reality, it is not likely that these phenomena will all happen at the same time. In order to make a good approximation of the combination of the effects, a probabilistic approach should be used. This is however not done in the scope of this project, because this is very time-consuming.

| Requirement | Initial | Adaptive |
|---|---|---|
| **Navigation** | | |
| Quantity seagoing vessels (n/year) | 36.300 | 49.500 |
| Quantity inland vessels (n/year) | 63.000 | 101.000 |
| Size seagoing vessels (l*b*d) (m) | 366*49*15,2 | 400*59*18 |
| Size inland vessels (l*b*d) (m) | VIb: (195*22,8*4,5) | VIIb: (285*34,2*4,5) (one lock) |
| | | |
| **Seaside hydraulic loads** | | |
| Max storm surge (including tide) (m + NAP) | 5,05 | 5,55 |
| Net seiche effect (m) | 0,47 | 0,57 |
| Sea level rise (m) | 0,85 | 1,20 |
| **Total governing high water (m + NAP)** | **6,37** | **7,32** |
| Governing low water level (m + NAP) | **-1,50** | **-1,50** |
| | | |
| **Inland side hydraulic loads** | | |
| Governing high water level (m + NAP) | **2,90** | **2,90** |
| Governing low water level (m + NAP) | **-1,50** | **-1,50** |

*Table 17 Adaptive and Initial requirements*

The above scenarios can result in different decisions. When the adaptive or an even more extreme scenario occurs, one can choose to adapt the existing lock complex. This means enlarging the existing locks. Another option is to keep the existing locks intact and build a new lock with the new requirements regarding vessel size and capacity. Both of the decisions will be taken into account when making the adaptive design.

**Other adaptive functional requirements**

Besides the above requirements regarding navigation and retaining water, other functional requirements can develop in the future which are not present in the current situation. These will only be evaluated qualitatively.

*Salt intrusion*

Requirements to prevent salt intrusion will not be taken into account for the initial design. The salt intrusion will already become less by closing the New Waterway compared to the current situation. It can however happen that the requirements will become more stringent and that measures have to be taken to prevent (too much) salt intrusion. It will be taken into account in the adaptive approach that some kind of measures need to be necessary.

*Dewatering through the lock*

It is possible that the level of extreme high discharges will increase in the future. This may cause capacity problems for the designed dewatering sluice. Therefore a future requirement could be that the navigation locks need to be used for dewatering by opening both gates when a high discharge occurs. This means that the gates need to be able to close and open in flowing water. Also, the bed protections need to be able to handle the high velocities in the navigation locks.

*Road connection*

It is possible to use the walls and the heads of the locks to connect the two river banks with each other by a local road. This is also the case with the lock complex in IJmuiden. In the New Waterway, this connection can replace the existing ferry between Rozenburg and Maassluis. On the other hand, a new tunnel is planned nearby (Schultz van Haegen, 2016), making this road connection probably less useful. This is however a highway. The possible demand for this local road connection is therefore taken into account, but the technical requirements regarding this are not further elaborated.

# 4

# Lock and barrier configuration

With the location and the boundary conditions derived, the overall solution can be derived on the system level for the barrier. This solution consists of several parts: the lock complex, the dewatering sluice, the pumping station and the dam. See Figure 17. The combination of those parts result in the barrier. In this chapter, the focus is on the lock complex. For the other parts, the dimensions as derived in chapter 2 will be used and are therefore not taken into account any further.

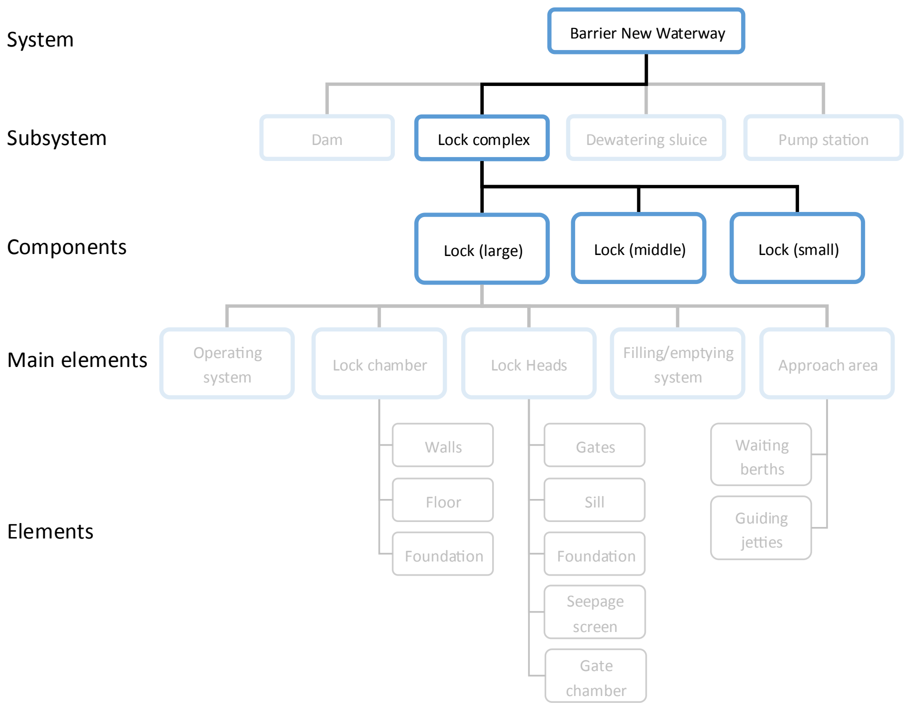



*Figure 17 Design tree, position of components in the design process*

The lock complex consist out of several components, to be known the different locks with varying sizes, see Figure 17. First of all, the required amount of locks for the different sizes is determined. Subsequently, the dimensions for the locks are derived. By combining the above factors, an overall solution for the barrier can be made. When the dimensions are known, partial solutions for the different components (gate, wall, floor, filling/emptying system) of the locks are chosen. This choice is based on different aspects. First of all, the technical feasibility of the principal solution for the desired element is considered. Next to this, the feasibility of the solutions regarding the adaptive boundary conditions are taken into account. A more thorough elaboration on each considered principal solution is given in Appendix B.

To make an overall solution, the principal solutions are combined with each other. The best option is composed taking the adaptive requirements and technical feasibility into account.

When designing a main barrier, several checks should be made regarding the safety of the barrier. These are the reliability of closing the barrier (closing the gates of the locks), the amount of allowed overtopping and leakage, the stability of the structure and the structural safety of the structure. To quantify this, an extended fault tree should be made, where all these factors are taken into account. When looking at similar reference projects like the lock in Terneuzen and in Ijmuiden, it is concluded that it is possible to ensure enough safety, according to the Dutch standards. Therefore, these aspects will not be further worked out in detail.

## 4.1 Vessel distribution

The lock complex consists out of several locks. The composition of seagoing vessels is very diverse. Therefore it is not efficient to make all the lock chambers the minimum size according to the largest vessel. Three different lock sizes are thus considered. A composition of the vessel sizes in the total fleet passing the New Waterway according to (Ecorys, 2012) is used, see Table 18. With the given estimates of the total intensity of passing vessels/year, the intensity of passing vessels for each vessel type can be derived.

| Vessel type | Portion (%) | Initial intensity/year | Adaptive intensity/year |
|---|---|---|---|
| <120 m | 25 | 9075 | 12375 |
| 120-200 m | 35 | 12705 | 17325 |
| 200-300 m | 15 | 5445 | 7425 |
| >300 m, depth <14,3 m | 4 | 1452 | 1980 |
| RoRo 215 m, depth 7 m | 18 | 6534 | 8910 |
| Rest | 3 | 1089 | 1485 |
| Total seagoing vessels | 100 | 36300 | 49500 |
| | | | |
| Total inland going vessels | | 63000 | 101000 |

*Table 18 Passings per vessel type*

With the capacity formulas given in chapter 2.1, the required number of locks per vessel type can be calculated (Glerum, et al., 2000).

A locking cycle is assumed to take 30 minutes for the small locks for inland going vessels and 50 minutes for the large and mid-sized locks.

The vessels are divided over the three different lock sizes, to use them as efficient as possible. Thus with the smallest amount of locks and smallest lock sizes as possible. This will give a required number of locks for the three different sizes. See the following tables for the results for the initial scenario and the adaptive scenario. With:

Req. cap. = The required capacity for a certain vessel type per week.

$n_{vessel}$ = The amount of vessels in a lock, depending on the size of the vessel and the size of the lock.

$n_{lock}$ = The amount of locks required for a certain vessel type. The summation of the required locks for the different vessel types gives the total required amount of locks.

| | | Large | | Middle | | Small | |
|---|---|---|---|---|---|---|---|
| | Req. Cap. | $n_{vessel}$ | $n_{lock}$ | $n_{vessel}$ | $n_{lock}$ | $n_{vessel}$ | $n_{lock}$ |
| <120 m | 249 | | | 2 | 0,15 | 1 | 0,19 |
| 120-200 m | 349 | | | 1 | 0,43 | 1 | 0,26 |
| 200-300 m | 150 | 1 | 0,37 | | | | |
| >300 m, depth <14,3 | 40 | 1 | 0,10 | | | | |
| RoRo 215 m, depth 7 m | 157 | 1 | 0,39 | | | | |
| remaining | 30 | 1 | 0,07 | | | | |
| Inland | 1514 | | | | | 2 | 1,13 |
| **Total** | | | **0,93** | | **0,59** | | **1,57** |
| **Number of locks** | | | **1** | | **1** | | **2** |

*Table 19 Required number of locks for the initial design*

In the initial design, a lock complex with a total of four locks is proposed: one large lock, one mid-sized lock and two small locks for inland going vessels (see Figure 18). The two smallest seagoing classes (<120 m and 120-200 m) are divided between the small locks and the mid-sized lock. It is assumed that 50% of these classes use the mid-sized lock and the other 50 % use the small locks. In reality this can vary, but both lock sizes have enough capacity to cope with a substantial variation. The proposed amount of locks in the adaptive design is given in the table below.

| | | Large | | Middle | | Small | |
|---|---|---|---|---|---|---|---|
| | Req. Cap. | $n_{vessel}$ | $n_{lock}$ | $n_{vessel}$ | $n_{lock}$ | $n_{vessel}$ | $n_{lock}$ |
| <120 m | 340 | | | 2 | 0,42 | | |
| 120-200 m | 476 | | | 1 | 1,18 | | |
| 200-300 m | 204 | 1 | 0,25 | 1 | 0,25 | | |
| >300 m, depth <14,3 | 54 | 1 | 0,13 | | | | |
| RoRo 215 m, depth 7 m | 214 | 1 | 0,53 | | | | |
| remaining | 41 | 1 | 0,10 | | | | |
| Inland | 2428 | | | | | 2 | 1,81 |
| **Total** | | | **1,02** | | **1,86** | | **1,81** |
| **Number of locks** | | | **1** | | **2** | | **2** |

*Table 20 Required number of locks for the adaptive design*

It can be seen that the vessel types are divided in a different way in the adaptive design. In the initial design, one large lock, one mid-sized lock and two small locks can fulfill the requirements. In the adaptive design, another mid-sized lock is needed, while the other locks are used more, see Figure 18 for a proposed lock configuration. The large lock will probably be at maximum capacity. It can also be considered to enlarge one of the small locks for the adaptive design. This will however result in a required maximum capacity for almost all of the locks, unless the small lock is enlarged very much (like the size of the large lock). This is therefore probably not cost-efficient compared to building a new lock and will not be taken into account any further.

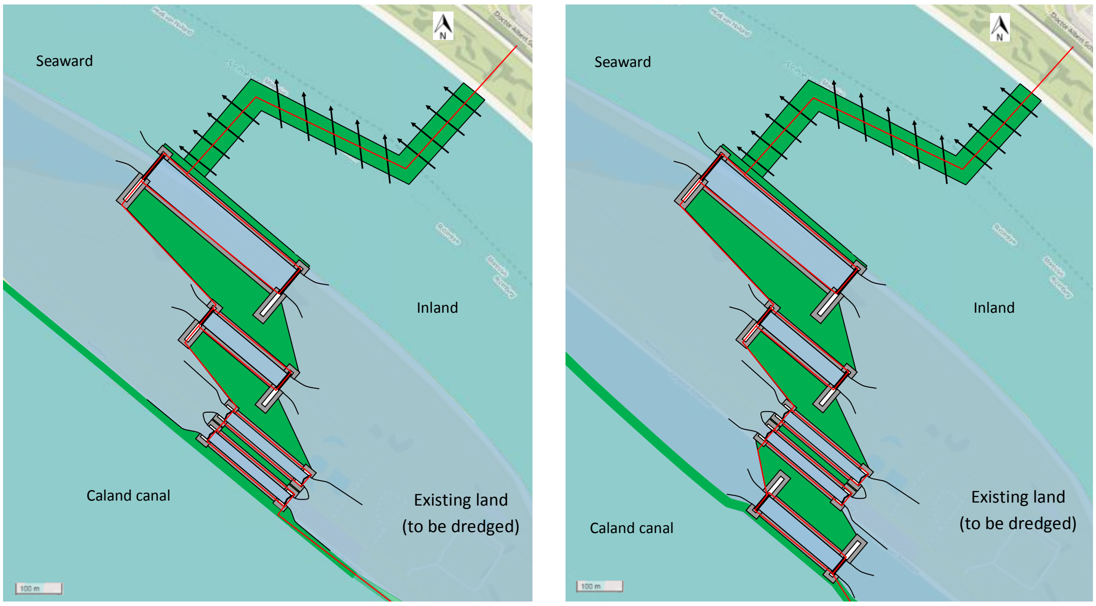



*Figure 18 Lock configuration, left: initial design, right: adaptive design*

## 4.2 Large lock

With the lock configuration known, the solutions for the different locks can be determined, starting with the large lock. The principal solutions given in appendix B are considered for the different elements.

When looking at the principal solutions, It is not easy to make such a design that every adaptive requirement can be implemented in the future. Therefore, a portion of these requirements will be implemented immediately in the initial design. This will result in higher costs. It is qualitatively determined whether this is acceptable. One of the most important reasons that enlarging the lock later on is not feasible is the availability of the lock. Certain types of work require a large down-time of the lock. Because there is only one lock for seagoing vessels, this is unacceptable.

**Wall and filling/emptying system**

The above problems regarding adaptivity are mainly the case for the replacement of the lock chamber walls. Therefore the adaptive requirement for the width of the chamber will be used immediately. This results in a higher amount of water that has to be locked every time and a larger chamber floor. The increase in water results in longer locking times, which is not preferable. To compensate for this, the relieving floor with longitudinal culverts will be used as a filling system. It is expected that this option results in lower filling and emptying times than openings in the gate. This is beneficial for future requirements regarding the capacity and locking levels as well. The principle of the filling/emptying system is given in Figure 19.

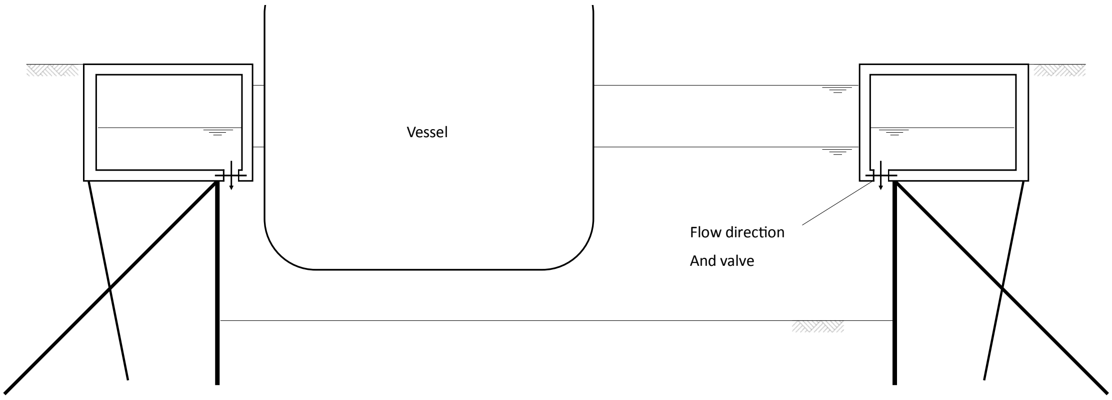



*Figure 19 Cross section of a lock with a longitudinal filling/emptying system through the relieving floor*

A small calculation gives an order of magnitude of the size of the culverts on top of the floor. A chamber size of 420*68 m and a maximum head ΔH of 3 meter gives a total water to be locked of 85680 $m^3$. With a locking time of 5 minutes an average discharge of 286 $m^3/s$ is required.
Using Bernoulli's law, the required cross sectional area of the culvert can be calculated:

$$A = \frac{Q}{\mu\sqrt{2 * g * \Delta H}} = 105\ m^2$$

With an assumed discharge coefficient μ of 0,5. The head difference changes in time. For now, an averaged difference of 1,5 m is assumed. An area of 105 $m^2$ is probably possible to incorporate in both the chamber walls.
A detailed design of the relieving floor in combination with a longitudinal culvert will be given in chapter 6.

**Lock head**
A straight rolling gate will be used in the design, because the gate recess can easier be adapted than the arched rolling gate. The initial gate will be made according to the initial requirements regarding depth and width. The required adaptive width will be taken into account in such a way that it can be adapted later on. The adaptive depth is not easy to incorporate in the initial design other than making the sill depth immediately on the required depth. Because this will lead to significantly higher building costs, this is not done. The depth is considered a smaller problem compared to the other dimensions. This is due to the fact that vessels can first lighten (resulting in a smaller draught) and subsequently can pass the lock.
Lengthening of the lock is only possible by replacing or rebuilding the head and lengthening the walls. This is in the initial design only a spatial requirement.
A more detailed design of the adaptive head will be given in chapter 5.

**Floor**
For the floor the concrete blocks mattress will be used in the design. This one has the highest score in the multi criteria analysis in the proof of concept study for the new sea lock in IJmuiden (van Sloten, et al., 2011). Besides, the block mattress is considered to have the same possibilities to be deepened later on as a loose rock floor.

**Dimensions large lock**

Resuming, In Figure 20 the way of implementation of the different requirements is visualised for the lock for seagoing vessels.

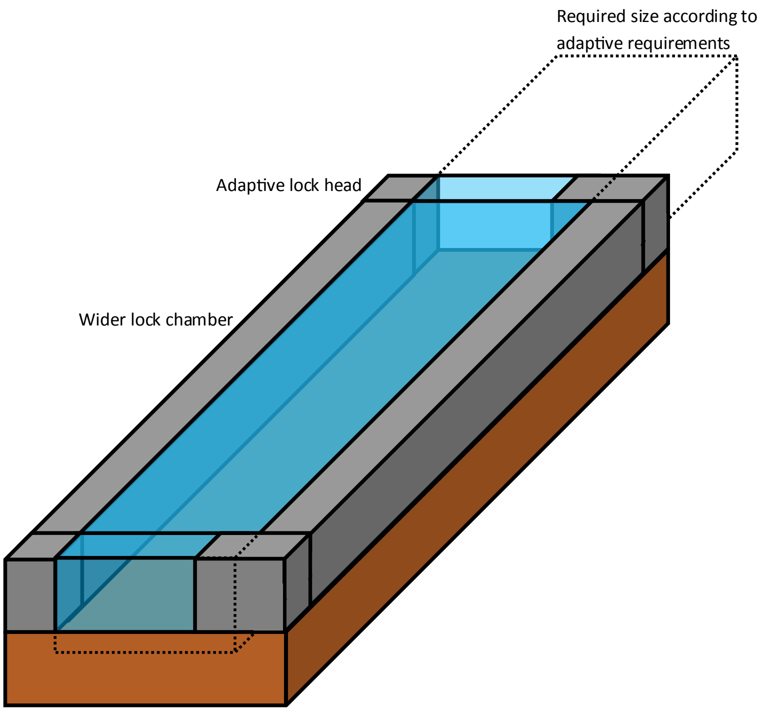



*Figure 20 Implementation of requirements in the large lock*

The sizes of the lock will be determined with the following formula (Glerum, et al., 2000):

Chamber width: $1{,}15 * width\ adaptive\ governing\ vessel = 1{,}15 * 59 = 68\ m$

Door width: $1{,}15 * width\ initial\ governing\ vessel = 1{,}15 * 49 = 56\ m$

Chamber length: $1{,}15 * length\ initial\ governing\ vessel = 1{,}15 * 366 = 420\ m$

Sill level: the sill level depends on the size of the governing vessel and the wet cross section of the lock. The wet cross section has to be 1,3 times bigger than the governing vessel cross section. A rectangular cross section of the vessel is assumed: $15{,}2 * 49 * 1{,}3 = 969\ m^2$. The necessary water depth is: $\frac{969}{56} = 17{,}3\ m.$

This gives a keel clearance of 2,1 m, which is 13 % of the maximum draught. This is enough regarding the lowering of the vessel and the hydraulic and safety margin. The minimum locking water level that is taken into account for this governing vessel is -1 m NAP. This is not the lowest water level regarding locking, but it is assumed that the combination of governing vessel and extreme low water level will not occur. This means that the sill level should be built on -18,3 m NAP.

The same method will be used for the adaptive sill level. 400*59*18. Required wet cross section: $18 * 59 * 1{,}3 = 1381\ m^2$. The required water depth: $\frac{1381}{68} = 20{,}3\ m$, which is 13 % of the maximum draught. This means with a locking level of -1 m NAP a sill level of -21,3 m NAP.

## 4.3 Mid-sized lock

The mid-sized lock will not have the same solutions as the large lock. A relieving floor is considered to be not cost-efficient, because of the smaller depth. Regarding the adaptive requirements, it is not necessary to enlarge this lock as stated before. Therefore a coffer dam will be used for the walls. This is more or less the same method as the vertical dam in the relieving floor and will provide potential scale advantages. Filling/emptying of the lock will be done with openings in the gate. For the other elements, the same solutions will be used as the large lock. That means that a rolling gate for closure and a concrete block mattress as floor will be used.

The dimensions for this lock are less stringent and clear than for the large lock. When looking at the distribution of the vessels between the locks, a maximum length of 200 meter can be seen in the initial design. In the adaptive design, a maximum length of 300 meter is considered. It is however assumed that these vessels can use the newly build lock.

A typical size for seagoing vessels slightly longer than 200 meter is a length of 215 meter and a beam of 32 meter (according to the Panamax class). With the same requirements for the lock size as for the large chamber this results in a chamber length of 250 m and a width of 37 meter. A draught of 9 meter is considered. The same method as for the large lock is considered:

Chamber/door width: $1{,}15 * width\ governing\ vessel = 1{,}15 * 32 = 37\ m$

Chamber length: $1{,}15 * length\ governing\ vessel = 1{,}15 * 215 = 250\ m$

Sill level: the sill level depends on the size of the governing vessel and the wet cross section of the lock. The wet cross section has to be 1,3 times bigger than the governing vessel cross section. A rectangular cross section of the vessel is assumed: $9 * 32 * 1{,}3 = 374\ m^2$. The necessary water depth is: $\frac{374}{37} = 10{,}1\ m$. This gives a keel clearance of 1,1 m, which is 12% of the maximum draught. The minimum locking water level is -1 m + NAP. This results in a sill level of -11,1 m + NAP

## 4.4 Small lock

The only difference in concepts between the small and the middle lock is the concept for the gate. To save space, mitre gates will be used for closing the locks. This is a proven method for these dimensions. Furthermore, coffer dams as chamber walls, openings in the gate for filling/emptying and a concrete blocks mattress as chamber floor will be used.

The size of the locks will be the same size as mentioned before: a length of 250 m and a width of 24 m. The maximum draught of these vessels is 4,5 m. A minimum keel clearance of 0,7 m is required (Glerum, et al., 2000). This results in a water depth of 5,2 m. The minimum water level that locking should be possible is -1,85 m + NAP. This is a lower level than the larger locks. It is assumed that the inland vessels have a higher probability to navigate with the maximum draught. The sill level will therefore be built on -7,05 m + NAP.

## 4.5 Overall solution

To summarize the partial solutions for the different components, they are put together in one overall solution for the barrier. The proposed lock sizes are given in Table 21. Together with the height of the structure and the length for the dewatering sluice and pumping station the complete barrier is determined (see Figure 18). The initial design with four locks will be taken into account in the further design.

| | Amount (n) | Length (m) | Width (m) | Sill level (m + NAP) |
|---|---|---|---|---|
| Large lock | 1 | 420 | 68 | -18,30 |
| Middle lock | 1 | 250 | 37 | -11,10 |
| Small lock | 2 | 250 | 24 | -7,05 |

*Table 21 Composed lock sizes*

With the required sizes of the locks known, they can be configured in the right way. The locks will be built on the existing land next to the New Waterway. In the existing channel the pumping station and dewatering sluice will be placed. This results in the most logical build order. In this way the lock complex can be built without blocking the waterway. When the locks are finished, the pumping station and dewatering sluice will be build.

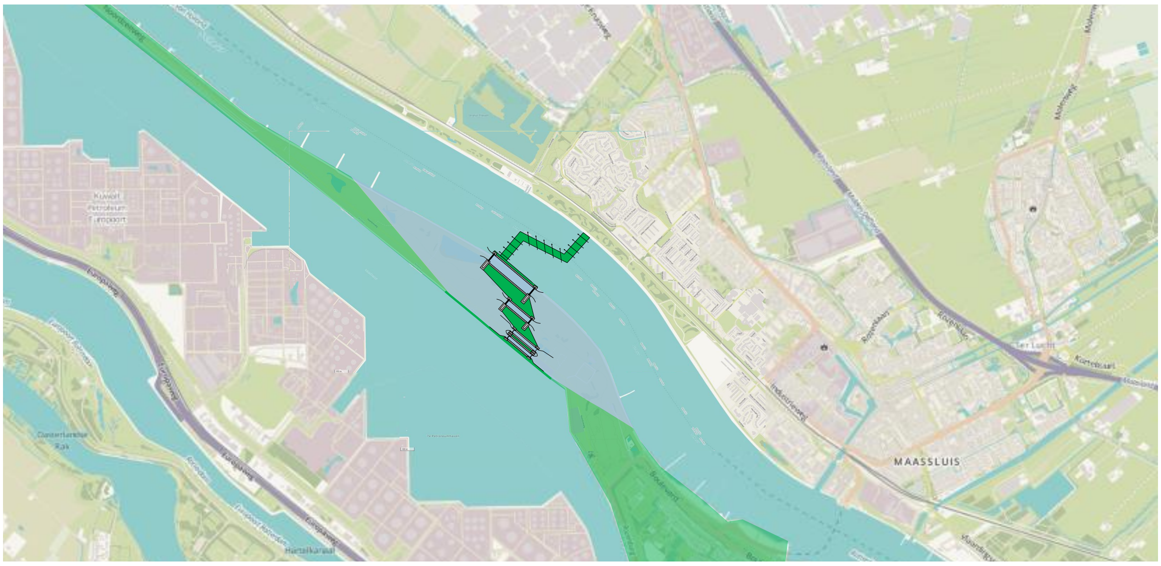



*Figure 21 Global overview lock complex*

**Navigation aspects lock complex**

Due to navigational purposes, the large lock will be placed on the north side of the complex, thus in the middle of the canal. Large vessels have less manoeuvrability, in this way they can enter the lock without making too much bends. The middle lock is situated south of the large lock and all the way to the south the two small locks are made. The locks are not placed right next to each other, but are shifted, also for navigational purposes. Only for the small locks this is not the case, see Figure 21 for a global overview. The small locks use the same approach channel with the adjacent waiting areas. When this is separated from each other, the spatial usage will increase a lot. Mostly inland vessels will use the small locks, so this is not likely to cause problems regarding navigation. A more detailed navigational study should prove this however. The small locks have relatively long berths. These are purposed for vessels awaiting for the lock to enter. The larger locks have short guiding berths, which is typical for locks for seagoing vessels.

A nautical study is not performed in this stage of the design, it should however be checked in more detail whether the above considerations are correct or that more space is required to make navigation possible.

**High water safety**

The red line in Figure 18 shows the flood barrier line of the whole structure. The locks are all made redundant. Both the lock heads and the chamber walls are built to retain the extreme high water. So when one gate fails to retain the high water or it fails to close, the other gate will take over the retaining function together with the chamber walls. This means however that both the gates should be made on more or less the same level to retain the high water. The exact level might differ between the gates, because of the requirements for overtopping caused by waves. The inland gate  The exact reliability of the closing of the locks is not determined. For a detailed design this should be checked by means of a fault tree analysis.

**Design (main) elements**

A more detailed design of the different elements is made. Due to academic interests, this is only done for the large lock. The other locks are more or less ‘standard’ locks, where the adaptive design approach is not present in the individual elements. The large lock consists of several innovative elements. These are the adaptive lock head and the relieving floor with the longitudinal culvert. Both will be elaborated further in the next chapters to make a substantiated estimation of the technical feasibility of the solutions.

# 5

# An adaptive design of a lock head

One of the most interesting elements where the adaptive requirements can be implemented is the lock head for the large lock. See Figure 22 for the position in the system. It is further elaborated how the head can be designed according to these changing requirements. This will be done with already proven technologies, but combined in such a way that the lock head can be adapted when necessary. The adaptive boundary conditions are taken into account for this element and results in the following dimensions that should be adaptive:

- Navigation:
    - Width lock head: 56 m → 68 m
    - Depth lock head: -18,3 m NAP → -21,3 m NAP
- Flood safety
    - Water retention height: +6,37 m NAP → +7,32 m NAP

The lengthening of the lock is not taken into account in the design, because this results in a complete new head. Furthermore, the proposed design as stated in section 4.2 is used, thus with a higher width of the chamber and a smaller initial width of the lock head.

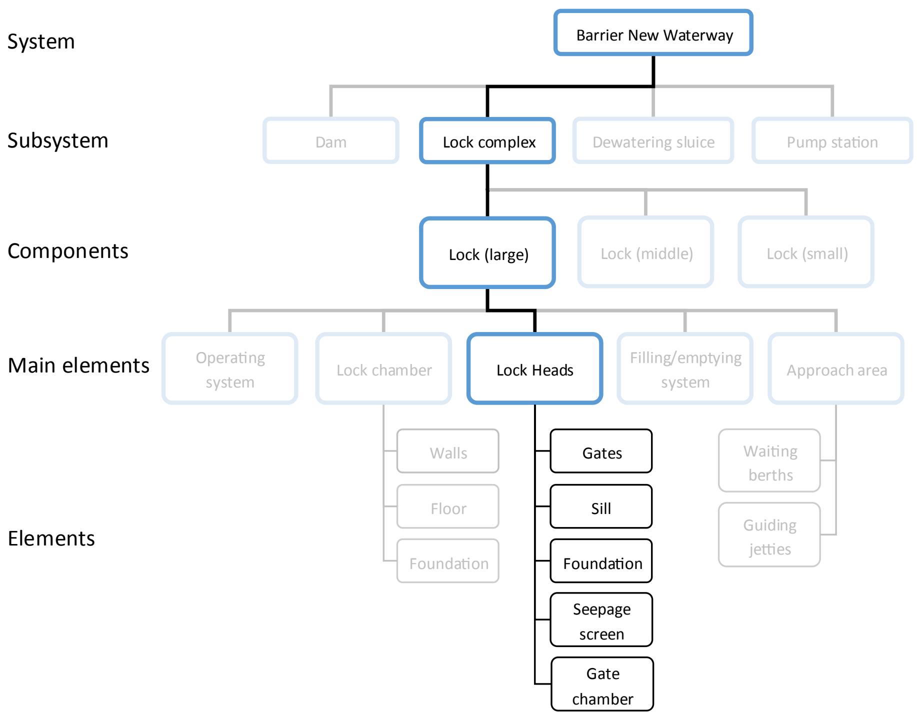



*Figure 22 Design tree, position of lock head in the design process*

## 5.1 Technical requirements

Because of the variations in water levels, several load combinations can be derived. These consists of combinations of water levels on the inside, on the outside and waves. The two main load combinations regarding water levels are given in Figure 23. The governing situation is with high water on the seaside of the structure. This will be taken into account in the further design.

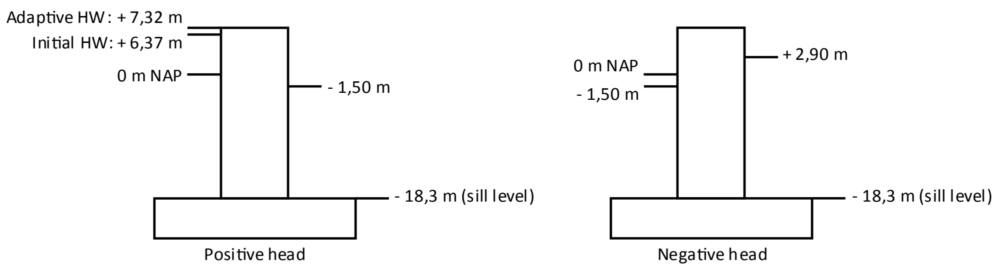



*Figure 23 Load combinations water levels*

## 5.2 Concept design

The proposed design consists of two main elements: the sill within the head and the gate recess. These elements will be elaborated separately.

### *Gate recess*

Several adaptive requirements influence the construction of the gate recess. First of all the width of the lock (58 m → 68 m). Furthermore, the depth of the lock (-18,3 m NAP → -21,3 m NAP) and the increased water level safety (+6,37 m NAP → +7,32 m NAP) should be taken into account for an adaptive design.

When looking at the width, a modular system could be used. The last part of the gate recess will then be made with separate elements, which can be removed when the head has to be widened. These elements has to be interlocked with the head itself, so that the loads can be transferred to the head. This will be done to make the elements in a T-shape. In this way they can be lifted out, but they are able to transfer the horizontal loads. The back of the head should be replaced as well (20 m). This can be done in more or less the same manner, by using a Berliner wall. The required space at the back of the head can be excavated and the head enlarged. The Berliner wall can then be removed. Besides of this, the operating mechanism of the gate must be replaced. These mechanisms are mostly placed above the high water level, and at the end of the gate recess. A new mechanism has to be installed, because a larger gate will be used. An option is to build this new mechanism on the new location and demolish the old mechanism when this is necessary. The building method is visualised in Figure 24.

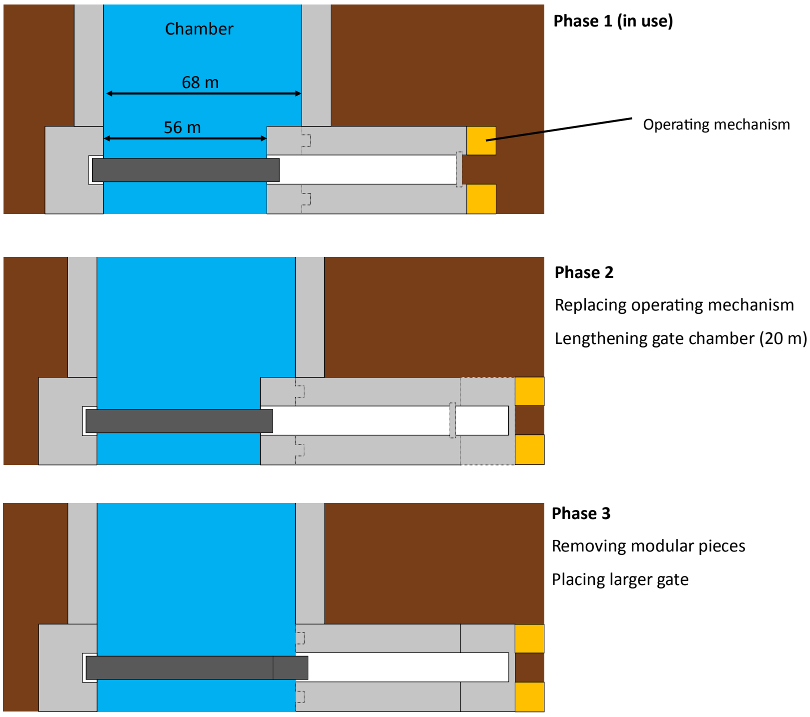



*Figure 24 Building method widening head*

The possibility of deepening of the head later on is easier to achieve. When using the caisson method, the head should be placed deep enough to prevent piping. This means that under the gate a 'useless' space is present. When a prefab element as a floor is placed in there, the head can first be used on the initial depth. When it has to be deepened, this prefab element can be removed.

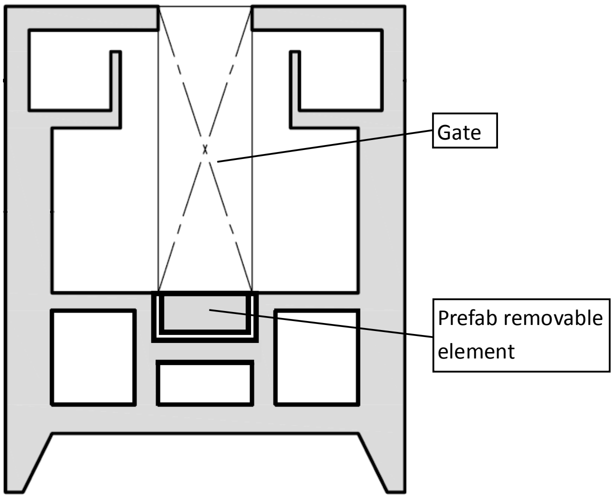



*Figure 25 Cross section gate recess, based on (van Sloten, et al., 2011)*

The increased water level safety results in higher loads on the head. These will be taken into account immediately. Besides, the structures has to be higher. This can easily be done later on on top of the head. It is assumed that the adaptive requirements will not result in a higher thickness of the gate.

### *Sill*

The important adaptive requirements for the sill are the required depth (-18,3 m NAP → -21,3 m NAP) and the increased water level regarding safety (+6,37 m NAP → +7,32 m NAP). The latter one results in a higher vertical load from the gate and higher requirements regarding piping.

The sill could be made with a prefab immersed element. This element will have a proposed width of 56 meter (the initial width of the chamber), a length of 30 meter and a height of 6 meter. It will be placed according to the initial sill level: -18,3 m NAP. When the lock has to be deepened, the sill can be raised by letting it float again. When this is done, the building pit can be excavated to the required depth. Subsequently, a new sill can be immersed in place.

An important point of interest is the requirement regarding piping. Several options can be proposed. First of all, a seepage screen can be used. A connection has to be made with the screen and the sill to prevent piping. The screen will be made in front of the sill. Therefore the space between the sill and the screen has to be made impermeable. This is done by grouting or underwater concrete. When the sill has to be removed for deepening of the lock, it has to be detached from the screen. This is done by burning the screen underneath the sill. The remaining screen can be used as a new seepage screen for the new sill. This will be placed deeper, so the connection can be made with the same method. The required depth of the screen will therefore be based on the adaptive requirement. The above method is however not proven to be practically possible.

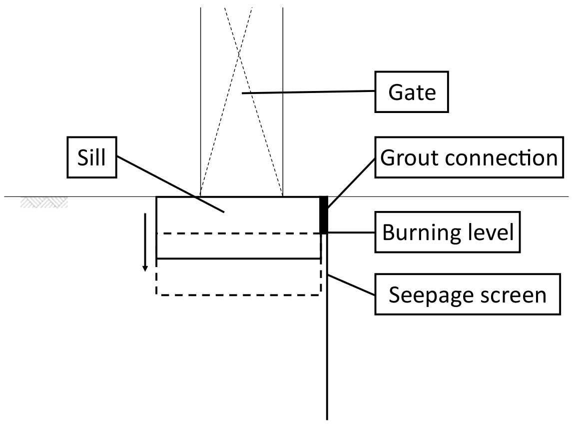



*Figure 26 cross section of the sill with seepage screen and measures for lowering of the sill*

Another option is to make a long horizontal underwater concrete floor adjacent to the sill. This will lengthen the seepage length and provides a bed protection as well. Deepening of this floor is however not easy. The floor has to be removed and a new one must be placed instead or it should be built on the adaptive depth in the initial phase. Further research needs to be done to prove that this will not cause too much blocking of the lock.

The method of lane will be used to determine the required seepage length (Vrijling, 2011):

$$L \geq \gamma * C_L * \Delta H$$

$$L = \sum L_{vert} + \sum \frac{1}{3} L_{hor}$$

With:
ΔH = 7,32+1,5 = 8,82 m

$C_L$ = Lane's constant, depending on soil type (5,0 for coarse sand)
γ = safety factor (1,5)
This gives a required seepage length of 60,15 m. The option with the vertical seepage screen gives a screen length under the sill of: $\frac{60{,}15-6-\frac{30}{3}}{2} = 22\ m$. Regarding the adaptive depth, this means it has to be installed up to -50 m NAP.
The option with the horizontal floor results in a total floor length of $\left(60{,}15 - 6 - \frac{30}{3}\right) * 3 = 132\ m$. The horizontal floor is also beneficial regarding horizontal stability. Both options are feasible. It has to be further researched which is the best option.

## 5.3 Costs

To draw a conclusion whether the above construction will be cost-effective, a decision tree is made. Three decisions are considered. One with an initial design, one with an adaptive design and the third extreme one where the adaptive requirements are immediately taken into account, see Figure 27. Two scenarios are considered. The initial scenario with the known boundary conditions and the extreme scenario where the stated adaptive boundary conditions will occur. This will lead together with the decisions to six scenarios that can possibly occur.

NB: in the following approach, only widening and deepening of the lock head is taken into account. Other adaptive requirements, like lengthening of the lock and a higher water level are not taken into account. Besides, it only counts for the consideration of implementing the adaptive requirements in the lock head. It does not count for the complete lock.

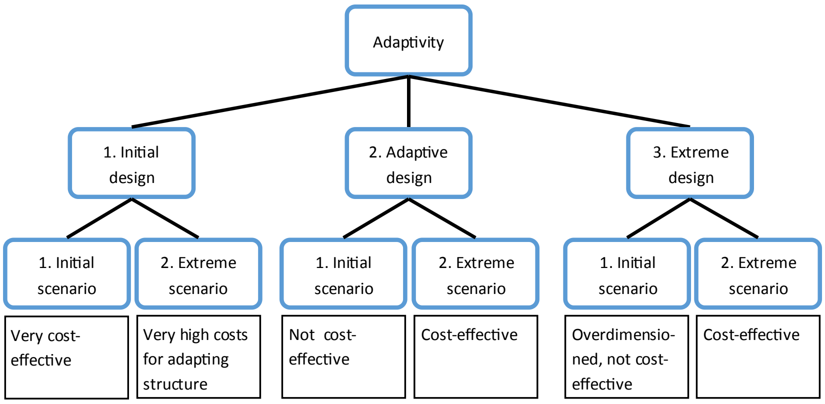


*Figure 27 Decision tree adaptivity*

To draw a conclusion whether it is useful to use an adaptive design for the lock head, the costs are estimated for the lock heads in the different scenarios. These are based on reference projects (mainly IJmuiden) and expert judgement. The costs are visualised in Figure 28. A discount rate *i* of 3 % is assumed. The net present costs (NPC) are calculated with $NPC = \int \frac{FC}{(1+i)^t}$, where FC stands for the future costs. It is considered that a reinvestment due to the extreme scenario must be done after 50 years. When this scenario does not occur, a reinvestment due to maintenance is still necessary. The life span of a gate is assumed to be 50 years, so replacement of the lock gates are always part of the reinvestment. The large investment for scenario 1.2 is caused by the fact that the building of two complete new lock heads is necessary. The costs for maintenance are assumed to be € 3 million for a small gate and € 3,5 million for a larger gate.

| | 1.1 | 1.2 | 2.1 | 2.2 | 3.1 | 3.2 |
|---|---|---|---|---|---|---|
| Initial building costs head [mil €] | 240 | 240 | 280 | 280 | 270 | 270 |
| Reinvestment costs after 50 years [mil €] | 80 | 400 | 80 | 190 | 90 | 90 |
| Net present costs (100 years) [mil €] | 354 | 431 | 393 | 422 | 401 | 401 |

*Table 22 Assumed costs for the different scenarios*


*Figure 28 Cumulative discounted costs different scenarios*

To summarize the costs: scenarios 3.1 and 3.2 are exactly the same. Scenarios 2.1 and 2.2 are respectively slightly cheaper and slightly more expensive than scenario 3.1 and 3.2.

## 5.4 Conclusion

Out of the costs alone it is not possible to conclude whether it is useful to make an adaptive design or to implement the adaptive requirements immediately. Only relatively small differences in total costs can be seen between the decisions 2 and 3. To draw a conclusion, an estimate of the probability of occurrence of the scenarios is made. The two scenarios is given both a probability of occurrence P(E) and P(I) for respectively the extreme and initial scenario. The expected costs for a certain decision can then be calculated with:

$$expected\ costs\ \ (decision\ x) = P(I) * NPC(x.1) + P(E) * NPC(x.2)$$

Experts are not in agreement about the probability of the above scenarios. Because it is not clear what the probability distribution of the scenarios is, several distributions will be researched to check where the breakeven point is.

| Probability distribution P(I)/P(E) | Decision 1 Initial design [mil €] | Decision 2 Adaptive design [mil €] | Decision 3 Extreme design [mil €] |
|---|---|---|---|
| 0,9/0,1 | **361** | 396 | 401 |
| 0,7/0,3 | **377** | 402 | 401 |
| 0,5/0,5 | **392** | 407 | 401 |
| 0,3/0,7 | 408 | 413 | **401** |
| 0,1/0,9 | 423 | 419 | **401** |

*Table 23 Expected costs different designs*

NB: this approach only takes into account the costs for the head. The added value of a larger lock or because of other reasons is not taken into account.

The above numbers show that the adaptive design for the head is probably not cost-effective. With a high probability of occurrence for the initial scenario, decision one is the most attractive decision. When an extreme situation is more likely to occur, it is more cost-efficient to implement the adaptive requirements immediately. Making an adaptive design seems to be never the cheapest option for the lock head, see Table 23. The differences between the decisions are however relatively low. The uncertainty in the estimation of the costs may not be neglected. Besides of that, the following factors are not yet taken into account:

- With decision 3 larger vessels are able to pass the lock immediately, causing less nuisance for new developments in the port areas, giving a higher value.
- The uncertainty whether it is possible to implement the adaptive requirements after several decennia is very high, because the considered approach is not a proven method.

Considering the above factors and the small difference in costs between the decisions, it is concluded that making an adaptive design for the lock head is not the best option. Implementing the higher adaptive requirements immediately will result in a higher value for the hinterland and the risk in costs of the proposed adaptive method is too high to be certain that it is cost-effective. Decision 1 might be the best future-proof option regarding costs. When the requirements will not increase, it is the cheapest option, but when the requirements increase, it is not easy to adapt the structure and the costs will increase very much. Besides, with the extreme design, the added value is higher, because larger vessels are able to pass the lock from the beginning. Therefore, in the further design of the large lock. the wider design will be taken into account (68 m). The sill depth is however chosen to be the initial depth (-18,3 m+ NAP). This is assumed to be a smaller problem, because vessels with a larger draft are able to lighten, before they enter the lock.

The wider lock results in more water to be locked each time. Therefore the proposed solution for the filling and emptying system and the chamber walls as stated before will be further researched. This is the relieving floor in combination with the longitudinal culvert. Because it is expected that this will result in faster filling times, the hydraulic aspects of the longitudinal culvert are explored first. Besides of that, the structural feasibility of the relieving floor is computed as well.

# 6

# Hydraulic design of a longitudinal culvert

As stated in chapter 4, the chamber wall for the large lock will be made with the use of a relieving floor. On top of this floor, a longitudinal culvert will be placed to fill and empty the lock. Filling is done vertically by openings in the floor of the culvert. The retaining wall underneath the floor must therefore be placed slightly outwards to make this possible, see Figure 29.

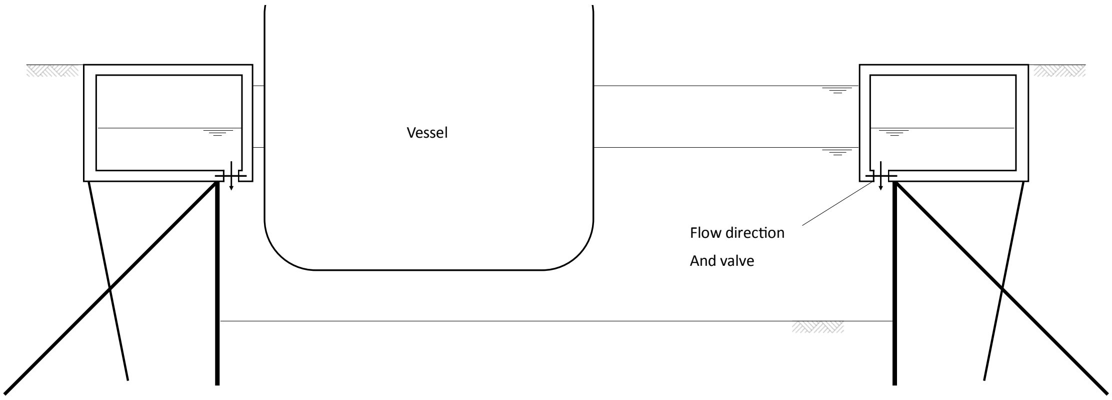



*Figure 29 Cross section of a lock with a longitudinal levelling system on top of the relieving floor*

This solution is not an adaptive solution by definition. It is however created out of the adaptive thoughts. By making the lock wider according to the adaptive requirements, more water has to be transferred with each locking. To compensate for this, a faster levelling system is designed, resulting in the combination of the relieving floor with a longitudinal filling system. A more detailed design of this combination will be computed below. First, the hydraulic aspects of the system will be elaborated. This results in certain required dimensions for the culvert and thus for the relieving floor. This is used as input for a structural elaboration of the relieving floor.

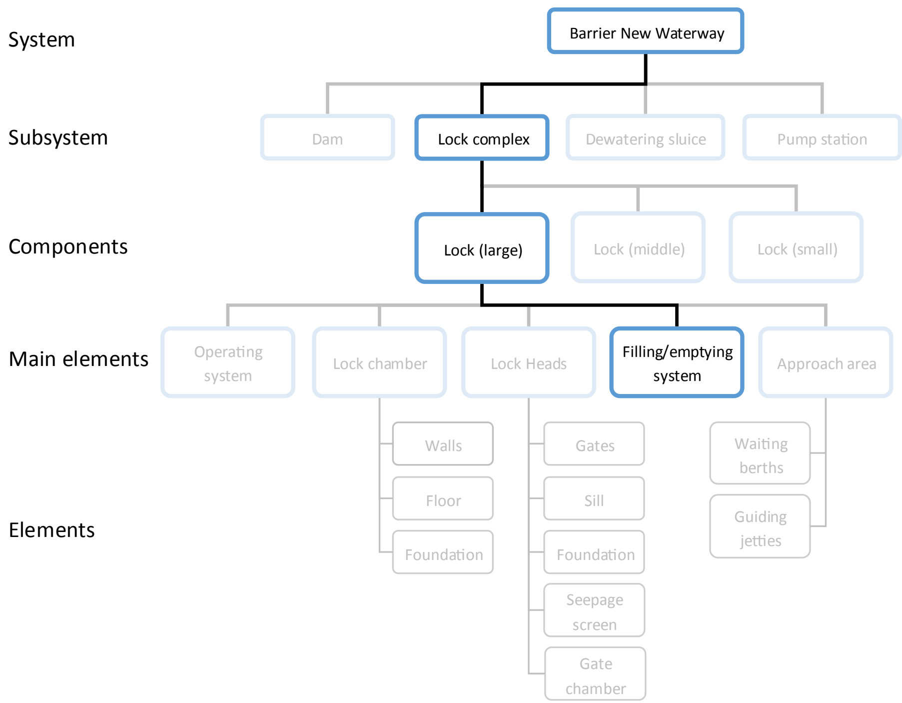



*Figure 30 Design tree, position of longitudinal culvert in the design process*

**Technical requirements**

The same requirements regarding water safety will be used as for the lock head. For the locking function however, other water levels are considered. The maximum and minimum locking water level are given in Figure 31.

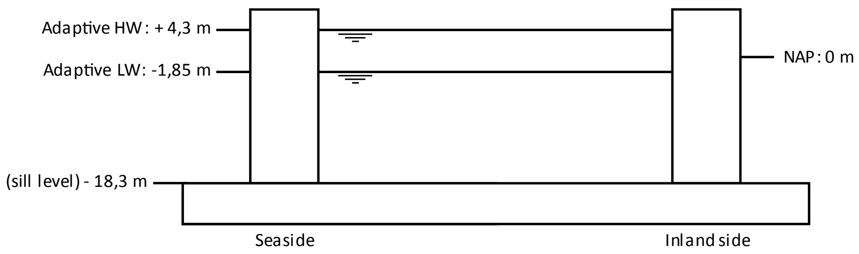



*Figure 31 Locking water levels*

## 6.1 Concept filling/emptying system

Regarding the hydraulic aspects of the filling system, a first estimate will be made with a simplified Bernoulli discharge formula:

$$A = \frac{Q}{\mu\sqrt{2 * g * \Delta H}}$$

Four different situations with different water levels will be considered. Two situations with the extreme locking water levels, which have a requirement of maximum filling time of 10 minutes and the other two situations are the ones representing the high and low water of the spring tide. These have a requirement for filling of 5 minutes. An average head of half the maximum head is assumed. The inland water level is set constant on 0 m + NAP. This will give a required cross section of the culvert. The results are given in the following table.

| | | $h_{sea}$ [m+NAP] | Filling time [s] | $Q_{avg}$ [m$^3$/s] | A [m$^2$] |
|---|---|---|---|---|---|
| Spring tide | High water | 1,53 | 300 | 146 | 75 |
| | Low water | -0,55 | 300 | 52 | 45 |
| Extreme locking water levels | High water | 4,30 | 600 | 205 | 63 |
| | Low water | -1,85 | 600 | 88 | 41 |

*Figure 32 Required culvert cross section*

The first situation with high water due to spring tide gives the highest required cross section of 75 m$^2$. This results in a wet cross section on each side of the lock of 38 m$^2$. This cross section will be used in the first design of the relieving floor.

The proposed system of filling is given in Figure 33. There is a difference between the two sides of the lock. The culvert has to go around the lock head. One has to ensure that the bends in the culverts are smoothened, to reduce the energy loss.

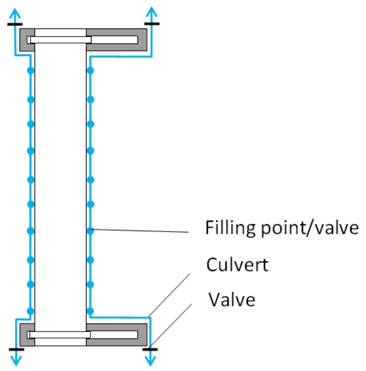



*Figure 33 Proposed filling system (top view complete lock)*

## 6.2 Filling/emptying through the head

The most important aspect that restricts the filling and emptying times is the horizontal load on the vessel due to the flowing water. The vessel is moored to the lock by a set of lines. Seagoing vessels are mostly moored with four lines (two lines and two springs, see Figure 34). When the horizontal load on the vessel will increase, the stresses in the lines will increase as well, eventually causing to failure of the lines. The forces on a vessel can be derived into two components, a longitudinal and a transversal component, the definitions of the directions are given in Figure 34. The conventional filling/emptying method used in the Netherlands (filling/emptying through the head) results mostly in longitudinal forces on the vessel. A sideways filling/emptying system results mainly in transversal forces on the vessel. Both will be elaborated below.

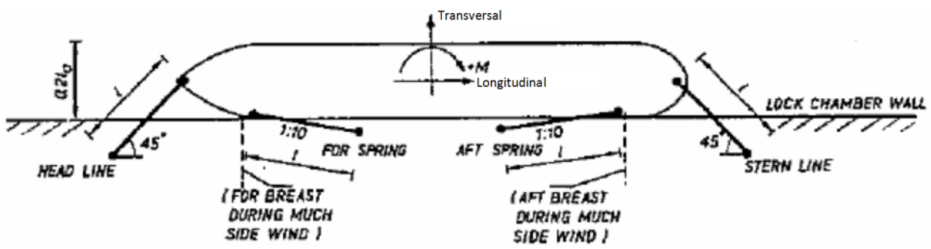



*Figure 34 Typical mooring method seagoing vessels (Vrijer, 1977)*

**Hawser force criteria**

The maximum allowable load on the vessel is determined by the hawser force criteria. This criteria is expressed as a relative value in ‰ of the weight of the water displacement of the vessel. For inland vessels this value for the longitudinal force lies about 1 ‰. The mooring capacity of large ocean going vessels are relatively smaller, resulting in a more stringent criteria for the longitudinal force. This is about 0,25 ‰ for a vessel with a DWT of 50.000 (Glerum, et al., 2000).

**Filling through the head**

In a conventional filling/emptying system (through the head) the filling and emptying of the lock mostly results in a load in longitudinal direction of the vessel. This load is created by several phenomenon, see also Figure 35:

a. Translatory waves
b. Momentum difference
c. Friction
d. Filling yet (only present when filling the lock)
e. Density differences (only present when filling the lock)

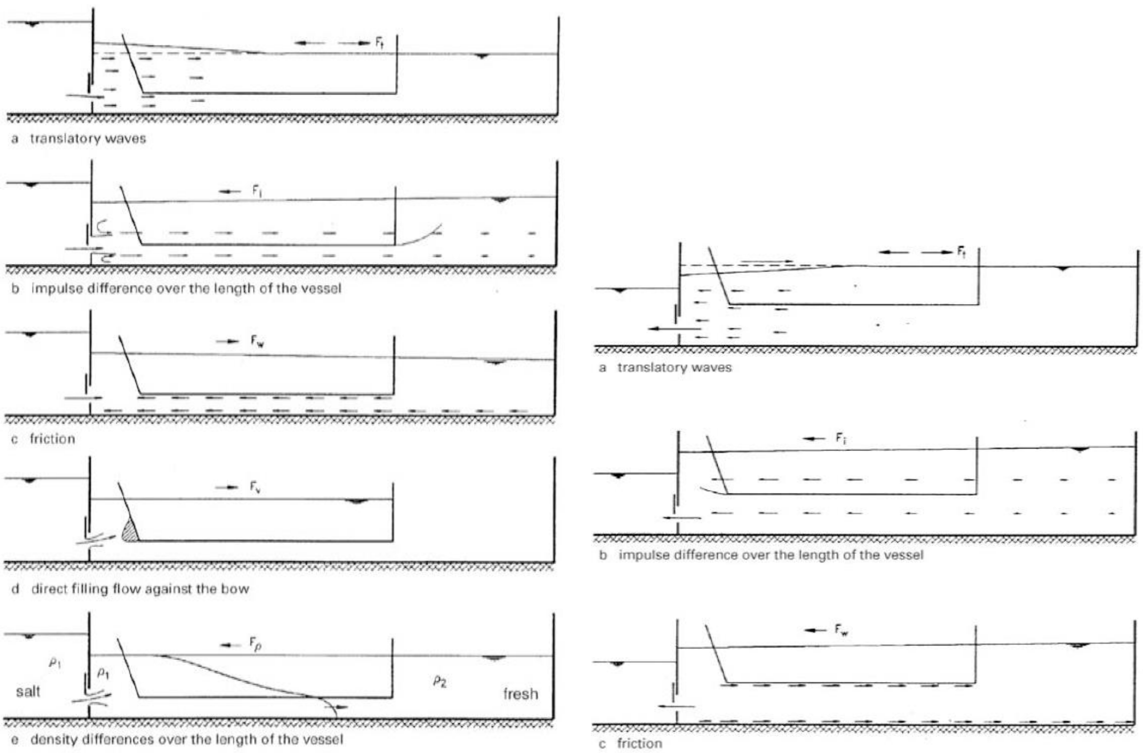



*Figure 35 Components of the longitudinal force during filling (left) and emptying (right) (Waterloopkundig laboratorium, 1994)*

The translatory waves and the momentum differences normally have the highest influence on the resulting load, except for the load due to density differences, which is not always present. The translatory waves can result in a negative and a positive load on the vessel. This load fluctuates highly in time. The momentum difference results normally in a negative load, directed to the filling/emptying side. The direct filling yet against the bow of the vessel and the density differences are not present during emptying of the lock. The resulting loads can be calculated with the software program LOCKFILL.

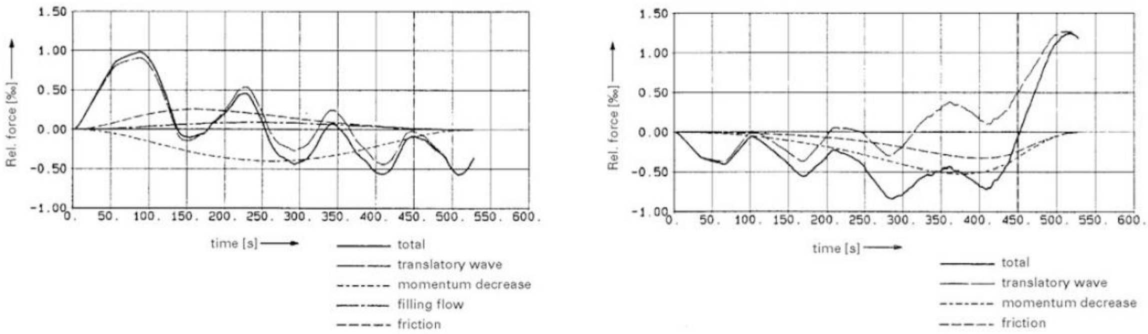



*Figure 36 Typical longitudinal load scheme on a vessel during filling (left) and emptying (right) through the head (Glerum, et al., 2000)*

## 6.3 Calculation method sideways filling/emptying

The situation with the relieving floor is different than a filling system through the head, because of the sideways filling and emptying of the lock. The LOCKFILL program only applies for a levelling system through the head, from one side only. The background formulas used in the program will be transformed in such a way so that they apply for a two-way sideways filling system. The phenomenon will not result in a longitudinal force on the vessel anymore, but in a transversal force, perpendicular to the vessel axis. It is however possible that a longitudinal load will develop due to uneven filling along the length of culvert. This longitudinal load is however neglected, because it is assumed that an even filling can be obtained. The longitudinal culvert on the sides of the lock should be made large enough to obtain this.

A transversal hawser force criteria should be determined. Because of the usual mooring method (two string lines under an angle of 1:10 and the head and stern lines under 45 degrees, see Figure 34), this criteria is more stringent than the longitudinal hawser force criteria. This mooring method is assumed to be used in the proposed lock as well. A longitudinal criteria of 0,25 ‰ results in a criteria for the transversal force of 0,11 ‰ with this line composition. It is hereby assumed that only two lines (for spring and stern line, or aft spring and head line) transfer the transversal load, the same lines that also transfer the longitudinal load. Because there will always be some longitudinal displacement of the vessel, the other two lines are not under tension anymore and are therefore not contributing in transferring any load. This is however not the case when a winch is used to keep the lines under tension.

The forces due to the filling yet will be neglected, because the flow is directed downwards, and not directly against the hull of the vessel. How the effects of the other phenomenon are determined is elaborated below. To visualise the effects separately, the results for a test case are determined. The used for this test case are given in Table 24.

| **Data test case** | | |
|---|---|---|
| Water level approach harbour | [m+NAP] | 1,53 |
| Initial water level lock | [m+NAP] | 0 |
| **Lock** | | |
| Lock width | [m] | 68 |
| Lock length | [m] | 420 |
| Sill depth | [m+NAP] | -18,3 |
| **Filling openings** | | |
| Maximum opening width | [m] | 1,2 |
| Total length | [m] | 100 |
| Opening speed valves | [mm/s] | 10 |
| **Vessel** | | |
| Beam | [m] | 49 |
| Length | [m] | 366 |
| Draught | [m] | 15,2 |
| Block coefficient | [-] | 0,8 |
| Distance between vessel and wall | [m] | 0,5 |

*Table 24 Data test case filling system*

A set of valves is used to close the connection between the culvert and the chamber. Similarly to the filling systems through the head, the valves are opened with a certain speed to fill the lock. This causes a linear increase of the cross sectional area of the filling openings. Together with the gradually decreasing head, this results in a parabolic shape of the discharge in time, see Figure 37. It turns out that the shape of this discharge line has a large influence on some of the load components, as will be shown below.

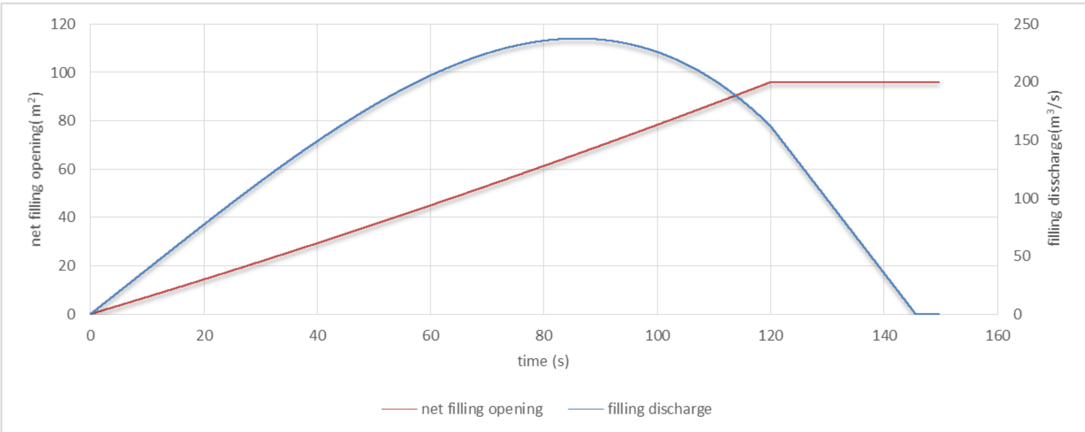



*Figure 37 The cross sectional area of the filling openings and the filling discharge during filling of the lock (test case)*

Not all of the load components that are present in the head filling systems are present in the sideways filling system as well. The load components that are taken into account are:

- Translatory waves
- Momentum difference
- Friction
- Density difference

This means that the direct filling flow against the bow is not taken into account. Due to the fact that filling is directed downwards, the flow is not directed against the bow.

### ***Translatory waves***

The theory of low translatory waves states that because of a sudden change in discharge or a sudden change in cross section, a wave will start to propagate in the lock. The propagating speed of the translatory wave can be calculated with the following formula (Battjes, et al., 2014):

$$c = \sqrt{gd}$$

With this formula and the known discharge the wave height can be computed as follows:

$$H = \frac{q}{c} = \frac{Q}{w * c}$$

This discharge is however not constant, but will change in time. The incoming discharge will be calculated with Bernoulli: $Q = \mu A\sqrt{2 * g * \Delta H}$, where the head will gradually decrease in time. At first, a wave will propagate from the filling point into the lock, it will partially be reflected when the wave ‘meets’ the vessel and completely when it meets the other wall of the lock. A new wave will therefore propagate from this particular cross section, to the next one. The resulting wave heights in a cross section are therefore a combination of the locally generated waves and the incoming waves generated a certain time step before at the adjacent cross section (the time the wave needs to propagate from one cross section to another).

Six important cross sections are defined in the lock where an incoming wave will be (partially) reflected: see Figure 38.

A. The left wall of the lock
B. Just before the left side of the vessel
C. Just after the left side of the vessel
D. Just before the right side of the vessel
E. Just after the right side of the vessel
F. The right wall of the lock

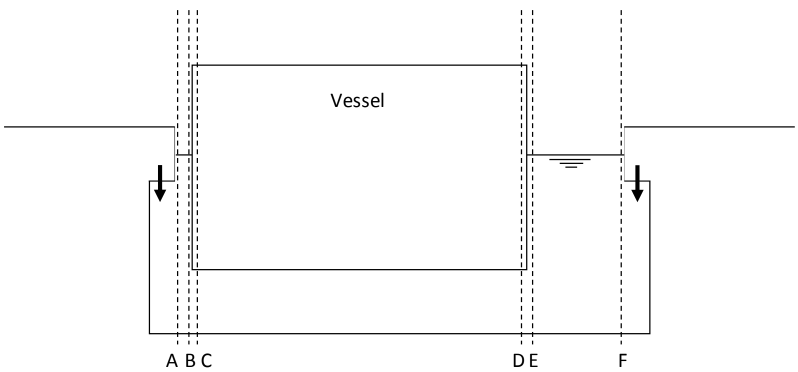



*Figure 38 Decisive cross sections in the longitudinal plane of the lock: A up to F*

At these locations the discharges caused by the translatory waves are computed. Subsequently, the wave heights and water levels can be calculated. Summarizing, for a lock being filled from two sides, this follows in the set of equations given in Table 25. The difference in water levels between the two sides of the vessel results in a horizontal load on the vessel.

| Generated discharges [$m^3/s$] | Incoming discharges [$m^3/s$] | Generated wave heights [m] | Incoming wave heights [m] | Water levels [m + NAP] |
|---|---|---|---|---|
| $Q_A(J) = Q_{inA} - Q_{At}$ | $Q_{At} = Q_B(J - NBJ)$ | $H_A = \frac{Q_A(J)}{w * c_l}$ | $H_{AT} = \frac{-Q_{AT}}{w * c_l}$ | $h_A = h_I + H_A + H_{AT}$ |
| $Q_B(J) = Q_{BC} - Q_{Bt}$ | $Q_{Bt} = Q_B(J - NBJ)$ | $H_B = \frac{-Q_B(J)}{w * c_l}$ | $H_{BT} = \frac{Q_{BT}}{w * c_l}$ | $h_B = h_I + H_B + H_{BT}$ |
| $Q_C(J) = Q_{BC} - Q_{Ct}$ | $Q_{Ct} = Q_D(J - NSJ)$ | $H_C = \frac{Q_C(J)}{w * c_v}$ | $H_{CT} = \frac{-Q_{CT}}{w * c_v}$ | $h_C = h_I + H_C + H_{CT}$ |
| $Q_D(J) = Q_{DE} - Q_{Dt}$ | $Q_{Dt} = Q_C(J - NSJ)$ | $H_D = \frac{-Q_D(J)}{w * c_v}$ | $H_{DT} = \frac{Q_{DT}}{w * c_v}$ | $h_D = h_I + H_D + H_{DT}$ |
| $Q_E(J) = Q_{DE} - Q_{Et}$ | $Q_{Et} = Q_F(J - NHJ)$ | $H_E = \frac{Q_E(J)}{w * c_l}$ | $H_{ET} = \frac{-Q_{ET}}{w * c_l}$ | $h_E = h_I + H_E + H_{ET}$ |
| $Q_F(J) = Q_{inF} - Q_{Ft}$ | $Q_{Ft} = Q_E(J - NHJ)$ | $H_F = \frac{-Q_F(J)}{w * c_l}$ | $H_{FT} = \frac{Q_{FT}}{w * c_l}$ | $h_F = h_I + H_F + H_{FT}$ |

*Table 25 Equations to calculate the water levels due to translatory waves (Waterloopkundig laboratorium, 1994)*

With the following used formulae:

Discharge at the left side of the vessel: $Q_{BC} = \frac{\frac{Q_{Bt}}{c_l} + \frac{Q_{Ct}}{c_v}}{\frac{1}{c_l} + \frac{1}{c_v}}$

Discharge at the right side of the vessel: $Q_{DE} = \frac{\frac{Q_{Dt}}{c_v} + \frac{Q_{Et}}{c_l}}{\frac{1}{c_l} + \frac{1}{c_v}}$

Propagating speed empty lock: $c_l = \sqrt{gd}$

Propagating speed next to vessel: $c_v = \sqrt{g \frac{dw - d_v b}{w}}$

Amount of raster points between left wall and vessel: $NBJ = \frac{x_v}{c_l dt}$

Amount of raster points between along vessel: $NSJ = \frac{L_v}{c_v dt}$

Amount of raster points between right wall and vessel: $NHJ = \frac{L_l - L_v - x_v}{c_l dt}$

The acting force (including damping) on the vessel will be derived with the following set of equations (Waterloopkundig laboratorium, 1994).
The (undamped) acting force can be calculated with the derived water levels, expressed relatively to the water displacement of the vessel:

$$F_z(J) = 0{,}5 * \frac{h_A + h_C - h_D - h_E}{L_v C_b}$$

With the following formula the oscillations are smoothened:

$$F_T(J) = 0{,}1F_z(J-2) + 0{,}3F_z(J-1) + 0{,}6F_z(J)$$

Subsequently, damping of the waves is implemented:

$$F_{PA}(J) = \frac{\left(L_l - L_v - \frac{x_v}{2}\right)\frac{dQ_{inA}}{dt}}{L_l g(dw - d_v b)C_b}$$

$$F_{PF}(J) = \frac{\left(L_v + \frac{x_v}{2}\right)\frac{dQ_{inF}}{dt}}{L_l g(dw - d_v b)C_b}$$

$$C_E = 0{,}07 + \frac{d_v b}{dw} * 0.4$$

$$T_k = 2\left(\frac{L_l - L_v}{c_l} + \frac{L_v}{c_v}\right)$$

The resulting force including damping is:

$$F_{SG}(J) = F_T(J) - (F_T(J) - \left(F_{PA}(J) + F_{PF}(J)\right)\left(1 - \exp\left(-C_E \frac{T}{T_k}\right)\right)$$

With:

| | |
|---|---|
| g | = gravitational acceleration = 9,81 [m/s$^2$] |
| w | = width of the lock [m] |
| b | = beam of the vessel [m] |
| dt | = time step [s] |
| $x_v$ | = distance from left wall to vessel [m] |
| $L_v$ | = length of the vessel [m] |
| $L_l$ | = length of the lock [m] |
| $h_l$ | = initial water level of the lock [m+NAP] |
| $d_v$ | = draught of the vessel [m] |
| J | = counter [-] |
| $C_b$ | = block coefficient of the vessel [-] |
| d | = water depth in the lock [m] |

The translatory waves only causes a resulting load when the vessel is moored to one side of the lock. When it is situated in the middle of the lock, the resulting load is zero, because the situation is symmetrical. The loads from both sides is exactly the same.

The resulting load for the test case due to translatory waves is given in Figure 39. The load will start positive (directed away from the wall) and gradually decreases to a negative load. A jump in the load can be seen at 120 s. This occurs due to the fact that the filling opening reaches its maximum opening area. This causes a rapid change in dQ/dt, resulting in a jump in the load. When a lock is being emptied, the load will start with a negative value and increases when emptying.

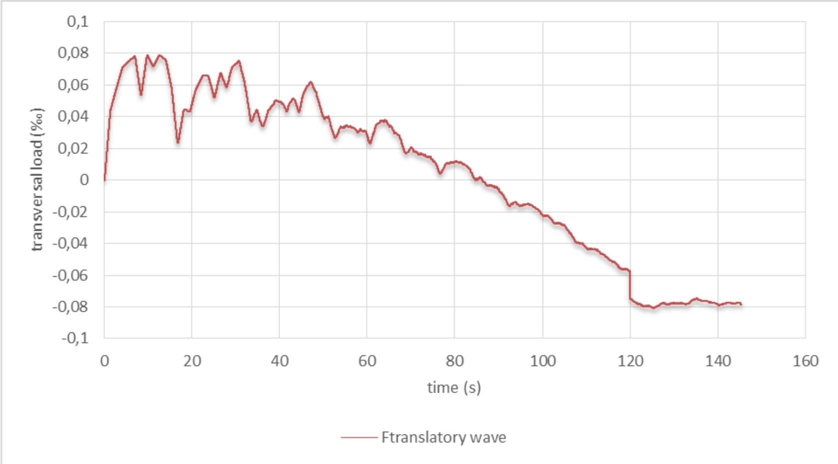



*Figure 39 Transversal load due to translatory waves for the test case*

### *Momentum difference*

The discharge because of filling or emptying of the lock causes an momentum force on the vessel. The momentum force is defined as (Battjes, 2002):

$$F_m = \frac{1}{2}\rho g d^2 l_v + \rho Q u$$

This momentum will change across the width of the lock, because of a changing discharge and velocity. The inflowing volume of water has to be distributed over the whole lock. Therefore the discharge is maximum at the walls and decreases linearly until zero in the middle of the lock, when the lock is being filled from two sides (Figure 40).

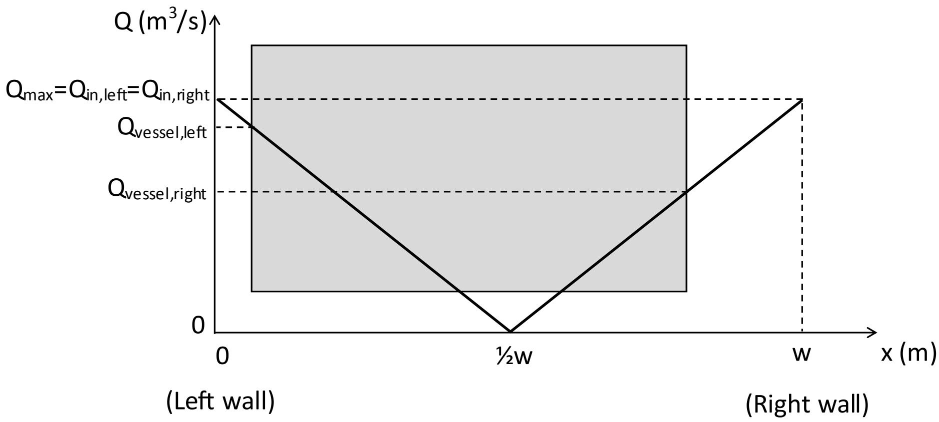



*Figure 40 Discharge distribution across the lock*

The resulting transversal force is the difference between the impulse on the left side and the right side of the vessel. This means:

$$F_{m,l} - F_{m,r} = \left(\frac{1}{2}\rho g d_l^2 l_v + \rho Q_l u_l\right) - \left(\frac{1}{2}\rho g d_r^2 l_v + \rho Q_r u_r\right)$$

The discharge just next to the vessel is determined by the distance between the vessel and the wall and the discharge at the wall:

$$Q_{vessel,left} = Q_{in,left}\left(1 - \frac{x_v}{\frac{1}{2}w}\right) \qquad Q_{vessel,right} = Q_{in,right}\left(1 - \frac{w-b-x_v}{\frac{1}{2}w}\right)$$

The average velocity u in a cross section is calculated by:

$$u = \frac{Q}{L_l * d}$$

In reality, the inflowing discharge on the left and the right side of the lock is not always the same. The translatory waves can result in a water level difference on both sides of the vessel due to blockage of the vessel. The head with the outside water level will therefore differ between the both sides of the lock, resulting in different discharges. The point of zero discharge in the cross section of the lock is not in the middle of the lock anymore, but will be shifted towards the middle of the vessel. The resulting momentum force will be smaller because of that. This is not yet taken into account in the used approach.

With:

| | |
|---|---|
| w | = width of the lock [m] |
| b | = beam of the vessel [m] |
| $x_v$ | = distance from left wall to vessel [m] |
| $Q_0$ | = incoming discharge from one side [$m^3/s$] |

Similarly to the translatory waves, a load due to the momentum difference will only happen due to the asymmetry of the vessel position in the lock. So when the vessel is not situated exactly in the middle of the lock.

The load due to the momentum difference during filling of the test case is given in Figure 41. It can be seen that the shape of the graph shows much resemblance with the shape of the discharge graph (Figure 37), but it is mirrored. The discharge is of great influence on the momentum load. The load is however always negative (directed to the wall), also when the lock is being emptied.

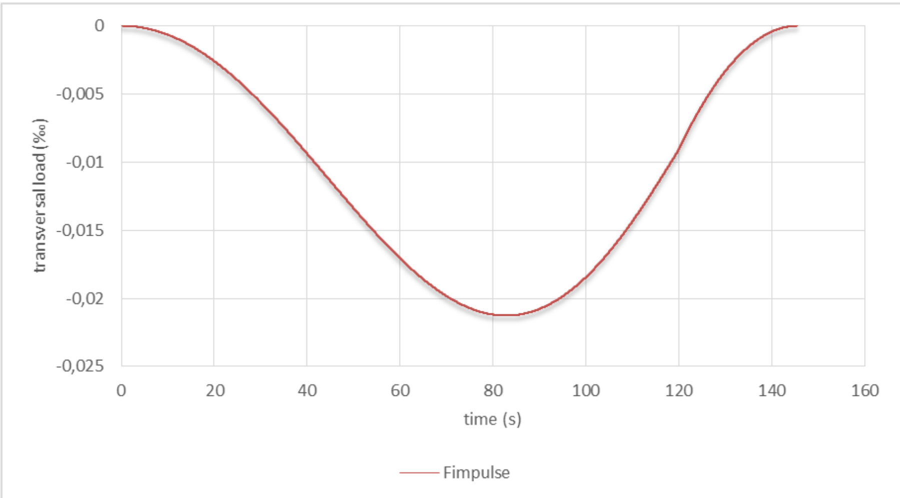



*Figure 41 Resulting load due to the momentum difference*

### *Friction*

Due to the friction resistance of the hull of the vessel, a transversal force will be created by the flowing water. This friction force can be calculated with the following formula:

$$R_f = C_f \frac{1}{2} \rho u^2 S$$

S is the wet surface of the hull. With Re being the Reynolds number and $C_f$ the friction coefficient according to the ITTC-1957 line (Journée, et al., 2015):

$$C_f = \frac{0{,}075}{\log(Re-2)^2} \qquad Re = \frac{uL}{\upsilon}$$

Because of the filling from two sides, two loads due to friction will occur on the vessel, in opposite directions from each other. The same discharge distribution used for the momentum difference is used here. A resulting load will therefore only occur due to the asymmetrical position of the vessel in the lock (similar to the momentum difference). The maximum velocity, which occurs at the side of the vessel, is used in the calculation. In reality, this velocity will decrease along the hull to zero in the middle of the lock. So this is a conservative assumption. The results of the test case are given in Figure 42. The friction depends very much on the discharge. The shape of the graph is comparable to that of the discharge development, but it is mirrored, so the load is negative. This is not the case for emptying of the lock, then the load will be mirrored. The maximum load is very small. The above conservative assumption will therefore be maintained.

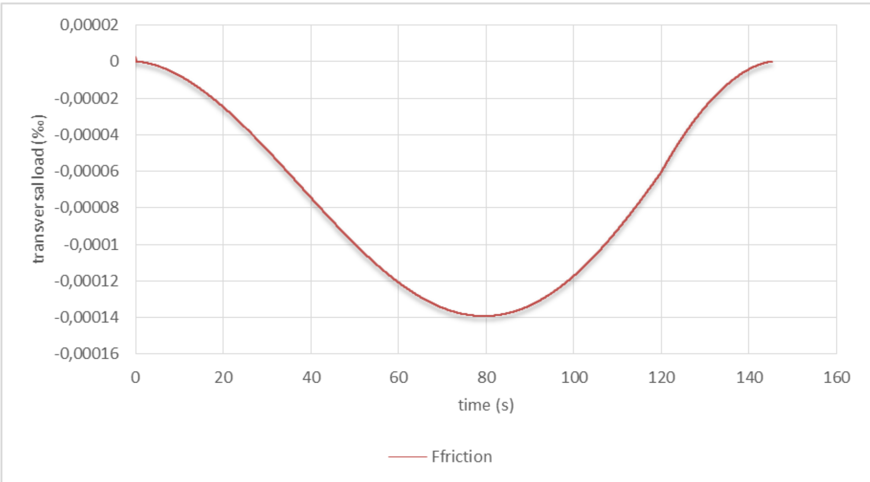



*Figure 42 Resulting load due to friction*

### *Density differences*

The proposed lock is situated at the boundary between sea and the inland waterways, this means that a density difference between the two sides of the lock is present. The sea contains mostly salt water with a density of approximately 1020 kg/m$^3$. The density of the fresh inland water is approximately 1000 kg/m$^3$. When locking with high water(upwards from the New Waterway to the sea), it may happen that the water within the lock is fresh, while the filling water is salt. Because of the larger density, this salt water will sink to the bottom and will form a layer that expands during filling of the lock. Mixing between the salt and fresh water can occur, but this depends on the blockage of the vessel. Most of the time this mixing is limited. Because of the relatively fast inflow of water the densities won't have time to level out completely during filling.

Therefore two separate layers will develop. Because in this case the lock is filled from two sides with an asymmetrical placement of the vessel, two separate salt layers will develop in a different way. The different densities may result in a load on the vessel. Of course, when a salt lock is filled with fresh water, the opposite situation occurs: a fresh layer will develop on top of the salt layer. When the lock is being emptied, density differences will not occur in the lock.

The case with an initially fresh lock being filled with salt water will be investigated further. In reality the development of density differences in a lock are very complicated. In this study, a simplified approach according to (Vrijburcht, 1991) will be used. This approach is originally made for a lock being filled through the head, but it is assumed that it also applies for sideways filling of the lock, because the relative blocking of the wet cross section by the vessel is more or less the same. There are however differences in the two situations. With filling through the head, the distance between the bow and the stern is much higher and therefore the distance for the incoming discharge wave to travel to the other side of the lock is larger. Instead of the situation with sideways filling, where the maximum distance that has to be travelled by the water the width of the lock is. On the other hand, the discharge could easier be distributed over the whole cross section in case of filling through the head. The sides of the wet cross section are closer to each other. Due to these differences, it has to be validated by laboratory research whether this assumption still can be applied.

Two different situations are considered: a vessel with a large blockage and a vessel with a limited blockage. The vessel is moored to one side of the lock. Therefore it is assumed that both situations occur, either on one side of the lock, see Figure 43.

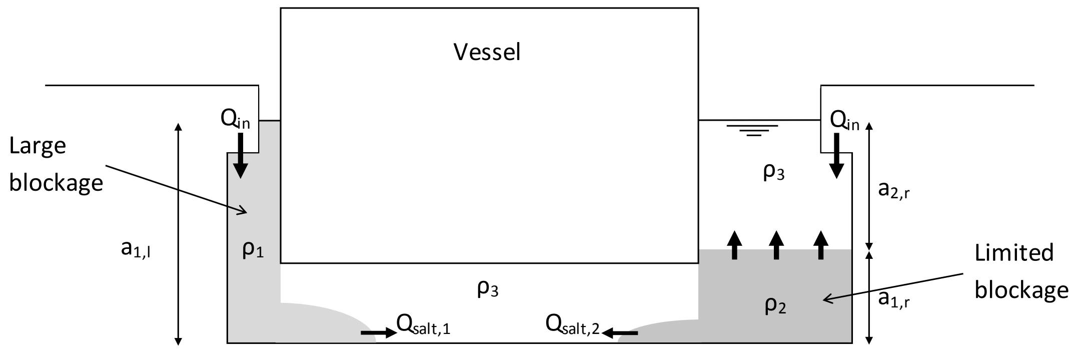



*Figure 43 schematization of salt layers in a lock due to sideways filling*

The situations with large and limited blockage have a separate approach. In the case of large blockage, the inflowing water doesn't have much space to distribute over the rest of the lock, because of the large blockage of the vessel. Therefore it is assumed that the inflowing water will mix completely with the column of water between the vessel and the lock wall. Initially this volume will be fresh water. The inflowing water with a higher density will cause the density of the volume to gradually increase until the density of the salt water. It is assumed that the density is constant over the entire water column. Because of the flowing water and the density difference between the volume and the remaining water, a salt water wave will start to propagate to the other side of the lock with a wave velocity of $c_1$. This means that a certain mass of salt will leave the volume again. The discharge of salt in kg/s leaving the volume depends on the distance between the wall and the vessel and the density of the volume at that moment:

$$Q_{salt,1}(t) = Q_{in}(t)\frac{0{,}5w - x_v}{0{,}5w}(\rho_1 - \rho_{water})$$

This salt water wave will eventually influence the salt layer at the other side of the lock. The front velocity of the propagating salt layer is (Deltares, 2015):

$$c_1 = c'_1\sqrt{\frac{1}{2}\epsilon g(h_a + h_l)}$$

With:

| | | |
|---|---|---|
| $c'_1$ | = | coefficient for front velocity (0,42 for a fresh lock, 0,46 for a salt lock) (-) |
| $\epsilon$ | = | relative density: $(\rho_1 - \rho_3)/\rho_3$ (-) |
| $h_a$ | = | water depth of the approach harbour (m) |
| $h_l$ | = | water depth of the lock (m) |

For the other side of the lock with limited blockage, a different approach is used. Because more space is available for the inflowing salt water, this will not completely mix with the entire water column next to the vessel. It will only partly mix and form a mixing zone on the bottom of the lock. This zone will have a constant density, depending on the level of entrainment of the vessel. This is defined by the mixing coefficient β. A value of 0,5 means that the density of the mixing zone is averaged between the densities of the approach harbour and the lock. The value of β is usually between 0,5 and 0,8. The thickness of the mixing zone ($a_{1,right}$) will develop in time, due to the inflowing salt water. The same as for the case with large blockage, a salt wave will propagate from the mixing zone to the other side of the lock. The velocity of the wave can be calculated with the same formula given for the case of large blockage. It is different however because of the relative density. The discharge of salt in this wave is given by:

$$Q_{salt,2}(t) = c_2 a_{1,r} L_l(\rho_2 - \rho_{water})$$

Summarizing, for the two sides of the vessel, two main factors change in time that are used to determine the load on the vessel: on the left side (large blockage) this is the density of the water column, on the right side (limited blockage) this is the height of the mixing zone. To determine the development of these factors, a mass balance is put up for large blockage, according to (Vrijburcht, 1991). The terms that determine this mass balance are the filling discharge of the lock, the change of mass (density change) in the zone, the generated salt wave and the incoming salt wave generated a certain time step before from the other side.

$$(\rho_a - \rho_{water})Q_{in}(t) - Q_{salt,1}(t) + Q_{salt,2}(t - \frac{b}{c_2}) = h_l x_v L_l \frac{d\rho_1}{dt}$$

For limited blockage (right hand side of Figure 43), a volume balance is used for the mixing zone. The terms that determine the volume are the filling discharge on that side of the lock, the volume change (change of mixing zone thickness) in the zone, the generated salt wave and the incoming salt wave generated a certain time step before from the other side.

$$Q_{in}(t) + Q_{in}(t)\frac{1-\beta}{\beta} = (w - b - x_v)L_l\frac{da_{1,r}}{dt} - \frac{Q_{salt,2}(t)}{(\rho_2 - \rho_{water})} + \frac{Q_{salt,1}\left(t - \frac{b}{c_1}\right)}{(\rho_2 - \rho_{water})}$$

To visualize the evolution of the two factors, they are determined for the test case with $\rho_a$=1020 kg/m$^3$ (approach harbour), $\rho_{lock}$,=1000 kg/m$^3$ and β=0,5, see Figure 44. It can be seen that the density of the large blockage mixing zone increases to the maximum density of the approach harbour. The height of the limited blockage mixing zone increases first, because of the increasing filling discharge, when the filling discharge decreases, the height of the mixing zone will start to decrease as well.

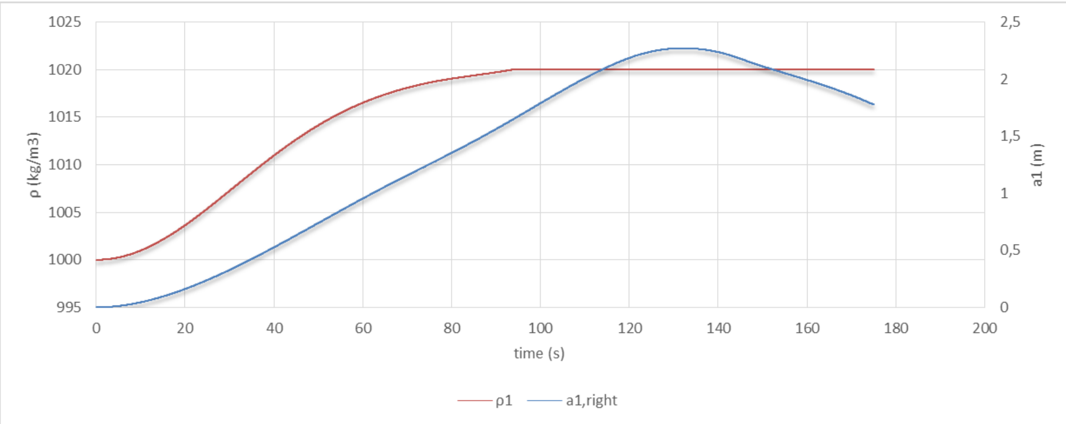



*Figure 44 Evolution of density of the mixing zone for large blockage and the thickness of the mixing zone for limited blockage*

With the densities and the thicknesses of the mixing zones derived, the transversal load on the vessel can be calculated. Several factors determine this load. First of all, the water level difference $\Delta h_{lr}$ between the two sides of the lock has to be determined. This water level difference is caused by the density differences, the thicknesses of the separate layers and the transport of momentum $I_{lr}$ along the vessel. Usually, the most salty layer has a lower water level. A set of continuity equations is used to determine the water level difference. A simplified solution is computed by (Vrijburcht, 1991) for the longitudinal direction, which corresponds closely to measurements. This equation is reformulated for the transversal direction:

$$\Delta h_{lr} = \frac{\frac{1}{2}\left(\epsilon_l {h_l}^2 L_l - \epsilon_r {a_{1,r}}^2 L_l - \epsilon_l {d_v}^2 L_v\right) + \frac{I_{lr}}{\rho_3 g}}{h_l L_l - d_v L_v}$$

The transport of momentum is determined by:

$$I_{lr} = \rho_1 a_{1l} {v_{1l}}^2 L_l + \rho_3 a_{2l} {v_{2l}}^2 L_l - \rho_2 a_{1r} {v_{1r}}^2 L_l - \rho_3 a_{2r} {v_{2r}}^2 L_l$$

Where $a_1$, $v_1$ and $a_2$, $v_2$ the thicknesses and velocity of respectively the salt and fresh layers are. The following values are used, corresponding to the two different mixing zones:

| | | |
|---|---|---|
| $a_{1l}$ | = | $h_l$ |
| $a_{2l}$ | = | 0 |
| $a_{2r}$ | = | $h_l - a_{1r}$ |
| $v_{1l}$ | = | $c_1$ |
| $v_{2l}$ | = | 0 |

$v_{1r}$ = $c_2$

$v_{2r}$ = $\frac{Q_{in}(t)\frac{0{,}5w-(w-b-x_v)}{0{,}5w}-c_2 a_{1,r} L_l}{(h_l-a_{1,right})L_l}$

When the thickness of the mixing zone of the limited blockage does not exceed the keel clearance of the vessel, the resulting force due to the density differences is as follows:

$$F_d = \frac{d_v \Delta h_{lr} + \frac{1}{2}\epsilon_l {d_v}^2}{b_v d_v C_b}$$

When the thickness of the mixing zone of the limited blockage side does exceed the keel clearance, another term is added to the force equation to take into account the different layers on the right side:

$$F_d = \frac{d_v \Delta h_{lr} + \frac{1}{2}\epsilon_l {d_v}^2 - \frac{1}{2}\epsilon_r (d_v - a_{2r})^2}{b_v d_v C_b}$$

The resulting load for the test case due to the density differences is given in Figure 45. The load is directed to the wall. It will start from zero and will gradually decrease to a minimum level at the moment that the density in the mixing zone of the large blockage has reached its maximum level. After that point the load will stay more or less constant until the end of the filling process.

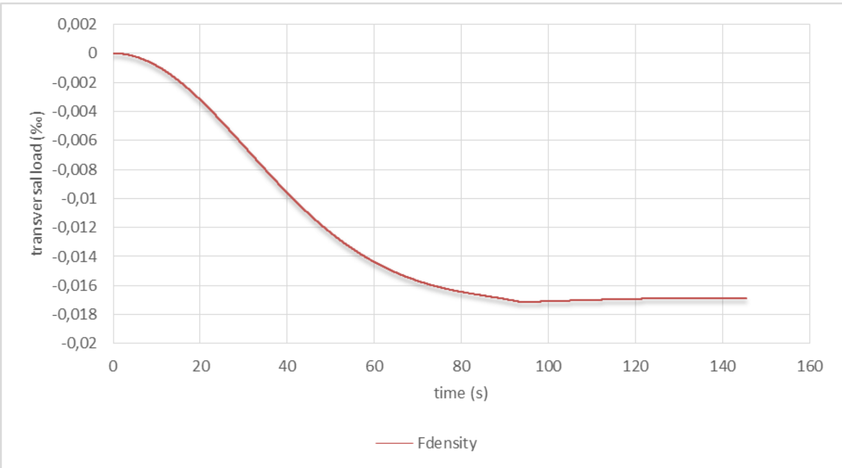



*Figure 45 Resulting load due to density differences*

NB: the above method is not yet validated in any way. The loads seem to be very small. The calculated water level difference between the two sides of the vessel, $\Delta h_{lr}$, is in the order of magnitude of 0,5 meter for the test case. With a maximum density difference, a short calculation gives an estimation of the occurred transversal load: $\frac{1000*15{,}7^2-1020*15{,}2^2}{366*15{,}2*49*0{,}8*1000} * 1000 = 9‰$. This is a factor 450 larger than the values calculated before. Therefore doubts are stated about the reliability of the calculation method for both the load and the water level difference. The method described above is still used in the results of the calculations, because a better calculation method is not yet available.

### *Total transversal load*

To determine the total transversal load on a vessel, the above components of the translatory waves, momentum difference, friction and density differences should be added up. This results in a total load that starts with a positive value, but decreases to a larger negative value. The main component is the translatory wave. The density difference and the momentum difference both have a negative influence on the transversal load, but less than the translatory waves. The influence on the minimum load can however not be neglected. The friction load is very small compared to the others and the contribution to the total load might be negligible.

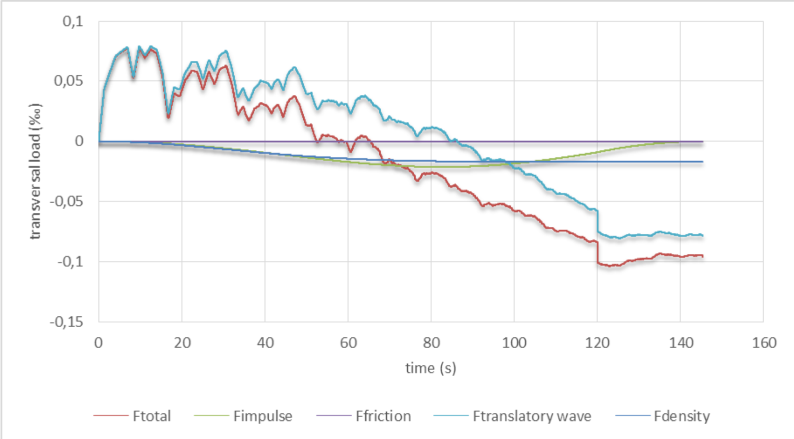



*Figure 46 Total transversal load for the test case with the separate components*

### *Influence of parameters*

Before any calculations are made with the above method, a sensitivity analysis is done. This is done to invest the influence of several parameters on the transverse load on the vessel. When this influence is known, more insight can be obtained to optimize the design of the filling system. The two main parameters that are researched are the eccentricity of the vessel and the opening valve dimensions. The influence of the dimensions of the vessel and the locks are not investigated, because it is considered that these are boundary conditions that cannot be changed by the designer, however he should take it into account. The load due to density difference are not taken into account in this part, because for certain cases the method will not apply anymore (for instance when the vessel is situated in the middle of the lock, both sides are the same, instead of one with large blockage and one with limited blockage).

**Eccentricity of the vessel**

The distance between the vessel and the wall influences the eccentricity of the vessel. In Figure 47 the influence is visualised for the test case. For the test case, the eccentricity is zero when the distance between the wall and the vessel is 9,5 m. The transversal load decreases linearly when the vessel's eccentricity becomes smaller.

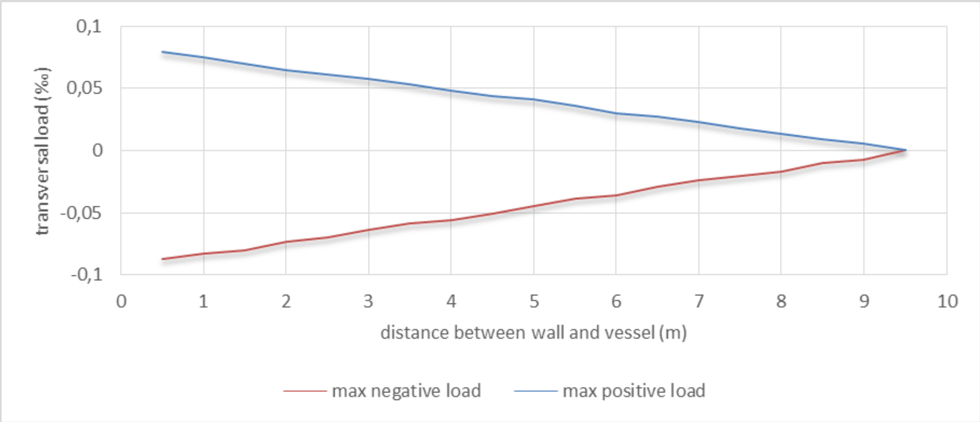



*Figure 47 Influence of the distance between the vessel and the wall on the transversal load*

The whole transversal load is only caused due to the eccentricity of the vessel in the lock. The eccentricity is made dimensionless with:

$$E = \frac{e}{\frac{1}{2}(w-b)}$$

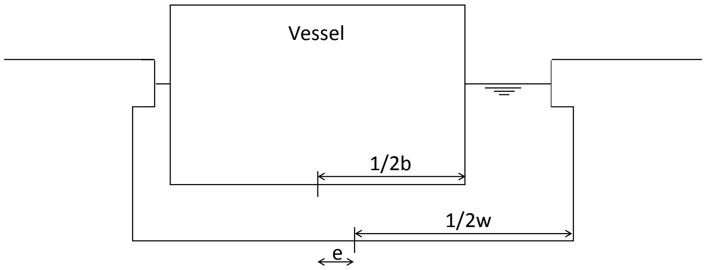



*Figure 48 Eccentricity of a vessel in a lock*

With e being the distance from the centre line of the vessel to the centre line of the lock. With E being zero, there is no transversal load. When E reaches one (maximum), the transversal load is maximum.

**Valve dimensions**

A similar valve system like for filling of a lock through the gate is used as stated before. The valves are opened with a certain speed, gradually increasing the discharge opening and therefore the discharge. Three parameters determine the discharge regime: The total opening width of the filling openings, the maximum opening height and the opening speed. These three factors define the blue graph shown in Figure 49, where the discharge is plotted together with the transversal load. Several parts in this graph determine the load on their hand. The graph can be divided into three parts: the increasing part (1), the maximum (2) and the decreasing part (3).

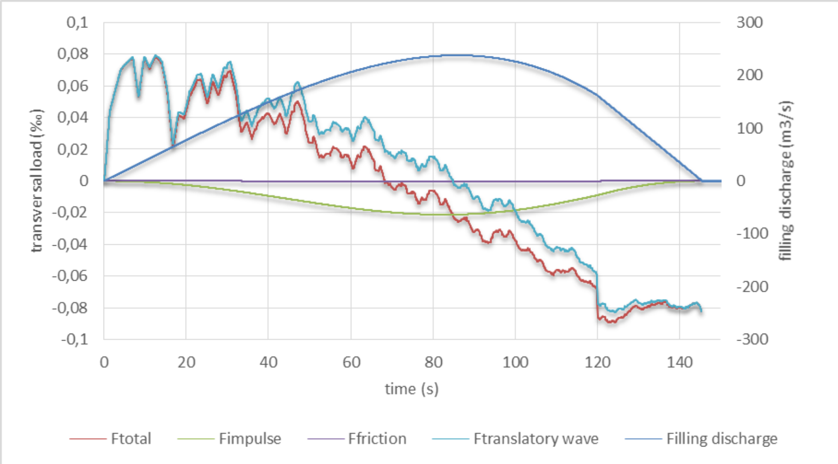



*Figure 49 The filling discharge and transversal load*

(1) First of all, the first peak in the load (usually the largest positive load) is highly influenced by the slope of the first part of the discharge. This means the opening speed and the total opening width determine the first peak.

(2) The height of the graph and thus the maximum discharge determine the maximum momentum load. The maximum discharge is also determined by the opening speed and the total opening width.

(3) In the third part the openings reach their maximum cross section. This causes kink in the discharge graph, which results in a jump in the load. Most of the time, afterwards this jump the most negative load can be seen. It is therefore important to let the kink be as small as possible.

Summarizing, to reduce the load as much as possible, the discharge graph has to be as low, long and smooth as possible. The first two are however in contrast with the discharge time, which has to be minimized as well. The total width and the opening speed has to be adjusted in such a way that the first peak meet the hawser force criteria. After that, the opening height has to be adjusted so that the discharge graph is relatively smooth and that the last peak in the graph meet the criteria as well.

## 6.4 Results

With the calculation method for the loads due to sideways filling and emptying known, the results of several combinations between vessels and lock sizes are derived. To investigate the influence of the new filling/emptying system on the levelling time, the conventional systems with filling through the head are elaborated as well. The levelling times for these systems are calculated using the software program LOCKFILL. For the test case, the results for these systems are given in Figure 50. The shape of the loads over time show much resemblance with the longitudinal culvert. The lockfill calculations results in a longitudinal force, instead of a transversal force for the longitudinal culvert. The main difference that can be seen is for filling through the gate, where the load due to translatory waves is smoother. Furthermore, with filling through the gate, the flow against the bow is also taken into account.

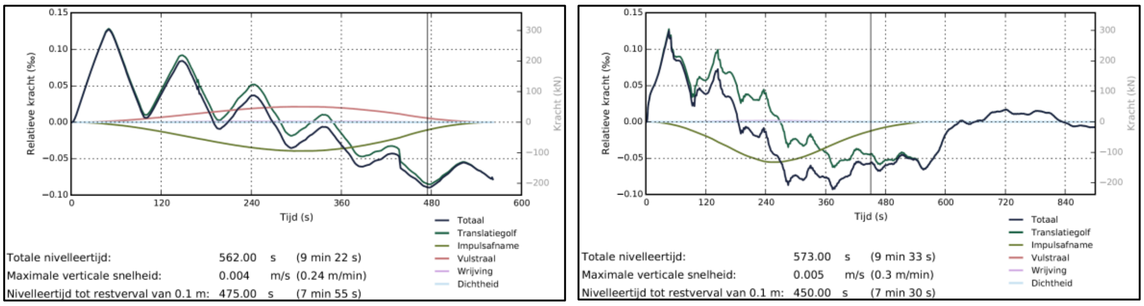



*Figure 50 Test case, left: filling through the gate, right: filling with short culverts*

Three different locks are being researched. First of all, the lock with the size according to the initial requirement is considered (l*b*d: 420m*56m*-18,3mNAP). Secondly, the wider lock as proposed in section 4.2 (l*b*d: 420m*68m*-18,3mNAP) is used as well. An even larger lock, for the extreme adaptive design is also researched.

- Initial: l*b*d: 420m*56m*-18,3mNAP
- Wider: l*b*d: 420m*68m*-18,3mNAP
- Adaptive: l*b*d: 460m*68m*-21,3mNAP

To determine the influence of different vessel, four different types of vessels are investigated. These are first of all the maximum vessel size according to the initial design: the New Panamax and the maximum vessel size for the adaptive design: the Malaccamax. Furthermore, the cruise vessel Oasis of the seas will be researched. This vessel has a relatively low draught compared to other freight vessels. This might influence the forces on the vessel. The last vessel to be researched is the smallest vessel which will probably use the lock to complete the range of vessels. This smallest vessel is assumed to have the same size as used for the design of the mid-sized lock. summarizing, the following vessel are taken into account:

- New Panamax: l*b*d: 366m*49m*15,2m
- Oasis of the seas: l*b*d: 361m*47m*9,3m
- Malaccamax: l*b*d: 400m*59m*18m
- Smallest vessel: l*b*d: 215m*32m*9m

The possible filling times for all of the combinations are given in Table 26. Several side notes need to be accounted for before looking at these results:

- In all of the situations, the water level in the approach harbour is 1,53 m + NAP, the initial water level in the lock is 0 m + NAP.
- For the calculation of the longitudinal culvert, the net discharge openings are never larger than the wet cross sectional area of the main culvert with a width of 15 meter and a depth of -4,5 m + NAP. The water level in the main culvert is assumed to be the same as the approach harbour.
- Pragmatic choices are made for the boundary conditions in Lockfill based on rules of thumb given in (Glerum, et al., 2000). The computations could therefore be further optimized.
- The maximum allowed longitudinal force (in case of filling through the gate or short culvert) is set on 0,12 ‰. This is lower than the value given before (0,25 ‰), but the used

version of Lockfill does not take density differences into account. It is assumed that the load due to density differences is also 0,12 ‰, based on expert judgement. This makes the total allowed longitudinal force 0,24 ‰.

- The maximum allowed transversal load is set on 0,11 ‰, as mentioned before.
- Only filling of the lock is considered.
- A potential maximum allowed vertical velocity of the water level is not taken into account. in reality this might however be the case due to handling of the hawsers.

| Filling times (s) | Initial | | | Wider | | | Adaptive | | |
|---|---|---|---|---|---|---|---|---|---|
| | Long. culvert | Gate | Short culvert | Long. culvert | Gate | Short culvert | Long. culvert | Gate | Short culvert |
| New Panamax | 97 | 668 | 630 | 141 | 562 | 573 | 122 | 553 | 528 |
| Oasis of the seas | 103 | 509 | 468 | 120 | 507 | 468 | 130 | 504 | 501 |
| Malaccamax | - | - | - | - | - | - | 122 | 722 | 688 |
| Small vessel | 108 | 474 | 474 | 134 | 450 | 484 | 135 | 448 | 501 |

*Table 26 Filling times (s) different filling systems*

When looking at the results, it can be seen that the longitudinal culvert system can ensure a much faster filling time. The detailed results are given in appendix C. A filling time of approximately two minutes seem to be possible with the longitudinal culvert, while the other systems need about 10 minutes to fill the lock. In most of the cases, the short culvert system is slightly faster than the filling through the gate. These are however minor differences. Besides of that, the filling time for the longitudinal culvert is not always limited by the transversal load on the vessel, but sometimes also by the wet cross section of the main culvert. When the size of this culvert is further optimized in the structural design, even faster filling times might be possible. The filling times for the New Panamax vessel are visualized in Figure 51. When filling of the wider lock is compared to other lock dimensions, it can be seen that the longitudinal filling system still results in much faster filling.

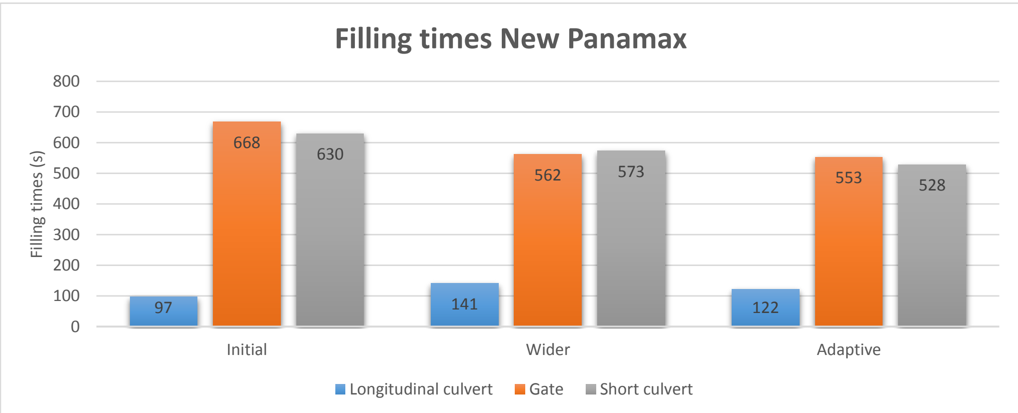



*Figure 51 Filling times different filling systems and different lock sizes, vessel: New Panamax*

In Table 27 the relative filling times are displayed, where the system with filling through the gate with the initial lock size is set as reference level. The average relative filling time of the longitudinal culvert is 23 %, more than 4 times as fast. The best results are obtained for the largest vessels, where the filling times are reduced the most, compared to other systems. This is expected, because with a large vessel, the eccentricity is smaller.

| Filling times (%) | Initial | | | Wider | | | Adaptive | | |
|---|---|---|---|---|---|---|---|---|---|
| | Long. culvert | Gate | Short culvert | Long. culvert | Gate | Short culvert | Long. culvert | Gate | Short culvert |
| New Panamax | 15 | 100 | 94 | 21 | 84 | 86 | 18 | 83 | 79 |
| Oasis of the seas | 20 | 100 | 92 | 24 | 100 | 92 | 26 | 99 | 98 |
| Small vessel | 23 | 100 | 100 | 28 | 95 | 102 | 28 | 95 | 106 |

*Table 27 filling times (%) different filling systems relative to filling through the gate in the initial situation*

An important comparison to make is the one with the head filling systems of the initial design with the longitudinal culvert of the wider design. This is an important factor to decide whether to implement the wider design in the initial design. The filling times for the longitudinal culvert for the wider design are on average 24% of the average filling time for the filling through the gate in the initial design. This means that even with a wider lock chamber, faster levelling can be obtained by a factor 4.

**Different water levels**

To investigate the influence of the difference in locking water levels, the filling times for several water levels are computed. These are first of all the water levels according to the maximum and minimum water levels for which locking is allowed (-1,85 m + NAP and 4,3 m + NAP). Besides, the water levels according to the spring tide are used as well (-0,55 m + NAP and 1,53 m + NAP). The results are given in Table 28.

| Water levels: approach harbour ; lock (m+NAP) | Longitud. culvert | Gate | Short culvert |
|---|---|---|---|
| 0,00 ; -1,85 | 220 | 709 | 674 |
| 0,00 ; -0,55 | 95 | 351 | 341 |
| 1,53 ; 0,00 | 141 | 562 | 573 |
| 4,3 ; 0,00 | 249 | 974 | 1085 |

*Table 28 Filling times (s) for different water levels, new Panamax, wider design*

The filling times for different water levels show similar results. The differences between the longitudinal culvert and the other systems for the lower water levels are however smaller, see Figure 52. This is mainly caused due to the lower wet cross sectional area of the main culvert, which is caused by the lower water level. The differences are however still considerable, filling with a longitudinal culvert could still be done 3 times as fast.

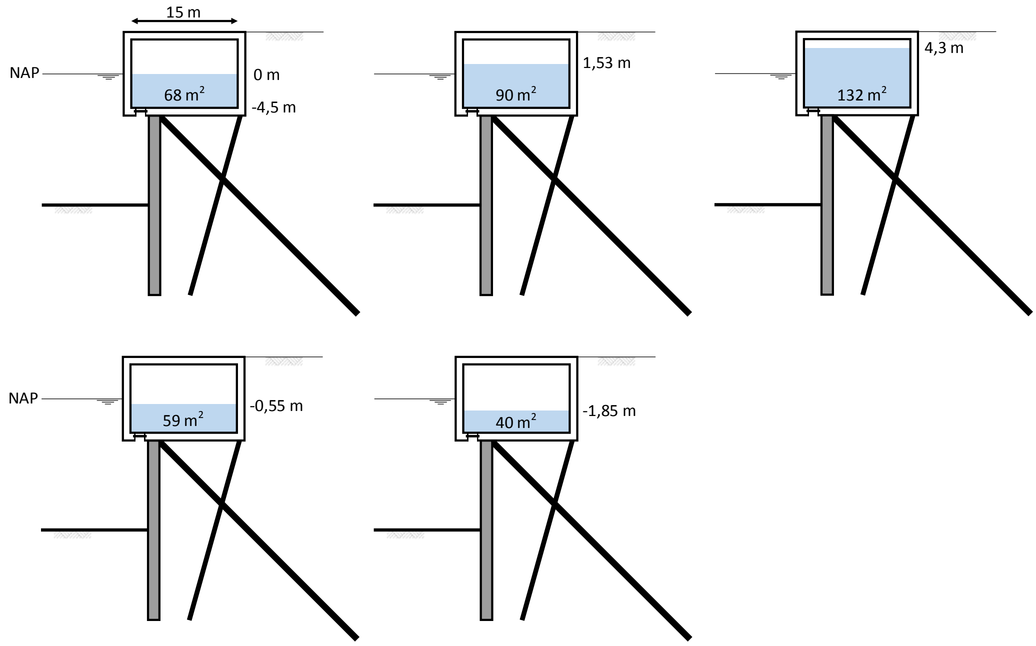



*Figure 52 Wet cross sectional areas different locking levels*

Summarizing, the longitudinal culvert results in much faster filling. Compared to filling through the gate, levelling times are reduced with a factor 3. In most of the cases, a factor 4 or even 5 is achievable as well.

**Total locking time**

When looking at the locking process, the levelling time is not the only aspect that determines the total locking time. The total locking process that determine the passing time described by the following subsequent processes:

1. Opening of the gate
2. Sailing in of the vessel
3. Closing of the gate
4. Levelling water level lock
5. Opening of the gate
6. Sailing out of the vessel

The total locking time is not derived in this research, but for large locks for seagoing vessels, this can increase up to 50 minutes, depending on the size of the vessel and the water level difference. It is therefore important to show the results in the light of the total locking process. When the levelling time is decreased with 75 % (in the case of the new Panamax vessel with a water level difference of 1,53 m), the total locking time is only decreased with 14 % for an initial locking time of 50 minutes.

**Conclusion**

Out of the above results it can be concluded that the longitudinal culvert levelling system results in smaller levelling times compared to other levelling systems. This is the case for all of the different situations regarding the size of the vessel and the dimensions of the lock. Therefore it is concluded that it is worthwhile to develop this system in more detail. Several aspects however, require more research to draw a better conclusion. These are mainly the following:

- The density differences. As mentioned before, the calculated force due to density differences should be validated.
- The longitudinal load. Due to uneven filling along the length of the lock, a longitudinal gradient could occur, causing a longitudinal load. This influence is not yet derived in this research.
- The energy losses (friction and energy dissipation due to bends and in- and outflow) in the longitudinal culvert. It should be researched how much energy is lost due to these aspects. This might influence the filling time negatively (Molenaar, 2011).
- The structural feasibility of the structure should be quantified.

The latter aspect will be researched in more detail in the following chapter. The structural feasibility of the culvert together with the relieving floor will be worked out, to prove that it is possible to build the structure with normal circumstances. The cross sectional wet surface area used in the hydraulic computations (floor on -4,5 m + NAP and a width of 15 m), is also used in the structural elaboration.

# 7

# Structural design of a relieving floor

As stated in chapter 4, the relieving floor with retaining wall will be used as chamber wall of the large lock. The relieving floor in combination with a longitudinal culvert has never been built before. Because the dimensions are different than a common relieving structure (larger superstructure and filling openings) a feasibility study of the structural integrity of the structure will be made in this chapter.

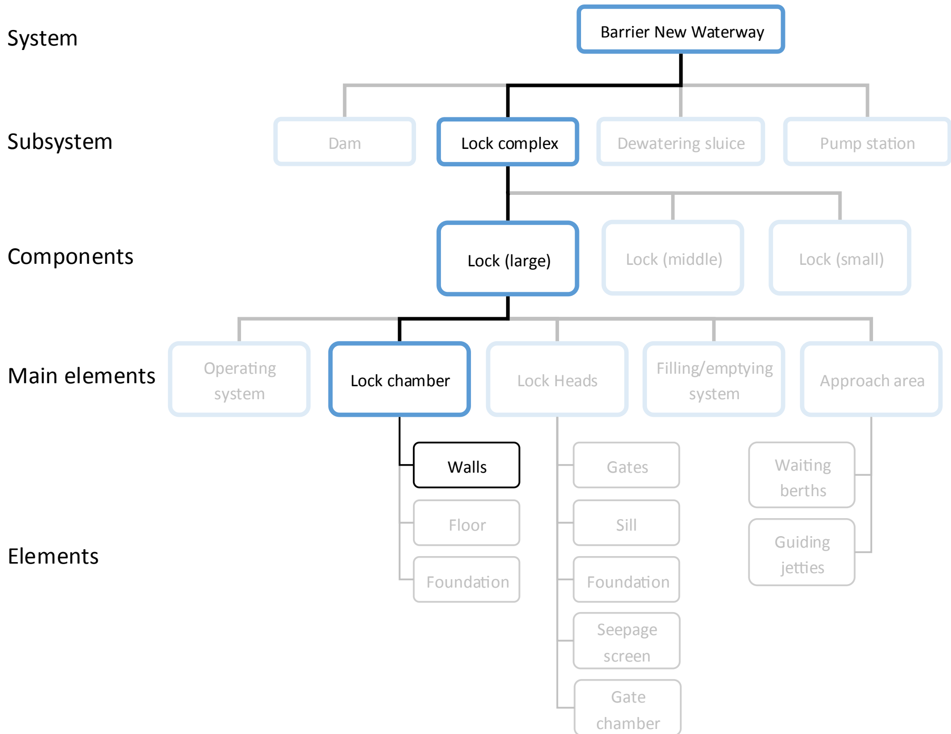



*Figure 53 Design tree, position of the relieving floor in the design process*

The goal of this chapter is not to give final dimensions for which the structure can be build, nor an optimised design will be given. The goal is to prove that the structure can be built without unrealistic design requirements for the different elements. Therefore the main aspects regarding the soil stability and the structural integrity will be researched on. First of all, the concept of the relieving floor as a chamber wall is discussed. Subsequently, the model approach for the superstructure and sub structure will be elaborated on. After that, the method to calculate the soil stability is given. Finally, the checks will be given that are done to ensure a safe structure given.

## 7.1 Concept relieving floor

An estimation for the design should be made as starting point. Therefore, first the important aspects of the concept of the relieving floor are discussed, before the dimensions are given.

**Working of a relieving floor**

The concept of a relieving floor is widely used for quay walls in deep ports. The combination of a relieving floor with a retaining wall is structurally beneficial compared to one continuous retaining wall from the top to the bottom. This is caused by the fact that the relieving floor reduces the horizontal load on the retaining wall. At the back of the relieving floor, the maximum horizontal pressure is present. Because of the internal friction angle of the soil, this horizontal pressure does not work directly on the retaining wall, see Figure 54. The soil pressure on the wall will develop gradually downwards, until it reaches the full pressure, which is also present at the vertical through the back of the relieving floor. Because the full horizontal pressure is not present everywhere, the bending moments in the retaining wall are reduced. The amount of reduction is influenced by the width and the depth of the relieving floor. A wider and deeper floor means a lower bending moment in the retaining wall. An optimum has to be found between structural feasibility of the relieving floor (super structure) and the retaining wall (sub structure).

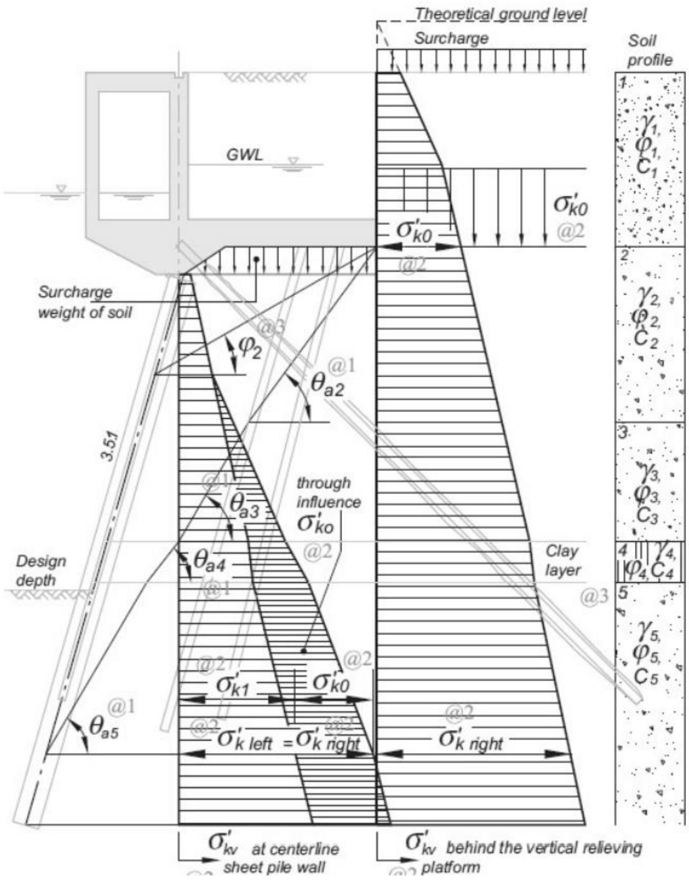



*Figure 54 working of a relieving floor (de Gijt, et al., 2013)*

**Concept structure**

Two different solutions are proposed for the culverts of the relieving floor. The first solution proposes a culvert with a roof. In this way the culvert is always filled with water under normal circumstances. This results in the fact that the water in the culvert is not a part of the water that has to be locked (this is not the case without the roof). Thus only one valve is necessary at the beginning of the culvert. A drawback of this solution that it has to be placed relatively deep, because of the required cross sectional area of the culvert.

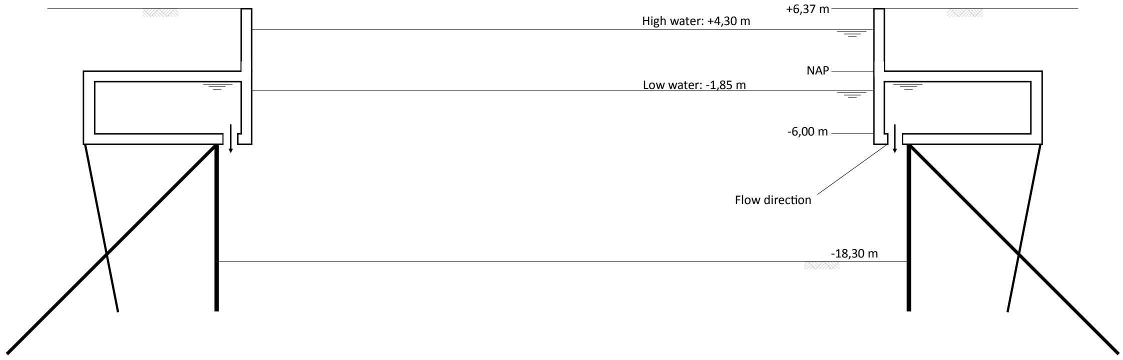



*Figure 55 Cross section lock (culvert with roof)*

The second solution doesn't have a roof in the culvert that restricts the flow. The culvert can therefore be placed higher, which is probably structurally beneficial. When only one valve is used at the beginning of the culvert, the water in the culvert is part of the water that has to be locked. To prevent this, numerous valves are placed in the openings between the culvert and the lock chamber. A drawback is that these valves are placed under water. This is not desirable regarding the maintainability. It is however probably possible to make an easy system to replace these valves. A lifting crane can be installed on top of the relieving floor. The valves can then be lifted out easily without causing too much problems for the availability of the lock. Besides, it is even possible to temporarily use only one culvert instead of two to make maintenance possible. the valves at the beginning and the end of the culvert are necessary as well, otherwise the inland canal and the sea are in open connection with each other. This last option is considered to be the best option and will therefore be used in the further elaboration.

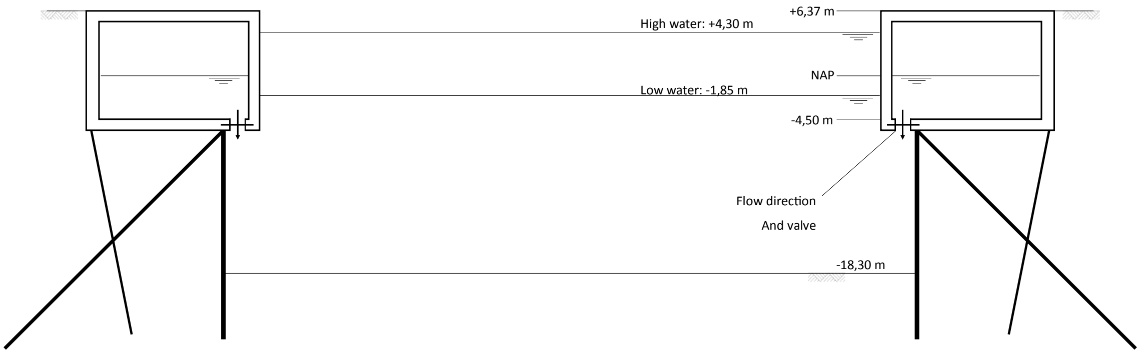



*Figure 56 Cross section lock (culvert without restricting roof)*

### *Dimensions*

First of all, the required dimensions for the hydraulic aspects are estimated. It turns out that the cross section of the culvert should be as big as possible and that this is the determining factor for the filling speed. This means a deeper and wider cross section. This cross section is however limited by the fact that the superstructure will be extremely expensive when it is enlarged too much. It will take over the retaining function of the wall. A practical choice is made for this cross section, taking both aspects into account and based on references in (de Gijt, et al., 2013). The dimensions are given in Figure 57. The floor will be placed at a depth of -5,5 m + NAP. This is also used in the hydraulic calculations in chapter 6.

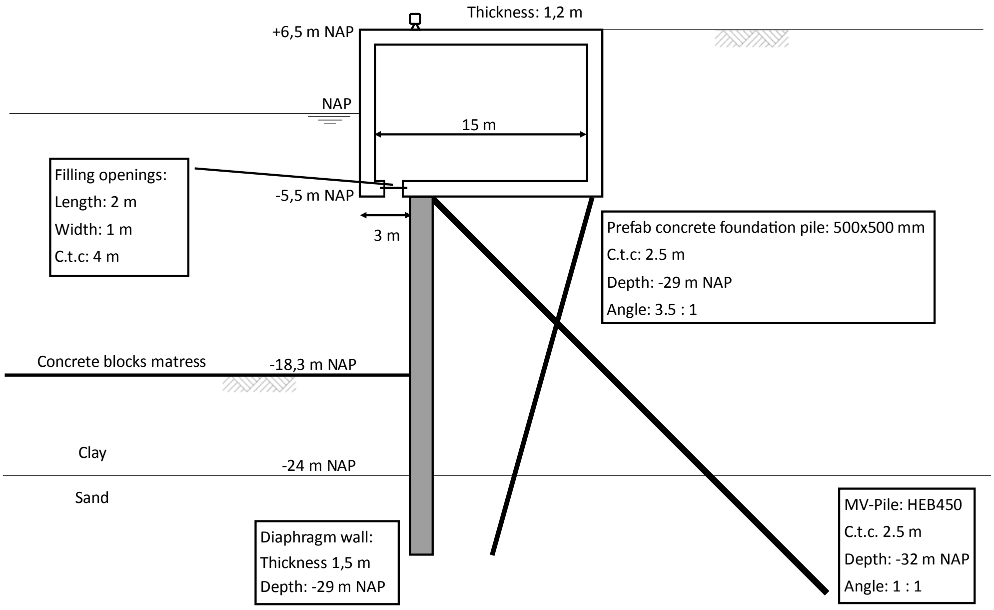



*Figure 57 Dimensions relieving floor and diaphragm wall*

The superstructure will be made out of concrete. The retaining wall will be made with the use of a diaphragm wall, which is also made out of concrete. It is also possible to make this wall with the use of a combined sheet pile wall, made out of steel. Because of the filling openings, a lot of water, with a high oxygen level will flow along the wall. Due to corrosion problems that could occur with a steel sheet pile wall, a concrete diaphragm wall is assumed to be more durable. This does not necessarily mean that a steel sheet pile wall cannot be used. With additional measures against corrosion, a steel sheet pile wall could be feasible as well.

At the back of the superstructure, a prefab concrete foundation pile will be used to retain the structure. This pile is placed under an angle of 16° with the vertical. At the front of the wall, an MV-pile will be placed under an angle of 45°. By placing these piles under an inclination, the horizontal loads on the structures can be transmitted. This is a common building method for these kind of structures. Lengthwise, the relieving floor will be made in slices of 44 meter. This is a common length to have an expansion joint. The bollards will be placed in the same configuration on every slice. They are placed in pairs with a centre to centre distance of 4 meter. The centre to centre distance of two pairs is 22 meter.

### *Building method*

To conclude whether the structure is technically feasible, the building method should be determined. The structure will mainly be built on land. Therefore it is considered that building will be done in a dry excavating pit. Regarding the use of equipment, this is probably less expensive than building from the water. The part of the wall that has to be built in where now water is, reclamation of land is necessary to make a dry building method possible. The following steps need to be followed in the building process:

1. Reclamation of land
2. Excavating building pit
3. Dewatering building pit
4. Placing diaphragm wall
5. Placing MV-piles and concrete foundation piles

6. Building superstructure
7. Dredging of soil within the lock and placing bed protection
8. Back fill of the superstructure.

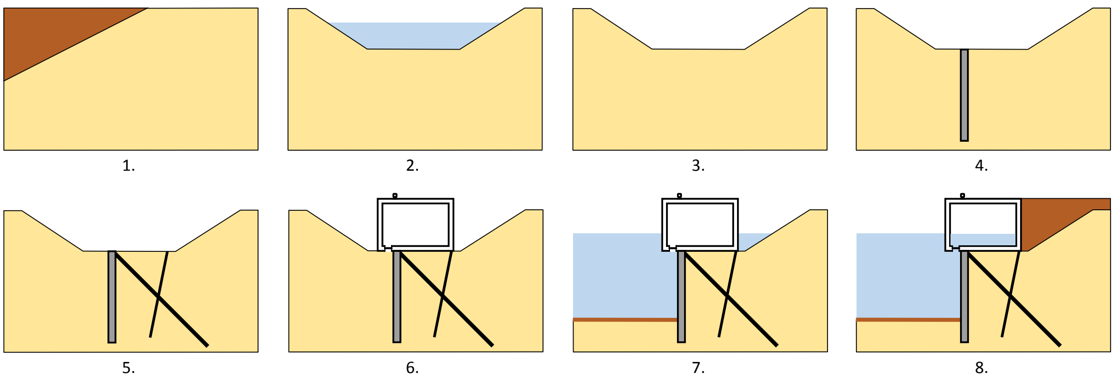



*Figure 58 Building method*

The different building phases of the structure results in different load combinations. One has to be aware that not always the final situation is the governing one, but one of the construction phases might that be as well. For example when the bed protection in front of the retaining wall is placed. Dredging should be done deeper than the final design, to make the placement of the bed protection possible. The final situation is however assumed to be the governing situation and is the only one which is taken into account.

Due to the excavation works, relaxation of the soil can occur. This might cause unexpected deformations of the front wall. Therefore the different building phases should be elaborated in the right order to take this relaxation into account. It is however expected that this influence is limited. Excavation will be done in a wet situation. The pressure of the water column is therefore still present on the soil. Because of that, the relaxation of the soil is limited. This is not further considered in this phase of the design.

## 7.2 Boundary conditions

When making a design, one has to take the surroundings into account. In this case, this means the loads that occur on the structure and the important soil aspects.

### *Soil profile*

A simplified soil profile will be used with only two different materials: clay and sand. The upper layer consists mostly of clay with a little bit of sand. The layer underneath consists of densely packed sand and will provide most of the retaining function. This sand layer will start at -24 m NAP (TNO). The soil parameters for soil are estimated based on references and usual values used for these kind of soils. These are given in Table 29.

| | Cohesion c (kPa) | Weight γ (kN/m$^3$) | Angle of repose ϕ (°) | Wall friction angle δ (°) |
|---|---|---|---|---|
| Clay | 10 | 17 | 23 | 13 |
| sand | 0 | 20 | 35 | 20 |

*Table 29 Soil parameters*

For the stability function of the different elements regarding the soil, a sounding (S37B02682) is used from (TNO) to estimate the bearing capacities of the soil. This sounding is taken in the New

Waterway, it is the closest one to the project location and will be used as the representative one for the soil.

### *Load combinations*

To make a design of the structure, different situations have to be taken into account. For this design, only the initial boundary conditions are taken into account. Any adaptive conditions that are mentioned before, will not be used because of the added complexity. The adaptive requirements will result in higher demands for the structure. It should be researched later on whether it is possible to meet these more stringent requirements. Three different situations are looked at for this design:

1. Maintenance situation, no vessel is present, the water level in the lock is just below the superstructure
2. Vessel situation, Vessel is present in the lock and moored at the relieving floor, the water level is situated at the lowest locking level of -1,85 m + NAP.
3. High water situation, no vessel is present, the water level in the lock and the culvert is at maximum water level of 6,35 m + NAP.

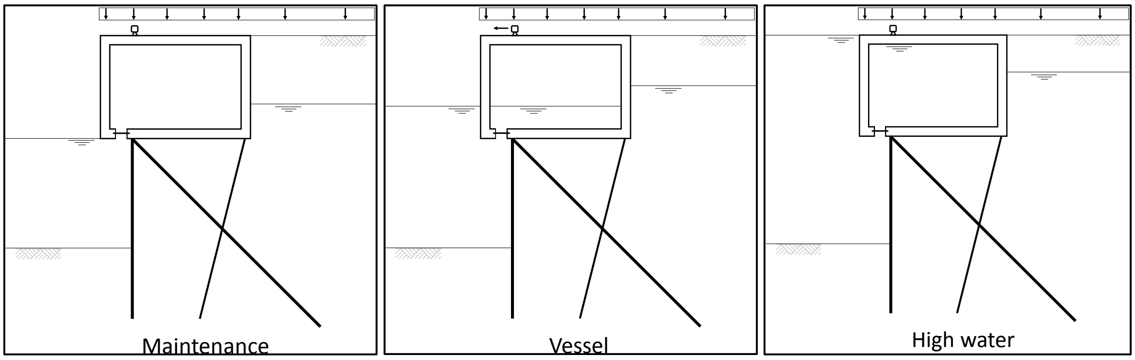



*Figure 59 Load situations, from left to right:1: Maintenance, 2:Vessel, 3: High water*

More situations can be thought of, but it is considered that the above are the most governing. All of these situations result in different load combinations and therefore in different deformations and force distributions in the structure. The main loads are the active ground and water pressures. Besides of that, bollard loads and variable vertical loads on top of the structure has to be taken into account. The bollard load is estimated at 1000 kN per bollard (Glerum, et al., 2000). A variable vertical load of 20 kN/m$^2$ is considered. This load consists of certain objects or material that can be present on top of the structure. For example a crane that is used for maintenance. This vertical load is present in all the situations.

## 7.3 Model approach

To make an estimation of the force distribution in the structure, computer models are used. The structure consists of two different elements: the superstructure and substructure. For both structures different models are used. For the substructure, the retaining diaphragm wall, the model D-sheet piling is used. For the superstructure, the relieving floor, the model Scia Engineer is used. Both structures are however connected to each other. Besides of this, both structures are connected to the MV-pile. Therefore, the models should be combined. This is done by putting a horizontal spring support at the connection in the two models. This spring represents the horizontal stiffness of the MV-pile. The two models are combined in such a way that the horizontal deformation and the horizontal force at this support are the same. The horizontal deformation does not only depend on the stiffness of the MV-pile, but also on the stiffness of the diaphragm wall, the soil, the back foundation piles and the superstructure.

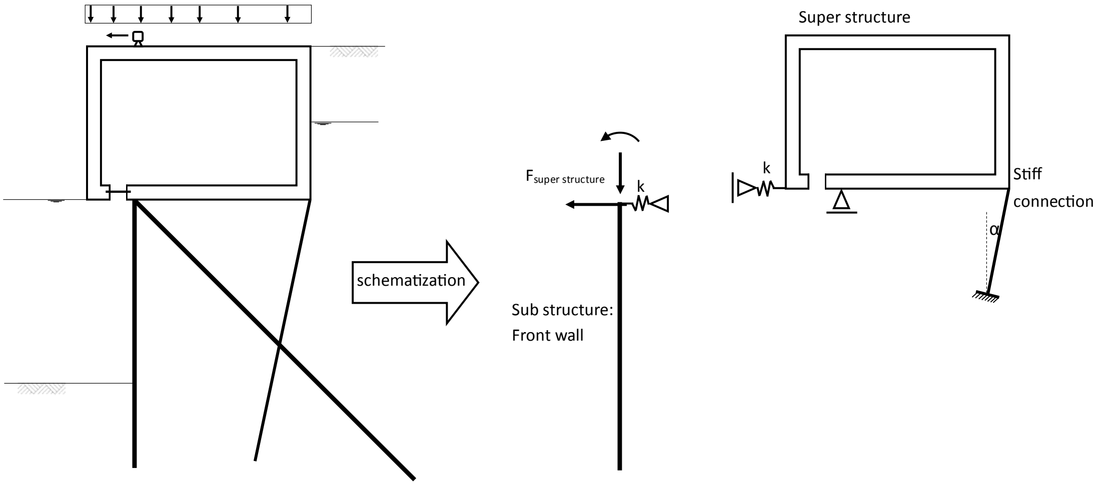



*Figure 60 Model approach, structure divided into sub- and superstructure*

Due to the construction method, it is likely that the MV-pile will be connected with the superstructure, see Figure 61. The superstructure is than supported vertically on the diaphragm wall. The horizontal displacement of the diaphragm wall is limited by the superstructure. The horizontal force in the diaphragm wall is thus transferred via the superstructure to the MV-pile. This is an important detailing aspect that has to be taken into account by making a final design.

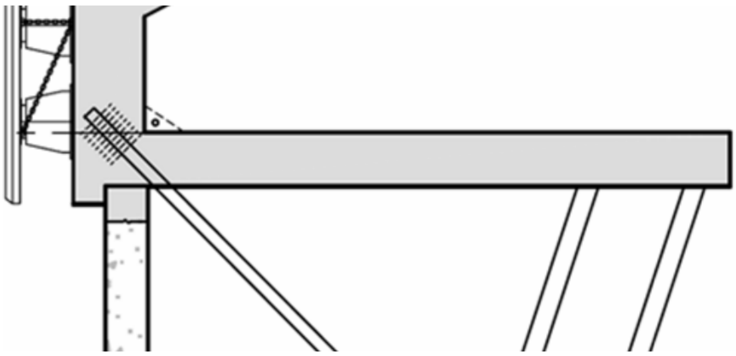


*Figure 61 Detail connection MV-pile with relieving floor (BAM/Delta Marine Consultants, 2010)*

## *Superstructure: relieving floor with culvert*

The superstructure consists of the concrete culvert on top of the diaphragm wall. This structure is supported on the lock side on top of the diaphragm wall and the MV-pile, which is modelled with a hinged support and a spring in horizontal direction. On the back side it is supported on the inclined foundation pile. The connection between the foundation pile and the superstructure is considered to be stiff. This makes it a statically indeterminate structure. To calculate the force distribution, the structure is modelled in the 3D computer program Scia engineer. Because it is a statically indeterminate structure, the stiffness is required to calculate the force distribution in the structure. The cracked stiffness of concrete is used in the model. The added stiffness due to the reinforcement steel is neglected here. Furthermore, the influence of the filling openings on the lock side can be taken into account with the program. The filling openings are 1 meter wide and 2 meter long. They have a centre to centre distance of 4 meter.

It is expected that the members in between the filling openings are subjected to larger forces, because the load has to be redistributed.

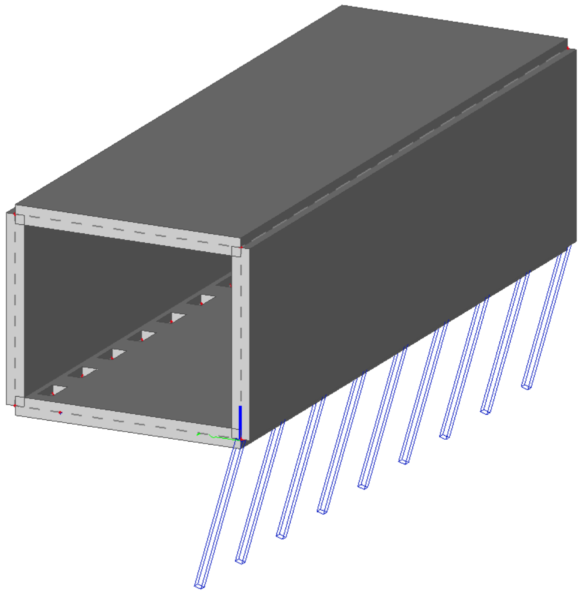


*Figure 62 3D-Model of one slice of super structure*

A slice of 44 meters is considered to be made in one piece. This is also used to make the model. This is a common distance to have an expansion joint. The deformations in longitudinal direction are however not exactly derived. It might be possible that due to thermal differences, the superstructure will deform in a different way than the substructure. When this deformation is obstructed, internal stresses might occur in the concrete. In the final design it should be investigated what the influence of the thermal differences is on the structure.

The back foundation piles are also modelled in Scia engineer. A length of 10 meter is used to take into account the stiffness of the piles. The stiffness of the soil has not be taken into account in this approach. This should be done however in the final design, because this stiffness might be more important than the stiffness of the foundation pile itself.

Several loads are entered in the model for the three different load situations. The exact calculation of the following loads are given in Appendix 0:

- Bollard load (vessel)
- Water lock side (vessel, high water)
- Water back side (vessel, high water, maintenance)
- Water in the culvert (vessel, high water)
- Soil pressure back side (vessel, high water, maintenance)
- Top load (vessel, high water, maintenance)

The results of the structure are obtained for two important cross sections. One that crosses a filling opening and one that crosses the member right in between the filling opening, see Figure 63. Right next to the filling opening the internal forces in the floor plate are extremely high. These are not taken into account, because it is assumed that these are caused by flaws in the computer model. Despite of this, attention should be given to this detailing aspect when the final design is made.

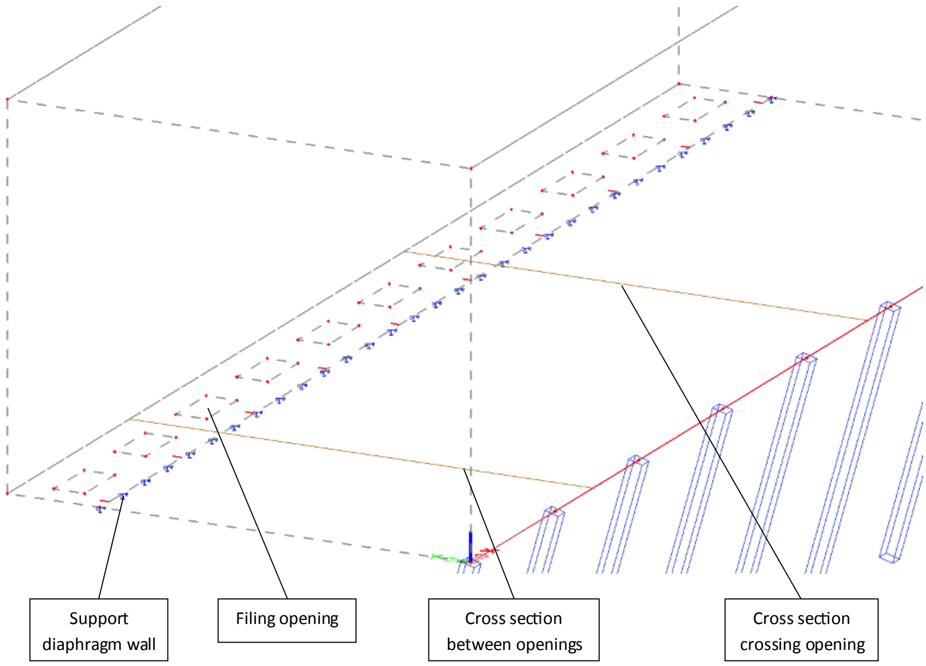



*Figure 63 Researched cross sections super structure*

### ***Substructure: diaphragm wall***

The deformations and the internal forces in the diaphragm wall depend mainly on the horizontal loads exerted by the soil on the wall. This horizontal soil load is known to be present in two different states: passive and active soil pressure. The passive soil pressure is the reaction pressure of the soil when a load works on a certain soil mass. The active soil pressure is the load that is generated by the soil itself because of the weight and any surcharge on the soil mass. In reality, there is not a clear boundary between these two states, but the pressure will develop gradually between active and passive soil pressure. This depends on the deformations of the soil and the retaining structure (when the retaining wall will deform, the horizontal active load on the wall will be smaller). A common method to take this into account, is to model the retaining wall as a beam on an elastic foundation (CURNET, 2012). The elastic foundation represents the soil stiffness as a set of springs. Because this is a laborious matter to do by hand, the computer program D-sheet piling is used for this. D-sheet uses a series of iterations to take into account the interaction between the soil and the diaphragm wall. The soil pressure coefficients (active and passive) are calculated according to (NEN, 2012). The exact calculation of these coefficients is given in Appendix 0.

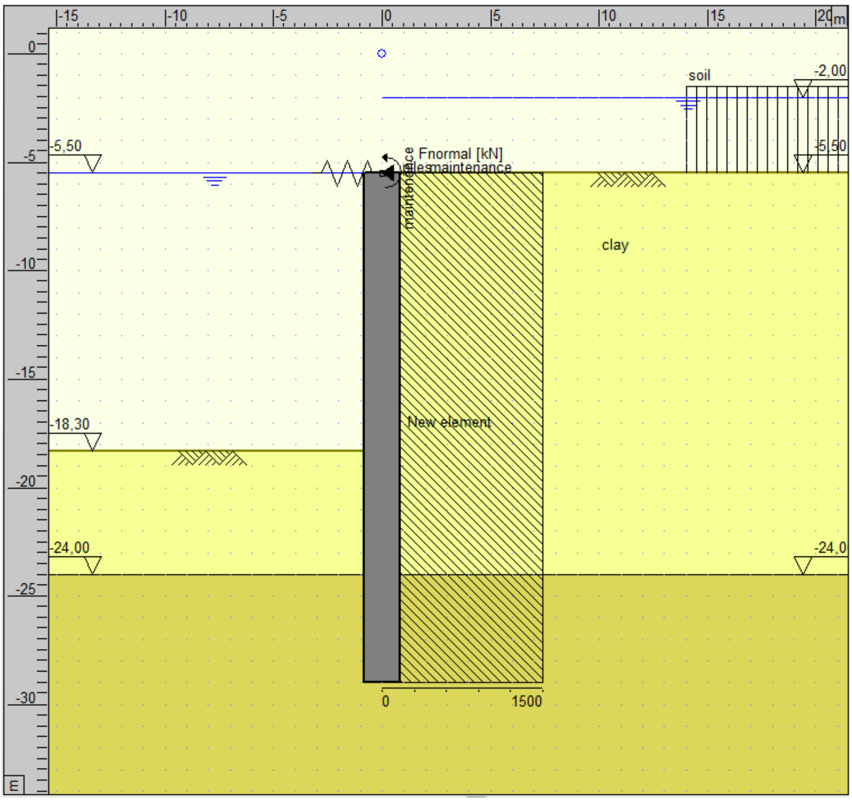



*Figure 64 D-sheet piling model*

**Input D-sheet piling**

The computer program requires a certain input to make the calculations. One of them is the stiffness of the diaphragm wall. Because a concrete wall will be used, this stiffness depends on the bending moment in the wall. To take this into account, an M-K diagram has to be made, where the deformation of the wall is described, depending on the bending moment. This diagram is divided into three branches (Soekhoe, 2015):

1. Elastic bending stiffness (uncracked state)
2. Cracked bending stiffness
3. Bending stiffness with yielding of the steel reinforcement

Four different points in the M-K diagram are used to determine the shape of the diagram:

0. Zero bending moment, zero deformation
1. Start cracking of the concrete under tension
2. Start yielding of the reinforcement steel
3. Failure concrete compression zone.

To determine the characteristic points, first of all, the strains in the concrete and reinforcement steel according to the four points are determined. After that, the total deformation of the cross section can be derived. With the horizontal equilibrium of the cross section, the height of the compression zone and the forces in the different members are computed (concrete and reinforcement steel under tension and compression). With these forces known, the associated bending moment is derived in the cross section. With the bending moment and the curvature in the cross section, the bending stiffness EI can be derived. See Appendix 0 for the complete calculation of the different points.

It is assumed that the cross section is constant over the whole height of the wall. This means that a reinforcement is applied on both sides of the cross section.

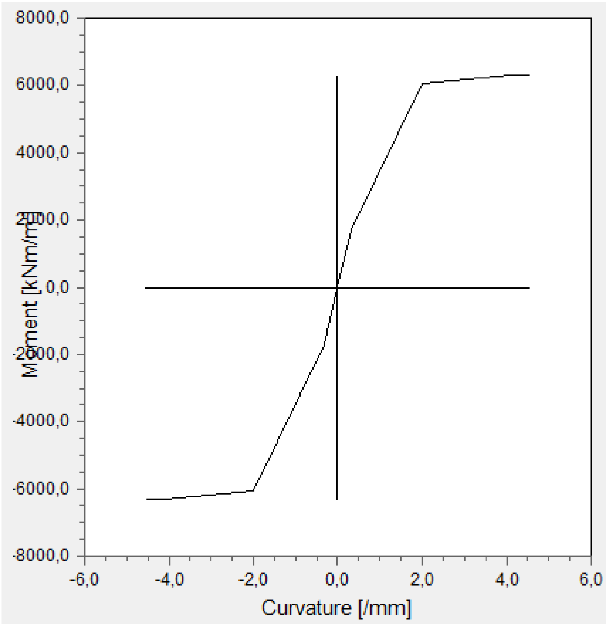



*Figure 65 M-Kappa diagram diaphragm wall*

The remaining input in the model are first of all the soil parameters, which are used as mentioned before. Besides, the horizontal stiffness of the MV-pile should be used as a horizontal spring support at the top of the retaining wall.

The loads in the computer program are as follows:

- Vertical load diaphragm wall: vertical support reaction derived from SCIA engineer
- Horizontal load diaphragm wall: horizontal support reaction derived from SCIA engineer
- Bending moment diaphragm wall: applied bending moment because of the eccentricity of 0,7 m of the vertical load. The superstructure is supported eccentrically to reduce the bending moment in the field of the retaining wall.
- Soil surcharge load: the soil next to the superstructure.

To calculate the stability and the force distribution in the ultimate limit state, D-sheet piling uses the CUR166 six steps design procedure (Deltares, 2015). Every step changes the soil parameters, water levels, and surface level, in order to determine the governing situation.

## 7.4 Soil stability

Several aspects should be researched to determine the stability of the structure in the soil. First of all the bearing capacity of the concrete foundation piles should be investigated. Besides of that, the tension capacity of the MV-pile is computed. For the total stability of the complete structure, the Kranz stability is researched. The stability of the diaphragm wall will not be considered in this part. The horizontal stability is determined with the computer program. Because of the relatively large tip of the wall, the vertical stability is assumed to be sufficient.

### *Bearing capacity foundation pile*

For the vertical stability of the concrete foundation piles at the back of the structure, the Koppejan method will be used (Verruijt, et al., 2011). This method divides the bearing capacity of the soil into three branches:

$q_{c,I}$: the minimum averaged value of the cone resistance in the first branche.
$q_{c,II}$: the averaged value of the cone resistance in the second branche.
$q_{c,III}$: the averaged value of the cone resistance in the third branche.

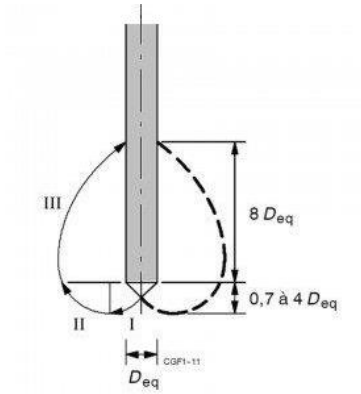



*Figure 66 Schematisation bearing capacity pile head (van Tol, 2006)*

The bearing capacity of the pile tip will be: $P_{r,max} = \frac{1}{2}\left(\frac{q_{c,I}+q_{c,II}}{2} + q_{c,III}\right)$. The shaft resistance of the pile is neglected in this stage of the design.

### *MV-pile*

The MV-pile is loaded under tension. Two different checks should be performed to ensure that the tension force capacity is enough (CURNET, 2012). These are a check based on the grout pressure and a check based on the cone resistance.

**Check on grout pressure**

This check is based on the difference in pressure between the hydrostatic pressure in the not yet hardened grout column and the hydrostatic groundwater pressure. This pressure difference results in a friction resistance along the surface of the grout body. The surface that may be taken into account is the determined by the circumference of the grout body and the effective length. Only the part of the pile in the supporting sand layer may be taken into account. Besides of that, the upper first meter of the sand layer may not be taken into account as well. The effective length of the pile is thus the length of the pile between -25 m NAP and the tip of the pile.

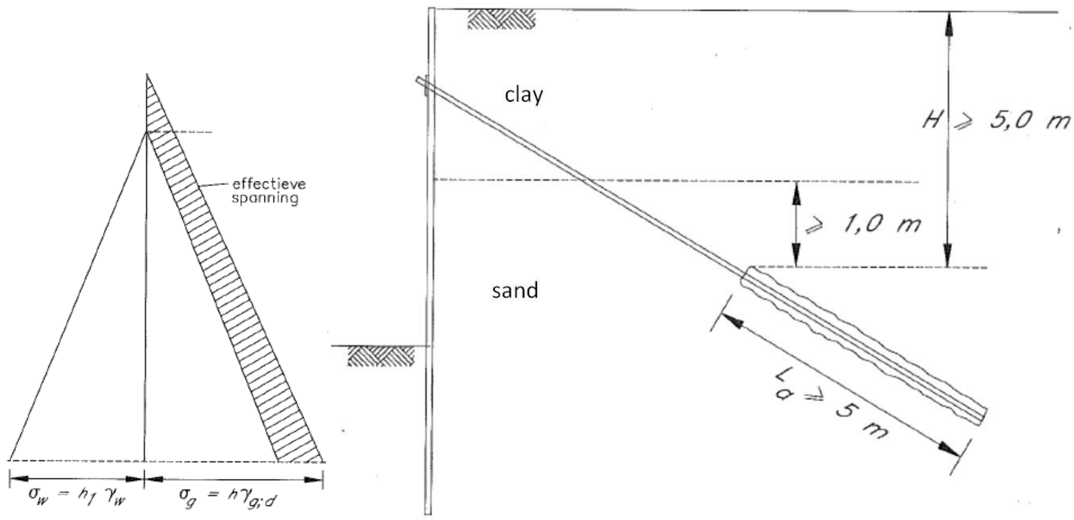



*Figure 67 water and grout pressure (left) and the requirements for grout anchors (CURNET, 2012)*

The shear stress between the grout body and the soil is given by:

$$\tau_{max} = (\sigma_g - \sigma_w) \tan \varphi$$

With ϕ being the internal angle of friction of sand. The total pulling force that can be retained in kN is given by:

$$F_{a,max} = \frac{\tau_{max} * O * L_{eff}}{\gamma_m}$$

O is the circumference of the grout body and a material safety factor $\gamma_m$ of 1,4 is used. The acting tension load may not exceed $F_{a,max}$.

**Check on cone resistance**

According to many test loads on MV-piles in the port of Rotterdam, it turns out that a maximum shear stress may be used of 1,4% of the pile resistance, with a maximum of 250 kN/m$^2$ (CURNET, 2012). The pulling force can be calculated in the same manner as in the check on grout pressure mentioned above.

### *Kranz stability*

When the anchor length of the MV-pile is not sufficient, it is possible that the complete structure will fail due to a developed sliding plane. The total stability of the structure is not assured anymore. To investigate this, the stability according to the Kranz method is calculated. This method assumes a straight sliding plane between the point where the shear force is zero in the retaining wall and the centre of the anchor body. In reality this sliding plane is not straight, but curved. By comparing calculations it is however shown that a straight sliding plane is an acceptable assumption.

To make sure that the structure is stable, the force equilibrium on the soil body enclosed by the points ABCD in Figure 68 is computed. This equilibrium consists of the following forces:

G = the gravitational mass of the soil body.
$E_A$ = the force working on the active slip wedge through the vertical of the retaining wall.
$E_1$ = the active force working on the vertical through the anchor.

$Q_1$ = the force working on the sliding plane BC, working under an angle ϕ.

$F_{kr}$ = the maximum anchor force that can be retained by the anchor.

The applied anchor force may not exceed the value of $F_{kr}$. A safety factor $\gamma_m$ of 1,5 is used for this calculation. A common way to calculate this is with the graphic method where the force equilibrium is drawn. G, $E_A$ and $E_1$ can be calculated. With the direction of $Q_1$ and $F_{kr}$ known, these two forces can be determined as well. If $F_{kr}$ is not sufficient, the MV-pile should be lengthened. The complete calculation for this case is given in Appendix 0.

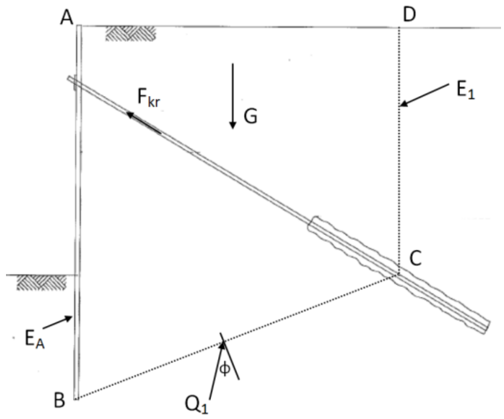



*Figure 68 Forces regarding Kranz stability*

## 7.5 Results

The above method is used to make a preliminary design for the retaining wall and relieving floor. Several checks according to the Eurocode are done however to prove the technical feasibility of the proposed design. An environmental class XS3 with a construction class S6 is considered (NEN, 2011). Besides, a concrete class C30/37 and reinforcement steel class B500 is used.

### *Checks proposed design*

The most important members of the structure are further evaluated. It turns out that the bottom slab is governing in almost all cases. This is expected, because of the cantilevering part and the openings in the slab. The required main reinforcement and shear force reinforcement are calculated for the bottom slab for the most extreme internal forces. The checks for the different members and the resulting proposed design is given below. The exact calculation of this is given in Appendix 0

#### Bending reinforcement

The main reinforcement is determined according to (NEN, 2011). The governing negative and positive bending moments can both be seen in the bottom slab, see Figure 69. The governing positive moment (tension in the bottom of the slab) is present at the back of the floor, above the foundation piles. The governing negative moment (tension in the top of the slab) is present above the diaphragm wall. Near the front wall, the internal bending moments should be redistributed, because of the openings in the floor. They are concentrated in the area in between the openings, where the governing bending moments are thus found.

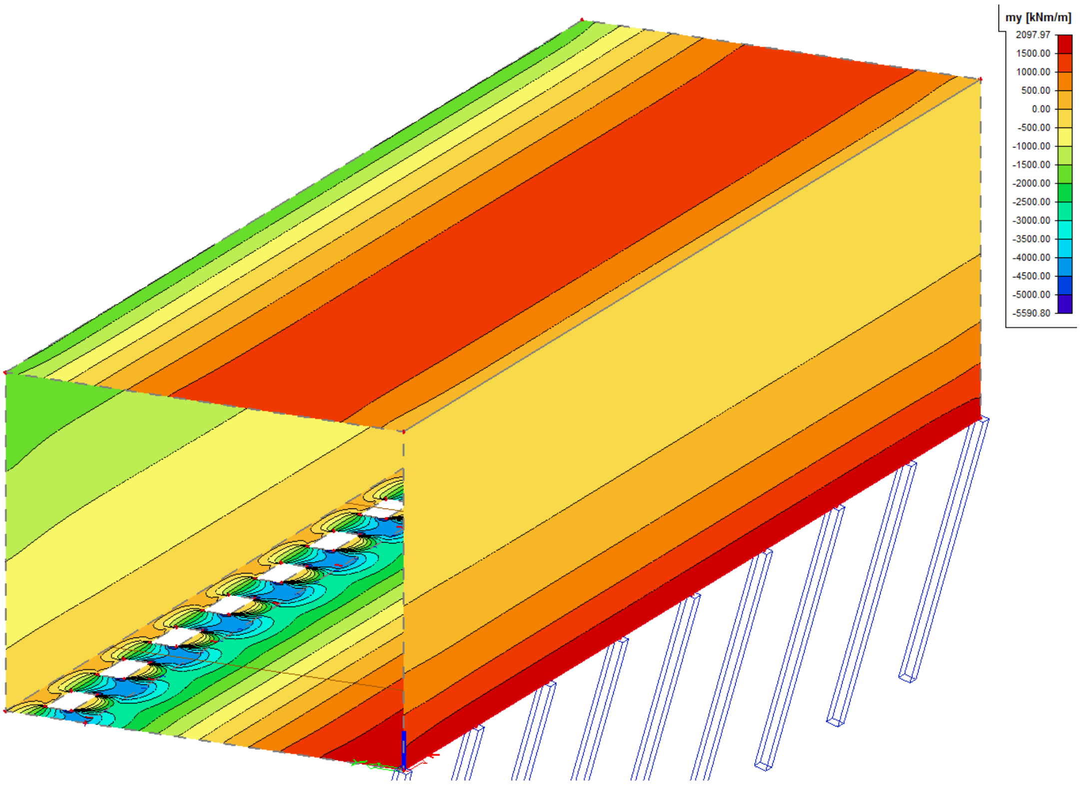



*Figure 69 Bending moment distribution in Y-direction for load situation: vessel*

To calculate whether the applied main reinforcement is sufficient, the bending moment resistance of the slab should be calculated. This is done per meter width of the slab. The bending moment resistance is determined by the moment equilibrium of the cross section where the force in the concrete and the reinforcement steel is taken into account. The concrete force is however determined by the height of the concrete compression zone: x. See Figure 70 for the cross sectional forces. The height of this zone is first calculated with the use of the horizontal force equilibrium of the cross section. This is the equilibrium of the concrete force under compression failure and the steel tension reinforcement force:

$$\lambda x * f_{cd} * 1000 = \varepsilon_s * E_s * A_s$$

The bending moment resistance is than calculated as follows around the point of action of the concrete compression force:

$$M_{Rd} = F_{st}(d - \frac{1}{2}\lambda x)$$

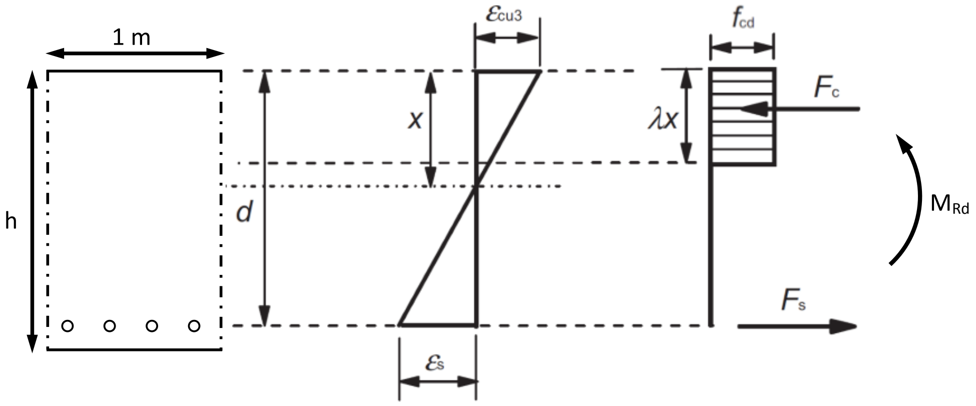



*Figure 70 cross sectional forces due to a positive bending moment for 1 meter width*

In the Ultimate Limit State check (ULS) a reinforcement cross section $A_s$ of 11000 $mm^2$ per meter width at the top of the slab and 5000 $mm^2$ per meter width at the bottom of the slab turns out to be sufficient enough to retain the governing bending moments. The proposed design is therefore 9 bars with a diameter of 40 mm on the top side and 11 bars with a diameter of 25 mm on the bottom side, both per meter width. See Figure 72 for the reinforcement configuration. The reinforcement ratios are $\rho_{Astop}$=0,9% and $\rho_{Asbottom}$=0,45%.

**Shear force reinforcement**

The shear reinforcement is determined according to the Eurocode (NEN, 2011). The governing shear force is given by the high water load situation (with the culvert full of water). This shear force is partly generated by the support reaction of the diaphragm wall. Therefore the same location as for the bending moment is governing: above the diaphragm wall in the cross section in between the openings. This gives a maximum shear force of 2065 kN/m (per meter width).

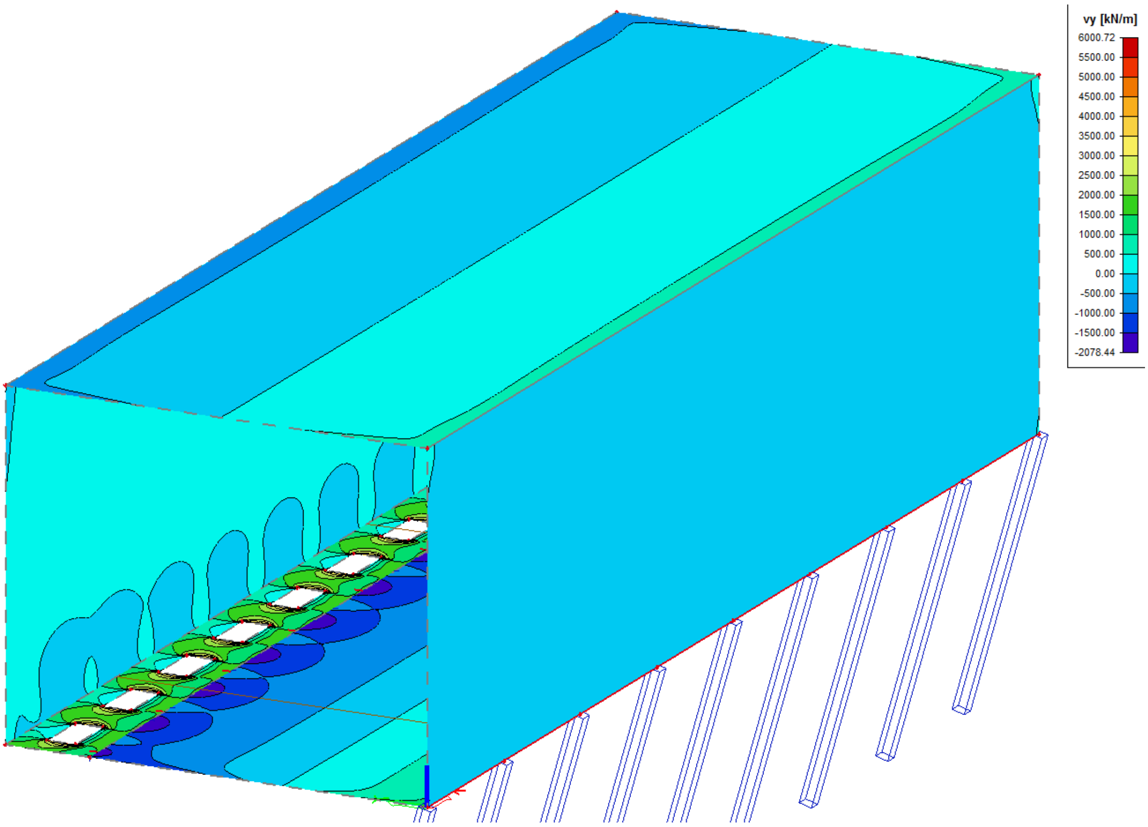



*Figure 71 Shear force distribution in Y-direction for load situation: high water*

First of all, the shear force capacity is calculated without any shear reinforcement. It turns out that this is not sufficient. Therefore shear force reinforcement has to be applied. The shear force resistance of a slab with reinforcement is calculated as follows:

$$V_{Rd} = \frac{A_{sw}}{s} z * f_{yd} * \cot\theta$$

Where:

| | | |
|---|---|---|
| $A_{sw}$ | = | The cross sectional area of the shear force reinforcement per unit width ($m^2/m$) |
| S | = | The ctc distance of the stirrup (m) |
| z | = | Internal lever arm (m) |
| θ | = | Angle between the pressure diagonal of the concrete and the axis of the beam perpendicular to the shear force (°). |

A shear force reinforcement with a diameter of 16 mm, a width of 350 mm and a distance in between two bars (s) of 300 mm gives enough shear force capacity.

It might have the preference to prevent shear force reinforcement, because it could cause difficulties regarding the execution during building of the formwork. There are several options to prevent the usage of shear force reinforcement:

- Larger thickness of the slab (in this case the thickness should be 3 times as large, which is not considered to be feasible).
- Prestressing of the bottom slab (prestressing should be done over the whole length of the wall, resulting in large executive costs as well).

- Higher concrete class (a higher concrete class might help to reduce the amount of shear reinforcement, but not enough to prevent shear force reinforcement completely).

Looking at the above factors, it is probably not possible to prevent shear force reinforcement with acceptable measures.

The calculated reinforcements are given in Figure 72 for both the location right above the diaphragm wall and the location right above the foundation piles. It is assumed that the shear force reinforcement will be applied everywhere. However, this can be further optimised.

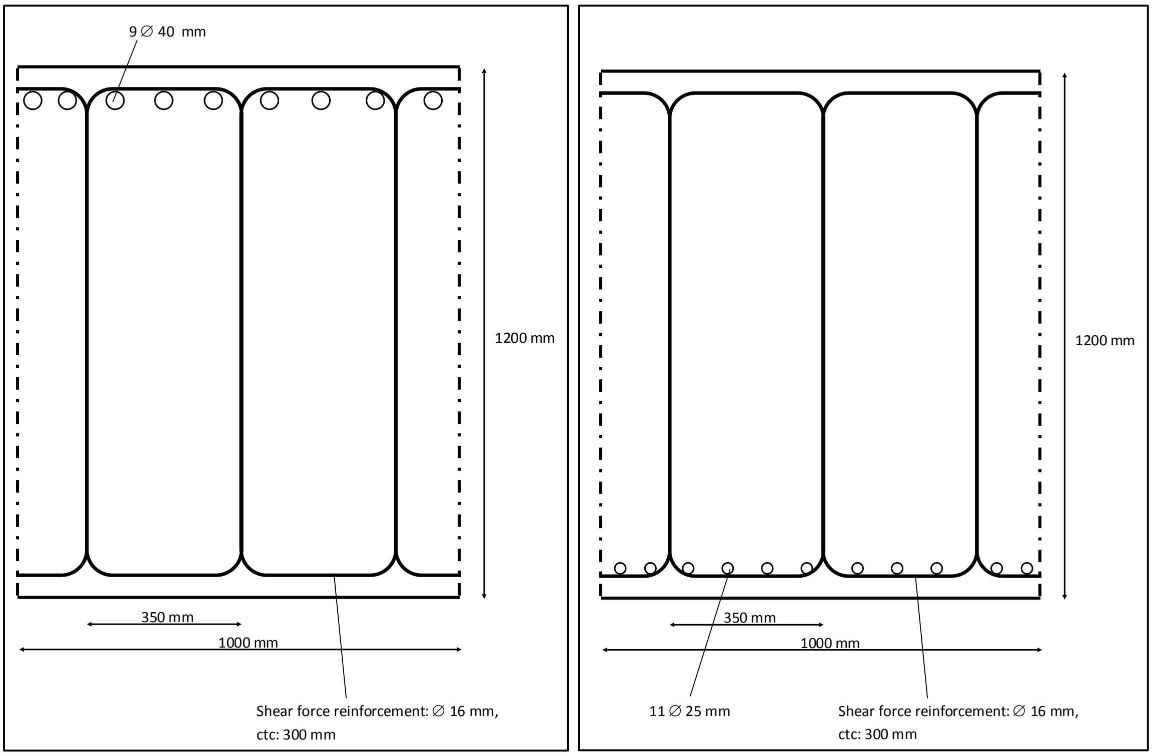



*Figure 72 Required reinforcement in the bottom slab (1 m width) Left: above diaphragm wall, right: above foundation piles*

**Crack width**

According to the Eurocode and the environment class, a maximum crack width in the Serviceability Limit State (SLS) is allowed of 0,3 mm. In the SLS the governing bending moment should be calculated without any load factors. The governing bending moment is now given by the maintenance load situation. The crack width $w_k$ is calculated with the following formulae:

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm})$$

$S_{r,max}$ is the maximum crack distance. The term: ($\varepsilon_{sm}$-$\varepsilon_{cm}$) is the difference in average strain of the reinforcement steel and the concrete in between the cracks. This is given by:

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}\left(1 + \alpha_e \rho_{p,eff}\right)}{E_s} \geq 0{,}6\frac{\sigma_s}{E_s}$$

With:

$\alpha_e$ = The ratio $E_s/E_{cm}$ (-)
$\rho_{p,eff}$ = $A_s/A_{c,eff}$ (-)
$k_t$ = factor depending on the load duration. Short: $k_t$=0,6; long: $k_t$=0,4

With the applied reinforcement the calculated crack width is 0,276 mm, which just satisfies the requirements of a maximum crack width of 0,3 mm.

**Foundation pile**
First of all, a centre to centre (ctc) distance of 5,5 meter and a cross section of 450x450 $mm^2$ was considered for the foundation pile. Due to the high load per pile, this is however changed to a centre to centre distance of 2,5 meter and a cross section of 500x500 $mm^2$. A piling depth of -29 m + NAP gives sufficient bearing capacity to retain the loads. The ctc should be the same as the MV-pile, to prevent any conflicts in the soil.

Other quay wall projects at the Maasvlaktes encountered problems regarding the drivability of these kind of foundation piles. It turns out that the force required to drive the pile into the soil was so large that the concrete of the top of the piles failed because of the impact of the driving installation. Therefore, instead of one pile, two piles next to each other could be considered. This results in a lower cross sections for the piles and therefore the pile driving load could be smaller.

**MV-pile**
As mentioned above, the ctc of the foundation piles and the MV-piles should be adapted to each other. With a ctc of 2,5 meter, a depth of -31 m + NAP is sufficient to develop a high enough pulling force to retain the tension loads. Besides, this length is enough for the stability of the structure regarding the Kranz stability.

**Diaphragm wall**
For the diaphragm wall a thickness of 1,5 meter and a reinforcement cross section of 10000 $mm^2$ per meter width on each side is used for the calculation of the M-K diagram. The load situation maintenance is governing for the diaphragm wall. It turns out that the governing bending moments in the diaphragm wall according to D-sheet are in the second branch of the M-K diagram. See appendix D3. This means that the stress in the tension zone of the concrete has exceeded the tensile strength of the concrete, but that the reinforcement steel is not yet yielding. For the ULS, this is very over dimensioned, so this might be further optimized in the final design.

When looking at the deformations of the wall, the maintenance situation is governing as well. in the SLS a maximum deformation of 25 mm can be seen. The retaining height is 12,8 m. The deformation ratio over the retaining height is 0,195 %. For the ULS according to the CUR-method in D-sheet piling, a maximum deformation of 60 mm can be seen. This is a deformation ratio of 0,469 %.

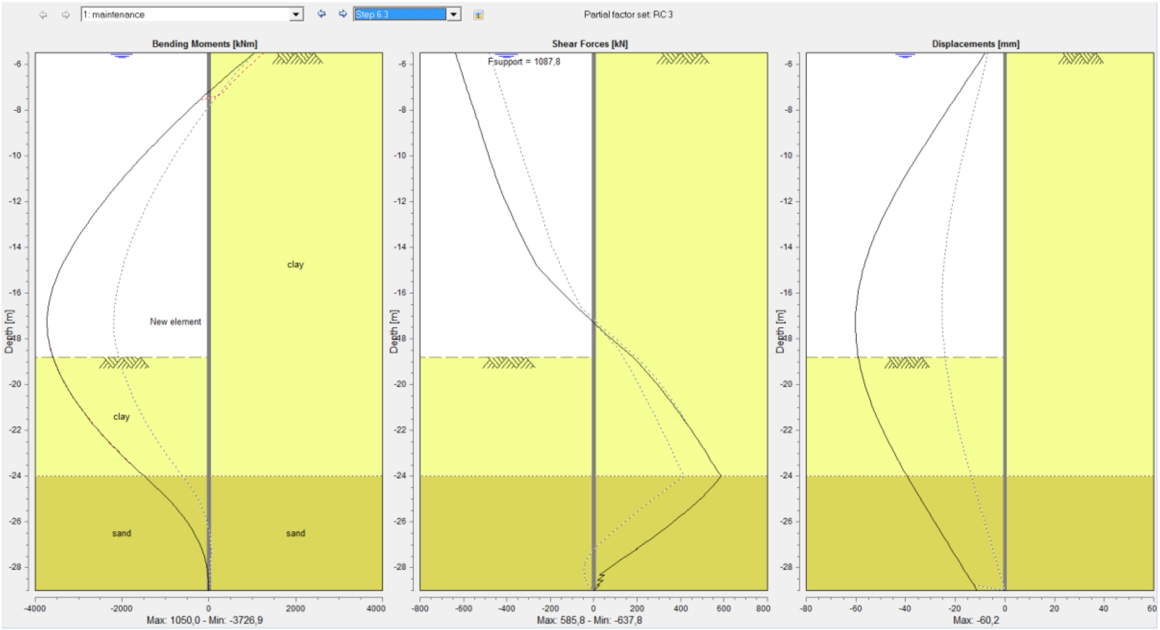



*Figure 73 Force and displacement distribution for the vessel load situation*

## *Costs*

The costs for the above retaining wall are slightly different than the costs for a conventional retaining wall with relieving floor. A small comparative estimation is made below. The main differences and aspects regarding the costs are named in a qualitative way:

- Soil supplementation to make dry building possible. Ground work is normally relatively cheap compared to other works, because specialised equipment is not necessary. It could however change when the amount of soil to be moved is very large or when it is necessary to execute from the water.
- Constructing filling openings in the bottom slab. The filling openings results in concentrated forces in the bottom slab. Therefore attention should be given to the detailing of the reinforcement around these openings and it might results in higher amounts of reinforcement. Furthermore, more formwork should be made, but this influence on the costs is probably limited.
- The size of the superstructure. Because of the required dimensions of the longitudinal culvert, the size of the superstructure is larger than one would usually consider by making a retaining wall. A larger structure results automatically in higher demands for the structure and therefore higher costs.

When looking at the differences between a usual filling/emptying system through the head and the longitudinal culvert in combination with the relieving floor several aspects regarding the costs can be named as well:

- The small valves over the length over the lock. These valves are small, but because there are numerous amounts of them (about 100 of them on each side), this can have a big impact on costs regarding both construction and maintenance.
- The big valves at the end of the culvert. The valves at the beginning and the end of the culvert are of a considerable size (width 15 m and height 10 m). the installation and maintenance of these valves can have a high influence on the costs.

- No openings in the gate (when compared to filling through the gate). By making use of the longitudinal culvert, openings in the gate are not necessary anymore, which is structurally beneficial and makes maintenance and production of the gate therefore cheaper.
- No short culvert with adjacent valves (when compared to filling by short culvert around the head). The same as for filling through the gate, short culverts are not necessary anymore, which has a high positive influence on the costs.

For the total costs, eventually a Life Cycle Costs (LCC) analysis could be executed. This takes into account the total costs over the lifetime of the structure. Thus also the maintenance, renovation and demolishing costs.

When making the decision whether to implement the above system, the costs is one of the most important aspects. It should be evaluated whether the benefits of faster locking weigh up against the costs.

### *Conclusion*

Resuming on the above structural calculations, it is concluded that it is technical feasible to build the relieving floor in combination with the longitudinal culvert. The above results does not provide in a design that can be used immediately to build the structure. The main components are considered and computed for both the superstructure (the culvert) and the substructure (diaphragm wall). However the superstructure is slightly different than the usual relieving structures (it is larger and has filling openings), the calculated required dimensions are acceptable. However, to determine whether this option could be used, the costs should be mapped out.

# 8

# Conclusion and recommendations

## 8.1 Conclusion

To draw a conclusion of the above report, the most important findings will be given that are found during the research. Subsequently, with the help of these findings, the main research question is answered.

**The best location to close the New Waterway is just east of the Maeslant Barrier**
Several locations to close the New Waterway could be considered. It turns out that the best location is at the headland just east of the Maeslant barrier. Compared to locations more to the east, this location has several main advantages:

- More unembanked areas are protected by the barrier
- The coast line is shortened more, resulting in a higher safety level
- The required spatial availability is higher

A drawback of this solution is the high demand of passing seagoing vessels. Therefore a relatively large lock should be made.

**The adaptive requirements for navigation have a larger impact on the structure than the requirements for flood safety**
It turns out that when looking at the flood safety, only a relatively small increase of the requirements will develop for the most extreme scenario. This means an increase of the high water level to be retained of about 1 meter. The other requirements are not likely to increase. The higher water level to be retained will of course lead to higher loads on the flood barrier. This load has to be taken into account when using an adaptive design approach. It is however not likely that this increase will influence the feasibility of the principal solutions.

The adaptive requirements for navigation have a larger impact on the structure. A lock complex is always built for a certain maximum sized vessel. If this size increases, the lock should be rebuild or adapted. For the initial design, the new Panamax class is taken into account. For the adaptive design, this is enlarged to the Malaccamax class. The existing lock should be enlarged when the demand for this class becomes reality. This results in high costs for rebuilding or adapting the existing structures like the lock head, the chamber walls and the chamber floor.

**An adaptive modular lock head is not cost-effective**
To take the adaptive requirements into account, a modular lock head is proposed. This lock head can be enlarged when this is necessary, by removing the modular pieces, after which a larger gate can be floated in. Doubts should be expressed about the cost-efficiency of the above solution. A reduction on costs is caused by the usage of a smaller gate in the initial situation. On the other

hand, a lot of additional measures should be done to make installing of a larger gate possible. When looking at the risk analysis for the costs, where several probabilities of occurrence are taken into account, it turns out that the adaptive design is never the most cost-effective solution. Therefore it is concluded that it is not advisable to incorporate this solution in the final design of the lock.

**The longitudinal levelling system results in faster levelling**

When a comparison of the levelling times is made for the longitudinal culvert with the usual levelling systems, it turns out that the longitudinal culvert results in much faster levelling. Levelling times can be reduced by a factor 4. This depends on the water level difference and the vessel size, but in all of the combinations, the longitudinal culvert is the fastest option. The decrease of levelling time has to be seen in the light of the total locking time for a vessel however. Because more processes determine the total locking time (sailing in and out of the vessel, closing of the doors, et cetera), the benefits of a faster levelling time should be taken into account for the whole process.

**The relieving floor in combination with the longitudinal culvert is structurally feasible**

Because the relieving floor with culvert is slightly different than other relieving floor structures, a feasibility study regarding the structural aspects of the relieving floor is done. The differences with a usual quay wall are the size of the superstructure, the filling openings in the floor and the cantilevering part into the lock. The proposed solution is to use a diaphragm wall as the retaining wall and to use a concrete box as superstructure. When comparing the results of the calculated dimensions with the dimensions of other relieving floors, it is concluded that it is technically feasible to build the relieving floor.

### *Main research question*

To draw a final conclusion, the main research question as stated in chapter 1 will be answered. The main research question is stated again:

*How can an adaptive navigation lock system be designed as part of a cost-effective solution to permanently close off the New Waterway?*

Considering the goal to develop a cost-effective solution, several factors in the design process should be taken into account. Different requirement scenarios for the different functions should be put up. With these adaptive requirements, solutions are put up that should meet the scenarios. However, it is not straight-forward to apply these requirements in making a technical feasible design for a certain element (like an adaptive design for a lock head). The adaptive design approach might however result in decisions where an adaptive design of the whole lock is used together with an element that is not adaptive by definition (like widening of the lock together with the relieving floor in combination with longitudinal culvert).

## 8.2 Recommendations

Not every aspect has been worked out on the same level of detail. In general most of the topics could be elaborated in more detail when a more final design is made. Therefore the following set of recommendations is advised for the further research on the elaborated topics.

- Computational fluid dynamics and physical scale model
  To make a better approximation of the hydraulic aspects of the longitudinal culvert system, it is advisable to use computational fluid dynamics. Besides of that, a physical scale model of the system might give even more insight in the properties of the system. Both methods could be used to validate the calculation method described in this thesis.

- Longitudinal force
  The longitudinal force on the vessels is neglected totally in the hydraulic design of the longitudinal culvert. In reality this might not be necessarily the case. A longitudinal force could occur due to the difference in filling discharges along the lock. The influence of this could be further evaluated.

- Nautical study
  Pragmatic choices are made in this report for the spatial usage of the several locks. A more detailed nautical study should be done to ensure that the proposed configuration acts in the same way as expected.

- Location choice
  For the location choice, it is advisable to map out the consequences of a certain choice in more detail. These are mostly regarding the costs of the solution, the benefits for the environment, the consequences regarding navigation and the added safety against flooding. A cost-benefit analysis of the location choice could give more insight in these aspects.

- Structural design
  A very rough structural design is made for the relieving floor. This design could be further optimized. This counts especially for the size of superstructure. Regarding the demanded wet cross sectional cross section of the culvert and maintaining the relieving aspect of the superstructure, more optimization can be done.

- Fault tree analysis
  Regarding the safety against flooding and failure of the structure, a fault tree analysis should be done to determine the exact retaining height and other safety aspects of the structure. In this way a funded estimation can be made for the safety, availability and reliability of the lock complex.

- Waves
  The influence of waves at the exact location is not derived. This might however influence the required height of the structure. Therefore the occurring waves should be computed.

These can be used to determine the amount of acceptable overtopping and this results eventually in a required structure height.

- Cost-benefit analysis
  For the longitudinal culvert in combination with the relieving floor an extended cost benefit analysis should be made to determine whether the decrease in total locking time weigh up against the probably higher costs for the structure compared to one of the other systems.

- Adaptive lock head
  It is concluded that the adaptive design of the lock head is not advisable to execute. There is however only one solution taken into account, namely the modular lock head. Other solutions regarding the adaptive design of a lock head might however be more feasible to implement. When one comes up with an innovative solution, it might be worthwhile to investigate the possibilities.

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# Appendices

# A. Analysis

## A1. South West of the Netherlands

The current water system is very diverse. It consists out of several elements and has several functions. First of all there is the port of Rotterdam with all kinds of vessels calling every day. The waterways have to make sure that shipping is possible. Both inland and seawards. Next, the waterways have the function of discharging the water from the Rhine and the Meuse because of the salt intrusion that occurs otherwise. Besides of this, one has to make sure that the safety against flooding in the Rhine-Meuse Delta is ensured. All these aspects have to be taken into account when designing a new structure to make sure that these functions will be remained in the future.

### *A1.1. Flooding safety and discharge*

**General safety approach**

The Rhine mouth-Drecht cities area plays an important role in the defence against flooding. First of all, the region itself is densely populated with the city of Rotterdam and the neighbouring Drecht cities. Besides of that, the region has a high economic value. This gives the high protection levels for the region. Another important aspect is the function of the region to discharge the Rhine and Meuse water to the sea. The New Waterway is the only open connection in the Netherlands between the rivers and the North Sea. This results in a situation where both the tide and storm surge from the sea and the discharge of the rivers influences the water levels. The region is roughly divided into three areas, the sea dominated area, the transition zone where both influences can be seen and the river dominated area. The transition between these areas can in reality not be described with a distinct line. In Figure 74 is however an indication given, together with the required flood safety levels for the dike sections in the region.

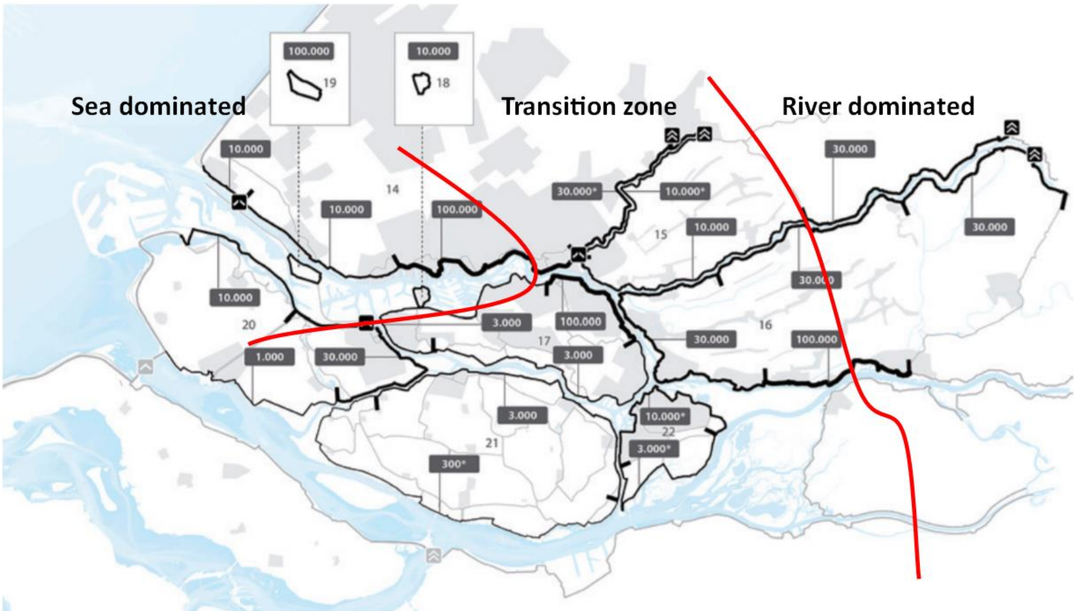



*Figure 74 The three zones of the Rhine-Meuse estuary with flood safety levels (return period) for the dike sections (Deltaprogramma Rijnmond-Drechtsteden, 2014)*

**Europoort barrier**

The New Waterway and the Hartel canal play an important role regarding the flood safety of the Rhine-Meuse Delta. The main elements making sure that the Rhine mouth-Drecht cities area will not be flooded are the two storm surge barriers, the Maeslant barrier and the Hartel barrier. These barriers together and the dike in between are called the Europoort barrier. The barriers will close when the water levels will be too high and will form a thread to the hinterland. Both barriers have the same requirements for closing. Namely when the expected water level will exceed a value of +3 m NAP at Rotterdam or +2,9 m NAP at Dordrecht. This results in a closing frequency of once in every 5 to 10 years. This will be higher in the future with the expectations of sea level rise. The usage of these barriers ensures that the hydraulic loads on the dikes in the Rhine mouth-Drecht cities area is reduced in extreme situations.

**Preference strategy delta program 2016**

In the Delta program of 2016, the preferred strategy is stated (Deltaprogramma, 2016). In this strategy, several decisions have been made regarding the flood safety of the Rhine-Meuse Delta. The strategy states that the current situation is a solid base to ensure the flood safety in the future. This means that the Europoort barrier will be used until the end of its lifetime. Regarding the high probability of failure of the Maeslant barrier when closing, research will be done to make the barrier more reliable.
Partly because of the renewed flood safety levels and the new high discharge scenario (18.000 $m^3/s$ at Lobith) it is expected that a lot of dikes in the region have to be strengthened in the future.
Besides of these strategies, research is done to investigate the effects of closing the New Waterway permanently. This is the ‘plan sluices’ which will be elaborated further.

**Dike rings**

Most of the land in the region is protected by a dike. An area protected by a dike completely all around is called a dike ring. Basically this means that when the dike breaches, the whole area will flood. This is in reality not completely true. It depends where the dike will breach. In the Rhine-Meuse Delta several dike rings are present with different safety levels. The most important one is dike ring 14. It covers a large part of the province South-Holland, with a lot of inhabitants and a high economic value. Other dike rings in the region are 15, 16, 17, 18, 19, 20, 21 and 22, see Figure 74.

**Discharge**

As said, The New Waterway is the only open connection to the North sea in the Netherlands, besides of the ‘half’ open connection of the Haringvliet. This means that a lot of the water of the Rhine is discharged to the sea via the New Waterway. This is also done to reduce the salt intrusion. The Haringvliet has enough capacity for the complete Rhine discharge. The distribution of the Rhine water at Lobith is as follows: 65 % to the Waal, 20 % to the Nederrijn and 15 % to the Ijssel (Brinke, 2013). The Waal and the Nederrijn both flow west. This will be distributed between the New Waterway and the Haringvliet. One of the scenarios that is taken into account in the delta program is that the Rhine will face more extreme values of the discharge in the future, both low (420 m3/s) and high (18.000 m3/s) extreme values (Bruggeman, et al., 2013). Both extreme values can cause different problems in the Rhine-Meuse Delta regarding safety and the availability of fresh water. In fact, the capacity of the Haringvliet dewatering sluices are designed for a discharge of 20.000 $m^3/s$, thus high enough for the Rhine discharge. This is however not done, because a minimum discharge is needed in the New Waterway to prevent salt intrusion.

**Unembanked areas**
Several areas nearby Rotterdam are not protected by a dike ring. These are mostly port areas where ships have to be moored. Nevertheless, several of these port areas are transformed into urban areas. Because of the very different heights and locations of these areas, the flood safety level differs as well. Besides of this, the Europoortbarrier does not cover all of these unembanked areas, resulting in different safety levels as well. Figure 75 gives a view of the unembanked areas in the Rhine mouth-Drecht cities area. A total amount of more than 60.000 people lives in the unembanked areas. The regions more to the west are mostly used for port activities, while the most densely populated unembanked areas are around the city centre of Rotterdam.

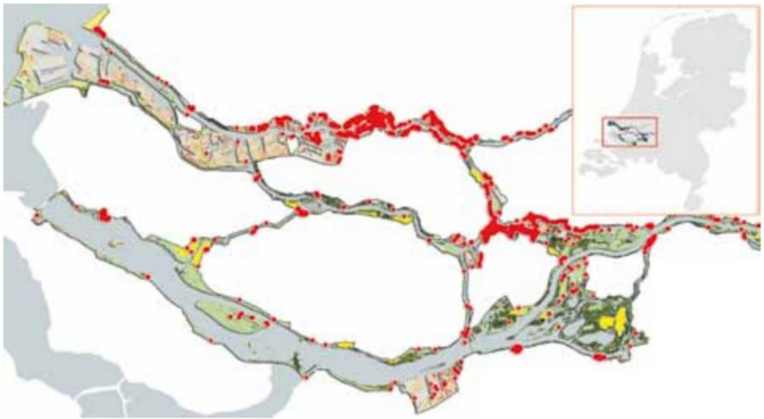


*Figure 75 Areas outside the dikes with in red highly populated areas (Veerbeek, et al., 2010)*

**Tide**
Because the Rhine mouth area is in open connection with the sea, the tide has an influence on the water level. The average water levels in Hook of Holland and Maassluis because of the tide can be found in Table 30. This counts however for the current situation. When the estuary will be closed, the situation will be different and other water levels may occur.

| | Hook of Holland | | Maassluis | |
|---|---|---|---|---|
| | Mean tide | Spring tide | Mean tide | Spring tide |
| High water(m + NAP) | 1,26 | 1,53 | 1,24 | 1,44 |
| Middle water (m + NAP) | 0,05 | -0,12 | 0,05 | -0,12 |
| Low water (m + NAP) | -0,39 | -0,55 | -0,27 | -0,41 |
| Amplitude (m) | 1,65 | 2,08 | 1,51 | 1,85 |

*Table 30 Tidal differences water level difference (Port of Rotterdam, 2012)*

## *A1.2. Plan sluices*
Two alternatives for the delta program are presented by two different groups of people. Both of them plead for permanent closure of the New Waterway. It has multiple reasons that this is beneficial. The two different plans are elaborated further below.

**Plan Spaargaren**
The main reason for closing the New Waterway is to ensure a higher and more efficient safety level for the hinterland. Closing it will probably lower the hydraulic loads on the dikes along the inland waterways (Dokter, 2015). This is mostly caused by the fact that the Europoort barrier will not be used anymore. The main barrier, the Maeslant barrier, has a probability of failure of 1/100 when closing. A dam with locks, dewatering sluices and pumping station can achieve a much higher safety level.

The plan Spaargaren states that a sluice complex should be built in the New Waterway, near the Beneluxtunnel (Rijkswaterstaat, 2015). This is quite far inland. The reason for this is that most of the Port of Rotterdam will remain available for seagoing vessels without passing a lock. For the inland going vessels a sufficiently amount of lock locks should be build, so that the passing time and therefore the delay is minimized. Because the location is at the east side of the parting between the New Meuse and the Old Meuse, another complex should be built in the Old Meuse. Both complexes consists of a navigation lock and a dewatering sluice. The one in the New Meuse will have the largest capacity, both for navigation and dewatering.

Because of the permanent closure, the high water levels in the sea can be accounted for at all time. On the other hand, the inland water levels can be more regulated. When a situation with extreme low discharges occurs, the dewatering sluices can be closed. This will make sure that the water levels in the inland waterways in the delta will be high enough for vessels to navigate. Furthermore, more water will be available for drinking water and irrigation. In an open situation this is not the case, salt water will intrude the waterways, making it less suitable for agricultural and drinking water usage. This is countered by 'flushing' the waterways by opening the weirs, but then the water levels will decrease, making navigation impossible. Until now, these situations have not caused large problems. But even lower extreme values of water discharge are predicted, making it more likely that problems will occur. By closing the New Waterway, the contradicting interests of fresh water usage and navigation are solved.

On the other hand, when an extreme high discharge occurs in combination with high sea water levels, dewatering may not be possible. This can be solved by installing pumps in the dewatering sluice. Besides of this, the Eastern Scheldt will be used as retention basin. The discharge water will be stored until the sea water level allows it to flow out. Therefore a dewatering sluice needs to be built in the Philips dam and an open connection between the Volkerak-Zoommeer and the Hollands Diep need to be achieved.

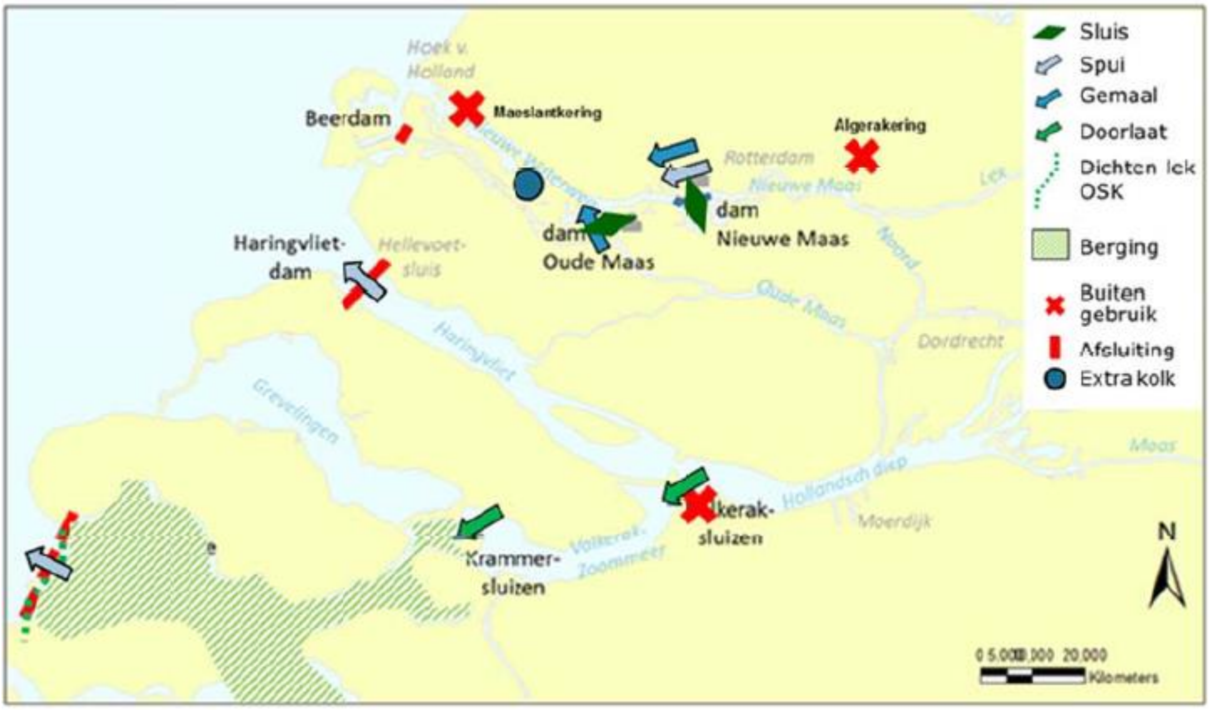



*Figure 76 proposed measures Plan Spaargaren (van Waveren, et al., 2015)*

**Borm and Huijgens**

Next to Spaargaren and his colleagues, the advising firm of Borm and Huijgens made a slightly different plan to close the New Waterway. The main idea is the same; the construction of a sluice complex and using the Eastern Scheldt as a retention basin. The location of the sluice complex is however situated more to the west near Hoek van Holland. Besides of this, the water will travel another route to the Eastern Scheldt. A new canal will be made between the Haringvliet and the Grevelingen. The retention area will be even bigger in this way (Adviesgroep Borm en Huijgens, 2011)

## A2. Rhine mouth area

### *A2.1. Shipping*

As can be seen in Figure 77, the New Waterway is a busy navigation channel. The amount of vessels passing differs very much within a small range, because of the many trade port areas along the New Waterway and the New Meuse. Both inland vessels and seagoing vessels are visiting the port of Rotterdam.


*Figure 77 Shipping movements per year (averaged 2005-2008) (Rijkswaterstaat, 2009)*

The total amount of inland vessels visiting the port of Rotterdam in 2014 is 111.000. Around 75.000 ship movements are made by sea-going vessels in 2014 (Port of Rotterdam, 2014). Besides of this, 32 cruise ships visited Rotterdam, including the largest cruise ship of the world, the Oasis of the Seas. This was however only for maintenance purposes. The cruise terminal houses in the city centre of Rotterdam. Other important port areas are the Botlek, the Petroleumhaven 1, 2, and 3, the Waalhaven and the Eemhaven. The main port areas more seawards are not considered in this study, because the activities in these areas will not influence the scope of the project. In Figure 78 the type of port activities in the area are shown. It can be seen that the area consists of very different types of cargo. This also means different types of vessels.

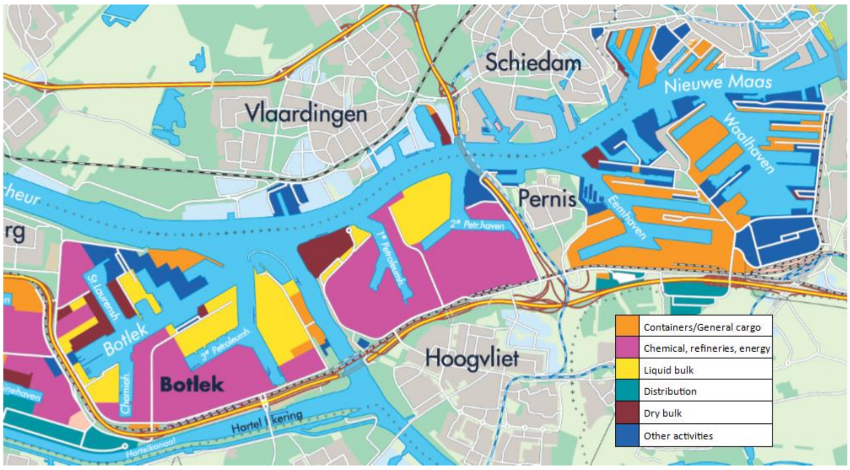



*Figure 78 Activities eastern part port of Rotterdam (Port of Rotterdam, 2014)*

The allowable inland vessels in CEMT-class are given in Table 31. The sea-going vessels are however not classed this way, so these are not by definition the largest allowable vessels. Besides of this, there is another type of exception; the shipping movements of offshore construction yards. The structures that these yards make, are sometimes unusual wide or high, which makes shipping very difficult. Several of these yards are on the east side of Rotterdam. This means that these constructions have to be shipped through the city centre before they can enter the sea.
An alternative route for sea-going vessels is to use the Goereese lock in the Haringvliet. This is however a relatively small navigation lock and will thus not be considered any further.

| | CEMT-class |
|---|---|
| Botlek | VIb |
| Petroleumhaven 1 | VIb |
| Petroleumhaven 2 | VIb |
| Petroleumhaven 3 | VIb |
| Waalhaven | VIc |
| Eemhaven | Va |
| New Waterway | Va |
| New Meuse | VIb |
| Old Meuse | VIc |

*Table 31 Allowable CEMT classes shipping areas (Rijkswaterstaat, 2016)*

## *A2.2. Area description*

**Structural elements**

Several structural elements like tunnels and bridges are present in the area of the port of Rotterdam. These elements restrict the shipping height, draft and width. When designing a navigation lock these aspects has to be taken into account. It might not be necessary to make a lock with less restrictions than the existing elements for vessels that pass both elements. Unless it is expected that the old structure will be demolished or renovated. In Table 32 the elements that restrict the navigation possibilities in some way in the area are given. These are dimensions of the current situation. These are however not the actual restrictions for the navigating vessels.

| | | Width (m) | Height (m + NAP) | Depth (m + NAP) | |
|---|---|---|---|---|---|
| | **New Waterway** | | | | |
| 1 | Maeslant barrier | 360 | - | -17 | |
| 2 | Blankenburgtunnel | - | - | - | Not yet realized |
| | | | | | |
| | **New Meuse** | | | | |
| 3 | Van Brienenoordbridge (moving) | 50 | - | - | Highway |
| | Van Brienenoordbridge (fixed) | 280 | 25,04 | - | Highway |
| 4 | Willemsbridge (fixed) | 260 | 12,30 | - | |
| | Willemsbridge (moving) | 49 | - | - | |
| 5 | Erasmusbridge (fixed) | 270 | 12,50 | - | |
| | Erasmusbridge (moving) | 50 | - | - | |
| 6 | Maastunnel | - | - | -11,5 | |
| 7 | Beneluxtunnel | - | - | -14,5 | |
| | | | | | |
| | **Old Meuse** | | | | |
| 8 | Botlektunnel | 240 | - | - | |
| 9 | Botlekbridge (moving) | 54 | 44,70 | - | Highway |
| | Botlekbridge (fixed) | 75 | 7,25 | - | Highway |
| 10 | Spijkenisserbridge (moving) | 87 | 45,00 | - | |
| | Spijkenisserbridge (fixed) | 40 | 12,52 | - | |
| | | | | | |
| | **Hartel/Calandkanaal** | | | | |
| 11 | Rozenburgse lock | 305 | 24 | - | |
| 12 | Hartel barrier | 95 | 14,00 | -5,30 | |
| 13 | Hartelbridge | 95 | 12,18 | - | |
| 14 | Hartellock | 280 | 24,00 | -5,30 | Usually not used |
| 15 | Calandbridge (moving) | 46 | 49,72 | - | |
| | Calandbridge (fixed) | 80 | 11,72 | - | |
| 16 | Thomassentunnel | - | - | - | |

*Table 32 Structural elements in the area of interest (van der Kaaij, et al., 2010) (Rijkswaterstaat, 2016)*


*Figure 79 Locations structural elements*

**Soil composition**

When designing a navigation lock or determining the right location for it, the soil composition is of importance. The type of foundation can heavily influence the costs for the construction of the navigation lock complex. In Figure 80 the soil composition is given along the New Waterway. It can be seen that a large layer of sand (the formation of Kreftenheye, purple in the figure) is present in the subsoil. This can probably be used to retain the vertical loads of the lock. This layer has not everywhere the same thickness. To the east, the layer is smaller. The other variations in the subsoil are negligible in the scope of this project.

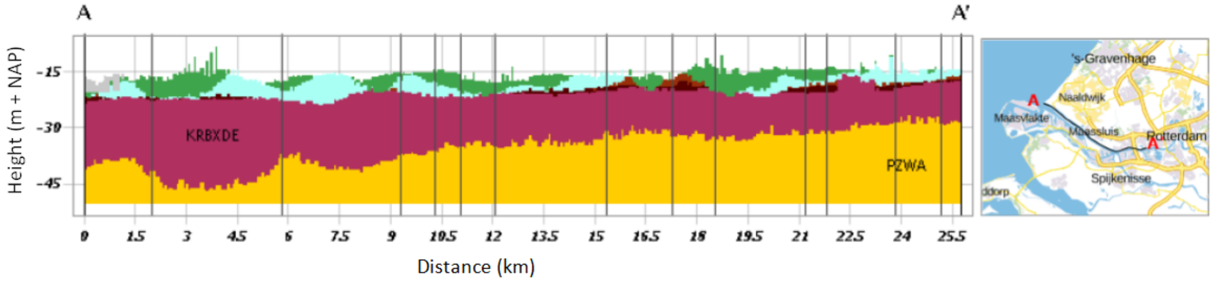



*Figure 80 Soil composition along the New Waterway*

**Spatial usage**

The usage of the land and water in the project area varies a lot. As can be seen in Figure 81 the largest difference can be seen between the south and the north of the New Waterway. The south is mainly covered with port basins and industrial areas. Several small urban areas are situated in between the port basins. On the north side there is a variety between a lot of urban activities to the east and rural areas to the west. Even further to the west a lot of greenhouses are build. The water itself is mainly used by the vessels visiting the port of Rotterdam. This means a wide variety of vessels, including recreational ones. A clear distinction should be made between the New Waterway and the Caland canal south of the New Waterway. Only a small strip of land divides the two canals. The Caland canal is the entrance to the Europoort basins. It therefore hosts different types of vessels. Besides, the Caland canal is not protected by the Maeslantbarrier.

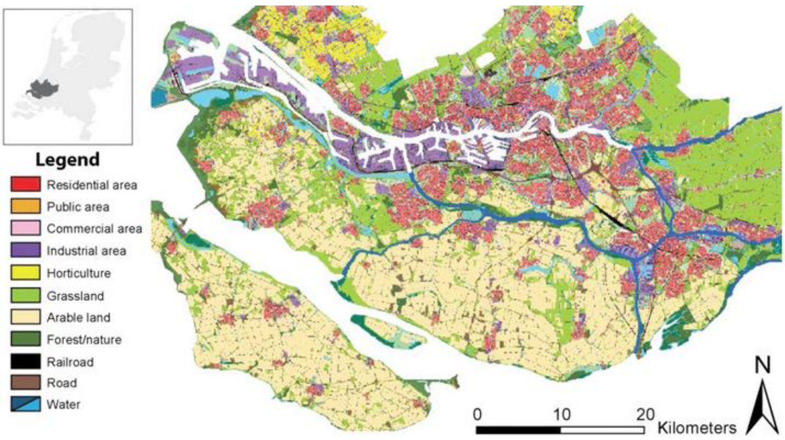



*Figure 81 Land usage Rotterdam area (Koks, et al., 2014)*

## A3. Design of a navigation lock

A navigation lock complex of this kind has several functions. First of all the navigation function. Besides of this, it has to retain a certain water level corresponding to extreme storm conditions both on the seaside and landside of the complex. A lock consists of several elements for which important design decisions have to be made. Both aspects will be evaluated further.

### *A3.1. Elements*

A lock consists of multiple elements. Only the most important civil ones will be elaborated on which choices should be made in the design process. A lot of different options are for the different elements, but only the most applicable ones will be named.

**Lock chamber**

The lock chamber is the element where the water level is changed according to the water level in the canal section where the vessel is heading. Multiple methods are available to construct the lock chamber. First of all, the dry excavation method can be used. This is the most commonly used method. Besides of this, another common method is the sheet pile wall or diaphragm wall with a underwater concrete floor. Both methods can be made on a shallow foundation or a pile foundation. More varieties can be thought, but the above are the most obvious ones.

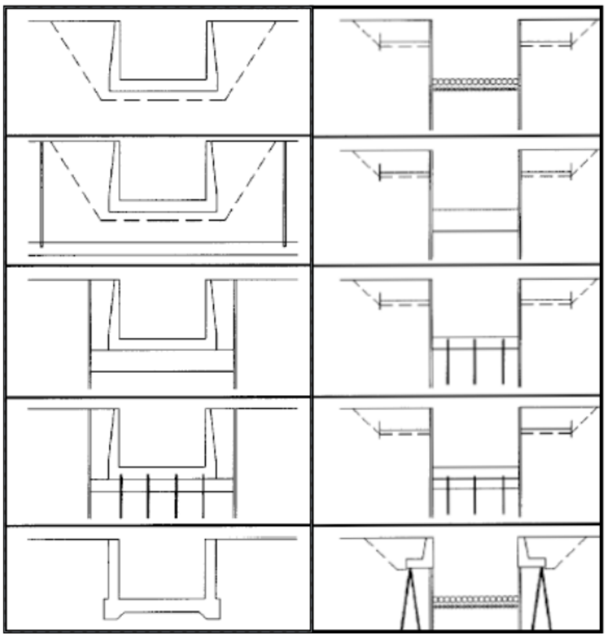


*Figure 82 Multiple construction methods lock chamber (Glerum, et al., 2000)*

**Lock head (seaward and landward)**

The lock heads are the retaining structures of the lock and the support structures for the gates, thus the place where the lock is closed and where an active water level difference needs to be retained. Partly the same design considerations can be made for the head as for the chamber. Another method that can be used for heads is to prefabricate the head in a separate building pit and float it to the designated location, instead of making the head in situ. Of course the choice for the type of gate is very important for the head design. The different types result in different loading schemes and different spatial requirements.

**Gates (seaward and landward)**

The gates are one of the most important elements of the navigation lock. It is the closing and opening element of a lock. A wide variety of types of gates is available. The choice for the gate depends on the functional requirements of the lock. The type influences the shipping width, the height and the depth of the lock. Not every gate is applicable in the same way. Three main types are widely used in navigation locks, the mitre gates, the rolling gate and the lift gate. The latter one is the only one with a vertical height restriction, which makes it unpopular for decision makers. This doesn’t have to be a problem however, when there are other restrictions (bridges) in the adjacent channel as well. The difference in applicability of the mitre gate and the rolling gate is the wideness of the lock. Mitre gates are mostly applied in narrow locks. When the locks becomes wider, the bending moment in the hinges will increase substantially. This makes the rolling gate more attractive in these cases. Drawback of the rolling gate is the large space that is required for the lock head perpendicular to the chamber axis.

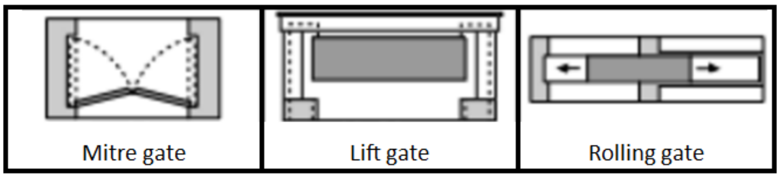



*Figure 83 Lock gate types (Hovingh, 2002)*

Another gate type that is not widely used in the Netherlands, is the pressure segment gate, see Figure 84. This gate is used in Germany in upper heads in river locks. The lock can be filled by rotating the gate downwards. Because of the shape of the gate, the water will flow down the gate in an energy dissipating chamber. The lock can be opened by rotating the gate completely downwards. For maintenance purposes, the gate can be rotated upwards, above the water level. Similar gates with a horizontal axis are also used for very large barriers in the Ems and the Thames with a width of about 60 m. These are however without the filling system.

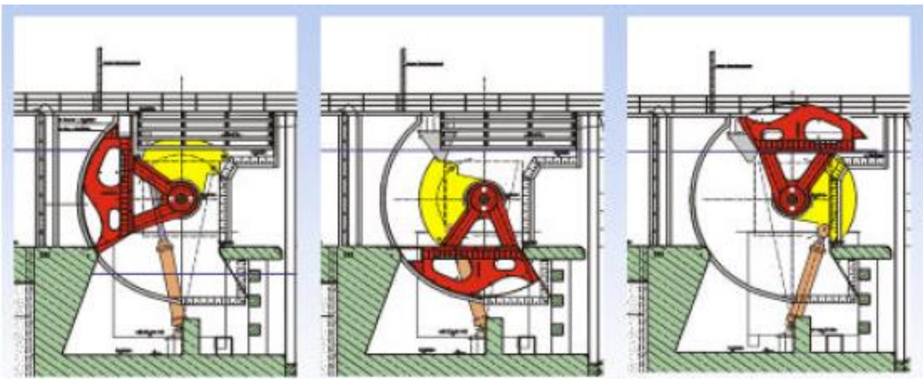


*Figure 84 Pressure segment gate (PIANC, 2009)*

**Filling and emptying system**

Three different mostly used types of filling/emptying systems can be derived. The most easiest is the one with openings in the gate. The construction costs are relatively low and maintenance is easy. Another option is the one with culverts around or under the gate through the head. The inflow can be obtained with a stilling chamber just behind the gate within the lock chamber, or larger longitudinal culverts with multiple openings along the lock chamber. The latter one is usually the most expensive one. The reason that these are applied instead of the gate openings, is that it makes possible to reduce the locking time while the turbulence caused by the inflow is minimized. Increasing the inflow through the gate opening will cause high local turbulence, resulting in high truss forces or troubles when handling smaller vessels. The culvert option is therefore mostly applied at locks with a very large head difference.

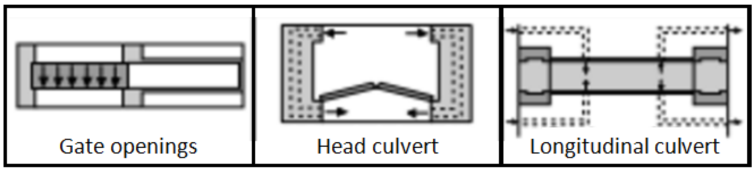



*Figure 85 Filling/emptying system types (Hovingh, 2002)*

**Guiding structure, approach area**

For the approach area a different approach should be made for locks for inland going vessels and locks for sea-going vessels. For inland navigation, guidelines can be used to determine the dimensions of the guiding structures (Glerum, et al., 2000). For sea-going vessels these guidelines don't exist. Usually nautical research is done to determine the requirements. Sea-going vessels are hard to manoeuvre, therefore the arrival of a vessel is always planned, so that the vessel can enter the lock chamber immediately. This is not the case with inland vessels. It is possible that a vessel has to wait before it can enter the lock. To make this possible, a waiting area with a mooring jetty have to be made where the vessels can be moored. These jetty are attached to a leading jetty, narrowing the approach channel to the width of the lock chamber. Sometimes an area have to be available where vessels can stay overnight. Besides of this, the approach area should be large enough to allow vessels for decreasing their speed.

Because sea-going vessels often need the assistance of tugboats, the mooring jetties are not desired in front of the locks, besides of the fact that they are not necessary. Usually, only a short leading jetty in front of the lock is made, to prevent collision damage of the lock head.

Another aspect where the mooring jetties can be used for, is for a harbour of refuge. This is a place where vessels can moor when an extreme situation like high or low water occurs. Research should be done to investigate whether it is necessary that the waiting area needs to facilitate this.

## *A3.2. Functional Requirements*

**Navigation**

One of the main functions of a navigation lock is of course making navigation possible. Multiple requirements can be made for navigation. One of them is capacity of the lock. This is the amount of vessels that is able to pass the lock within a unit of time. Secondly there is the size of the chamber. This is determined by the largest vessels that has to pass the lock.

**Safety**

Several safety requirements can be made for a navigation lock. There are different reasons that unsafe situations will occur.

One of them is structural failure. The lock structure has to retain certain loads. In order to do so, it has to be strong enough. The requirements for strength are determined by the prevailing standards.

Besides of that, the flooding safety should be taken into account. The lock has to retain certain water levels to make sure the hinterlands is safe against flooding. Occurring of the design water levels, does not necessarily mean structural failure, so a distinction is made herein. Furthermore, the general safety during usage should be taken into account, for instance the navigational possibilities, the width of the lock chamber, flaws in the operating system, etcetera.

**Maintenance**

Before a structure is designed, the level of maintenance should be taken into account. Some types of structures need more maintenance than others. Besides of that, some parts (like the foundation) are hard to reach and therefore should be designed in such a way that maintenance is not necessary. Maintenance is almost always considered as a large nuisance and expense.

**Reliability and availability**

For the users of the lock, the nuisance of passing the lock should be reduced to a minimum. Therefore the user requirements should take this into account. This results in requirements for signalling, the shape of the lock and the approach channel and the operating system. Besides of that, the availability of the lock should be derived beforehand, so that an estimation for the downtime can be made.

**Life time**

The lifetime of a structure influences the different requirements for strength and usage of the lock. The future developments should be taken into account regarding the size and amount of vessels and the deterioration of the structure.

**Loads**

Several loads should be taken into account when designing a navigation lock.

- Wind
- Hydraulic
    - Waves
    - Current
    - Hydrostatic
    - Seiches
    - Tide
- Vessels
    - Ship collision
    - Thrust forces
    - Mooring forces
- Ground pressure (buoyancy)

## A4. Adaptive design

Within the Delta program an adaptive delta management approach is developed (van Rhee, 2012). This holds that the band width of the future scenarios are taken into account. There is a large uncertainty in the climate and the social-economic development of the Netherlands during the lifetime of a structure. Both of them influences the requirements of the lock complex. The approach is that newly designed dikes and hydraulic structures are made in such a way that it is safe for now, but that it can easily and relatively cheap be adapted when another situation will develop in the future, see Figure 86. (However a different approach is needed for the dikes and for the hydraulic structures, they don't have the same level of adaptivity). On the other hand, the structure needs to be robust enough. It is not desirable that every year a structure has to be adapted. For a navigation lock such as in the New Waterway, several aspects can be taken into account in this. Not only regarding the safety of the people in the hinterland, but other functional requirements as well.

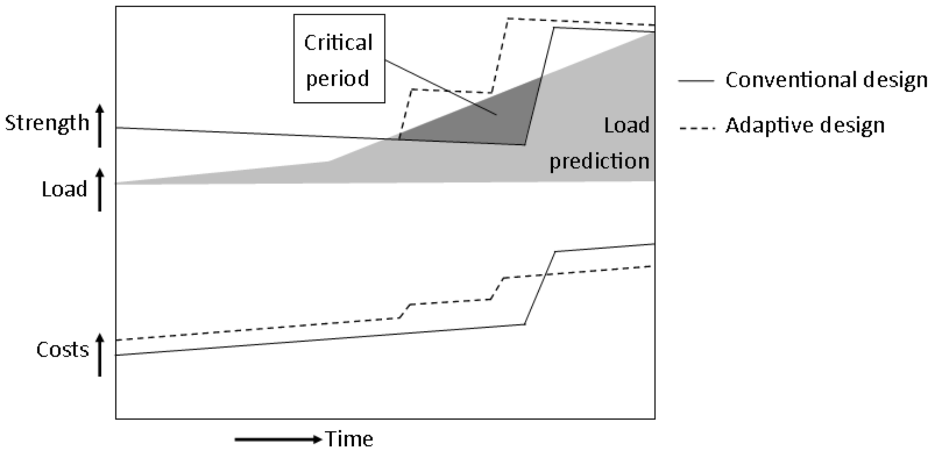



*Figure 86 Strength and costs relation in time*

## *A4.1. Scenario aspects*

**Navigation**

The navigational requirements depend on the passing vessels. The port of Rotterdam plays a major role in this aspect. Landwards of the considered lock locations there are several port areas for sea-going vessels. Because of the growing size of vessels, port development was focused more and more to the west to the Maasvlakte areas. Smaller port basins in the city centre of Rotterdam were modified for urban activities. It is however not quite certain that this will happen with the remaining eastern port basins. These are however smaller than the basins in the western port area, but because of the far inland location they can still be competitive. Building a navigation lock will of course change this competitive position. Furthermore, it is possible that the usage of inland vessels will grow.
Summarizing, the lock has to be designed in such a way that it can handle the current and future amount and size of vessels without over dimensioning too much. With a decreasing number of vessels, the lock will be over dimensioned in the future. This is probably a waste of money. The only operating problem that will occur due to this is the large locking time, because the lock chamber is much bigger than the vessel size. This can be solved by making a third lock head in the middle of the chamber. This will result however in even higher costs.

When the vessel requirements will increase, large operating problems can occur: Large waiting times and vessels which are too large to pass the lock. The lock has to be expanded when this happens. There are several possibilities for lengthening a lock chamber. One of the heads can be demolished, where after the lock chamber can be lengthened and a new head will be placed adjacent to the chamber. Furthermore a complete new lock chamber and lock head can be placed adjacent to one of the old heads. This requires more spatial availability, but less demolition costs. The latter one is the most applicable when the capacity needs to be increased. The first one when the size of the vessels requires lengthening.
Widening of the lock is hard to do. The gates and therefore the heads needs to be redesigned. Besides, the walls of the lock chamber needs to be relocated. In fact almost a complete new lock has to be constructed.
Deepening of the lock is not easy as well. The walls of the lock chamber can be designed in such a way that a larger depth is possible. This is however not possible for a conventional head. The gates need to be changed, and therefore also the head.

**Safety**

Regarding the water safety a complicated approach is needed. The investments in water safety depend on the probability that an extreme situation occurs and the consequences of a failure of the dikes. In fact: risk = probability of failure * consequences. Both of these aspects are hard to predict for the future. The consequences depend on the social-economic development. More people in the hinterland with a higher economical value will make the consequences in case of a failure much higher. Regarding the aspects of the economic crisis and the aging population of the Netherlands it is not certain that a higher safety level is necessary.

For the probability of failure the same complicated situation occurs as for the probability of occurrence of an extreme situation. This depends however on the nature and climate change. The current predictions results in higher water levels and a higher discharge from the rivers. See Table 33 for the current Deltascenarios for the Netherlands: Druk, stoom, rust and warm, translated respectively busy, stoom, quite, warm. These four scenarios are a combination of social-economic and climate changes. There is a lot of discussion concerning the uncertainty of this topic. Besides of this, there are different consequences of climate change. First of all there is the sea level rise. This is a slowly graduating process and can be monitored easily. On the other hand, the development in probability of exceedance of extreme water levels cannot be monitored and this process is thus hard to predict. An extreme situation occurs always unexpected. The argument to delay strengthening measures because it is not necessary in the current situation is therefore very hard to maintain. This is one of the reasons that Spaargaren initiates the building of the lock in the New Waterway (Rijkswaterstaat, 2015). The same accounts for the extreme discharges from the river.

| | | | DRUK | | STOOM | | RUST | | WARM | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | 2000 | 2050 | 2100 | 2050 | 2100 | 2050 | 2100 | 2050 | 2100 |
| Sea level rise | (cm) | | 15 | 35 | 35 | 85 | 15 | 35 | 35 | 85 |
| Extreme discharge Rhine 1/100 year | (m3/s) | 12.500 | 13.000 | 14.000 | 14.000 | 15.000 | 13.000 | 14.000 | 14.000 | 15.000 |
| Extreme discharge Meuse 1/100 year | (m3/s) | 2.900 | 3.000 | 3.200 | 3.200 | 3.600 | 3.000 | 3.200 | 3.200 | 3.600 |
| Return period Rhine discharge >15.000 m3/s | (year) | 1.250 | 1.000 | 400 | 400 | 100 | 1.000 | 400 | 400 | 100 |
| Return period Meuse discharge >3.600 m3/s | (year) | 1.250 | 1.000 | 400 | 400 | 100 | 1.000 | 400 | 400 | 100 |
| Extreme low discharge Rhine 1/10 year | (m3/s) | 630 | 650 | 670 | 520 | 420 | 650 | 670 | 520 | 420 |
| Extreme low discharge Meuse 1/10 year | (m3/s) | 18 | 18 | 18 | 10 | 6 | 18 | 18 | 10 | 6 |

*Table 33 Deltascenarios (Bruggeman, et al., 2013)*

Regarding the discharge that has to be handled by the sluice complex, the same problems as with the water levels count. It is very hard to predict the development of probability of exceedance of discharge levels. When a high discharge level occurs and one did not take this into account, dangerous high water levels on the rivers can occur.

To make a sophisticated design regarding water safety, the above aspects have to be weighed to what extent they will be considered in the design process.

**Remaining future functional requirements**

More future functional requirements have to be taken into account. For example when a large discharge occurs and the capacity of the dewatering sluice is not enough, it might be a feasible option

to open all the doors of the navigation lock and use it as a dewatering sluice. Navigation is not possible in that situation and the bed protection have to retain the large current.
Another future requirement that can arise is the preventing of salt intrusion. Additional measures in the navigation lock might be needed to minimize the salt intrusion in the New Waterway.

## *A4.2. Implementing adaptive delta management in the design elements*

When adaptive delta management is used in the design of a structure, a balance has to be found between doing nothing and over dimensioning. The approach can be used for multiple elements. Especially the ones that are easy to reach and don't cause much trouble for shipping and the neighbourhood when working on it. The most important elements where this approach will be considered can be found in Figure 4. A first approach can be made by deriving several scenario's which has to be taken into account. In this exploratory part a distinction will be made in a qualitative way between two scenario's: one where the current assumptions will stay valid and another one where the requirements will increase.

**Foundation**

A foundation is not easy to adapt in a later stage. It is hard to reach and will probably cause long blockings of the lock when work has to be done. Therefore the most plausible option is to make the foundation as robust as might be reasonably necessary in the uncertain future.

**Heads**

The lock heads are easier to reach. Depending on the type of head and the type of work that needs to be done blocking of the lock can be substantial. This should be reduced to a minimum or to a manageable amount. Several options are available to do this. For instance by replacing the complete head. This can be done when the caisson method is used where the head is floated to its place. Another method is to make the lock head modular. In this way elements of the head can be removed to widen or deepen the head. Of course the rest of the head should have enough strength to bear the loads. The same, but reversed, approach applies for heightening of the head, than a modular element can be placed on top of the existing head.

**Gate**

The gate is a separate element that can be lifted out of the head when necessary. When this is done, another gate can be placed in the lock instead. This is the fastest way and will only cause a small blocking time of the lock. Construction costs are however high when a complete new gate is made. When the gate is made of steel, it can be adjusted in size when still in its place in the lock if the heads allow this. This is however a time consuming operation, where the lock needs to be blocked all the time.

**Chamber**

Adjusting the chamber will likely cause blocking of the lock for a long time. A possible method to widen the chamber is to build a new retaining wall behind the existing wall. After this is done the old wall can be removed. The use of anchors is than however probably not possible in the initial design. One of the most applicable methods to deepen the chamber, is to use a floor of loose rock. These are easy to dig out and can be replaced deeper.

**Filling/emptying system**

The suitability of the filling and emptying system for an adaptive approach depends on the type of system. The build in culvert systems are not easy to adapt. Openings in the gates can be adjusted, but they are probably already designed for maximum allowed capacity regarding the allowed turbulence in the chamber. Besides of this, the only aspect that adapting the filling and emptying system can improve, is the locking time and thus the capacity. This will probably not be a high improvement, especially when compared to the costs. Therefore the filling/emptying system will not be a subject to an adaptive strategy.

**Guiding structures**

The guiding structures or approach area are not a part of the lock itself. Therefore work that needs to be done will not highly influence the working of the lock. Besides of this, it is not likely that the requirements for the guiding structures will increase, apart from lengthening the mooring structure. So it is not likely to take an adaptive approach into account when looking at the guiding structures.

## A5. Construction innovations

Different types of innovations can be considered. An innovative idea can be very elegant, which results in slender structures with a lot of savings in terms of material usage and therefore costs. On the other hand, innovation can also mean a simple, pragmatic approach, resulting in a low-risk structure. Multiple innovative construction methods can be taken into account when designing a navigation lock, but Of course not all of them can be applied in the situation of the sea navigation lock in Rotterdam. Several interesting options will be stated here.

**Immersion technologies**

Immersion is a proven method for tunnel elements. This can also be used for the construction of locks. The sizes of an lock for inland going vessels chamber are more or less the same as for the immersed tunnel elements. The advantage of this method is that it is not necessary to make a building pit. The work on the building site itself will cause less nuisance. Furthermore, because the elements will be made in a controlled situation, the quality can be better guaranteed. This is however probably not applicable for large locks, because the dimensions will be too large to make in one piece.
Another option to using immersion technology is by not making the locks as elements, but the walls of the locks. A stiff structure can be made in this way, because the top of the elements can be closed as well, which is not the case with the lock chamber itself. Besides of this, it can be used for larger widths of a lock chamber. The floor of the chamber can be made with underwater concrete or with loose blocks.

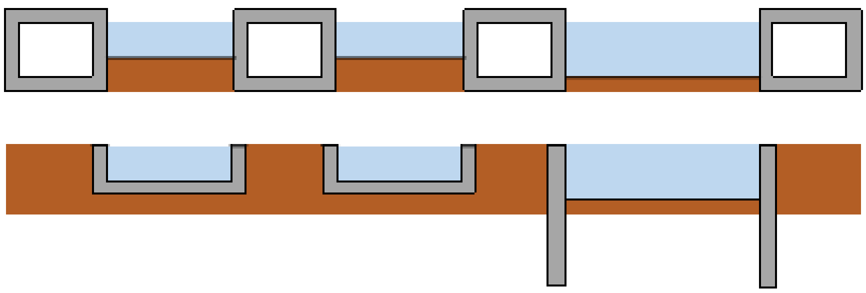


*Figure 87 Immersion construction options for lock locks (upper: immersed walls, lower: immersed locks)*

**Combined lock chamber walls**

Because multiple lock locks have to be build next to each other, it is possible to combine the walls of each other. If this is done, a very strong and stiff wall can be realized. This can be beneficial regarding the soil retaining function and the mooring forces of the locking vessels. The above one with immersion is one of the options. Another one is to make diaphragm walls or sheet pile walls and connect them to each other. In regular designs, ground anchors are made to reduce the bending moments in the sheet pile. This function can be overtaken by the other wall of the other chamber. Several configuration can be thought of. When the distance between the locks will be too high, an intermediate wall can be made in between. This can be combined with a soil relieving floor, to reduce the horizontal soil pressures at the sheet pile.

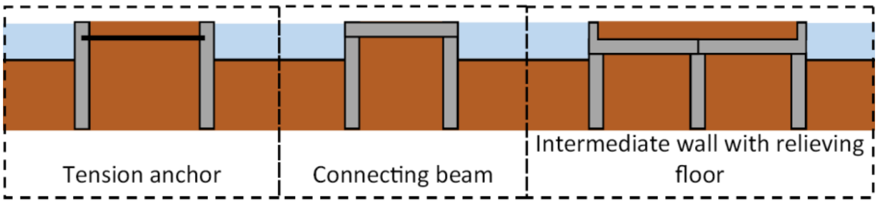



*Figure 88 Chamber wall connection options*

**Locks as water saving basins**

One of the benefits of closing the New Waterway is that salt intrusion is reduced. However, with low discharges this can still cause a problem. This will cause low water levels on the landward side of the lock. Together with high vessel traffic and thus many locking cycles, a substantial amount of salt water will flow into the fresh New Waterway. Furthermore, the outflow of fresh water into the sea needs to be reduced to a minimum in case of a dry period. Both problems can be solved by using water saving basins. Every locking cycle a part of the outflow water will be saved in the basin, to be used in a later cycle again. The total volume of used water will decrease in this way. Building separate water saving basins is an expensive and spatial defining matter. To prevent this, the lock locks can use each other as water saving basins. A culvert system has to be built together with the locks. Besides, this can only be done when the other chamber has a low water level in the chamber and needs to be risen. This is of course not always the case. It is however possible to control this by the lock manager, but this will result in larger locking times. This is only necessary in extreme situations with a very low river discharge. The problems of salt intrusion should be researched in more detail to make a decision if this measure is more cost-effective than other solutions. It is likely that these problems will occur later in time, with changing of the climate.

**Emptying/filling system through relieving floor**

When a filling and emptying system with openings in the gate is not sufficient in terms of locking times and turbulence problems, one can think of a longitudinal culvert system. The conventional ones through the walls or along the floor are very expensive. Another option is to use a relieving floor as a culvert. A relieving floor is hollow which can be used to transport water. The floor does not lose its function as it is meant to relieve the horizontal forces on the chamber wall. This wall has to be placed a little bit backwards to make vertical outflow possible. This will slightly widen the chamber. The total volume of used water each locking will however not be larger, because (under normal circumstances) the water level will not drop under the relieving floor. This is one of the requirements of the functioning of this system. The inlet is relatively high. This can cause problems regarding the availability of the lock when a situation occurs with very low water levels outside the lock.

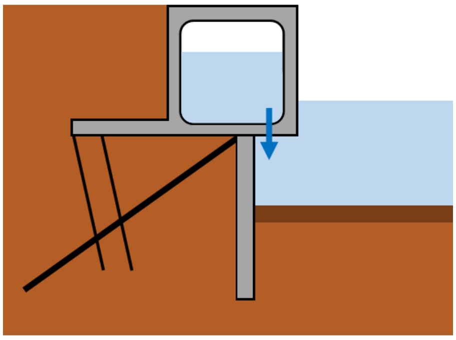


*Figure 89 Emptying and filling system through relieving floor*

**Rolling arc gate**

One of the most promising gate types for very wide locks is the rolling arc gate. It is comparable with the conventional straight rolling gate. The arched shape however reduces the bending moments in the gate itself. This causes that the gate can be lighter and thinner. Besides of that, the required space for the lock head will be smaller, making it a feasible option for situation with large space limitations, like in the port of Rotterdam. Several studies have been executed regarding this type of gate (Doeksen, 2012) (v Rossum, 2015) (Zel taat, 2012). Out of these studies it can be concluded that this is a feasible option that can be further elaborated. Special aspects of interest are the supports of the gate, to make sure that it stays in place when loaded. Besides of that, the shape of the gate will result in lateral forces which have to be transmitted to the head.

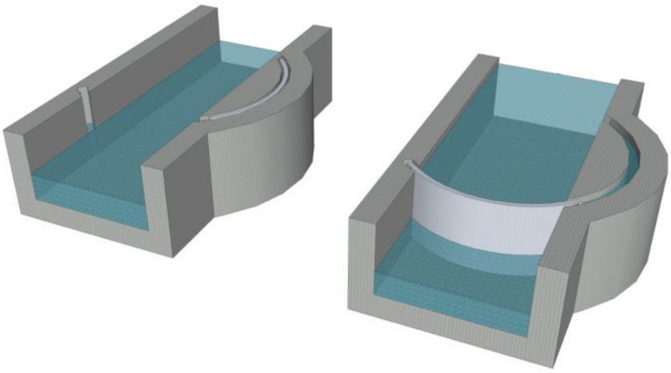


*Figure 90 Rolling arc gate (Doeksen, 2012)*

**Modular building**

The total length of locks and the amount of heads and gates are considerably. Because of this total volume of work, it can be beneficial to build the structure out of certain elements with a determined dimension. Because of the amount of elements this would generate, it will be cost effective to make a special designed construction site where these elements can be prefabricated. The construction process can be eased with re-use of formworks and easier placement of formwork. In this way it can be cheaper than in situ construction and the quality can be better guaranteed, because it is constructed in a controlled environment. A drawback is that the elements should be transferred to the construction site and placed adjacent to the other elements.

# A6. Reference projects

## *A6.1. IJmuiden*

In IJmuiden, the existing lock complex needs to be renovated to handle the future expectations of vessels calling in in Amsterdam. A new lock chamber will be built to replace the current largest lock chamber. The new lock will be built in between the other ones. When it is finished, it can be named the largest lock in the world with a width of 70 m, a length of 500 m and a depth of 18 m. A point of interest is how the water retaining function is ensured. The conventional method is to make two gates in each head. In this way maintenance can be done without losing the safety against flooding. With this project they have chosen to use two identical gates for the two heads, with one spare gate. When one of the gates needs to be lifted out because of ship collision or maintenance, the spare one can be easily and fast replaced. In this way the lock can be built much cheaper.

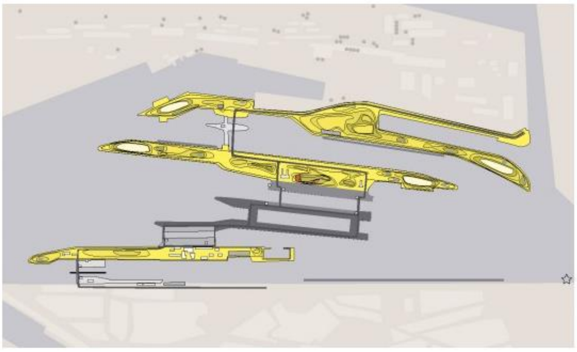


*Figure 91 Navigation lock complex with in black the new lock chamber*

**Differences between IJmuiden and Rotterdam:**

- Building a new chamber in an existing complex

**Similarities between IJmuiden and Rotterdam:**

- Sea lock complex
- Similar maximum size of the passing vessels
- Primary water retaining function
- Small spatial availability

## *A6.2. Terneuzen*

In Terneuzen a new lock will be built in the existing lock complex. The lock complex provides the connection between the ports of Gent and Terneuzen and the Western Scheldt. It consists of three lock locks. The middle one (140 m long and 18 m wide) will be removed and a new, bigger one will be constructed in its place (van Calsteren, et al., 2015). The new one will be 427 m long, 55 m wide and 16 m deep. In this way the lock can transfer most of the seagoing vessels. The construction of the lock is in the planning phase, so the final design is not yet made. The preference preliminary design is a lock with 2x2 rolling gates. The emptying and filling of the lock will be done with openings in the gate.

*Figure 92 New lock chamber in between the old locks (Schultz, 2015)*

**Differences between Terneuzen and Rotterdam:**

- Dewatering function much smaller in Terneuzen
- Building a new chamber in an existing complex
- The amount of passing vessels is smaller in Terneuzen

**Similarities between Terneuzen and Rotterdam:**

- Sea lock complex
- Similar size of the passing vessels
- Primary water retaining function
- Small spatial availability

### *A6.3. Meppelerdieplock*

The Meppelerdieplock is a lock that is changed from a water retaining sluice to a navigation lock. The dimensions of the lock are probably much smaller than the dimensions of the lock in the New Waterway. Interesting is however the method that is used to ensure the water retaining function. Under normal circumstances, both gates of the lock are opened. Vessels can pass the lock without passing a water level difference. When high water occurs, the gates will close and the structure will function as a normal navigation lock. The nuisance for the shipping will be minimized in this way. It may be possible to implement the same method for the new navigation lock in Rotterdam.

The new navigation lock will be made with rolling gates. The lock chamber is wider than the heads. This will make navigation easier when the lock is not functioning as a navigation lock. The locking time will be however bigger, because more water needs to be transferred.

**Differences between Meppelerdieplock and Rotterdam:**

- The amount and size of passing vessels is smaller in the Meppelerdieplock

**Similarities between Meppelerdieplock and Rotterdam:**

- Primary water retaining function
- Small spatial availability

# B. Principal solutions elements

Several principal solutions will be discussed whether they are suitable for an adaptive approach. This is done in a qualitatively way. After this step they will be combined into an overall solution.

## B1. Gate and head types

For the gate types a distinction will be made between the seagoing chamber and the locks for inland going vessels, because not every type is feasible for the lock for seagoing vessels.

### *B1.1. Lock for seagoing vessels*

**Rolling gate**

The rolling gate is the most applied gate for locks with these kind of dimensions.

*Widening:* Widening of the lock is possible, the head needs to be adapted for that. The gate recess need to be enlarged. This can be taken into account easily in the initial design, by making the head modular. A new, larger gate can then be installed.

*Deepening:* To deepen the lock, the sill must be lowered. This can be done by making the floor of the head modular and remove the upper part when necessary. This will however cause an over dimensioned design in the first phase. A new, higher gate must be installed.

*Lengthening:* If the lock need to be lengthened, a complete new head has to be made. or the head has to be replaced.

*Heightening:* If the gate has to retain a higher water level, it can be heightened or a new gate can be installed. The head has to be heightened as well. The foundation must be able to retain this.

*Salt intrusion:* -

*Dewatering:* For dewatering, the gate must open and close during a high current in the lock. The rolling gate is not feasible for this.

*New lock:* The rolling gate has a high spatial usage perpendicular of the lock axis, which can make the placement of a new lock difficult.

**Rolling arc gate**

The rolling arc gate is never applied before as a navigation lock gate, but it is considered a feasible option. The main advantage is the spatial usage in perpendicular direction from the lock axis.

*Widening:* Widening of the lock is not easy, the whole lock head need to be replaced, because it is situated in the length of the lock. Besides, a new gate has to be installed.

*Deepening:* To deepen the lock, the sill must be lowered. This can be done by making the floor of the head modular and remove the upper part when necessary. This will however cause an over dimensioned design in the first phase. A new, higher gate must be installed.

*Lengthening:* If the lock needs to be lengthened, a complete new head has to be made or the head has to be replaced.

*Heightening:* If the gate has to retain a higher water level, it can be adapted or a new gate can be installed. The head has to be heightened as well. The foundation must be able to retain this.

*Salt intrusion:* -

*Dewatering:* For dewatering, the gate must open and close during a high current in the lock. The rolling arc gate is not feasible for this.

*New lock:* The rolling arc gate has a low spatial usage perpendicular of the lock axis, placement of a new lock is a possible option.

## *B1.2. Locks for inland going vessels*

**Rolling gates**

The rolling gates for the locks for inland going vessels have the same properties regarding the adaptivity as the rolling gates for lock for seagoing vessels.

**Mitre gates**

*Widening:* For widening of the lock, part of the lock head need to be replaced. Besides, new gates has to be installed. The head must be able to retain the new loads.

*Deepening:* To deepen the lock, the sill must be lowered. This can be done by making the floor of the head modular and remove the upper part when necessary. This will however cause an over dimensioned design in the first phase. New, higher gates must be installed.

*Lengthening:* If the lock needs to be lengthened, a complete new head has to be made or the head has to be replaced.

*Heightening:* If the gate has to retain a higher water level, it can be adapted or new gates can be installed. The head has to be heightened as well. The foundation and the head must be able to retain this.

*Salt intrusion:* -

*Dewatering:* For dewatering, the gate must open and close during a high current in the lock. The mitre gate is not feasible for this.

*New lock:* The mitre gate has a low spatial usage perpendicular of the lock axis, placement of a new lock is a possible option.

**Segment gate with horizontal axis**

*Widening:* Widening of the lock is not easy, part of the lock head need to be replaced. Besides, a new gate has to be installed.

*Deepening:* To deepen the lock, the sill must be lowered. Therefore the head must be adapted drastically and a new gate must be installed.

*Lengthening:* If the lock needs to be lengthened, a complete new head has to be made or the head has to be replaced.

*Heightening:* If the gate has to retain a higher water level, a new gate has to be made and the head has to be adapted for this. The head has to be heightened as well. The foundation must be able to retain this.

*Salt intrusion:* To lower salt intrusion, the gate can have a variable height in open position to retain the water at the seaside.

*Dewatering:* For dewatering, the gate must open and close during a high current in the lock. The segment gate is highly feasible for this.

*New lock:* The segment gate has a low spatial usage perpendicular of the lock axis, placement of a new lock is a possible option.

**Lifting gate**

*Widening:* Widening of the lock is not easy, a new lifting tower has to be constructed and the old one demolished.

*Deepening:* To deepen the lock, the sill must be lowered. This can be done by making the floor of the head modular and remove the upper part when necessary. This will however cause an over dimensioned design in the first phase. A new, higher gate must be installed. The towers must be able to lift this higher gate high enough.

*Lengthening:* If the lock needs to be lengthened, a complete new head has to be made or the head has to be replaced.

*Heightening:* If the gate has to retain a higher water level, it can be adapted or a new gate can be installed. The head has to be heightened as well. The foundation must be able to retain this. The towers must be able to lift this higher gate high enough.

*Salt intrusion:* -

*Dewatering:* For dewatering, the gate must open and close during a high current in the lock. The lifting gate is highly feasible for this.

*New lock:* The lifting gate has a low spatial usage perpendicular of the lock axis, placement of a new lock is a possible option.

## B2. Chamber walls

**Diaphragm wall with relieving floor**

*Widening:* Widening of the lock is not easy, a new wall has to be constructed and the old one demolished.

*Deepening:* If the chamber is deepened, the wall should be able to retain the higher horizontal loads, or it should be strengthened.

*Lengthening:* If the lock needs to be lengthened, the wall has to be lengthened as well. The existing wall can still be used.

*Heightening:* Heightening of the walls is possible, if the walls are able to retain the increased loads.

*Salt intrusion:* -

*Dewatering:* -

*New lock:* The relieving floor uses most of the time skewed piles. This means that building near the relieving floor lock can be difficult.

**Coffer dam**

A coffer dam consists of two diaphragm walls with a connection on top with anchors or a beam.

*Widening:* Widening of the lock is possible, but a new wall has to be constructed and the old one demolished.

*Deepening:* If the lock is deepened, the coffer dam should be able to retain the higher horizontal loads, or it should be strengthened.

*Lengthening:* If the lock needs to be lengthened, the wall has to be lengthened as well. The existing wall can still be used.

*Heightening:* Heightening of the coffer dam is possible, if the dam is able to retain the increased loads.

*Salt intrusion:* -

*Dewatering:* -

*New lock:* The coffer dam only has vertical walls. Building right next to the coffer dam is possible. The coffer dam can also be used as new chamber walls.

**Immersion locks**

*Widening:* Widening of the lock is not easy, a new wall has to be constructed and connected to the floor, besides, the old wall has to be demolished.

*Deepening:* because of the concrete floor, deepening of the lock is impossible without completely rebuilding the chamber.

*Lengthening:* If the lock needs to be lengthened, the wall has to be lengthened as well. The existing wall can still be used.

*Heightening:* Heightening of the walls is possible, if the walls and floors are able to retain the increased loads.

*Salt intrusion:* -

*Dewatering:* -

*New lock:* The immersion locks only has vertical walls. Building right next to the existing lock is possible.

**Immersion walls**

*Widening:* Widening of the locks is possible by replacing the walls.

*Deepening:* If the lock is deepened, the wall should be able to retain the higher horizontal loads, or it should be strengthened.

*Lengthening:* If the lock needs to be lengthened, the wall has to be lengthened as well. The existing wall can still be used.

*Heightening:* Heightening of the walls is possible, if the walls are able to retain the increased loads.

*Salt intrusion:* -

*Dewatering:* -

*New lock:* The immersion walls only has vertical walls. Building right next to the existing lock is possible. The walls can even be used as new chamber walls.

## B3. Chamber floor

**Underwater concrete floor**

*Widening:* Widening of the lock is possible, the floor has to be extended sideward.

*Deepening:* If the lock is deepened, the floor should be removed and a new one should be build, which is a laborious matter.

*Lengthening:* If the lock needs to be lengthened, the floor has to be lengthened as well. The existing floor can still be used.

*Heightening:* Heightening of the locks is possible, the floor will not suffer from an increased load.

*Salt intrusion:* -

*Dewatering:* When dewatering through the lock, high velocities can occur, the concrete floor is suitable to remain stable with these velocities.

*New lock:* -

**Loose rock floor**

*Widening:* Widening of the lock is possible, the rock layers has to be extended.

*Deepening:* If the lock is deepened, the rock should be removed and replaced deeper. This is a possible solution.

*Lengthening:* If the lock needs to be lengthened, the open floor has to be extended as well. This does not cause problems regarding the existing floor.

*Heightening:* Heightening of the locks is possible, the floor will not suffer from an increased load.

*Salt intrusion:* -
*Dewatering:* The loose rock has to remain stable during dewatering with high velocities.
*New lock:* -

**Concrete blocks mattress**

*Widening:* Widening of the lock is possible, a new mattress should be placed in the new part of the floor.
*Deepening:* If the lock is deepened, the mattress should be removed and replaced deeper. This is a possible solution.
*Lengthening:* If the lock needs to be lengthened, the mattress has to be extended as well. This does not cause problems regarding the existing floor.
*Heightening:* Heightening of the locks is possible, the floor will not suffer from an increased load.
*Salt intrusion:* -
*Dewatering:* The mattress has to remain stable during dewatering with high velocities.
*New lock:* -

## B4. Emptying/filling types

**Through the gates**

When a filling system with valves through the gates is chosen, it must be ensured that the capacity is high enough for future sizes of the locks, regarding the locking time and water levels. Most of the times this capacity is limited by the turbulence and velocity causing loads on the vessel.

**Longitudinal culvert in relieving floor**

With longitudinal culverts a high capacity is possible without causing problems regarding velocity and turbulence in the chamber. To adapt this capacity in a later stage, the culverts openings and valves need to be enlarged. When a lock has to be widened, this system has to be replaced as well. With a longitudinal culvert, the locks can be connected with each other. In this way the locks can be used as water saving basins to prevent salt intrusion.

## B5. Lock configuration

The lock complex can be configured in different ways. The most logical option is to build the lock for seagoing vessels at the outside (north or south) of the complex. When it is built at the north side, widening of the lock might be difficult, because it is enclosed by the pumping station/dewatering sluice and the other locks. On the other side, the lock for inland going vessels is not enclosed and can be expanded easily when needed. When the situation is mirrored (lock for seagoing vessels at the south side), this is of course the other way around.

# C. Hydraulic computations

## C1. Lockfill, filling through the gate

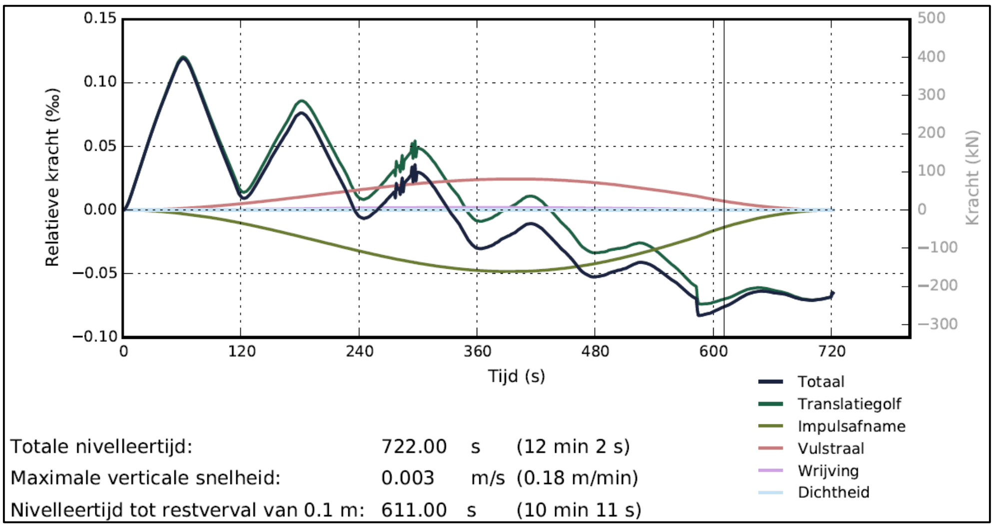



*Figure 93 Malaccamax, adaptive design*

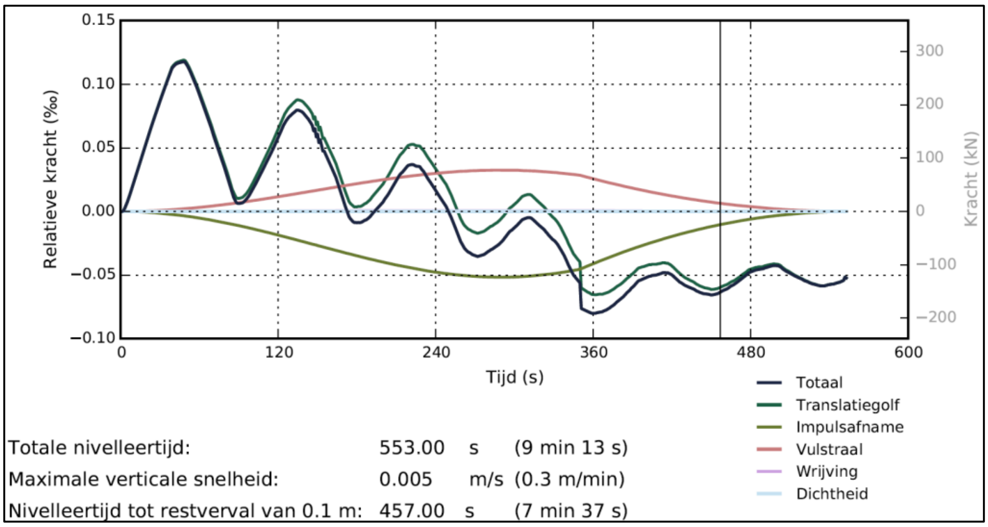



*Figure 94 New Panamax vessel, adaptive design*

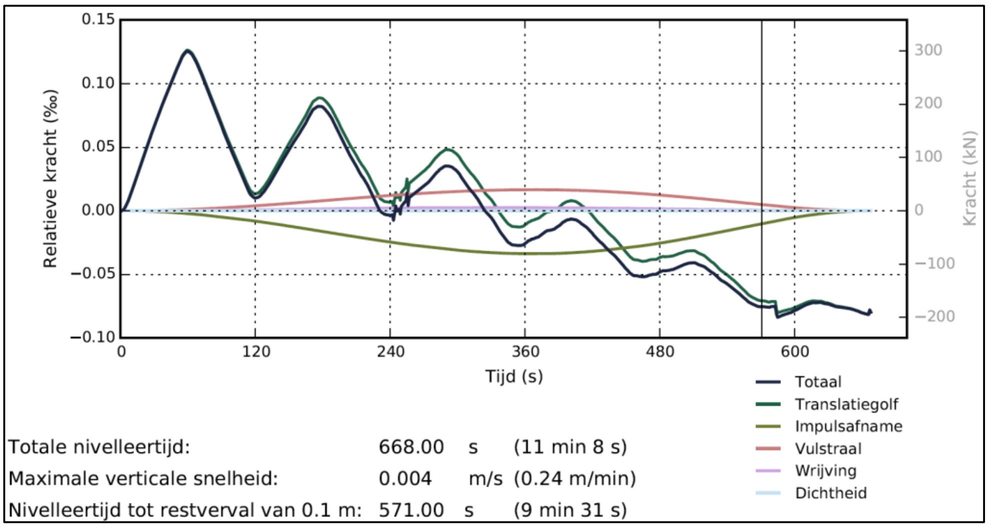



*Figure 95 New Panamax, initial design*

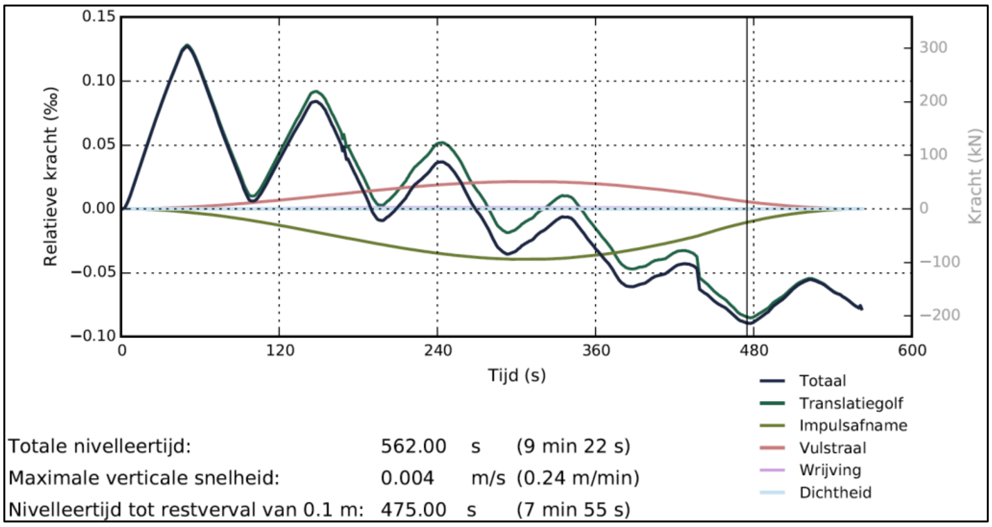



*Figure 96 New Panamax, wider design*

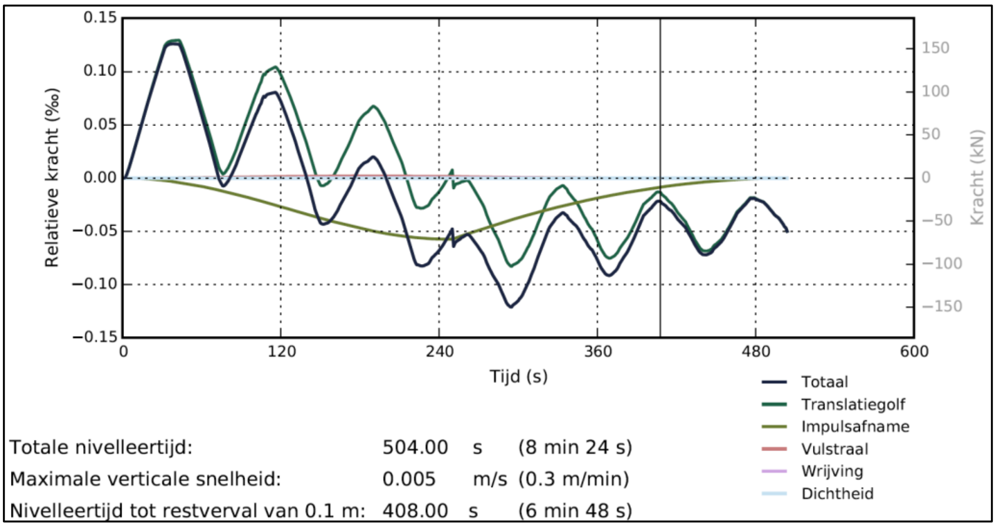



*Figure 97 Oasis of the seas, adaptive design*

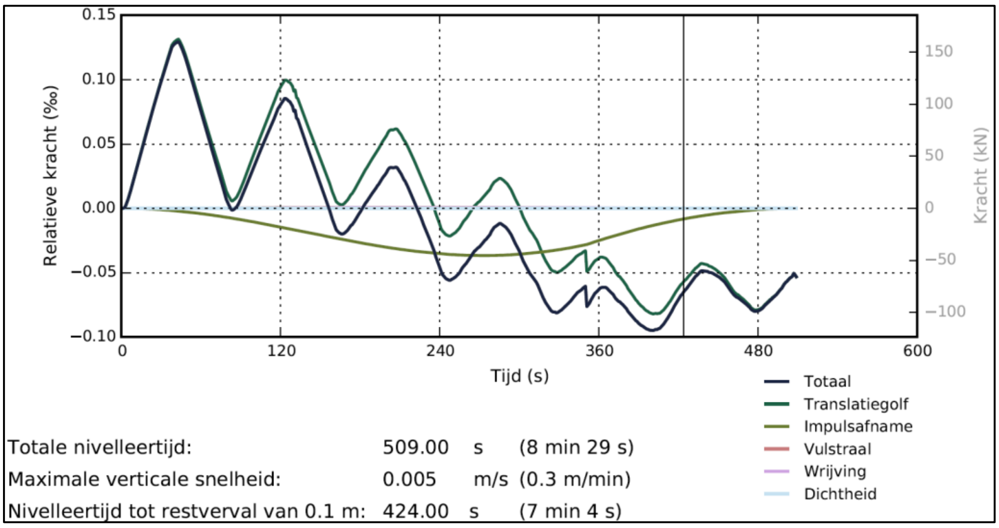



*Figure 98 Oasis of the seas, initial design*

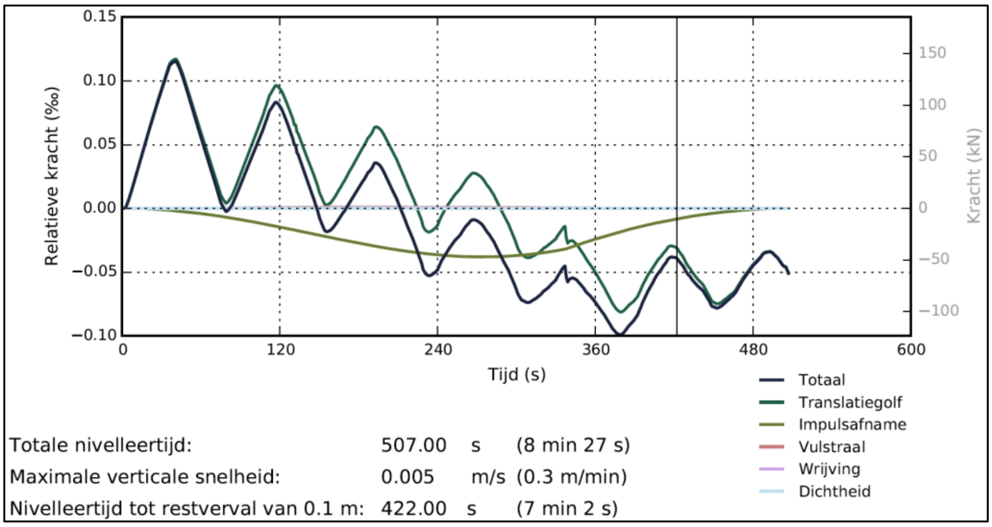



*Figure 99 Oasis of the seas, wider design*

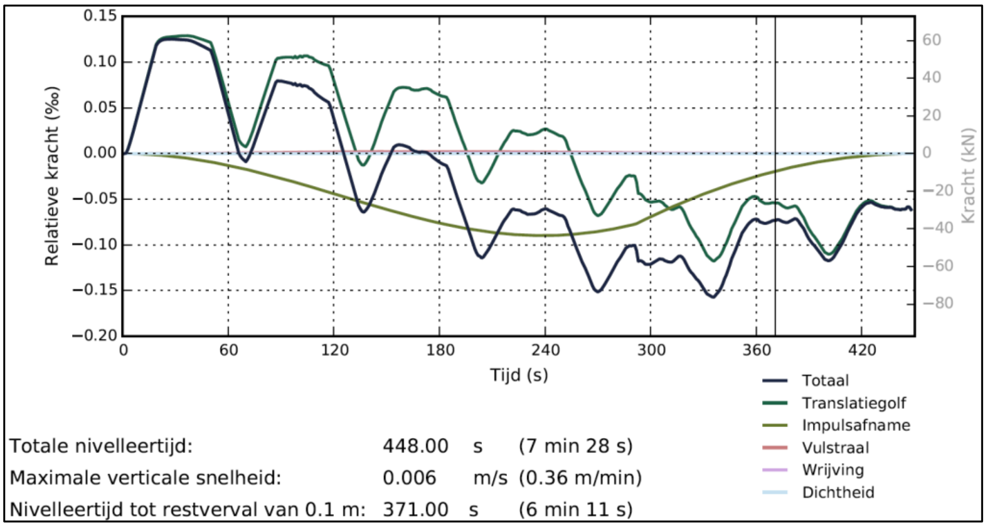



*Figure 100 Small vessel, adaptive design*

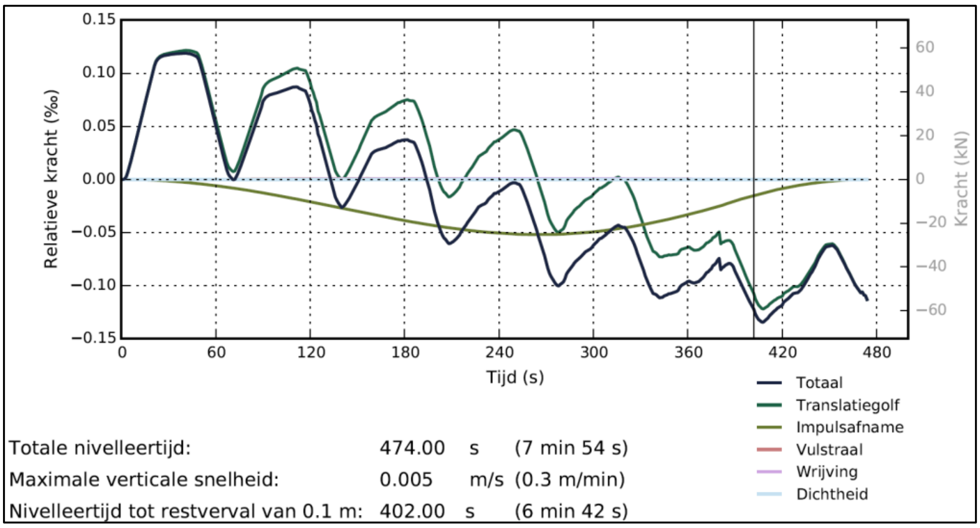



*Figure 101 Small vessel, initial design*

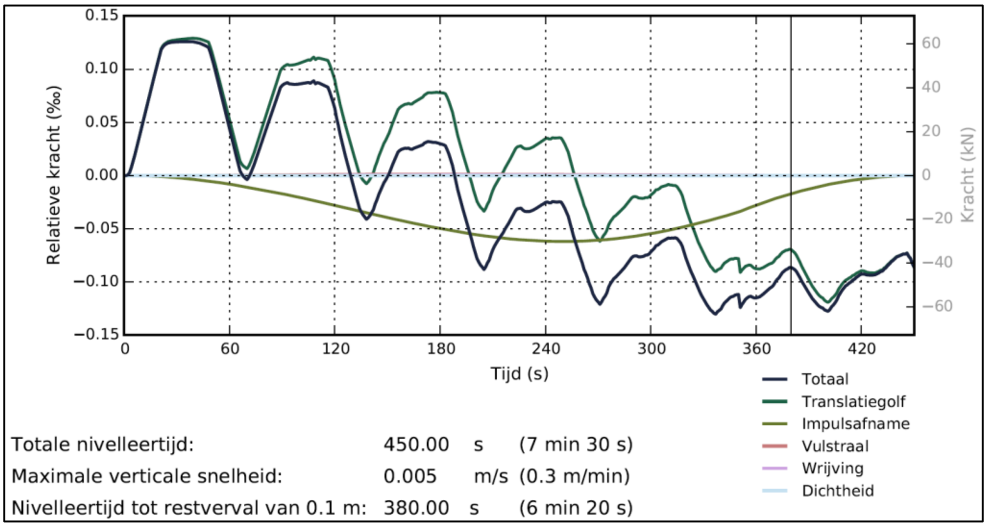



*Figure 102 Small vessel, wider design*

## *Other water levels*

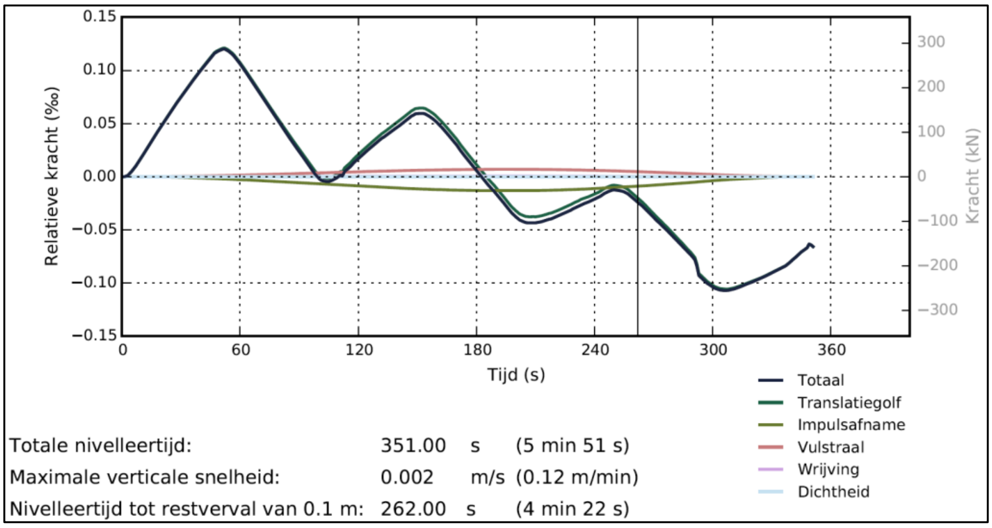



*Figure 103 New Panamax, wider design, approach harbour: 0 m + NAP, initial lock level: -0,55 m + NAP*

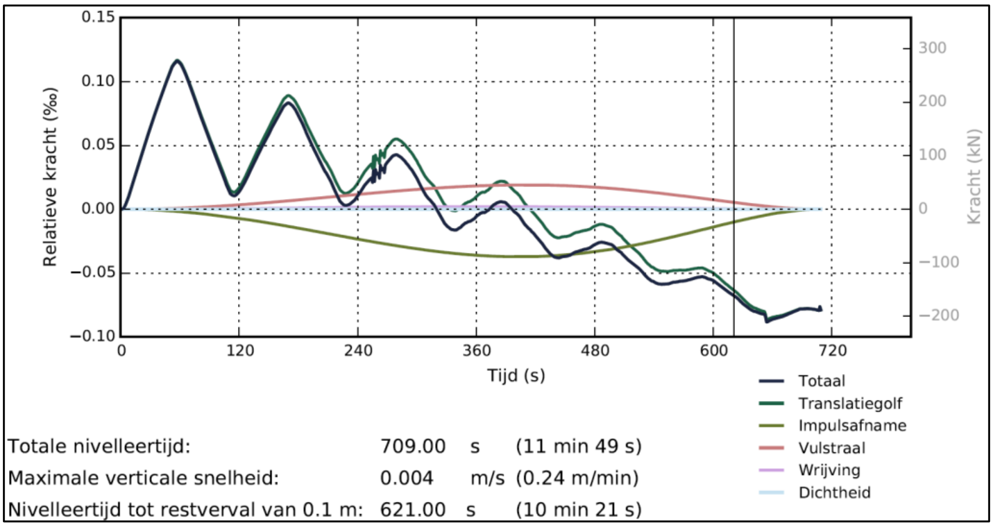



*Figure 104 New Panamax, wider design, approach harbour: 0 m + NAP, initial lock level: -1,85 m + NAP*

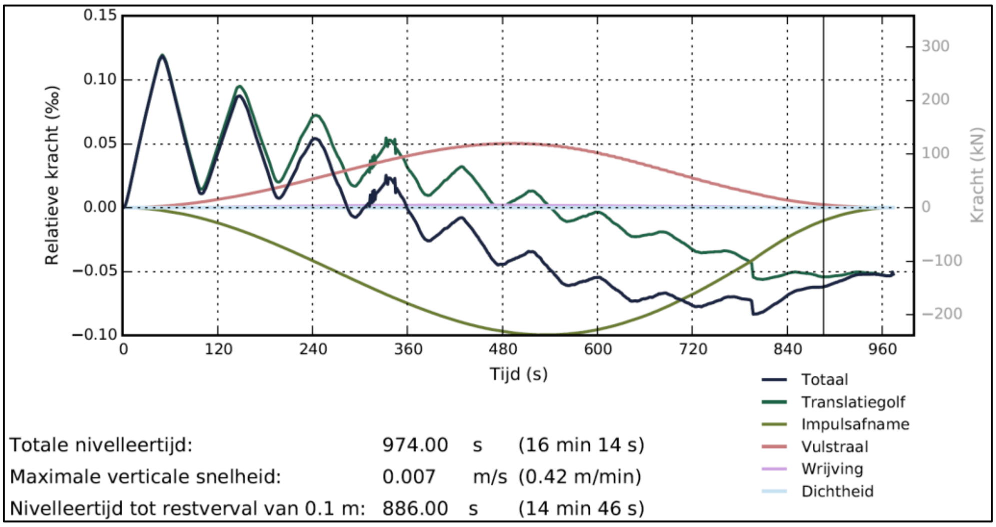



*Figure 105 New Panamax, wider design, approach harbour: 4,3 m + NAP, initial lock level: 0 m + NAP*

## C2. Lockfill, filling short culverts

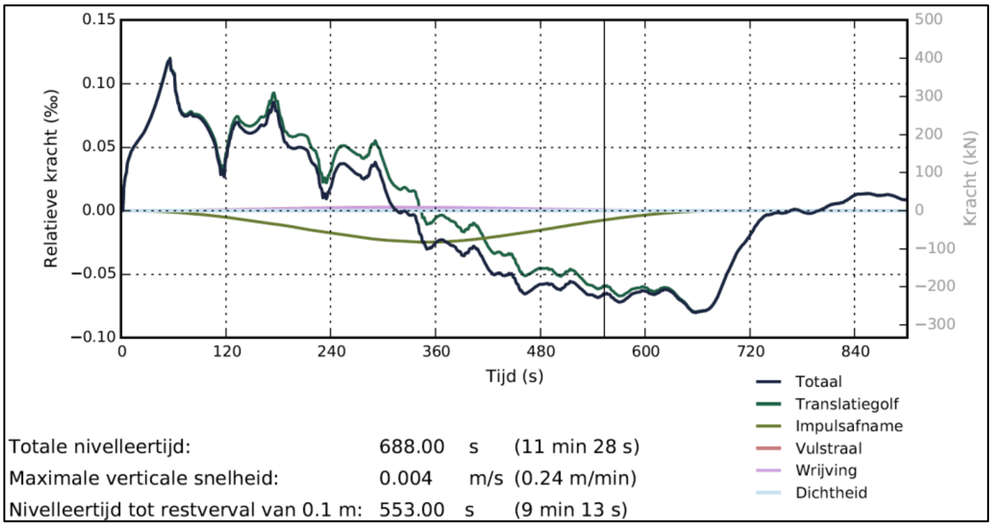



*Figure 106 Malaccamax, adaptive design*

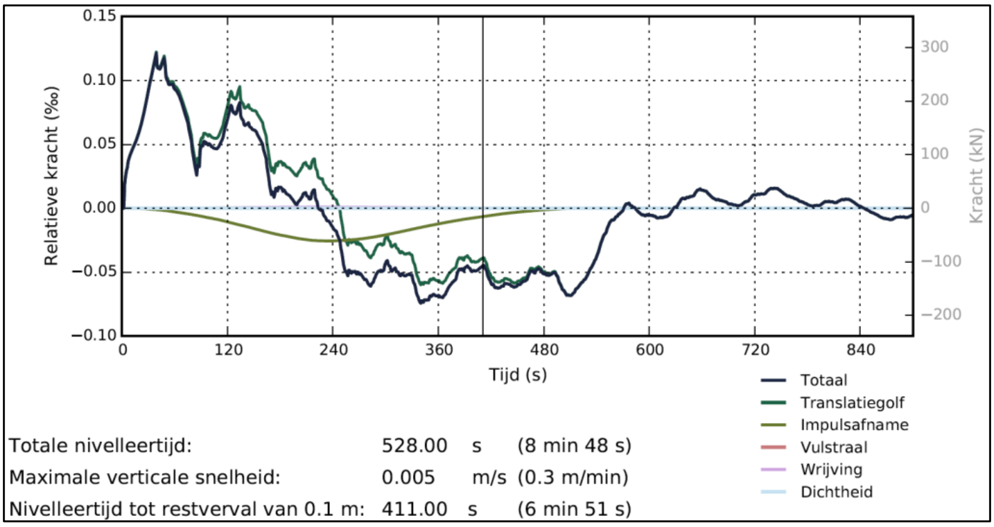



*Figure 107 New Panamax, adaptive design*

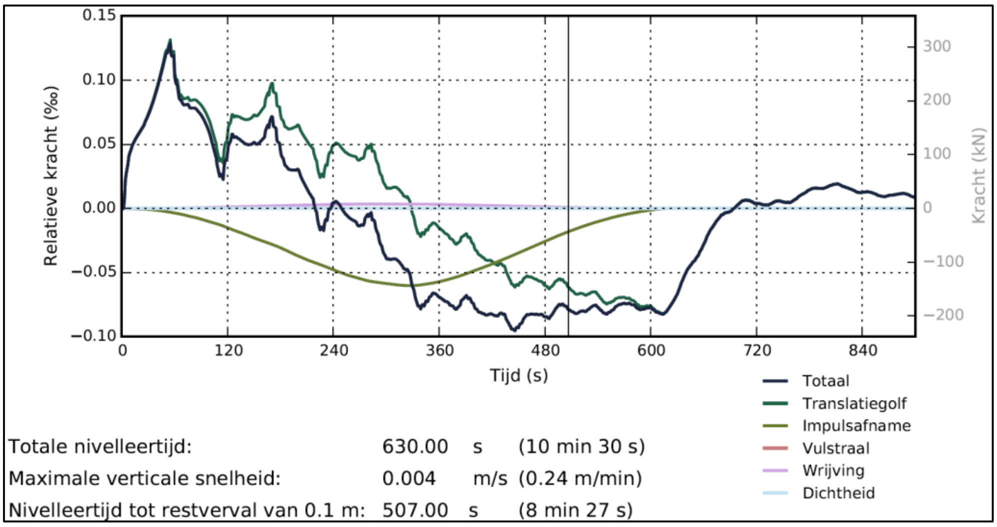



*Figure 108 New Panamax, initial design*

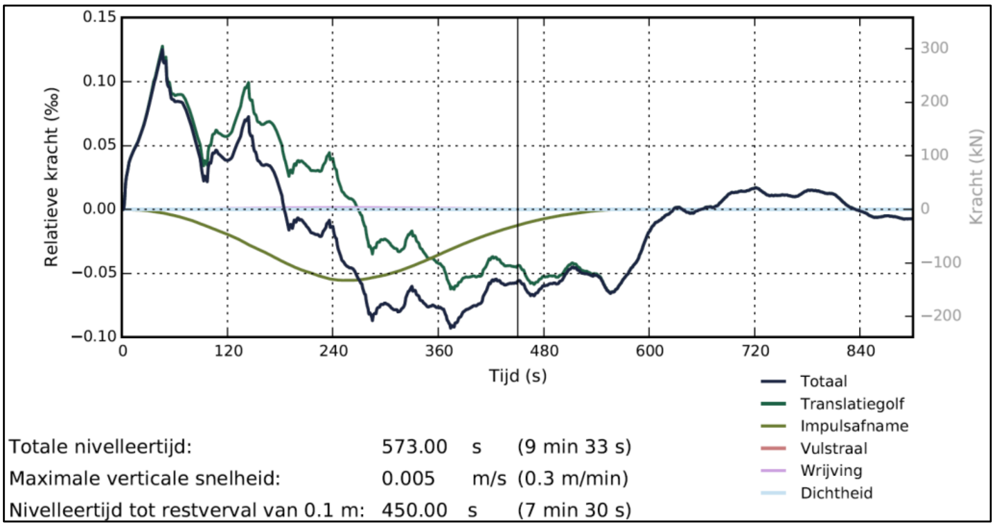



*Figure 109 New Panamax, wider design*

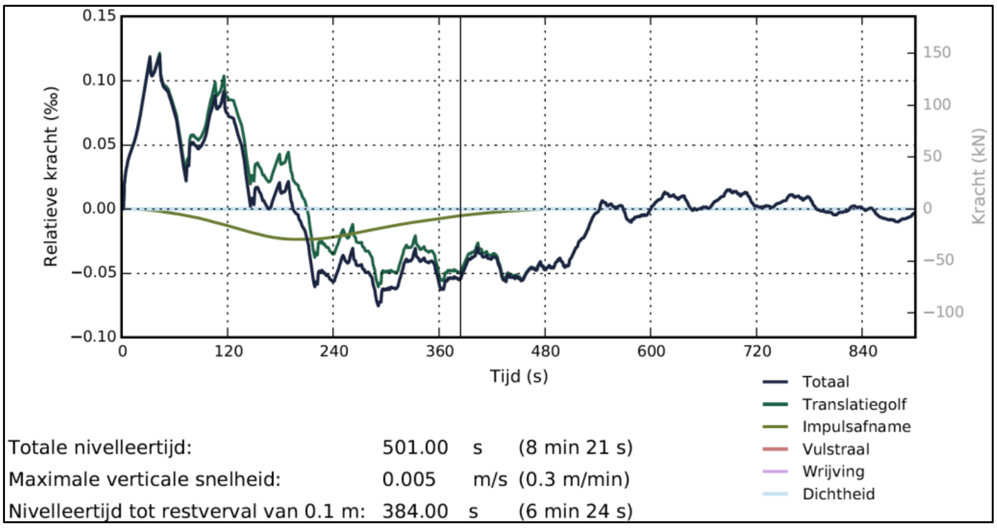



*Figure 110 Oasis of the seas, adaptive design*

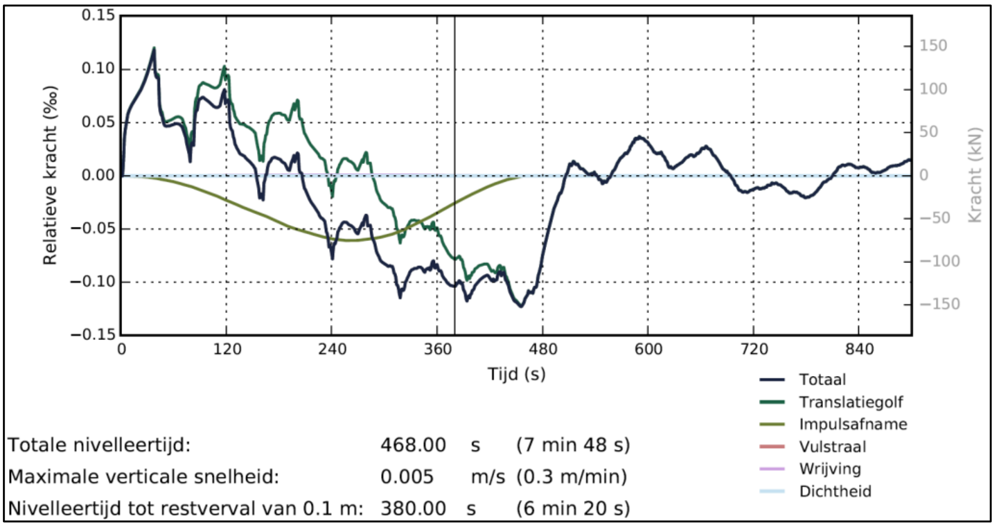



*Figure 111 Oasis of the seas, initial design*

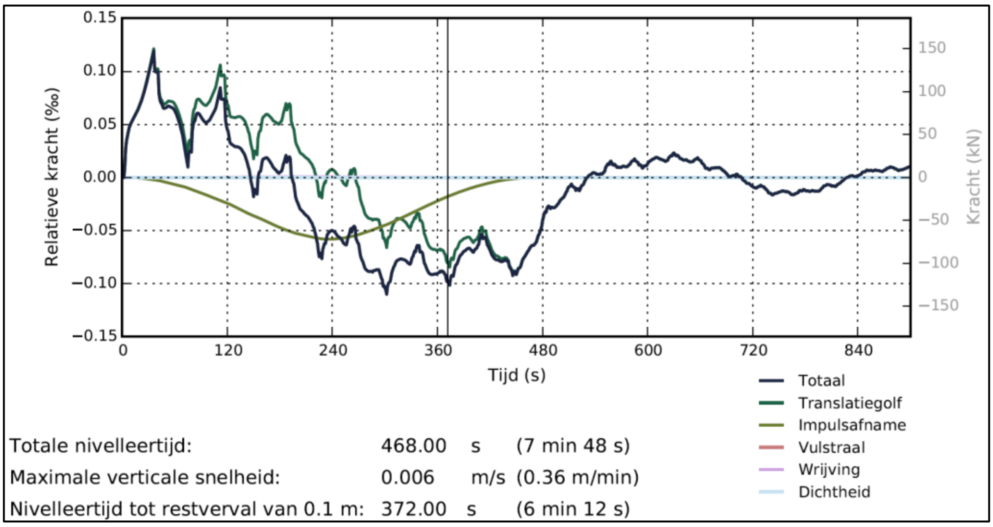



*Figure 112 Oasis of the seas, wider design*

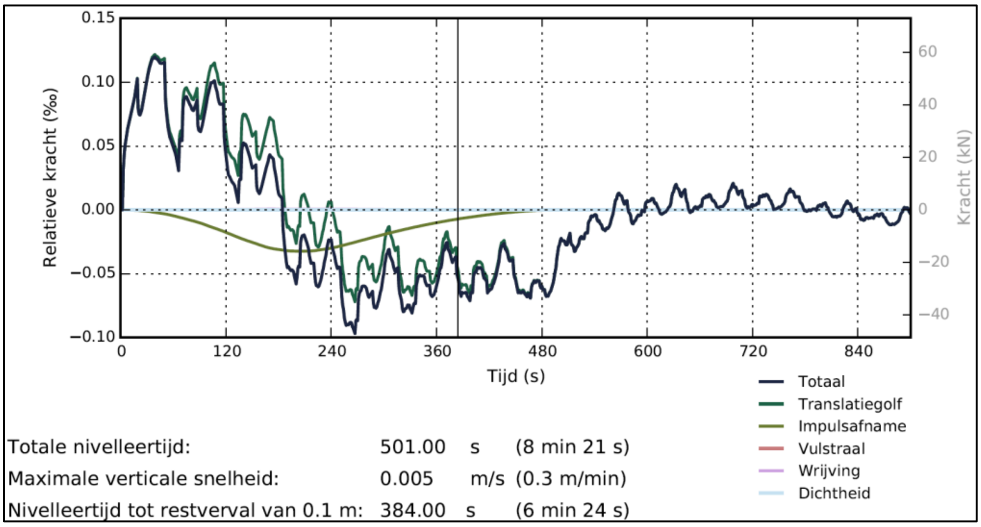



*Figure 113 Small vessel, adaptive design*

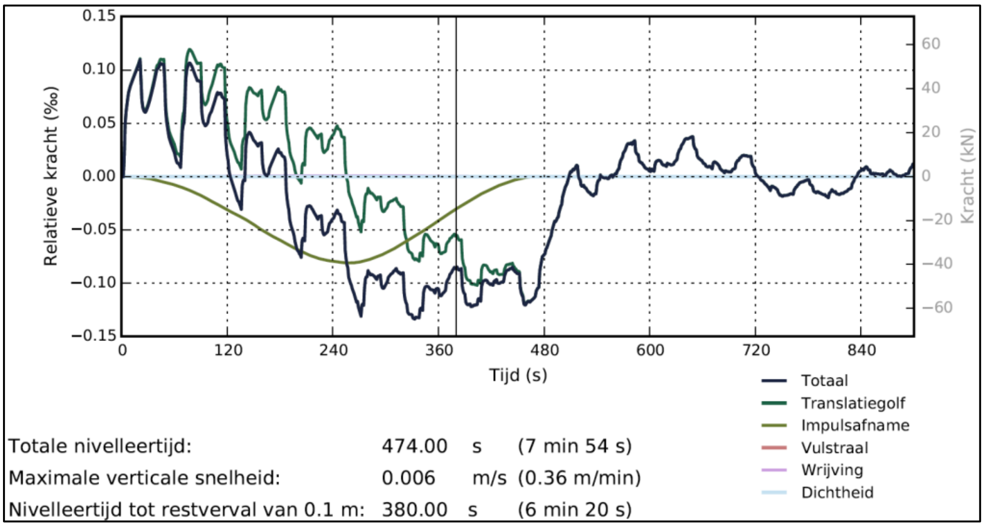



*Figure 114 Small vessel, initial design*

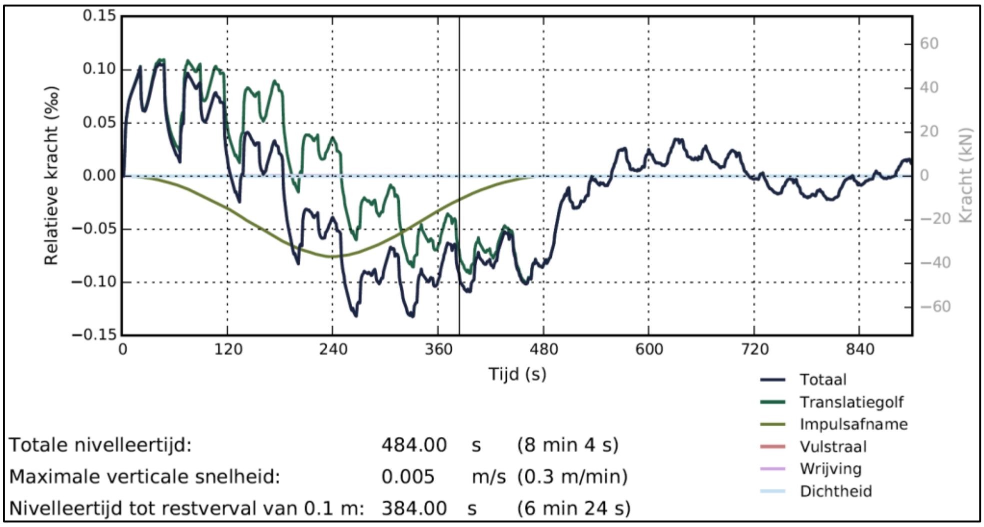



*Figure 115 Small vessel, wider design*

## *Other water levels*

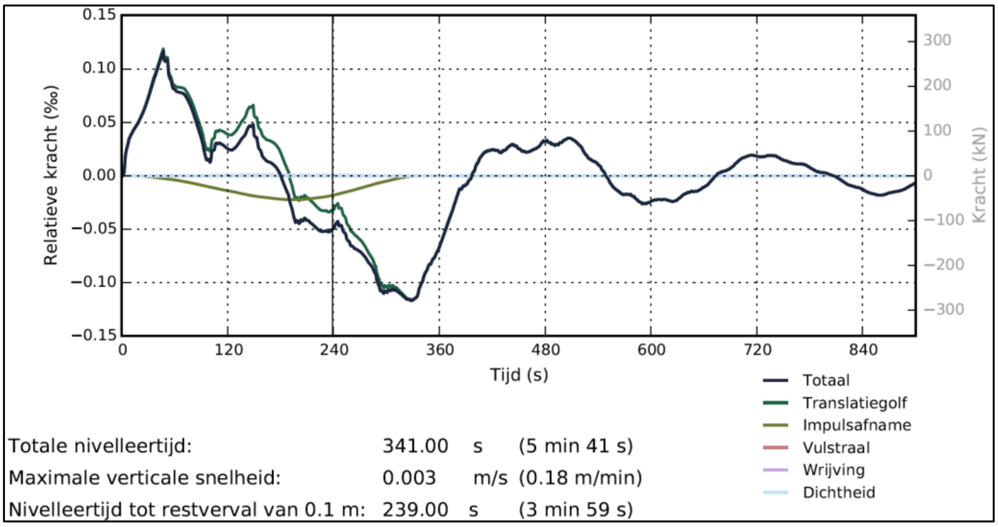



*Figure 116 New Panamax, wider design, approach harbour: 0 m + NAP, initial lock level: -0,55 m + NAP*

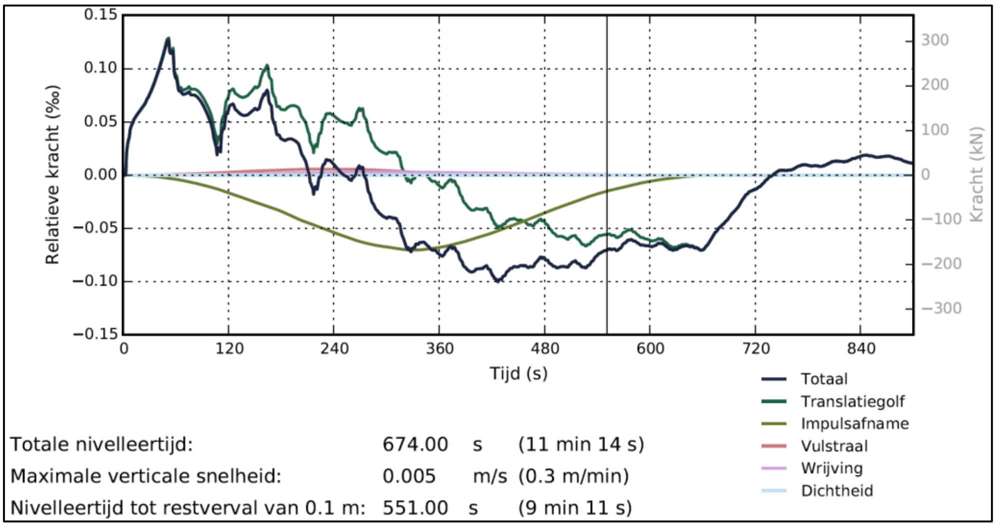



*Figure 117 New Panamax, wider design, approach harbour: 0 m + NAP, initial lock level: -1,85 m + NAP*

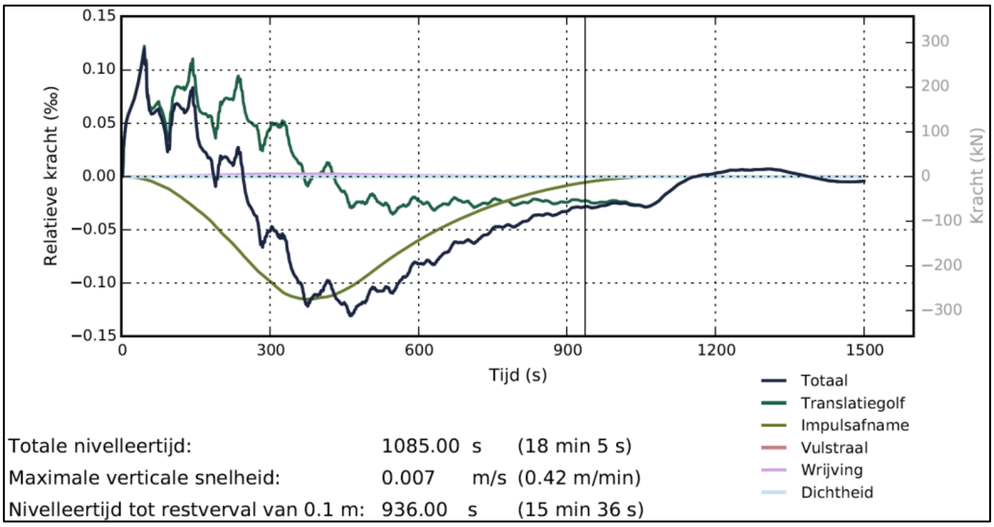



*Figure 118 New Panamax, wider design, approach harbour: 4,3 m + NAP, initial lock level: 0 m + NAP*

# C3. Longitudinal culvert

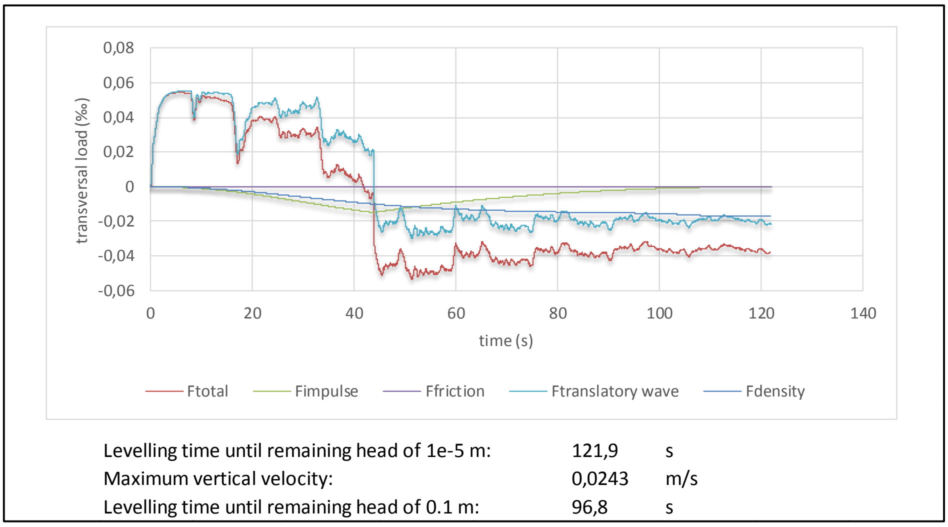
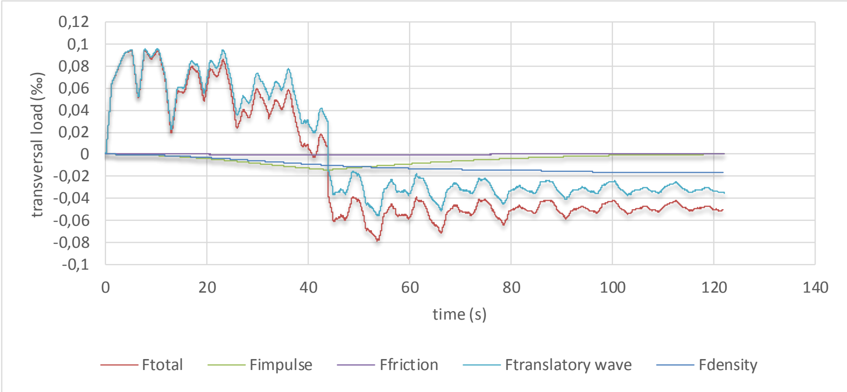
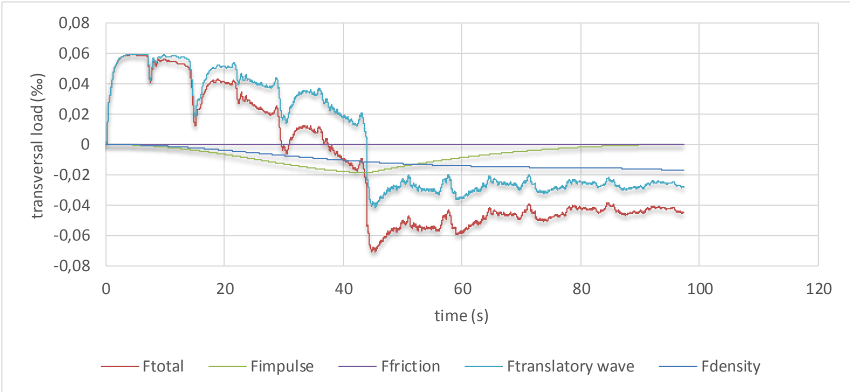



*Figure 119 Malaccamax, adaptive design*



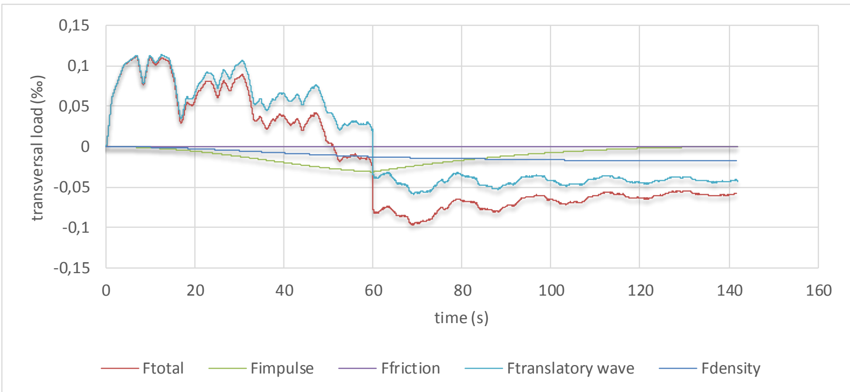

| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 121,9 | s |
| Maximum vertical velocity: | 0,0243 | m/s |
| Levelling time until remaining head of 0.1 m: | 96,8 | s |

*Figure 120 New Panamax, adaptive design*



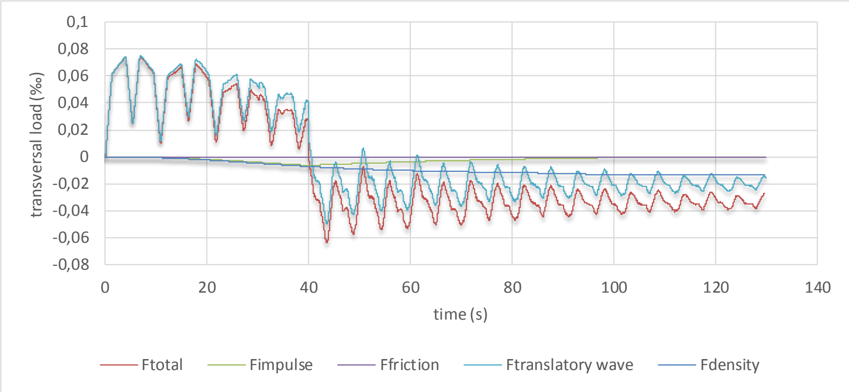

| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 97,4 | s |
| Maximum vertical velocity: | 0,0294 | m/s |
| Levelling time until remaining head of 0.1 m: | 78,5 | s |

*Figure 121 New Panamax, initial design*



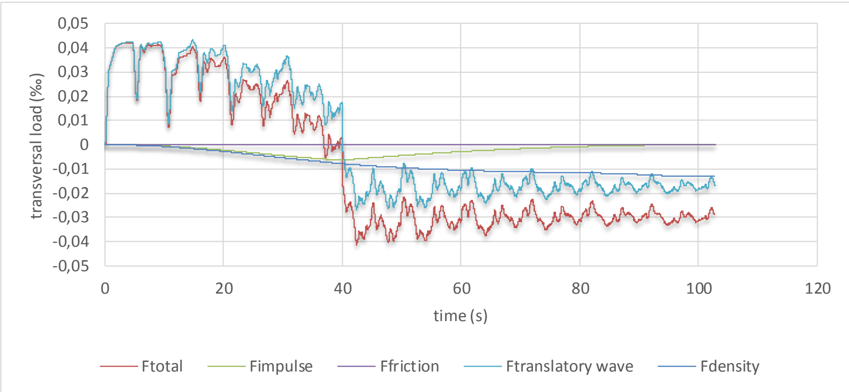

| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 141,7 | s |
| Maximum vertical velocity: | 0,0205 | m/s |
| Levelling time until remaining head of 0.1 m: | 113,7 | s |

*Figure 122 New Panamax, wider design*



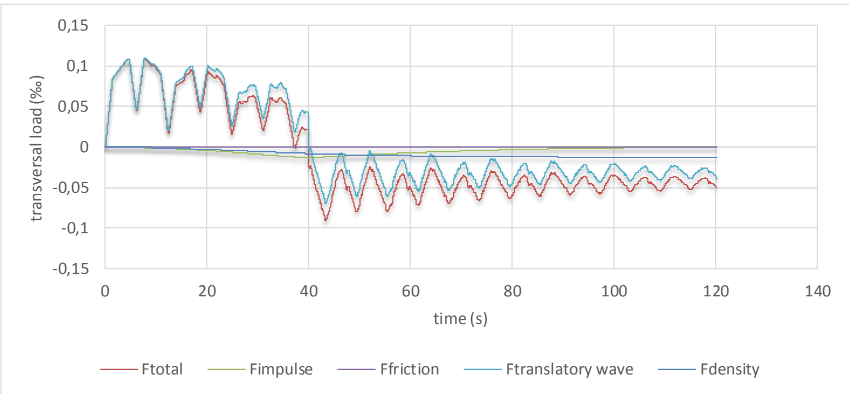

Levelling time until remaining head of 1e-5 m: 129,7 s
Maximum vertical velocity: 0,0231 m/s
Levelling time until remaining head of 0.1 m: 102,1 s

*Figure 123 Oasis of the seas, adaptive design*



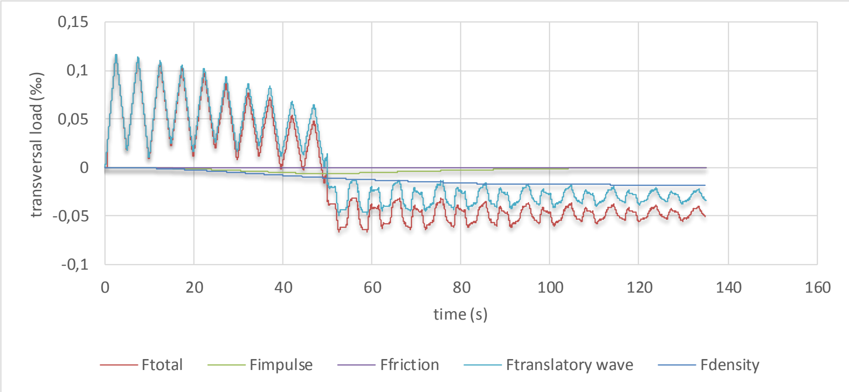

Levelling time until remaining head of 1e-5 m: 102,7 s
Maximum vertical velocity: 0,0286 m/s
Levelling time until remaining head of 0.1 m: 82,0 s

*Figure 124 Oasis of the seas, initial design*



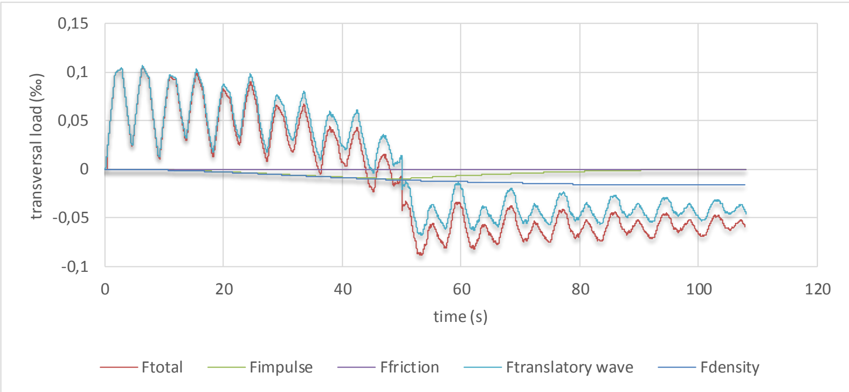

Levelling time until remaining head of 1e-5 m: 120,3 s
Maximum vertical velocity: 0,0248 m/s
Levelling time until remaining head of 0.1 m: 95,0 s

*Figure 125 Oasis of the seas, wider design*



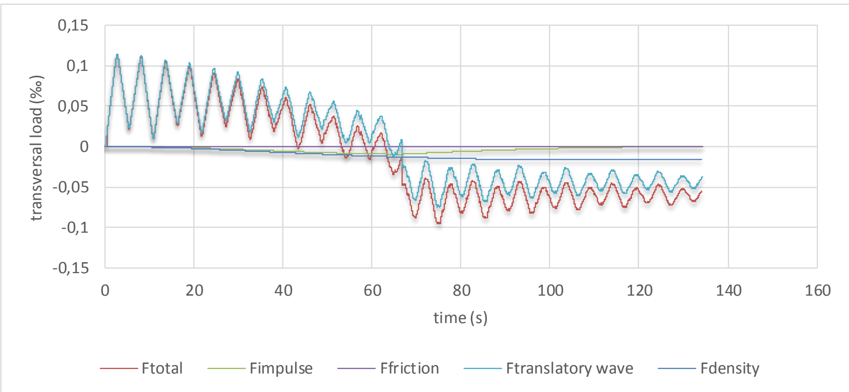

| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 134,9 | s |
| Maximum vertical velocity: | 0,0219 | m/s |
| Levelling time until remaining head of 0.1 m: | 107,3 | s |

*Figure 126 Small vessel, adaptive design*




| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 107,9 | s |
| Maximum vertical velocity: | 0,0264 | m/s |
| Levelling time until remaining head of 0.1 m: | 87,2 | s |

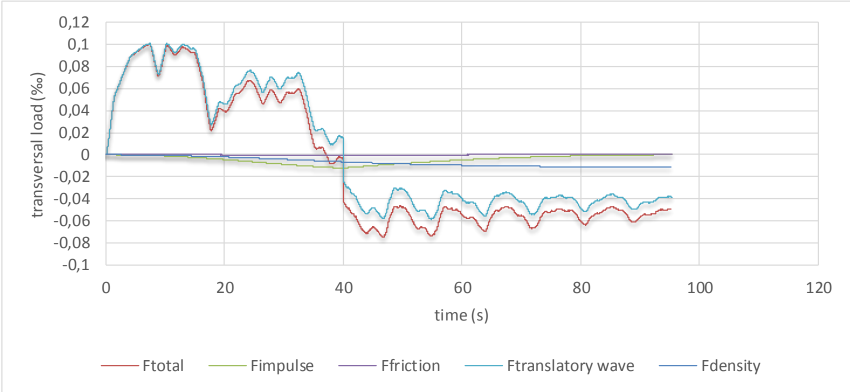
*Figure 127 Small vessel, initial design*




| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 134,1 | s |
| Maximum vertical velocity: | 0,0209 | m/s |
| Levelling time until remaining head of 0.1 m: | 108,9 | s |

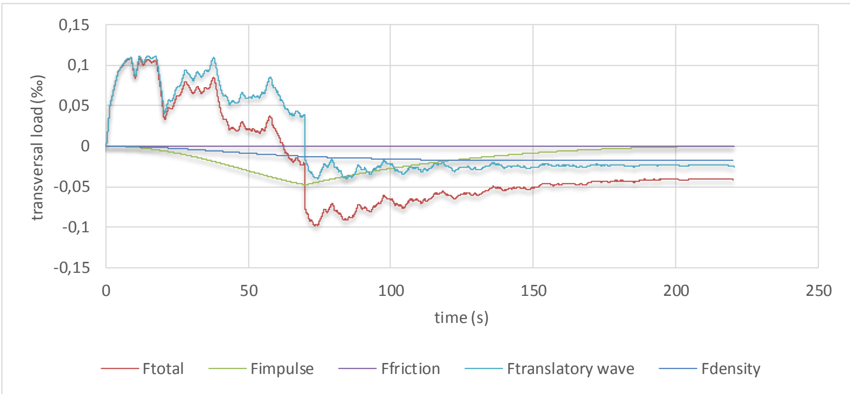
*Figure 128 Small vessel, wider design*

## ***Other water levels***



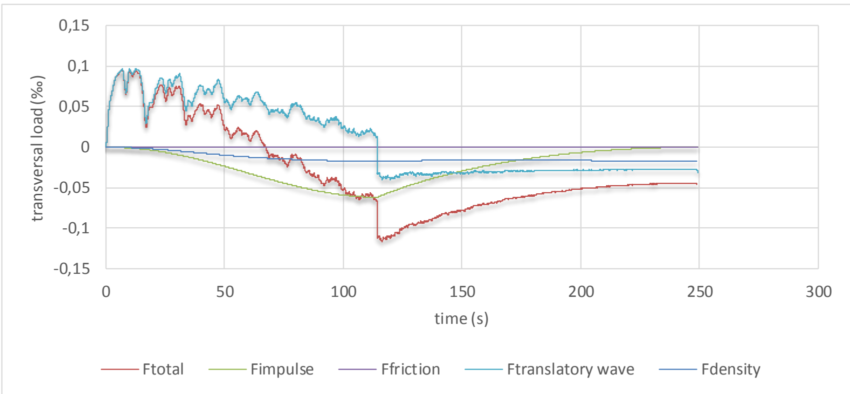

| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 95,2 | s |
| Maximum vertical velocity: | 0,0109 | m/s |
| Levelling time until remaining head of 0.1 m: | 63,7 | s |

*Figure 129 New Panamax, wider design, approach harbour: 0 m + NAP, initial lock level: -0,55 m + NAP*




| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 220,3 | s |
| Maximum vertical velocity: | 0,0164 | m/s |
| Levelling time until remaining head of 0.1 m: | 177,9 | s |

*Figure 130 New Panamax, wider design, approach harbour: 0 m + NAP, initial lock level: -1,85 m + NAP*




| | | |
|---|---|---|
| Levelling time until remaining head of 1e-5 m: | 249,1 | s |
| Maximum vertical velocity: | 0,0322 | m/s |
| Levelling time until remaining head of 0.1 m: | 220,5 | s |

*Figure 131 New Panamax, wider design, approach harbour: 4,3 m + NAP, initial lock level: 0 m + NAP*

# D. Structural design

## D1. Results Scia Engineer

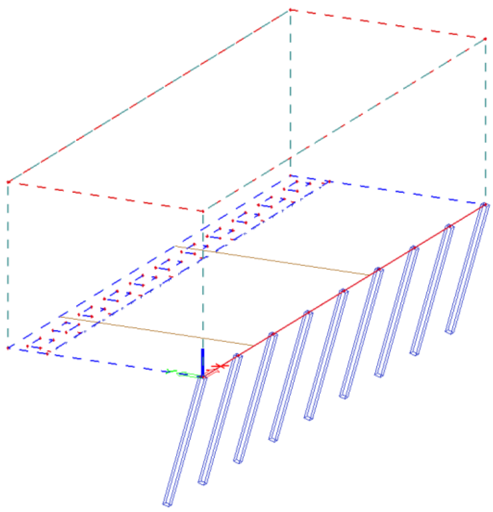


*Figure 132 Scia model overview with the two cross sections on the floor*

### ***Bending moments Y-direction***

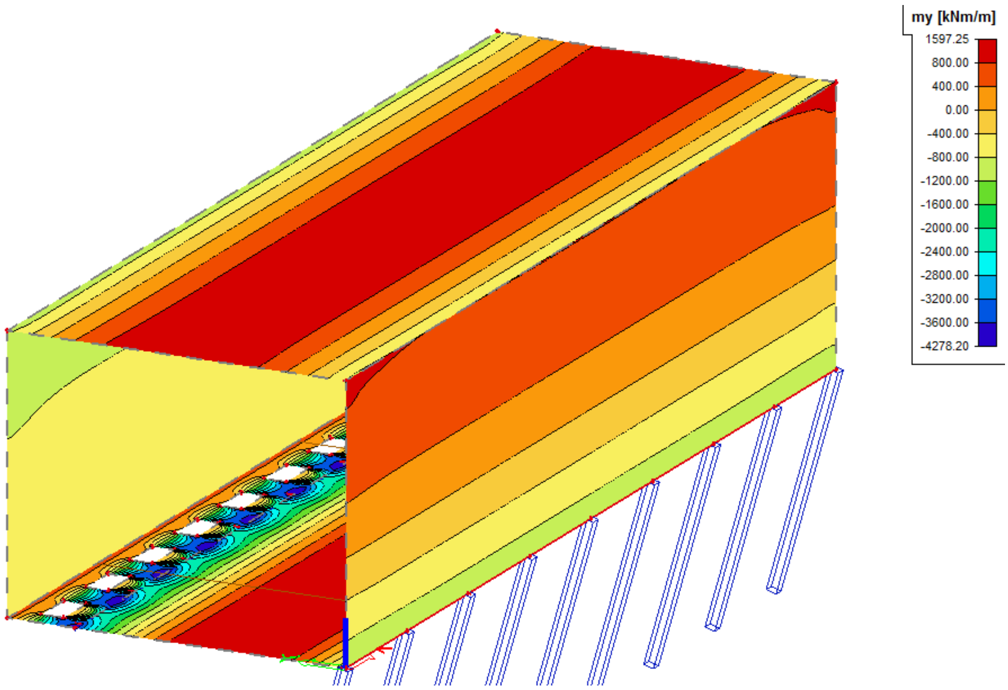



*Figure 133 Bending moments y-direction, load situation: High water*

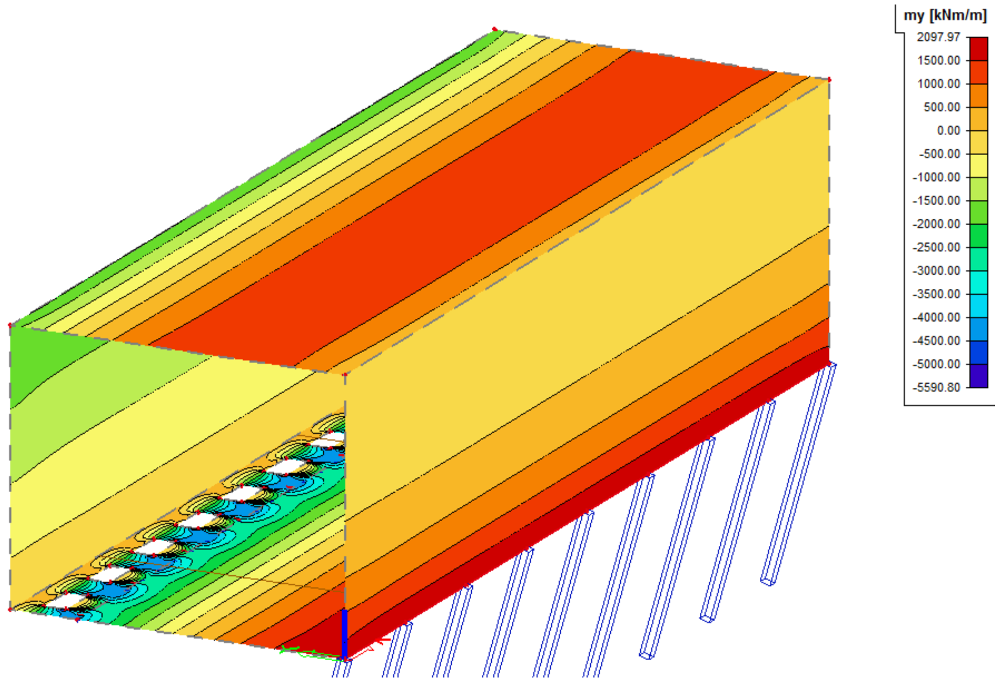



*Figure 134 Bending moments y-direction, load situation: Vessel*

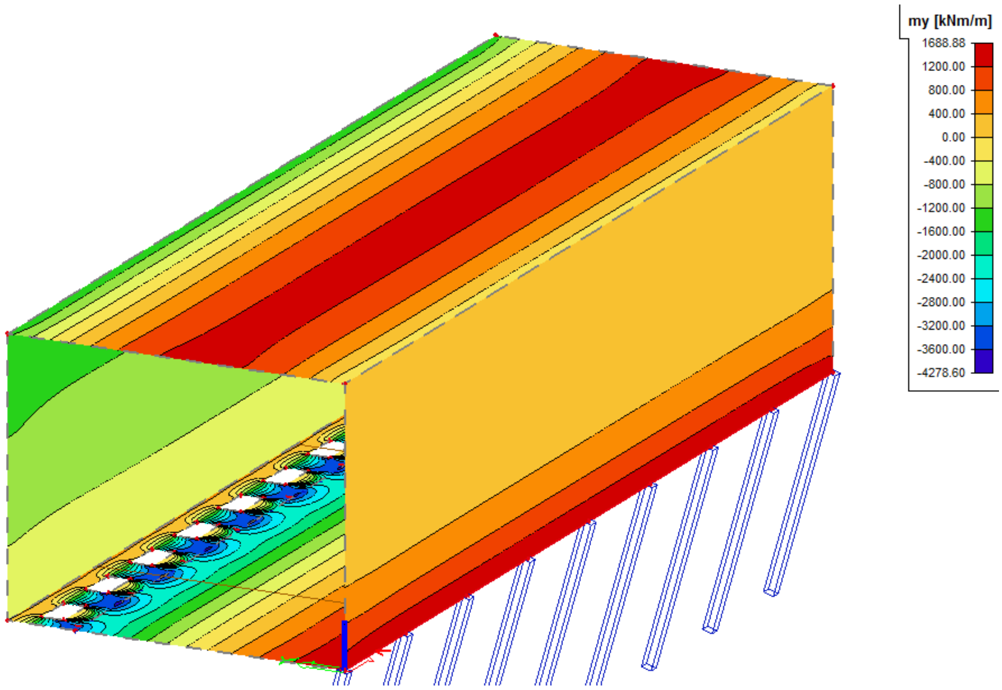



*Figure 135 Bending moments y-direction, load situation: Maintenance*

***Shear forces Y-direction***

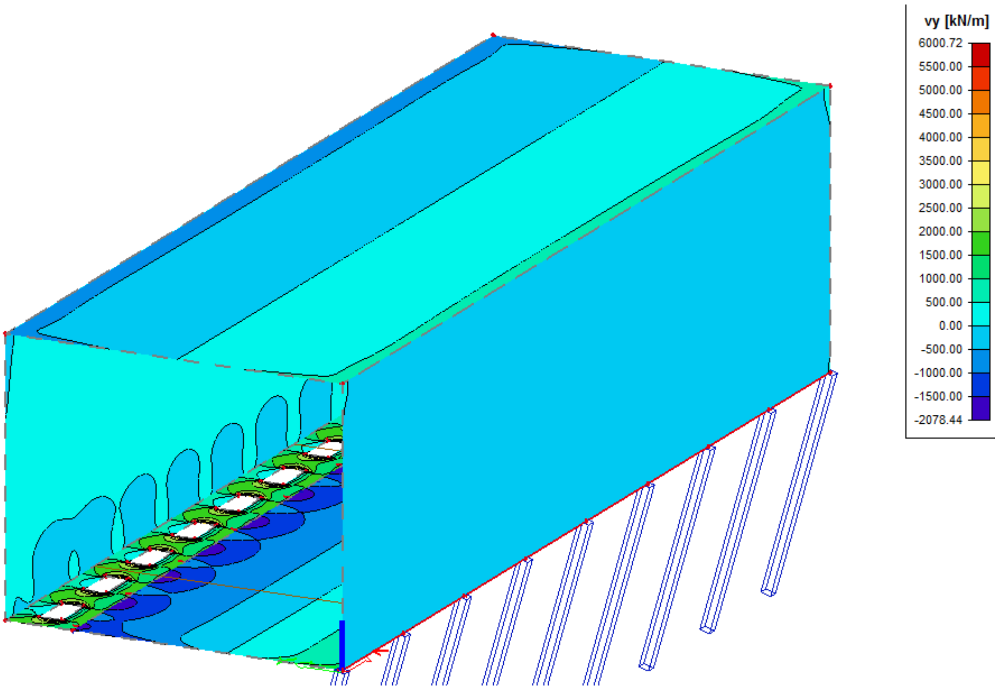



*Figure 136 Shear force y-direction, load situation: High water*

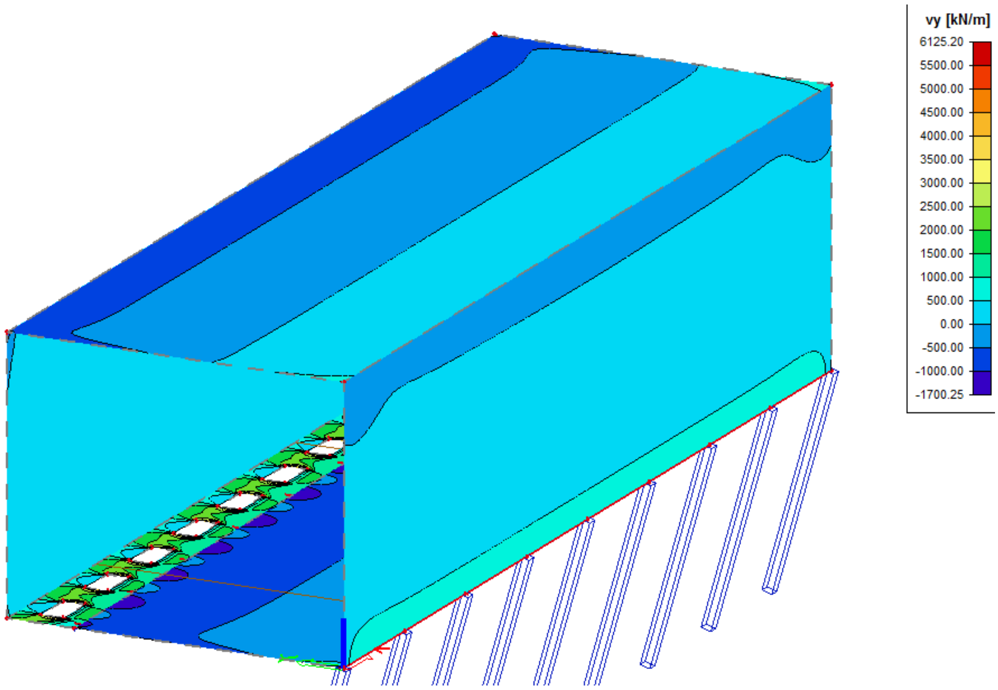



*Figure 137 Shear force y-direction, load situation: Vessel*

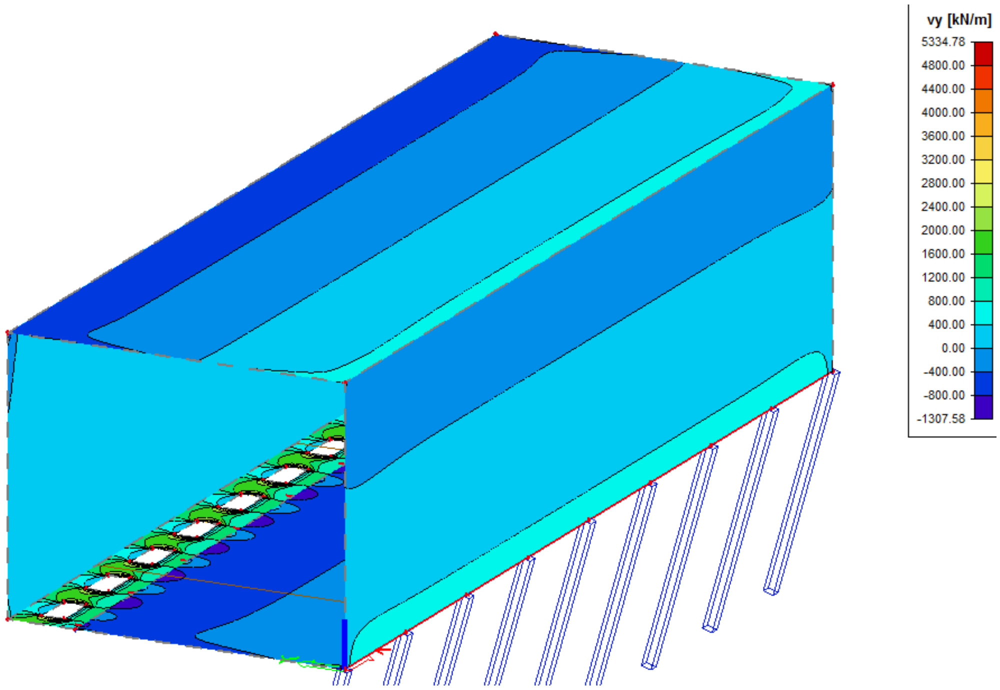



*Figure 138 Shear force y-direction, load situation: Maintenance*

## *Governing cross sections*

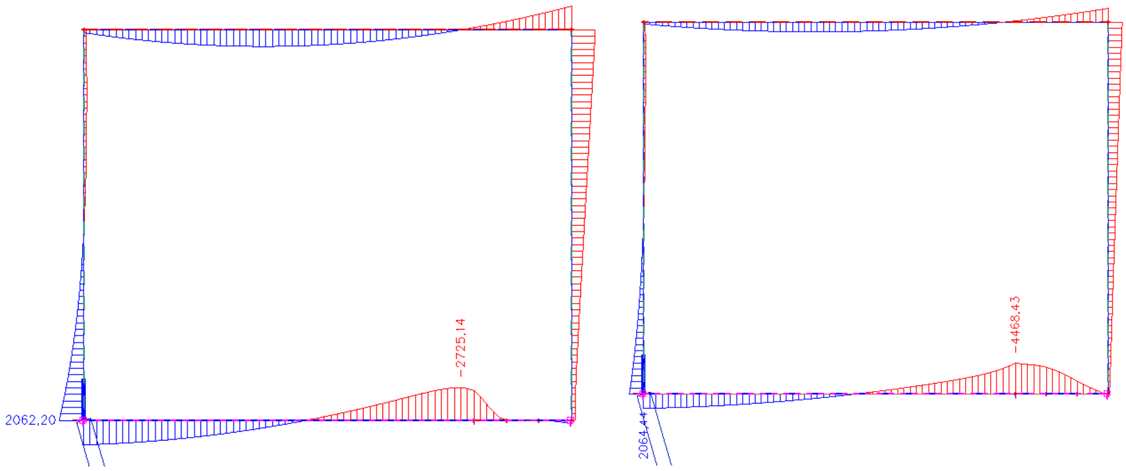



*Figure 139 Moment diagram, load situation: Vessel, left: cross section through the opening, right: cross section between openings*

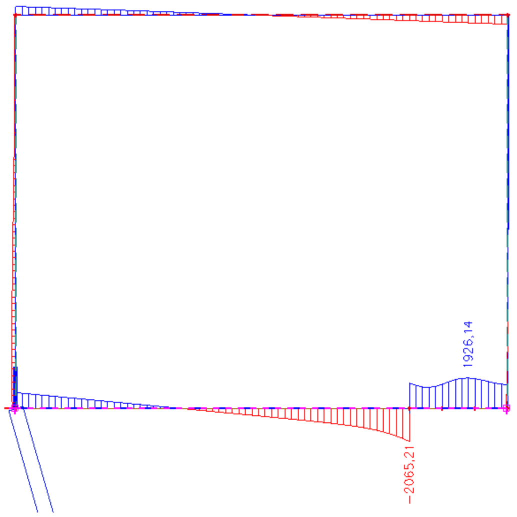



*Figure 140 Shear force diagram, Load situation: High water, cross section between openings*

## D2. Results D-sheet piling

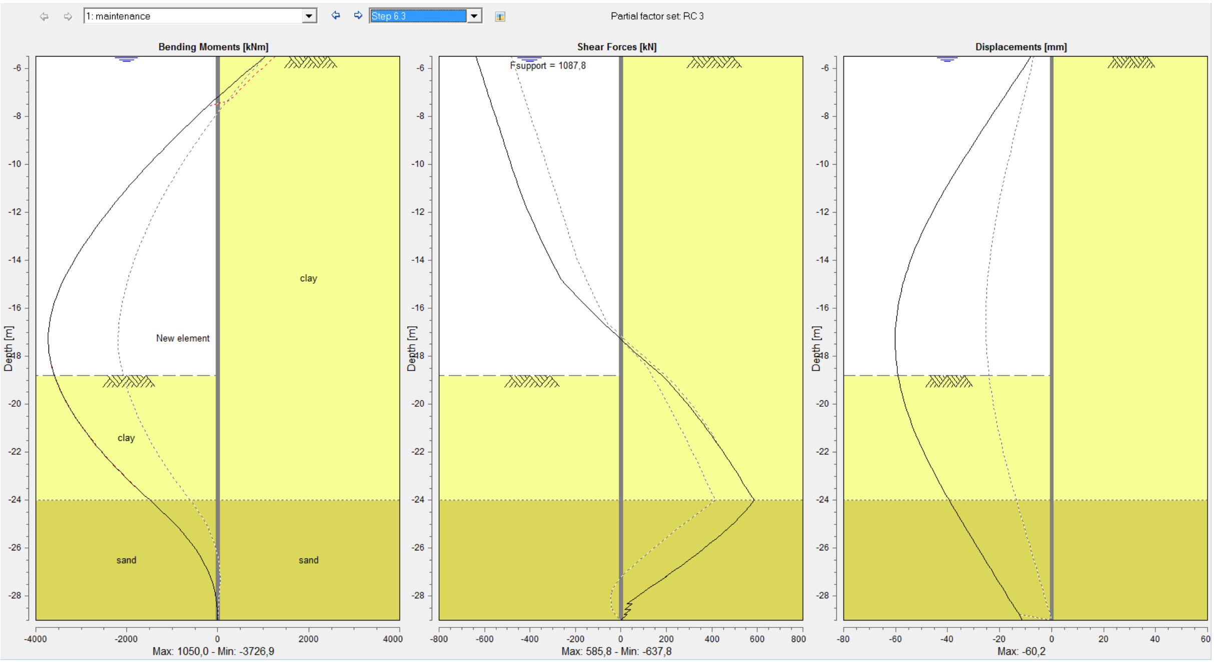



*Figure 141 Maintenance, step 6.3*

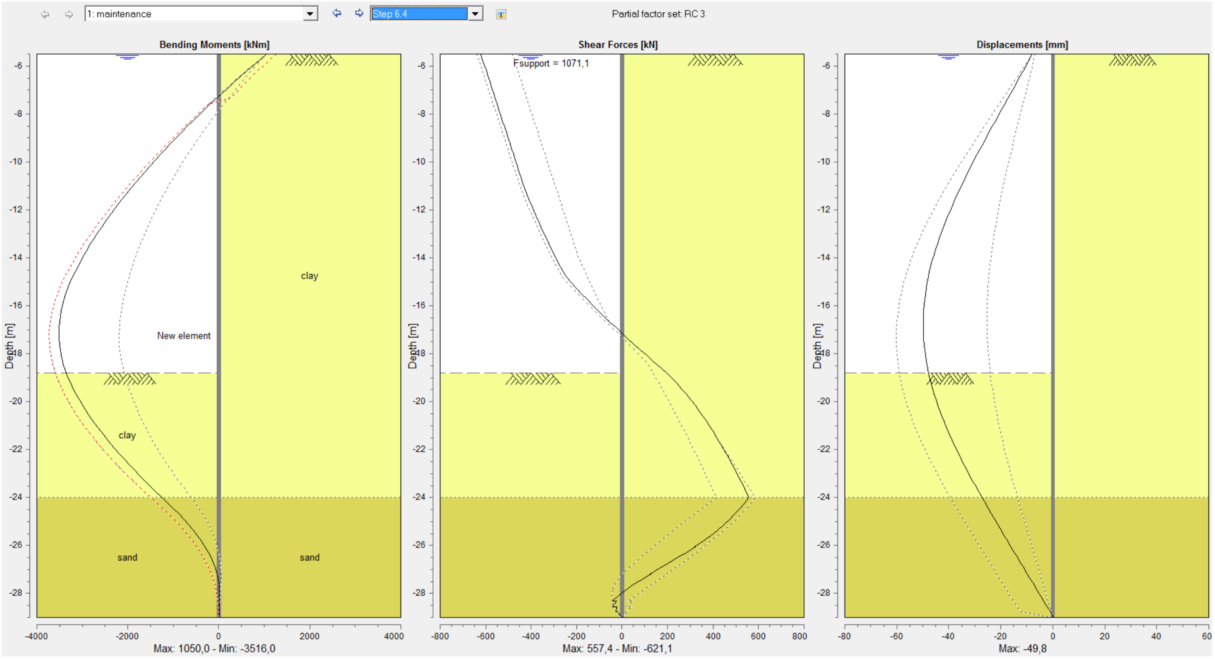



*Figure 142 Maintenance, step 6.4*

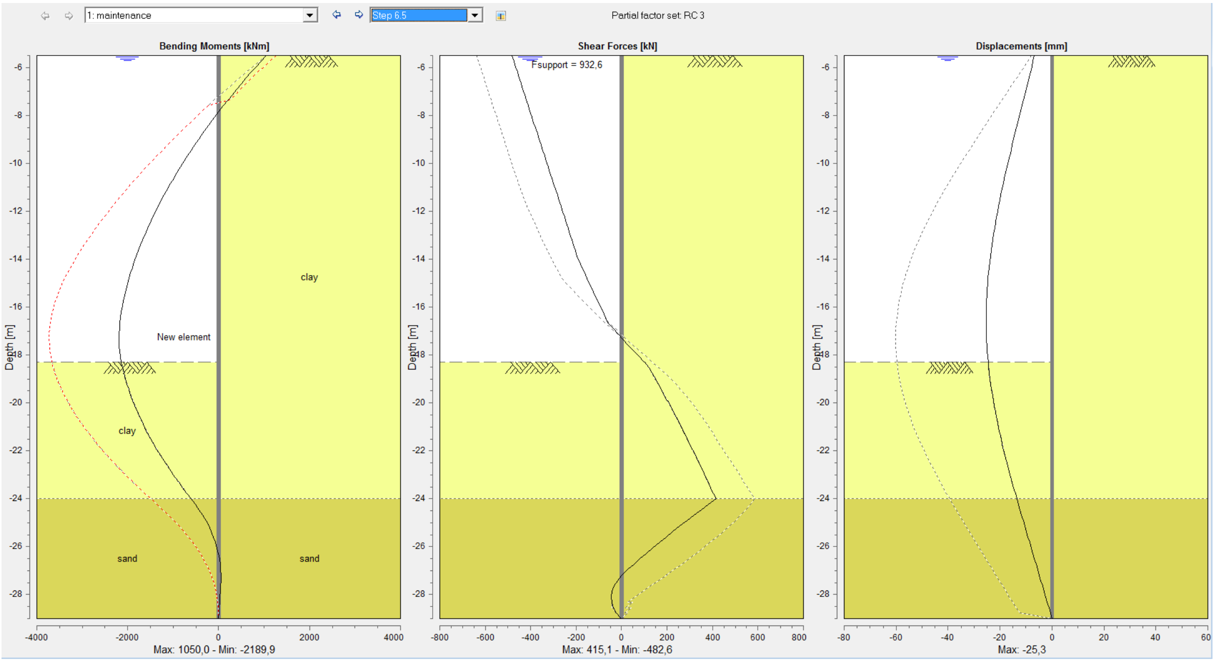



*Figure 143 Maintenance, step 6.5*

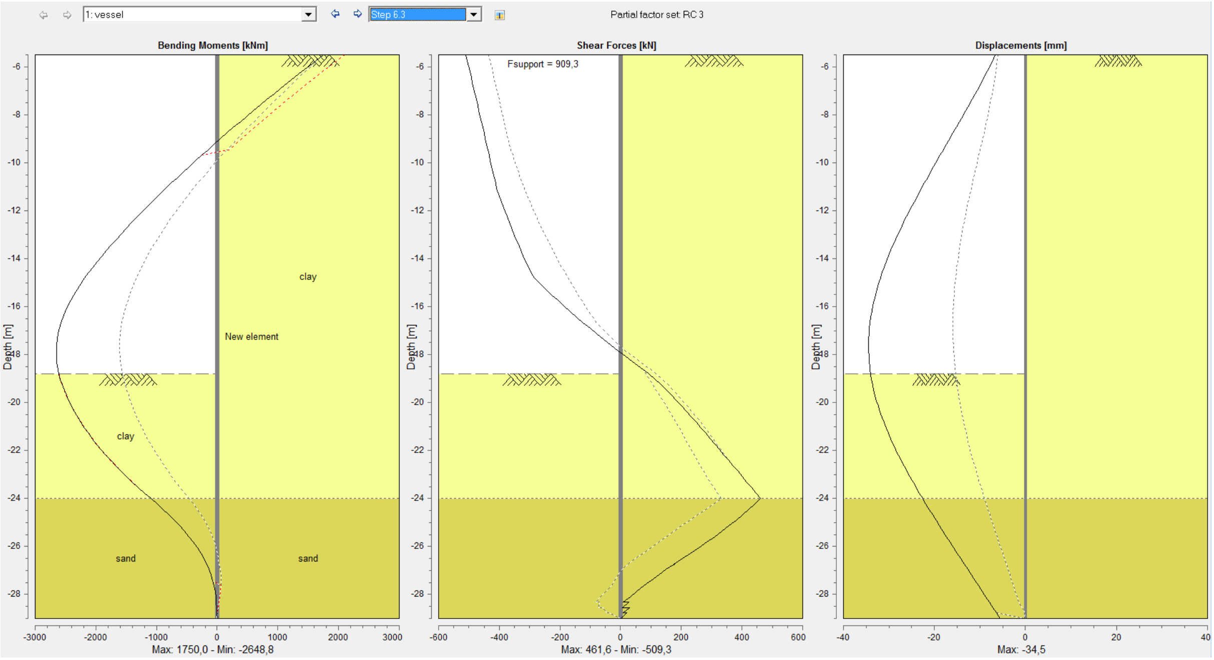



Figure 144 Vessel, step 6.3

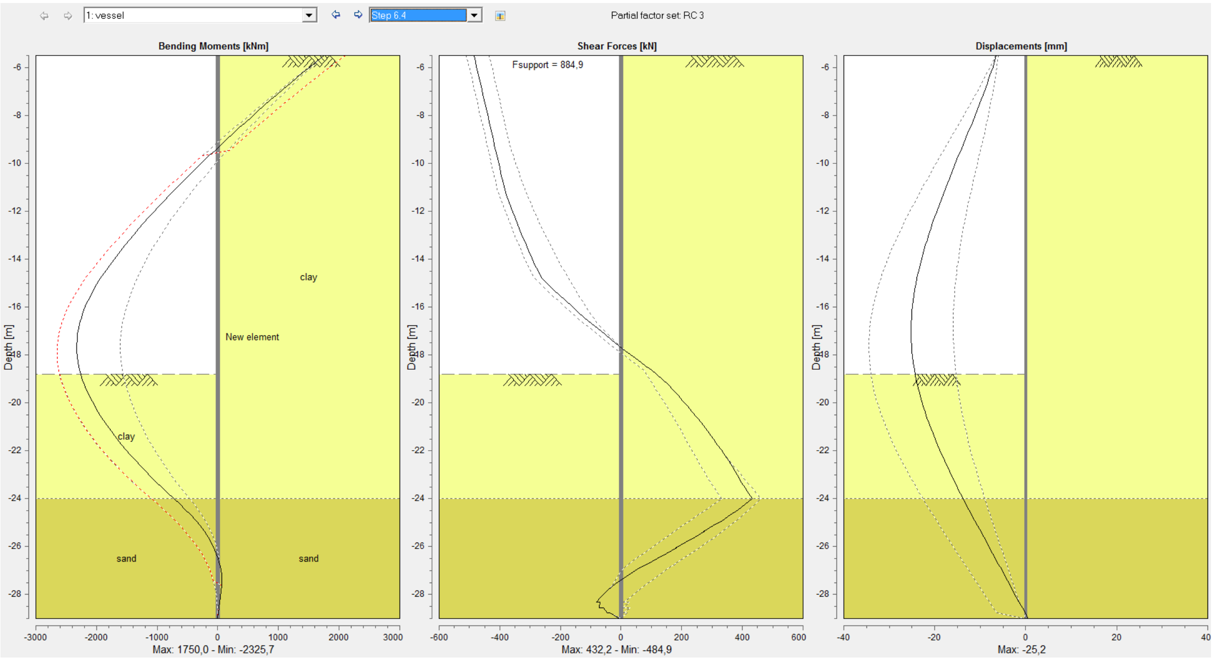



Figure 145 Vessel, step 6.4

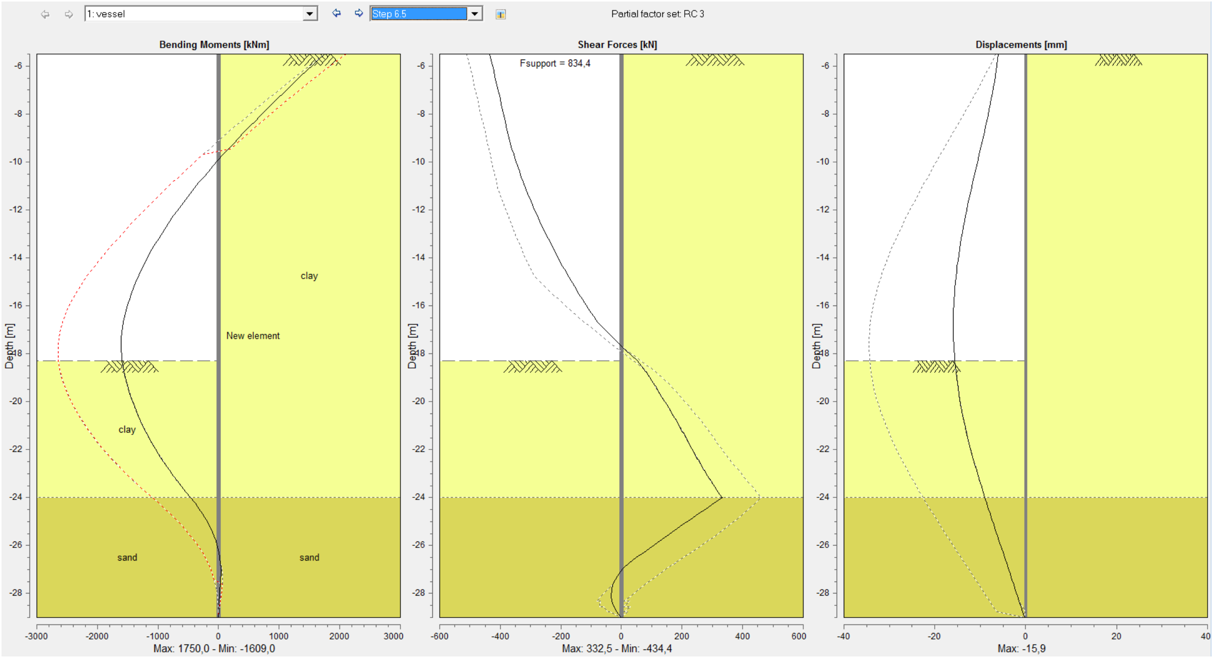



*Figure 146 Vessel, step 6.5*

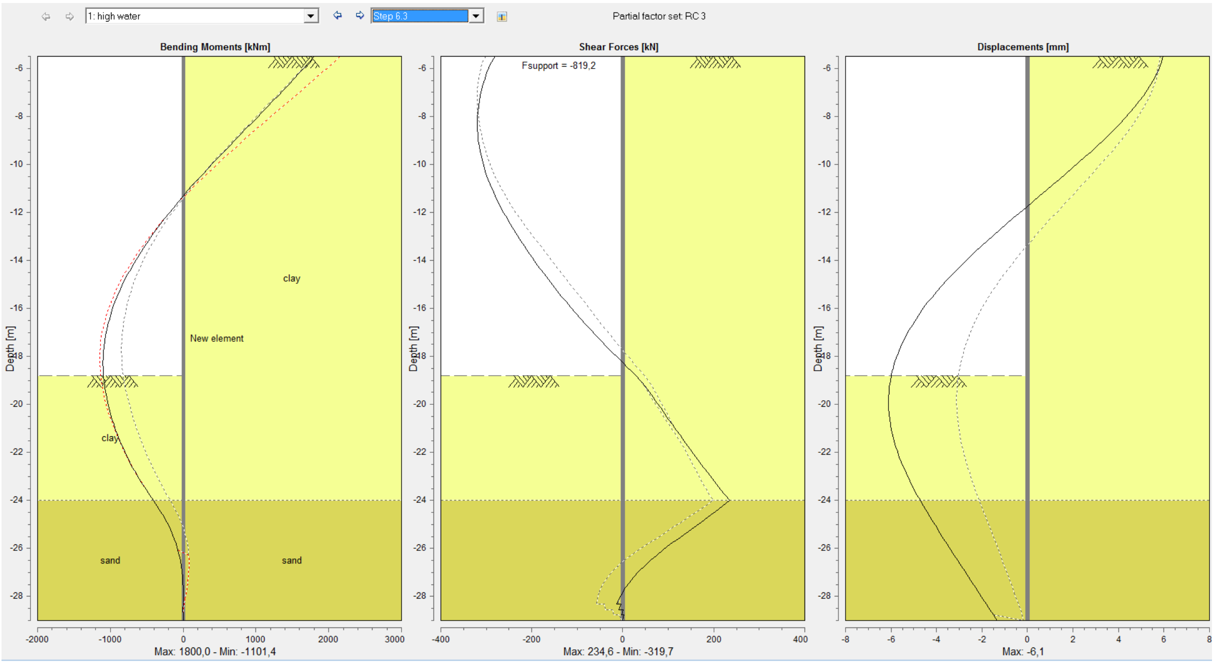



*Figure 147 High water, step 6.3*

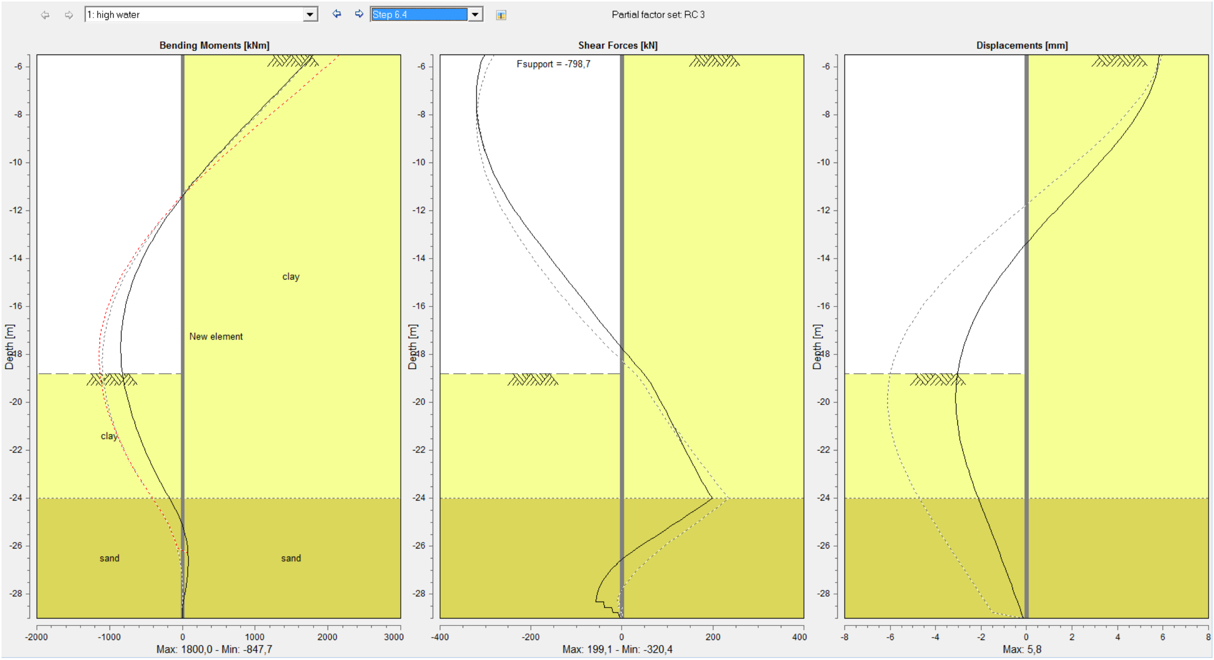



*Figure 148 High water, step 6.4*

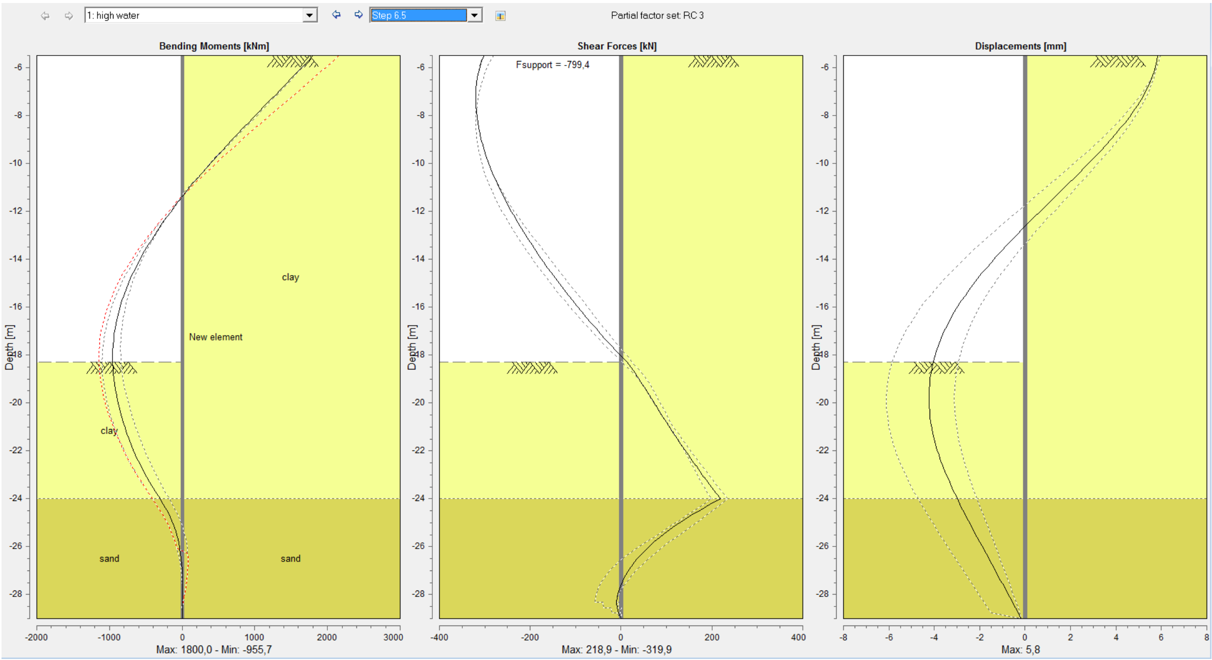



*Figure 149 High water, step 6.5*

## D3. Calculation checks (Mathcad)

# Active and passive ground pressure coefficients

**sand**

$$\varphi := \frac{35\cdot 2\cdot \pi}{360} = 0.611\cdot\text{rad}$$

$$\delta := 20\cdot 2\cdot\frac{\pi}{360} = 0.349\cdot\text{rad}$$

$$k0 := 1 - \sin(\varphi) = 0.426$$

$$kasand := \frac{\cos(\varphi)^2}{\left(1 + \sqrt{\frac{\sin(\varphi)\sin(\varphi+\delta)}{\cos(\delta)}}\right)^2} = 0.23$$

$$kpsand := \frac{\cos(\varphi)^2}{\left(1 - \sqrt{\frac{\sin(\varphi)\sin(\varphi+\delta)}{\cos(\delta)}}\right)^2} = 7.822$$

**clay**

$$\varphi := \frac{23\cdot 2\cdot \pi}{360} = 0.401\cdot\text{rad}$$

$$\delta := 13\cdot 2\cdot\frac{\pi}{360} = 0.227\cdot\text{rad}$$

$$k0 := 1 - \sin(\varphi) = 0.609$$

$$kaclay := \frac{\cos(\varphi)^2}{\left(1 + \sqrt{\frac{\sin(\varphi)\sin(\varphi+\delta)}{\cos(\delta)}}\right)^2} = 0.384$$

$$kpclay := \frac{\cos(\varphi)^2}{\left(1 - \sqrt{\frac{\sin(\varphi)\sin(\varphi+\delta)}{\cos(\delta)}}\right)^2} = 3.201$$

# Loads

### Extreme high water super structure

$hstructure := 6.35m$

$hbottom := -5.5m$

$hwlock := 6.25m$

$hwground := 0m$

$wstructure := 12m$

$\rho w := 1025 \frac{kg}{m^3}$

$\rho clay := 1700 \frac{kg}{m^3}$

$tfloor := 1m$

$Pwater := \rho w \cdot g \cdot (hwlock - hbottom) = 118.109 \frac{1}{m^2} \cdot kN$

$Pinside := \rho w \cdot g \cdot (hwlock - hbottom - tfloor) = 108.057 \frac{1}{m^2} \cdot kN$

$Pclay := (\rho clay) \cdot g \cdot (hstructure - hbottom) \cdot kaclay = 75.857 \frac{1}{m^2} \cdot kN$

$Pwground := \rho w \cdot g \cdot (hwground - hbottom) = 55.285 \frac{1}{m^2} \cdot kN$

### Maintenance super structure

$hstructure := 6.35m$

$hbottom := -5.5m$

$hwlock := -5.5m$

$hwground := -3m$

$wstructure := 12m$

$\rho w := 1025 \frac{kg}{m^3}$

$\rho clay := 1700 \frac{kg}{m^3}$

$tfloor := 1m$

$Pwater := \rho w \cdot g \cdot (hwlock - hbottom) = 0 \frac{1}{m^2} \cdot kN$

$Pinside := \rho w \cdot g \cdot (hwlock - hbottom - tfloor) = -10.052 \frac{1}{m^2} \cdot kN$

$Pclay := (\rho clay) \cdot g \cdot (hstructure - hbottom) \cdot kaclay = 75.857 \frac{1}{m^2} \cdot kN$

$Pwground := \rho w \cdot g \cdot (hwground - hbottom) = 25.13 \frac{1}{m^2} \cdot kN$

### Vessel super structure

$hstructure := 6.35m$

$hbottom := -5.5m$

$hwlock := -1.85m$

$hwground := 0m$

$wstructure := 12m$

$\rho w := 1025 \frac{kg}{m^3}$

$\rho clay := 1700 \frac{kg}{m^3}$

$tfloor := 1m$

$Pwater := \rho w \cdot g \cdot (hwlock - hbottom) = 36.689 \frac{1}{m^2} \cdot kN$

$Pinside := \rho w \cdot g \cdot (hwlock - hbottom - tfloor) = 26.637 \frac{1}{m^2} \cdot kN$

$Pclay := (\rho clay) \cdot g \cdot (hstructure - hbottom) \cdot kaclay = 75.857 \frac{1}{m^2} \cdot kN$

$Pwground := \rho w \cdot g \cdot (hwground - hbottom) = 55.285 \frac{1}{m^2} \cdot kN$

# Material parameters

concrete C30/37

$$fck := 30\frac{N}{mm^2}$$

$$fcd := 20\frac{N}{mm^2}$$

$$fctm := 2.9\frac{N}{mm^2}$$

$$fctd := 1.35\frac{N}{mm^2}$$

$$Ec := 33000\frac{N}{mm^2}$$

$$Eccracked := 11000\frac{N}{mm^2}$$

Reinforcement steel B500B

$$Es := 2.1\cdot 10^5\frac{N}{mm^2}$$

$$cover := 55mm$$

$$fyd := 435\frac{N}{mm^2}$$

# Input D-sheet

## M-κ diagram diaphragm wall

### Elastic bending stifness

$h := 1500mm$

$\varepsilon max := \frac{0.2}{1000} = 2 \times 10^{-4}$

$As := 10000mm^2$ one side

$Nstr := 1000000N = 1 \times 10^3 \cdot kN$

$f(x) := Nstr + Fct - Fcc + Fst - Fsc$

$$f(x) := Nstr + \frac{1}{2} \cdot fctm \cdot (h - x) \cdot 1 \cdot m - \frac{1}{2} fctm \cdot \frac{x^2}{h - x} \cdot 1m + \varepsilon max \cdot \frac{(h - x - cover)}{h - x} \cdot Es \cdot As - \varepsilon max \cdot \frac{x}{h - x} \cdot \frac{(x - cover)}{x} Es \cdot As$$

$x := 871mm$

$root(f(x), x) = 0.871m$

$Fct := \frac{1}{2} \cdot fctm \cdot (h - x) \cdot 1000mm = 9.12 \times 10^5 \cdot N$

$Fcc := \frac{1}{2} fctm \cdot \frac{x^2}{h - x} \cdot 1000mm = 1.749 \times 10^6 \cdot N$

$Fst := \varepsilon max \cdot \frac{(h - x - cover)}{h - x} \cdot Es \cdot As = 3.833 \times 10^5 \cdot N$

$Fsc := \varepsilon max \cdot \frac{x}{h - x} \cdot \frac{(x - cover)}{x} Es \cdot As = 5.449 \times 10^5 N$

$\kappa := \frac{\varepsilon max}{h - x} = 3.18 \times 10^{-4} \frac{1}{m}$

$$M := -Nstr \cdot \left[\frac{1}{2}h - (h - x)\right] + Fsc \cdot \frac{2}{3}(h - x) + Fcc \cdot \frac{2}{3} \cdot x + Fst \cdot (h - x - cover) + Fsc \cdot (x - cover) = 1.788 \times 10^3 m \cdot kN$$

$EI := \frac{M}{\kappa} = 5.622 \times 10^6 m^2 \cdot kN$ elastic bending stifness

**Bending stifness 2: steel yielding**

$$\varepsilon steel := \frac{fyd}{Es} = 2.071\times 10^{-3}$$

$$f(x) := Nstr - Fcc + Fst - Fsc$$

$$f(x) := Nstr - \varepsilon steel\cdot\frac{x}{(h - x - cover)}\cdot Eccracked\cdot\frac{1}{2}\cdot x\cdot 1000mm + fyd\cdot As - \varepsilon steel\cdot\frac{x}{h - x - cover}\cdot\frac{(x - cover)}{x}Es\cdot As$$

$$x := 513mm$$

$$root(f(x), x) = 0.513m$$

$$Fcc := \varepsilon steel\cdot\frac{x}{(h - x - cover)}\cdot Eccracked\cdot\frac{1}{2}\cdot x\cdot 1000mm = 3.217\times 10^{6}\cdot N$$

$$Fsc := \varepsilon steel\cdot\frac{x}{h - x - cover}\cdot\frac{(x - cover)}{x}Es\cdot As = 2.138\times 10^{6}N$$

$$Fst := fyd\cdot As = 4.35\times 10^{6}N$$

$$\kappa := \frac{\varepsilon steel}{h - x - cover} = 2.223\times 10^{-3}\frac{1}{m}$$

$$M := -Nstr\cdot\left[\frac{1}{2}h - (h - x)\right] + Fcc\cdot\frac{2}{3}\cdot x + Fst\cdot(h - x - cover) + Fsc\cdot(x - cover) = 6.37\times 10^{3}m\cdot kN$$

$$EI := \frac{M}{\kappa} = 2.866\times 10^{6}m^{2}\cdot kN$$ bending stifness 2

**Bending stifness 3: compression failure concrete**

$$\varepsilon\text{concrete} := \frac{1.75}{1000} = 1.75\times 10^{-3}$$

$$\varepsilon\text{concrete}\cdot\text{Eccracked} = 1.925\times 10^{7}\,\frac{1}{m^2}\cdot N$$

$$f(x) := \text{Nstr} - \text{Fcc} + \text{Fst} - \text{Fsc}$$

$$f(x) := \text{Nstr} - \varepsilon\text{concrete}\cdot\text{Eccracked}\cdot\frac{1}{2}\cdot x\cdot 1000mm + \text{fyd}\cdot\text{As} - \varepsilon\text{concrete}\cdot\frac{(x-\text{cover})}{x}\text{Es}\cdot\text{As}$$

$$x := 256mm$$

$$\text{root}(f(x), x) = 0.256m$$

$$\text{Fcc} := \varepsilon\text{concrete}\cdot\text{Eccracked}\cdot\frac{1}{2}\cdot x\cdot 1000mm = 2.464\times 10^{6}N$$

$$\text{Fst} := \text{fyd}\cdot\text{As} = 4.35\times 10^{6}N$$

$$\text{Fsc} := \varepsilon\text{concrete}\cdot\frac{(x-\text{cover})}{x}\text{Es}\cdot\text{As} = 2.885\times 10^{6}N$$

$$\kappa := \frac{\varepsilon\text{concrete}}{x} = 6.836\times 10^{-3}\,\frac{1}{m}$$

$$M := -\text{Nstr}\cdot\left[\frac{1}{2}h - (h-x)\right] + \text{Fcc}\cdot\frac{2}{3}\cdot x + \text{Fst}\cdot(h - x - \text{cover}) + \text{Fsc}\cdot(x - \text{cover}) = 6.667\times 10^{3}m\cdot kN$$

$$EI := \frac{M}{\kappa} = 9.752\times 10^{5}m^{2}\cdot kN$$ bending stifness 3

# Spring support

MV-pile HEB450

$EI1 := 252000000N \cdot m^2$

$EA1 := 4577580000N$

$L1 := 20m$

concrete foundation pile 450*450 mm^2

$EI2 := 374000000N \cdot m^2$

$EA2 := 22275000000N$

$t := 29m$

$L2 := 0.65 \cdot t = 18.85m$

$\alpha := 15.9 \cdot 2 \cdot \frac{\pi}{360} = 0.278$

horizontal combination stifness

$$k := \frac{3 \cdot EI1}{L1^3 \cdot 2} + \frac{3 \cdot EA1}{L1 \cdot 2} + \cos(\alpha) \frac{12EI2}{L2^3} + \sin(\alpha) \frac{3EA2}{L2} = 4.405 \times 10^5 \frac{1}{m} \cdot kN$$

per unit width

$ctc := 2.5m$

$$k := \frac{k}{ctc} = 1.762 \times 10^5 \frac{1}{m^2} \cdot kN$$

$$kmvpilehor := \frac{\frac{3 \cdot EI1}{L1^3 \cdot 2} + \frac{3 \cdot EA1}{L1 \cdot 2}}{ctc} = 1.373 \times 10^5 \frac{1}{m^2} \cdot kN$$

$$kbackpilehor := \frac{\cos(\alpha) \frac{12EI2}{L2^3} + \sin(\alpha) \frac{3EA2}{L2}}{ctc} = 3.887 \times 10^4 \frac{1}{m^2} \cdot kN$$

$$kbackpilevert := \frac{\sin(\alpha) \frac{12EI2}{L2^3} + \cos(\alpha) \frac{3EA2}{L2}}{1} = 3.41 \times 10^5 \frac{1}{m} \cdot kN$$

# Combination Scia and D-sheet ULS:

Maintenance

$Fhdsheet := 1100kN$ $Fhscia := 1126kN$

$udsheet := 7.9mm$ $uscia := 8.2mm$

high water

$Fhdsheet := 819kN$ $Fhscia := 833kN$

$udsheet := 5.97mm$ $uscia := 6.1mm$

Vessel

$Fhdsheet := 900kN$ $Fhscia := 800kN$

$udsheet := 6.6mm$ $uscia := 5.9mm$

# Max Dsheet

Maintenance

$MaxMdsheet := -3727kN \cdot m$ Branch 2 (not yet yielding steel)

high water

$MaxMdsheet := 1800kN \cdot m$ Branch 2 (not yet yielding steel)

Vessel

$MaxMdsheet := -2648kN \cdot m$ Branch 2 (not yet yielding steel)

# Max SCIA

$Medneg := -4468kN \cdot m$ Vessel floor above diaphragm wall

$Medpos := 2064kN \cdot m$ Vessel floor back corner above piles

$Ved := 2065kN$ High water floor above diaphragm wall

$Fpile := 9942kN$ High water

$Fhmv := 1100\frac{kN}{m}$ Maintenance

$Mmaxsls := 2757kN \cdot m$ Maintenance floorabove diaphragm wall

# ULS check bottom floor super structure maintenance

### main reinforcement bending

$Asbottom := 5000mm^2$

$Astop := 11000mm^2$

$\varepsilon c3 := \frac{3.5}{1000} = 3.5 \times 10^{-3}$

$h := 1200mm$

$f(x) := -Fcc + Fst$

$\lambda := 0.8$

$dshear := 16mm$

### Tension at top

$f(x) := -\lambda \cdot fcd \cdot x \cdot 1000mm + fyd \cdot Astop$

$x := 299mm$

$root(f(x), x) = 0.299m$

$Fcc := \lambda \cdot fcd \cdot x \cdot 1000mm = 4.784 \times 10^6 N$

$Fst := fyd \cdot Astop = 4.785 \times 10^6 N$

$Fsc := \varepsilon c3 \cdot \frac{(x - cover)}{x} Es \cdot Asbottom = 2.999 \times 10^6 N$

$\beta := 0.5$

$y := \beta \cdot x = 0.15m$

$Mrd := Fst \cdot (h - y - cover - dshear) = 4.687 \times 10^3 m \cdot kN$

$uc := \frac{|Medneg|}{Mrd} = 0.953$ sufficient

reinforcement to apply:

$nbars := 9$
$dtop := 40mm$

$Astopapplied := dtop^2 \cdot \frac{\pi}{4} \cdot nbars = 1.131 \times 10^4 \cdot mm^2$

$\rho as := \frac{Astopapplied}{h \cdot 1000mm} = 0.942 \cdot \%$

**Tension at bottom**

$f(x) := -\lambda \cdot fcd \cdot x \cdot 1000mm + fyd \cdot Asbottom$

$x := 136mm$

$root(f(x), x) = 0.136m$

$Fcc := \lambda \cdot fcd \cdot x \cdot 1000mm = 2.176 \times 10^{6} N$

$Fst := fyd \cdot Asbottom = 2.175 \times 10^{6} N$

$Fsc := \varepsilon c3 \cdot \frac{(x - cover)}{x} Es \cdot Astop = 4.815 \times 10^{6} N$

$\beta := 0.5$

$y := \beta \cdot x = 0.068m$

$Mrd := Fst \cdot (h - y - cover - dshear) = 2.308 \times 10^{3} m \cdot kN$

$uc := \frac{|Medpos|}{Mrd} = 0.894$ sufficient

reinforcement to apply:

$nbars := 11$

$dbottom := 25mm$

$Asbottomapplied = dbottom^{2} \cdot \frac{\pi}{4} \cdot nbars = 5.4 \times 10^{3} \cdot mm^{2}$

$\rho as := \frac{Asbottomapplied}{h \cdot 1000mm} = 0.45 \cdot \%$

**Shear force reinforcement**

Capacity without reinforcement

$dshear := 16mm$

$fck := 30$

$\gamma c := 1.5$

$$Crd := \frac{0.18}{\gamma c} = 0.12$$

$$d := h - cover - dshear - \frac{1}{2} \cdot dtop = 1.109 \times 10^{3} \cdot mm$$

$$k := 1 + \sqrt{\frac{200mm}{d}} = 1.425 \qquad \leq 2.0$$

$$\rho 1 := \frac{Astopapplied}{1000mm \cdot d} = 0.01$$

$Ned := 0$

$Ac := 1200$

$$\sigma cp := \frac{Ned}{Ac} = 0$$

$k1 := 0.15$

$$vmin := 0.035 k^{\frac{3}{2}} \cdot fck^{\frac{1}{2}} = 0.326$$

$$Vrdmin := (vmin + k1 \cdot \sigma cp) \cdot 1000 \cdot \frac{d}{1000mm} = 361.519 \ kN$$

$$Vrd := \left[ Crd \cdot k \cdot (100 \cdot \rho 1 \cdot fck)^{\frac{1}{3}} + k1 \cdot \sigma cp \right] \cdot 1000 \frac{d}{1000mm} = 592.98 \qquad kN$$

$$uc := \frac{Ved}{Vrd \cdot kN} = 3.482$$ Not sufficient

capacity with reinforcement

$b := 350mm$

$Asw := dshear^2 \frac{\pi}{4} \cdot \frac{1000 \cdot mm}{b} = 574.463 \cdot mm^2$

$z := d - \beta \cdot x = 1.041m$

$\theta := \frac{20 \cdot 2 \cdot \pi}{360} = 0.349$

$s := 300mm$

$Vrd := \frac{Asw}{s} \cdot z \cdot fyd \cdot \cot(\theta) = 2.382 \times 10^3 \cdot kN$

$uc := \frac{Ved}{Vrd} = 0.867$ sufficient

# SLS check maintenance

$Mmaxsls = 2757kN \cdot m$

$f(x) := -Fcc + Fst - Fsc$

$\varepsilon bot := 0.785 \cdot 10^{-3}$

$$f(x) := -\varepsilon bot \cdot Eccracked \cdot \frac{1}{2} x \cdot 1000mm + \varepsilon bot \cdot \frac{(h - x - cover)}{x} Es \cdot Astop - \varepsilon bot \cdot \frac{(x - cover)}{x} Es \cdot Asbottom$$

$x := 459mm$

$root(f(x), x) = 0.459m$

$Fcc := \varepsilon bot \cdot Eccracked \cdot \frac{1}{2} x \cdot 1000mm = 1.982 \times 10^{6} \cdot N$

$Fsc := \varepsilon bot \cdot \frac{(x - cover)}{x} Es \cdot Asbottom = 7.255 \times 10^{5} N$

$Fst := \varepsilon bot \cdot \frac{(h - x - cover)}{x} Es \cdot Astop = 2.71 \times 10^{6} N$

$Med := -Fcc \cdot \left(\frac{1}{3} x - cover\right) + Fst \cdot (h - 2cover) = 2.76 \times 10^{3} \cdot kN \cdot m$ $= \blacksquare \cdot Mmaxsls$

$\sigma s := \frac{Fst}{Astop} = 246.377 \cdot \frac{N}{mm^2}$

$fctm = 2.9 \times 10^{6} Pa$

$\alpha e := \frac{Es}{Eccracked} = 19.091$

$kt := 0.6$

$hcef1 := 2.5(h - d) = 0.227m$ $hcef2 := \frac{h - x}{3} = 0.247m$ $hcef3 := \frac{h}{2} = 0.6m$

$hcef := \min(hcef1, hcef2, hcef3) = 0.227m$

$Aceff := hcef \cdot 1000mm = 0.227 m^2$

$\rho peff := \frac{Astop}{Aceff} = 0.048$

$$\varepsilon smminacm := \frac{\sigma s - kt \cdot \frac{fctm}{\rho peff}(1 + \alpha e \cdot \rho peff)}{Es} = 8.437 \times 10^{-4} \quad \geq \quad 0.6 \cdot \frac{\sigma s}{Es} = 7.039 \times 10^{-4}$$

$k1 := 0.8 \quad k2 := 0.5 \quad k3 := 3.4 \quad k4 := 0.425$

$$Srmax := k3 \cdot cover + \frac{k1 \cdot k2 \cdot k4 \cdot dtop}{\rho peff} = 0.328m$$

$wk := Srmax \cdot (\varepsilon smminacm) = 0.276 \cdot mm$

$wmax := 0.3mm$

$wmax \geq wk$ sufficient

# Bearing capacity foundation pile

$$Atip := 500^2 mm^2 = 0.25\,m^2$$

$$deq := \sqrt{4 \cdot \frac{Atip}{\pi}} = 0.564m$$

$$fsoil := \frac{Fpile}{Atip} = 39.768 \cdot \frac{N}{mm^2}$$

Pile depth: -29 mNAP
Sounding: S37B02682_00
koppejan

$$qc1 := 24MPa$$

$$qc2 := 18MPa$$

$qc3 := 14MPa$ but max: $qc3 := qc2 = 18 \cdot MPa$

$$Prmaxtip := \frac{1}{2}\left(\frac{qc1 + qc2}{2} + qc3\right) = 19.5 \cdot MPa$$

$$uc := \frac{fsoil}{Prmaxtip} = 2.039$$

not sufficient, change ctc from 5.5 m to a smaller value: 2.5 m. Fpile will decrease.

$$Fpilenew := \frac{2.5}{5.5} \cdot Fpile = 4.519 \times 10^3 \cdot kN$$

$$fsoil := \frac{Fpilenew}{Atip} = 18.076 \cdot MPa$$

$uc := \dfrac{fsoil}{Prmaxtip} = 0.927$ sufficient

# MVpile HEB450 CUR 166

$Fhmv = 1.1 \times 10^{3} \cdot \frac{kN}{m}$

in line MV-pile $Fax := \frac{Fhmv}{\sqrt{2}} = 777.817 \frac{1}{m} \cdot kN$

$ctc = 2.5m$

per pile $Fpile := Fax \cdot ctc = 1.945 \times 10^{3} \cdot kN$

bottom effective grout section

$hbot := 31m$

$\gamma g := 22 \frac{kN}{m^3}$

$\sigma g := hbot \cdot \gamma g = 682 \frac{1}{m^2} \cdot kN$

$h1 := -2m + hbot = 29m$

$\gamma w := 10 \frac{kN}{m^3}$

$\sigma w := h1 \cdot \gamma w = 290 \frac{1}{m^2} \cdot kN$

$\phi d := 35 \cdot 2 \cdot \frac{\pi}{360} = 0.611$

$\tau maxbottom := (\sigma g - \sigma w)\tan(\phi d) = 274.481 \frac{1}{m^2} \cdot kN$

top effective grout section

$htop := 25m$

$\gamma g := 22 \frac{kN}{m^3}$

$\sigma g := htop \cdot \gamma g = 550 \frac{1}{m^2} \cdot kN$

$h1 := -2m + htop = 23m$

$\gamma w := 10 \frac{kN}{m^3}$

$\sigma w := h1 \cdot \gamma w = 230 \frac{1}{m^2} \cdot kN$

$\phi d := 35 \cdot 2 \cdot \frac{\pi}{360} = 0.611$

$\tau maxtop := (\sigma g - \sigma w)\tan(\phi d) = 224.066 \frac{1}{m^2} \cdot kN$

$\tau maxaverage := \frac{1}{2}(\tau maxbottom + \tau maxtop) = 249.274 \frac{1}{m^2} \cdot kN$

$Leff := (hbot - htop)\sqrt{2} = 8.485\,m$

$\gamma m := 1.4$

$O := 450mm \cdot 4 = 1.8m$

$Famax := \frac{\tau maxaverage \cdot Leff \cdot O}{\gamma m} = 2.719 \times 10^3 \cdot kN$

$uc := \frac{Fpile}{Famax} = 0.715$ sufficient

according to cone resistance

$qc := 15 \frac{N}{mm^2}$

$\tau max := 0.014 \cdot qc = 210 \cdot kPa$ $\tau max \le 250kPa$

$Famax := \frac{\tau max \cdot Leff \cdot O}{\gamma m} = 2.291 \times 10^3 \cdot kN$

$uc := \frac{Fpile}{Famax} = 0.849$ sufficient

# Kranz stability

depth points A-D

$A := 5.5m$

$B := 28.5m$

$C := htop + \frac{hbot - htop}{2} = 28m$

$D := 0m$

$\gamma clay := 17 \frac{kN}{m^3}$

$\gamma sand := 20 \frac{kN}{m^3}$

$G := 24m \cdot \gamma clay \cdot C - A \cdot 15m \cdot \gamma clay + (C - 24m) \cdot C \cdot \gamma sand = 1.226 \times 10^4 \frac{1}{m} \cdot kN$

$p1 := 24m \cdot \gamma clay = 4.08 \times 10^5 Pa$

$p2 := 24m \cdot \gamma clay + (C - 24m) \gamma sand = 4.88 \times 10^5 Pa$

$E1 := \frac{p1}{2} \cdot kaclay \cdot 24m + \frac{(p2 - p1)}{2} kasand \cdot (C - 24m) = 1.917 \times 10^3 \frac{1}{m} \cdot kN$

$Ea := 1000 \frac{kN}{m}$ from D-sheet

$Fkr := \frac{5500}{1.5} \frac{kN}{m} = 3.667 \times 10^3 \cdot \frac{kN}{m}$ graphic method

$uc := \frac{Fax}{Fkr} = 0.212$ sufficient