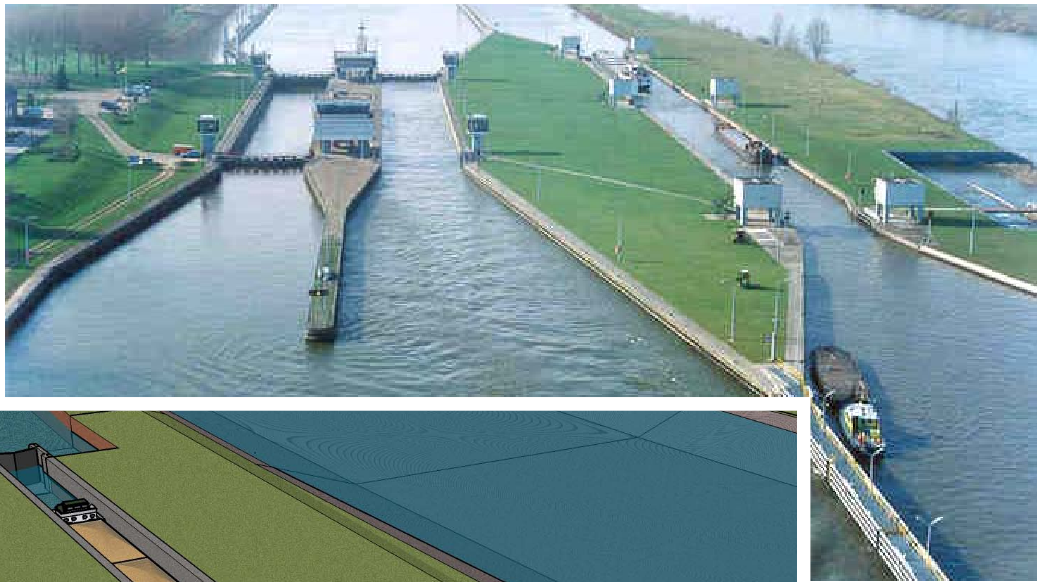
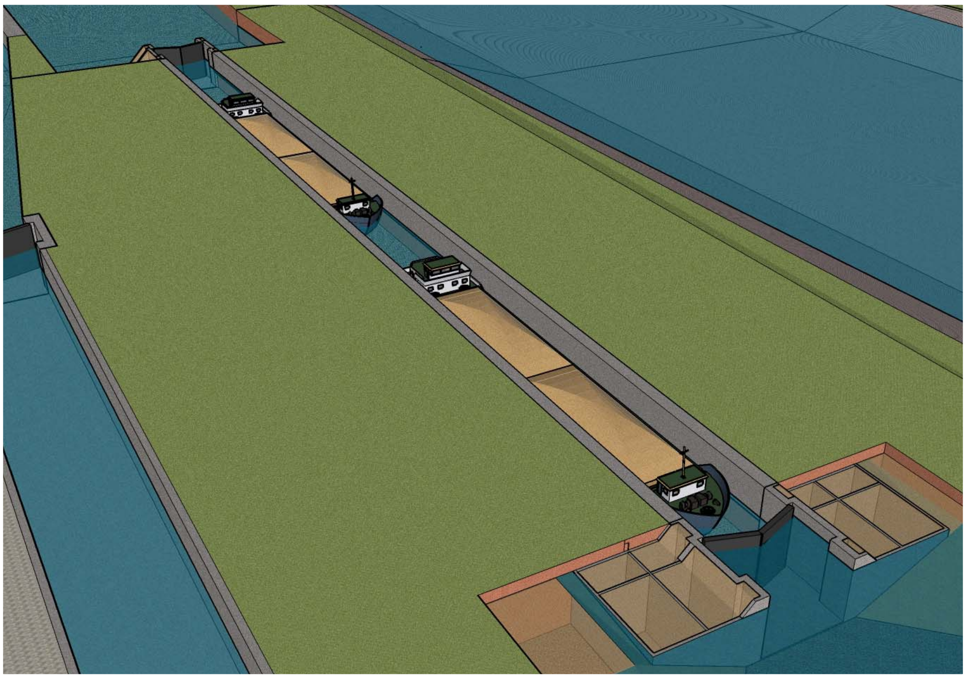
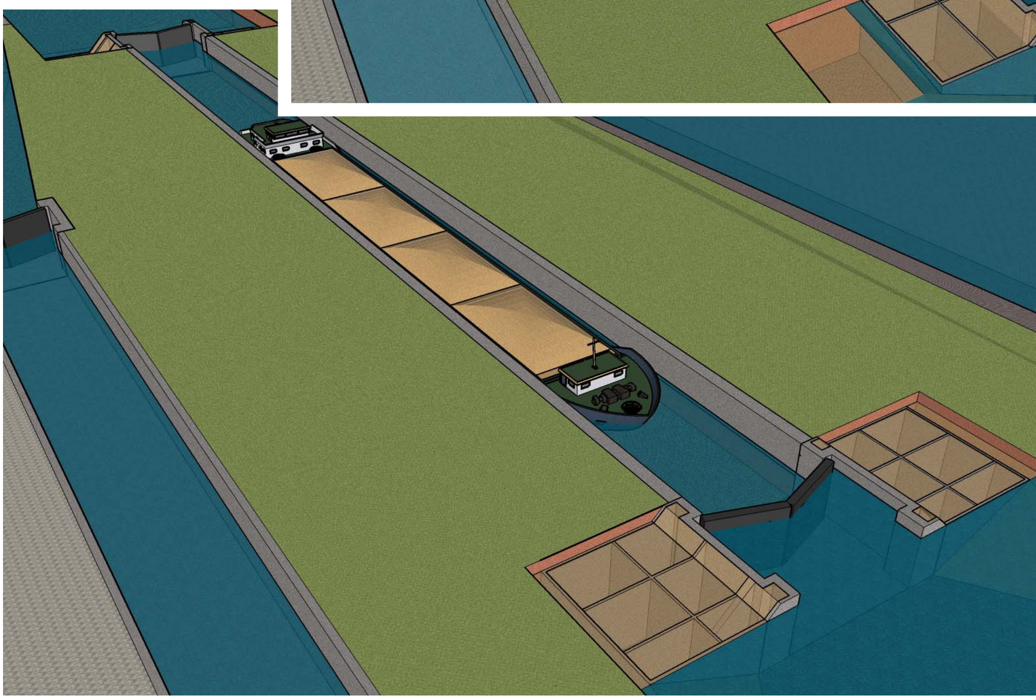
The functional flexibility of lock design

Applied on the Meuse route







Ramon de Groot

June 2009

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Applied on the Meuse route

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Preface

This graduation report is written as a part of the master Hydraulic Structures, faculty of Civil Engineering and Geosciences at Delft University of Technology. With the graduation report "The functional flexibility of lock design, applied on the Meuse route" I will finish my study at the university. This report is executed in cooperation with BAM Infraconsult bv in Gouda, the Netherlands.

This study is carried out under the supervision of my thesis committee: Prof. drs. ir. J.K. Vrijling, Prof. ir. A.Q.C. van der Horst, Ir. W.F. Molenaar and Ir. J.F.M. van Rijen. I want to thank them for the support and guidance during my graduation project. I also want to thank the employees of BAM Infraconsult bv and the other students that performed their graduation at this company for the great time and the help I received. Moreover, my thanks goes to B. Hogendonk (BAM Civiel) for the help with the cost variables and to T. Burhenne (RWS Limburg) for providing data about lock complex 'Sluis Sambeek' and showing me round the complex.

Furthermore, I am grateful to my parents for the support and the freedom they gave me during my study. Last but not least, I want to thank Janet for the support during the difficult phases of my study and for the correction of the final version of my graduation report.

Ramon de Groot,
Gouda, June 2009

Abstract

In general, an inland navigation lock has a structural (technical) lifespan of about 100 years, while in most cases this lock is not big enough anymore after 25 - 50 years. This lose of functionality (economic value) is caused by the growing dimensions and/or the growing intensities of the passing ships.
To extend the functional lifespan to the structural life of about 100 years, a functional flexible lock is proposed. This type of lock is, with or without a few structural adjustments, capable of serving the shipping traffic for its whole structural lifespan. This new lock design approach will be applied on one of the locks of lock complex 'Sluis Sambeek', which is located in the Meuse route.

To determine whether a functional lock will make the difference, the first part of the study is carried out on the basis of the Life Cycle Management (LCM) approach.
Before the LCM approach can be used, a trend forecast is performed to complete the required 'Basis of Design'. From the trend forecast, which is based on data of the past, follows that the future ship sizes will increase and that the ship intensity will slowly decrease. Therefore, the future ship sizes are normative for the required lock dimensions. In addition, the (re)construction of the (flexible) lock is expected to be required in 2020 to fit CEMT-class Vb, vessels and in 2052 to fit CEMT-class VIa vessels. Only an extension in the width direction appears to be necessary in 2052.
On the basis of this trend forecast, three alternatives are elaborated, namely:

1. Zero-alternative: renovation of the lock as planned by the department of public works.
2. Functional flexible lock: standard lock that is built large enough to facilitate passages for the maximum expected ships in the next 100 years.
3. Structural flexible lock: a relatively easy extendable navigation lock, which can be enlarged when it is required. Thus, the maximum dimensions are reached step-wise.

While often only the initial costs are taken into account, this study uses a Whole Life Costing (WLC) analysis with a risk inventory to compare the alternatives. In this analysis, alternative 2 appears to be the least expensive option for 'Sluis Sambeek'.
Although alternative 3 is the most expensive option, this alternative is elaborated further, because it is the most innovative option. Alternative 3 could also be more beneficial at lock locations where the spilling of water is a bigger problem, where the growth in the lock dimensions is expected less, or where the obstruction of one of the locks of the complex is more far-reaching than is the case for 'Sluis Sambeek'.

The structural flexibility in alternative 3 is formed by the lock chamber which exists of relatively simple replaceable sheet pile walls, and by floatable lock heads that can be replaced by wider lock heads.
The innovative parts that have to be designed or considered for alternative 3 are:

- The float up of the small lock heads after 40 or 50 years
- The construction planning
- The structural lock head design
- The lock chamber and the lock heads connection

Only two of these parts are elaborated in this report, namely the construction planning and the structural lock head design. The construction planning is just worked out partly to provide load combinations for the reconstruction in 2052. The large structural lock heads (2052) will be checked and optimised on the basis of three critical cross-sections. From this checks it can be concluded that the floatable lock heads satisfy and that they could be optimised further.

In this report, it was shown that a functional flexible lock is the cheapest option over the whole life time. Moreover, the obstruction time during the life time of the functional flexible lock and the structural flexible lock will be shorter, this will be beneficial for the transportation per ship. Consequently, it was concluded that it is useful to consider the possibilities of a functional or structural flexible lock design for 'Sluis Sambeek' or for another lock reconstruction.

Table of contents

1. Introduction

Nowadays, durability is a big issue in the world of navigation lock design. Hydraulic structures are designed to have a lifetime of about 100 years [Glerum, 2000], but in most cases the lock is not big enough anymore after 25 - 50 years. This is caused by the growing dimensions and/or the growing intensities of ships that want to pass the lock. The solution for reaching a real lifetime of 100 years is a functional flexible lock. This type of lock is, with or without a few structural adjustments, capable of serving the shipping traffic for its whole structural lifespan. This new lock design approach will be applied on the Meuse route in this report, but can also be used at other locations. More specifically, in this report the lock complex 'Sluis Sambeek', which is located in Meuse route, will be adjusted. 'Sluis Sambeek' contains an 80 year old lock, which is already planned to be reconstructed or rebuild in the next few years.

The possibilities of a functional and a structural flexible inland navigation lock will be studied and these design approaches will be applied on the Meuse route ('Sluis Sambeek'). The lock design must be able to cope with the ship sizes and intensities that are expected to occur in the next 100 years.
The design of a functional flexible lock consists of two parts in this report. In the first part, different alternatives, which are based on a trend forecast, are worked out conceptual and compared on the basis of the risks, the probable benefits and the initial and lifetime costs. Three alternatives are compared, namely a zero-alternative, a functional flexible lock and a structural flexible lock. This alternative comparison is worked out roughly to give a view of the situation. In the second part, the structural flexible lock alternative is designed and calculated into detail. The structural flexible lock alternative will be worked out regardless the outcome of the alternative comparison, because this alternative is the most innovative one. Only the most interesting and innovative parts of this alternative are treated in this lock design.

This report starts with an explanation of the Life Cycle Management (LCM) approach in Chapter 2, because the first part of this study is based on this approach (Chapter 5 till 9). In Chapter 3, it is discussed how the elements and components fit into the system of a navigation lock, and in which way they interact with the different functions of a lock. After that, the problems are defined, and the objective is formulated in Chapter 4.
After the introducing chapters (Chapter 1 till Chapter 4), it is explained why the location 'Sluis Sambeek' is selected, and this location is introduced in Chapter 5. The trend in ship traffic for this location is forecasted in Chapter 6. Then, the required lock dimensions, which follow from the forecast, are combined with the boundary conditions in the 'Basis of Design' (Chapter 7). On the basis of Chapter 7, the three alternatives are worked out conceptual in Chapter 8 and these alternatives are compared by a Whole Life Costing (WLC) analysis in Chapter 9 to complete the LCM analysis.
In the second part of this report, the innovative parts of a structural flexible lock are discussed and explained (Chapter 10). In this chapter it is also determined that the construction planning and the structural design of the lock heads are the most interesting and innovative parts. Therefore, these parts will be worked out in more detail. First, the construction planning of the reconstruction of the future structural flexible lock will be elaborated in Chapter 11. Second, the structural design of the lock heads will be worked out in detail in Chapter 12.
Finally, the conclusions and recommendations will be represented in Chapter 13.

2. Life Cycle Management (LCM)

As stated in the introduction (Chapter 1), an inland navigation lock has a structural (technical) lifespan of about 100 years, while his functional (economic) lifespan is already questioned after 25-50 years.
The aim of this graduation report is to design a lock with a functional lifespan that is equal to the structural lifespan. Therefore, the functional flexible lock is introduced. For this type of lock, the structural lock design will be made adjustable to make the lock functional for its whole structural lifespan. To compare the different functional flexible lock designs and a standard lock design, a Life Cycle Management (LCM) analysis is performed. This LCM analysis takes all the costs and risks into account to compare the different design scenarios.
Before the LCM analysis is implemented, this chapter will explain the general working of a LCM approach in paragraph 2.1. In paragraph 2.2 the LCM approach is applied on a functional flexible navigation lock.

2.1. LCM in general

LCM is a management approach to achieve an optimum quality and minimum Whole Life Costing (WLC) [PIANC, 2009]. All the costs that arise in the lifetime of the structure are included. Besides the initial costs, like construction and maintenance costs, also the risks, the lifetime costs and probable benefits related to its use are taken into account. The costs of the structure are analysed from design till demolition. Therefore, the cost of any major rebuilding or component replacement during that period will also be part of the WLC. Different examples for achieving a minimum WLC are: constructing an almost maintenance free construction, use a lot of standardisation in construction, and optimise the lock design on the basis of the traffic intensity. In Figure 2.1 the general LCM procedure can be seen.

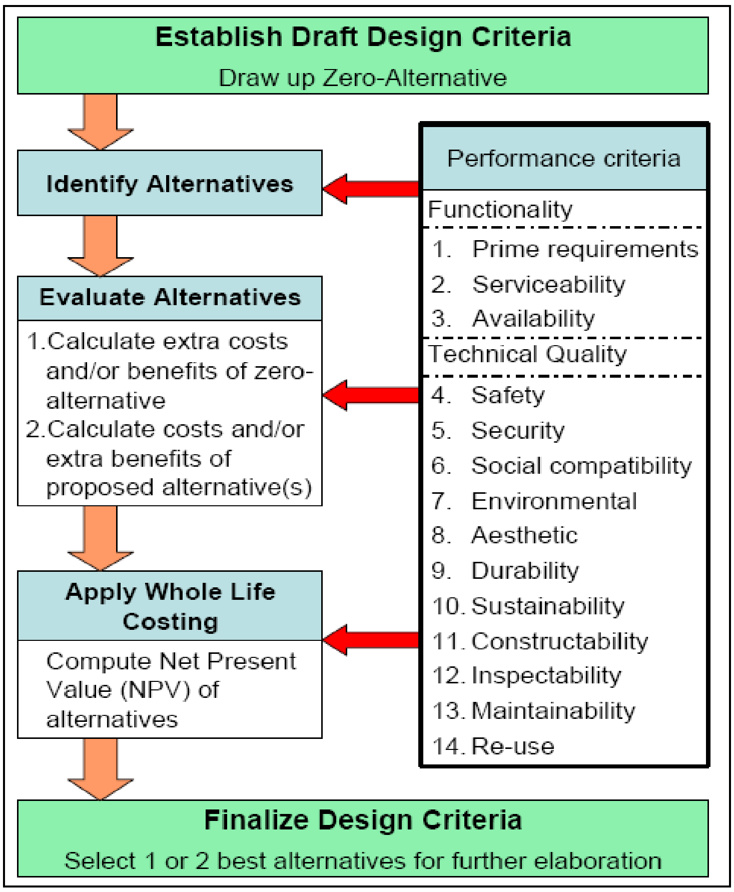

Figure 2.1: The LCM procedure [PIANC, 2007]

An LCM approach can be used during the design and construction of all types of structures and it can be used at different levels. For instance, the LCM approach can be used on the level of a complete waterway (scenario development), but also on the level of a single sheet pile wall (material selection). For each type of structure or level, the LCM is separated into four different phases [PIANC, 2007], namely:

- Planning and design phase
- Construction phase
- Operational and maintenance phase
- Re-use and/or disposal phase

In this report, only the 'Planning and design phase' is used, because this report only concerns a design. For this phase two main documents are needed, namely the 'Client's Brief' and the 'Basis of Design'.

The 'Client's Brief' should include in any case [PIANC, 2007]:

- The type of the facility required
- Where the facility is to be located
- When the facility should be commissioned – programme/phasing of facilities
- Planned performance of the facility – throughput and phasing
- Planned economic life and implementation of LCM
- Potential future use for the facility at the end of its economic life or possible alternatives
- Likely external influences e.g. planning consents
- The available budget/required phasing of costs

The 'Basis of Design' should include in any case [PIANC, 2007]:

- A recital of the 'Client's Brief'
- Local and site specific physical and environmental conditions
- Site geotechnical investigations
- The design criteria and design loadings to be adopted
- Impacts from external sources e.g. planning conditions or operational conditions
- The results of any investigations undertaken and their impacts
- A maintenance strategy
- Anticipated re-use/removal of the structure at the end of its economic life

On the basis of these two documents and the LCM procedure (Figure 2.1), a LCM analysis can be performed.

2.2. LCM applied on a functional flexible navigation lock

In this report, the LCM approach is used to make a comparison between different design scenarios of a functional flexible navigation lock. For this purpose, the LCM analysis is not executed in total, but only for the parts which are expected to make a difference in the 'Planning and design phase'.

No real 'Client's Brief' is available, because there is no client for the lock considered in this report ('Sluis Sambeek', see Chapter 5). Nonetheless, a trend forecast is produced in this report (Chapter 6), which delivers the additional requirements for the 'Basis of Design'. In this trend forecast, the expected ship sizes and intensities are determined. The lock dimensions and the assumed end of the functional (economic) life, result from this data and can be used instead of the 'Client's Brief'.

The 'Basis of Design' in Chapter 7 of this report consists of the following parts.

- A recital of the 'Client's Brief' => Trend forecast in the lock dimensions
- Local and site specific physical and environmental conditions
- Site geotechnical investigations
- The design criteria and design loadings to be adopted

- Impacts from external sources e.g. planning conditions or operational conditions
- The results of any investigations undertaken and their impacts
- The assumed end of its economic life

After finishing the 'Basis of Design', the zero lock alternative and the functional flexible lock alternatives are drawn up in Chapter 8. In addition, these alternatives are evaluated and the Whole Life Costing (WLC) is calculated for each alternative in Chapter 9. On the basis of these evaluations and the WLC calculations, an alternative can be selected.
In Figure 2.2 is shown how the LCM procedure is used in this report. This figure also gives insight in the structure of this report. The LCM procedure will be used on the level of an alternative study to compare the different lock development scenarios.

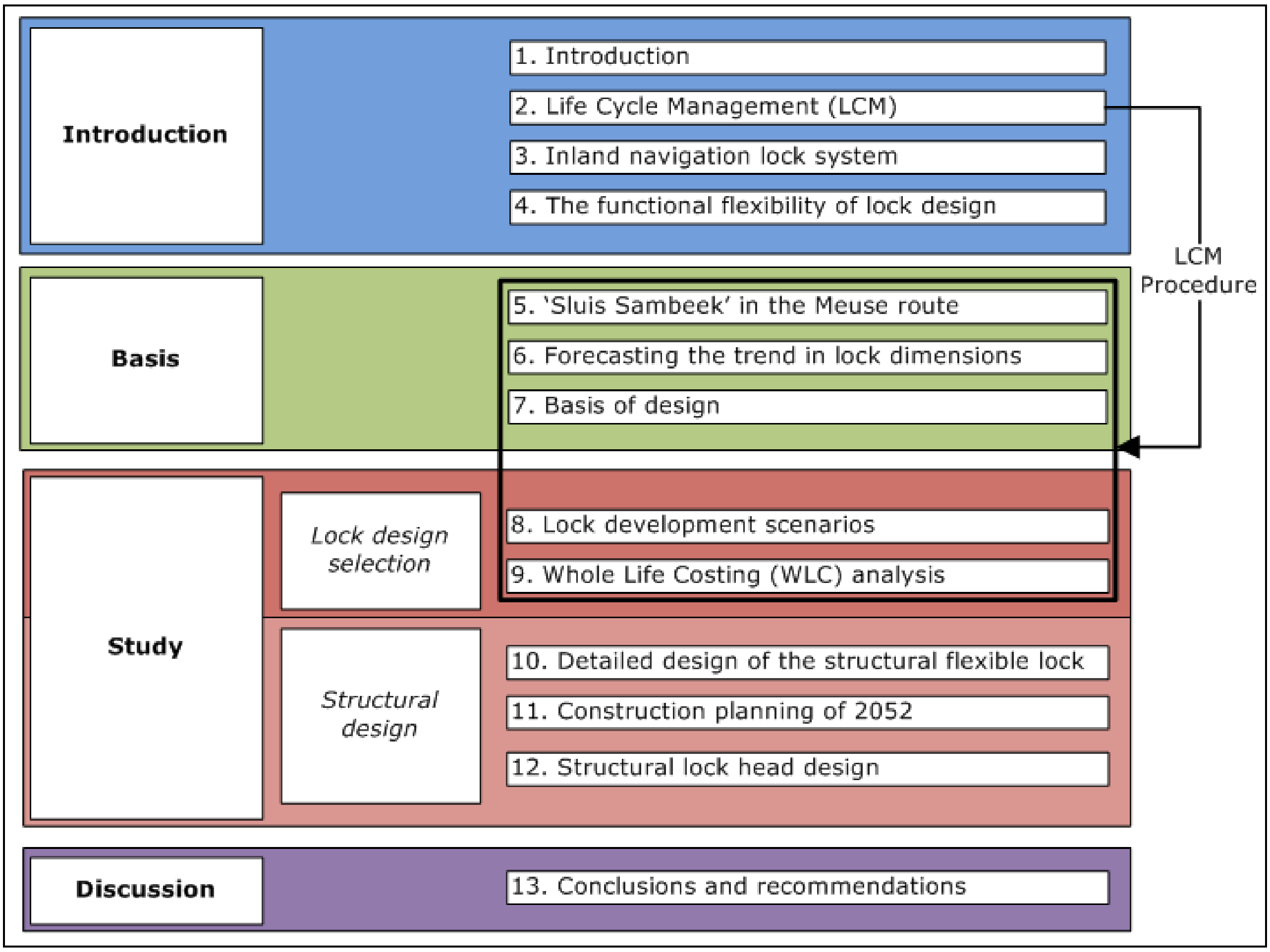

Figure 2.2: Structure of this report and the role of the LCM procedure

3. The inland navigation lock system - short analysis

3.1. The functions of an inland navigation lock

An inland navigation lock has four main functions:

- the navigation function
- the water retaining function
- the water management function
- the cross waterway connection

Not all these functions are representative for the lock considered in this report (lock complex 'Sluis Sambeek', see Chapter 5). There is no cross waterway connection in the area and the lock complex is not part of a primary dike ring, so it has no primary water retaining function. Therefore, the navigation function and the water management function are the only two main functions of 'Sluis Sambeek'. These functions are described below. In the 'Basis of Design' (Chapter 7) these functions are quantified.

3.1.1. The navigation function

The main function of a navigation lock is creating a passage for ships to reach a different water level. The two conditions to fulfil this function are:

- Let ships pass
- Overcome the water level difference

3.1.2. The water management function

This function provides a minimum water level to keep the upstream waterway navigable. In times of a low river discharge it is important to spill as little water as possible. Though, in case of a high discharge, the water has to be flushed in order to reduce the water levels upstream.

3.2. The system of an inland navigation lock

The purpose of the functional flexible lock is adjusting the structural components of the lock to serve the ship dimensions and intensities and consequently satisfy the navigation function. This navigation function is represented by the system capacity. The system capacity is based on two parts.

1. The structural system of a lock (paragraph 3.2.1)
2. The locking process (paragraph 3.2.2)

The combination of these two parts determines the capacity of a lock system. This is shown in Figure 3.1. In the figure it can also be seen that the structural system influences the locking process. This influence results from the fact that the locking time depends on the structural element 'Filling and emptying system'.

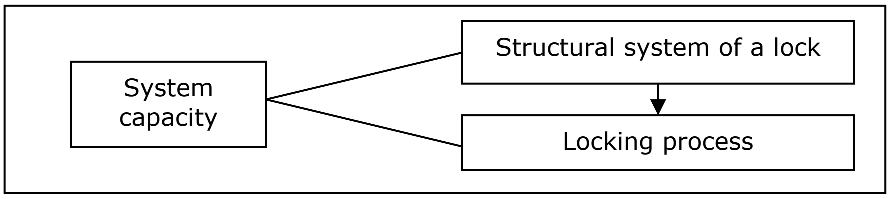

Figure 3.1: System capacity of an inland navigation lock

3.2.1. The structural system of a functional flexible lock

The structural system of a navigation lock is formed by four different elements:

1. Approach (upstream and downstream approach)
2. Lock head (upper head and lower head)
3. Lock chamber
4. Filling and emptying system

In Figure 3.2 these elements are shown in a basic navigation lock layout. The approach and the lock chamber (elements 1 and 3) are serving only the navigation function. The lock head and the filling and emptying system (elements 2 and 4) are serving both the navigation and the water management function.

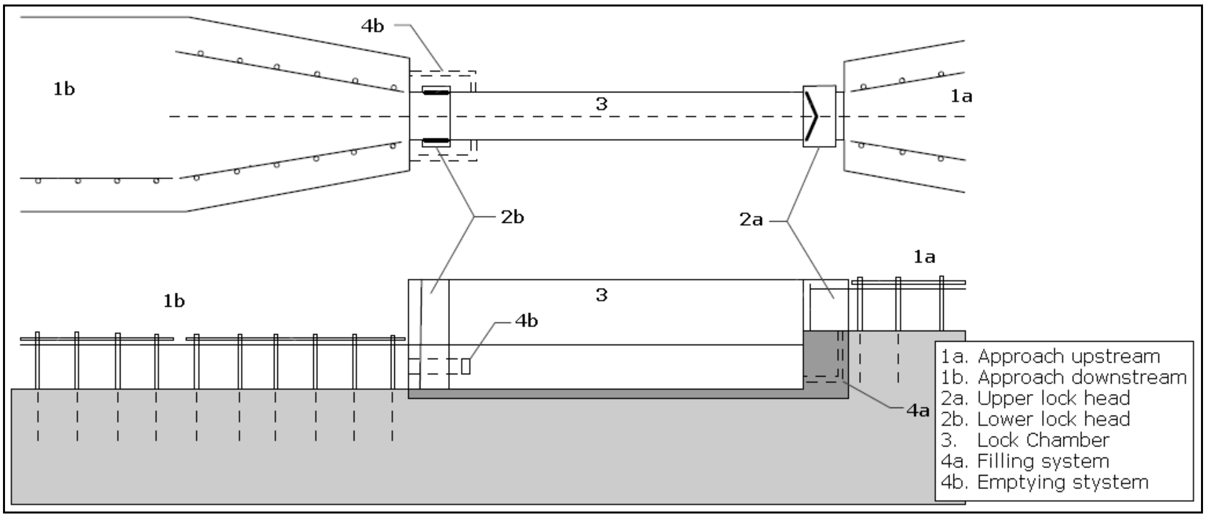

Figure 3.2: Basic navigation lock layout [Hovingh, 2002]

The elements and the components that form the system of an inland navigation lock can be seen in Figure 3.3. In this figure it is also shown which components are related to a change in one of the three flexible lock dimensions.

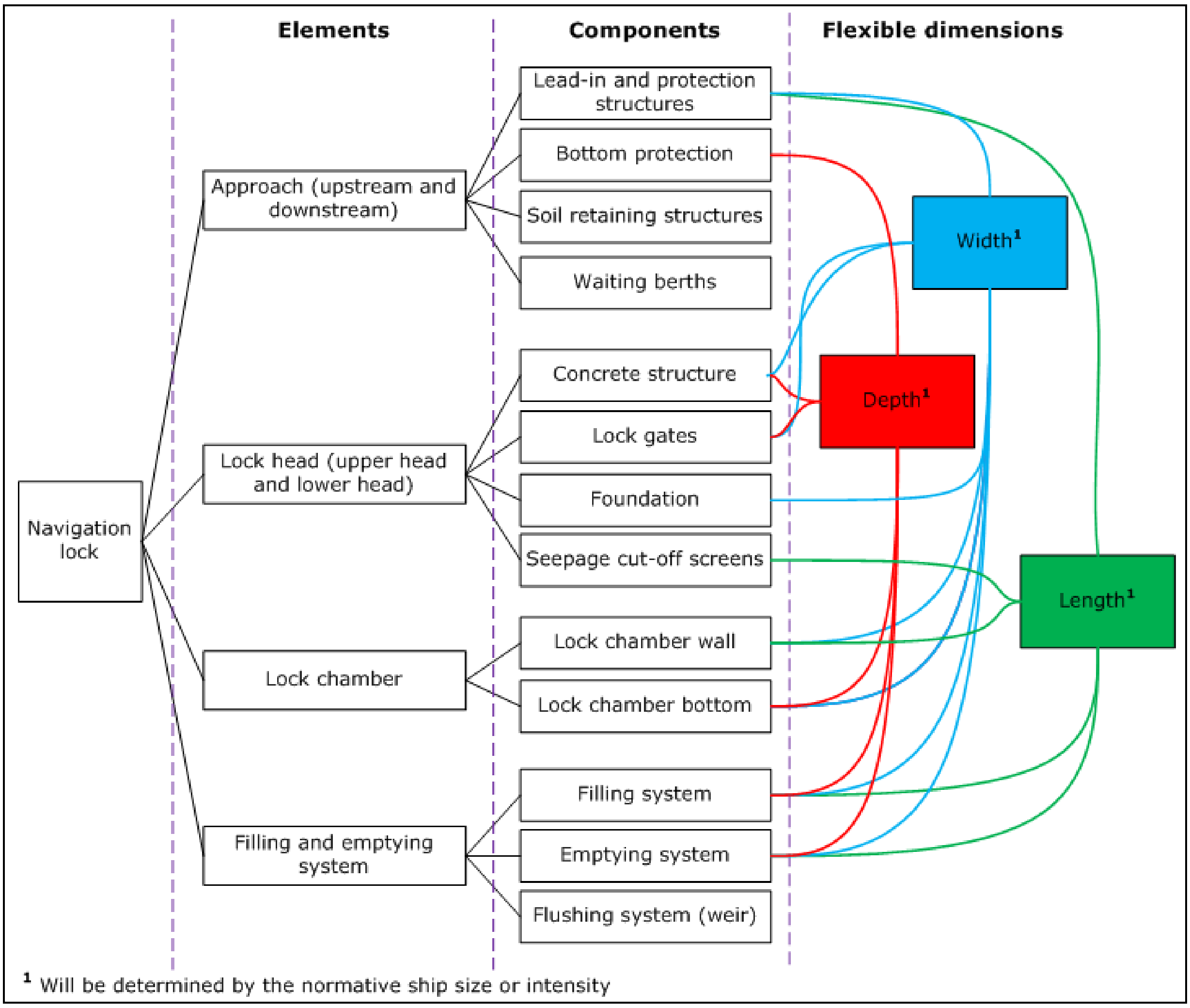

Figure 3.3: The structural system of a flexible navigation lock

The requirements for the functions and their corresponding elements are determined in Chapter 7. The influences of the flexible lock dimensions on the components are specified in Chapter 8 in a sensitivity analysis.

3.2.2. The locking process

The locking process as a function of time is the locking cycle. The locking cycle is shown in Figure 3.4. The figure represents a normal locking cycle. This locking cycle is standardized and can differ from the real situation, because the cycle depends on the number of ships that have to be locked at one moment. This means that a larger number of ships results in a relatively longer navigate in and navigate out time than the service time. Shipping traffic from both directions is taken into account. T_c is the cycle period divided in $T_{d;upstream}$ and $T_{d;downstream}$, which both represent the time one ship needs to pass the lock (with exception of the waiting time).

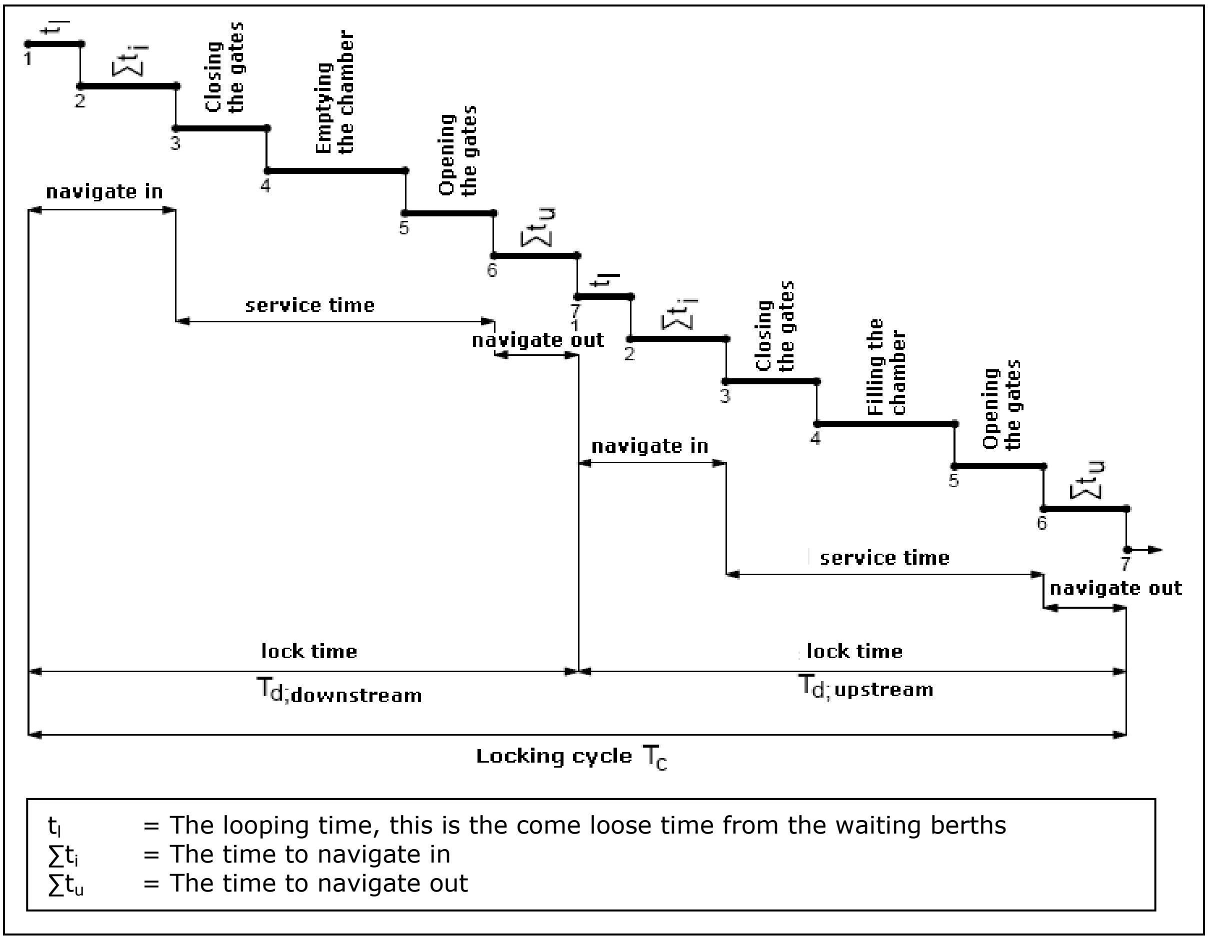

Figure 3.4: The locking cycle [Hovingh, 2002]

4. The functional flexibility of lock design

4.1. *The functional flexible lock*

The functional flexible lock is a new approach in navigation lock design. As stated before, the structural (technical) lifespan of a navigation lock is about 100 years, while the functional (economic) life of a lock is in most cases not longer than 25 to 50 years.
The functional flexible lock is designed as a navigation lock that has a functional life of 100 years. This means that the lock's structural life is equal to the lock's functional life. As a result, the lock will be demolished at the end of its structural life. To achieve this functional life of 100 years, the designers have to anticipate on the future traffic situation. Two approach methods can be used to do this.

1. The first possibility is designing a lock that can cope with the ship sizes and intensities that are expected for about 100 years. This lock is built large enough at once.
2. The second possibility is taking into account the future extensions of the lock dimensions in the initial design. This lock structure is enlarged step-wise.

To determine the functional flexible dimensions of a lock, a forecast of ship passages and ship sizes is performed in Chapter 6. This forecast can be used to estimate the needed lock size for about 100 years. The dimensions of the navigation lock can be determined by the expected traffic intensity and by the expected maximum ship size. After the trend is forecasted, the best path to reach the needed lock dimensions can be determined. The different paths are shown in Figure 4.1.

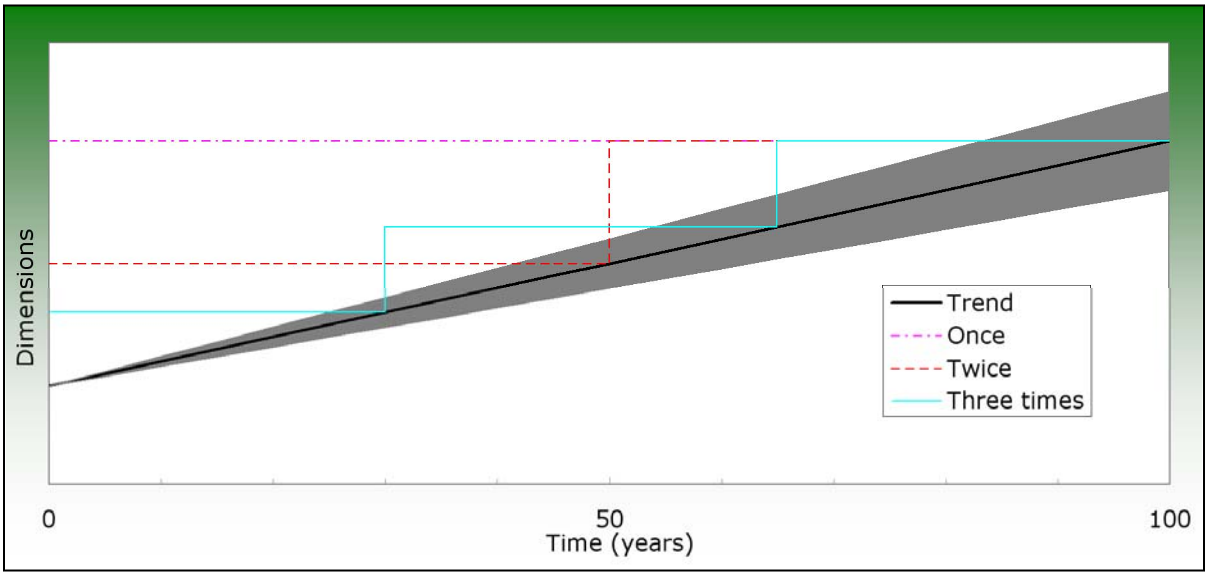

Figure 4.1: Lock dimension options

The different paths are explained below

- *Once* represents a functional flexible lock, without flexible structural elements. At point t=0, a lock is built with the dimensions for the expected ship intensities and sizes in about 100 years (method 1).
- In path *twice* the lock is constructed for the expected dimensions required in 50 years. After these 50 years, the lock has to be reconstructed. The initial build lock is made suitable for reconstruction, so the final required lock dimensions will be reached step-wise (method 2).
- The *three times* path is the same as the twice path, except from the fact that the lock has to be reconstructed two times (method 2).

There is a risk that the investment made at point t=0 is not necessary in the future anymore. This risk is much higher for the *once* path than for the *twice* and the *three times* paths. For instance, it is possible, that a smaller lock will satisfy when the traffic situation become less favourable. When the *once* path was selected, this would be a loss

of investment. In contrast to the *once* path, in the *twice* and *three times* path it is possible to throw out the plan to enlarge the lock or to delay the enlargement.
The advantages and disadvantages of the different design paths of a functional flexible lock are listed below.

Functional flexible lock advantages
Once

- The investments in the future will be restricted to a minimum.
- There is no obstruction time due to reconstruction.

Twice and three times

- In the future, the obstruction time, due to reconstruction, of a functional flexible lock will be less compared to a standard lock.
- The structural flexibility provides the possibility to anticipate on the ship traffic, because the reconstruction can take place either sooner or later than was planned.
- Less water is spilled than in case of a larger not structural flexible lock (the *once* path).

Functional flexible lock disadvantages
Once

- The initial costs will be higher due to the construction of a larger lock than initially is necessary.
- More water is spilled than in case of a standard lock or a smaller structural flexible lock.

Twice and three times

- There are uncertainties about the problems that can arise in the design or construction phase, because of the new construction and design method.
- During the extension phase the flexible lock is temporarily out of use.
- Extra space has to be reserved in the lock complex to provide future extensions.
- The structural flexible lock is more expensive than a standard lock, because of the structural flexible components.

In this report, the *once* path is called a functional flexible lock further on. The *twice and three times* path is called a structural flexible lock. In Chapter 8 the structural design of a standard lock, a functional flexible lock (*once* path) and a structural flexible lock (*twice* path) is performed. After that, in Chapter 9, the best design path is determined on the basis of the Whole Life Costing (WLC).
Besides the design path that has to be selected, also the governing direction of the extension has to be determined. This means that it has to be determined which of the lock dimensions has to be flexible: the depth, the length, the width or a combination of these dimensions. The decision which flexible dimension(s) is/are necessary is decided in combination with the trend forecast in Chapter 6.

4.2. Definition of the problem

After these four introducing chapters, the problems that will be treated in this graduation report can be formulated. These problems are represented in research questions. These questions will lead to the objective that is formulated in the next paragraph. In this report the following questions will be answered.

1. Meuse route trend forecast (Chapter 6)
    - What is the expected trend in ship sizes and intensities for the next 100 years on the Meuse route ('Sluis Sambeek')?
    - Which inland navigation lock dimensions are required and when are they needed?
    - Which of the lock dimensions has to be flexible: the depth, the length, the width or a combination of these dimensions?

2. Alternative selection (Chapter 8 and 9).
    - How can the following alternatives deliver the right lock dimensions at the correct time?
        - The construction of a standard lock that will be demolished and rebuild when required by the ship traffic (zero-alternative).
        - The construction of a functional flexible lock that is large enough to handle the expected ship intensities and sizes for the next 100 years (*once* path).
        - The construction of a structural flexible lock that will be reconstructed when required by the ship traffic (*twice* path).
    - What is the best option of the three mentioned alternatives, based upon the initial costs, the lifetime costs and the risks of these alternatives?

3. Structural feasibility of a structural flexible lock (Chapter 10, 11 and 12)
    - What is the structural feasibility of a structural flexible lock?
    - Which parts of a structural flexible lock are innovative and need some extra attention?

4.3. Objective

Design and study the possibilities of a functional and structural flexible inland navigation lock for the Meuse route ('Sluis Sambeek') that is able to cope with the ship sizes and intensities that are expected to occur in the next 100 years.

5. 'Sluis Sambeek' in the Meuse route

In this report, the functional flexible lock design will be situated on a specific location. Selecting a location is needed because the functional flexibility of a lock depends on the actual and the future ship intensities and dimensions. However, it is still possible to use the solutions of this design for another case with only a few adjustments. When the design was only made on a theoretical basis, the possibilities were too extensive to represent a good and clear solution. In the next two paragraphs, a waterway and a specific navigation lock are selected to accommodate the design of a functional flexible lock.

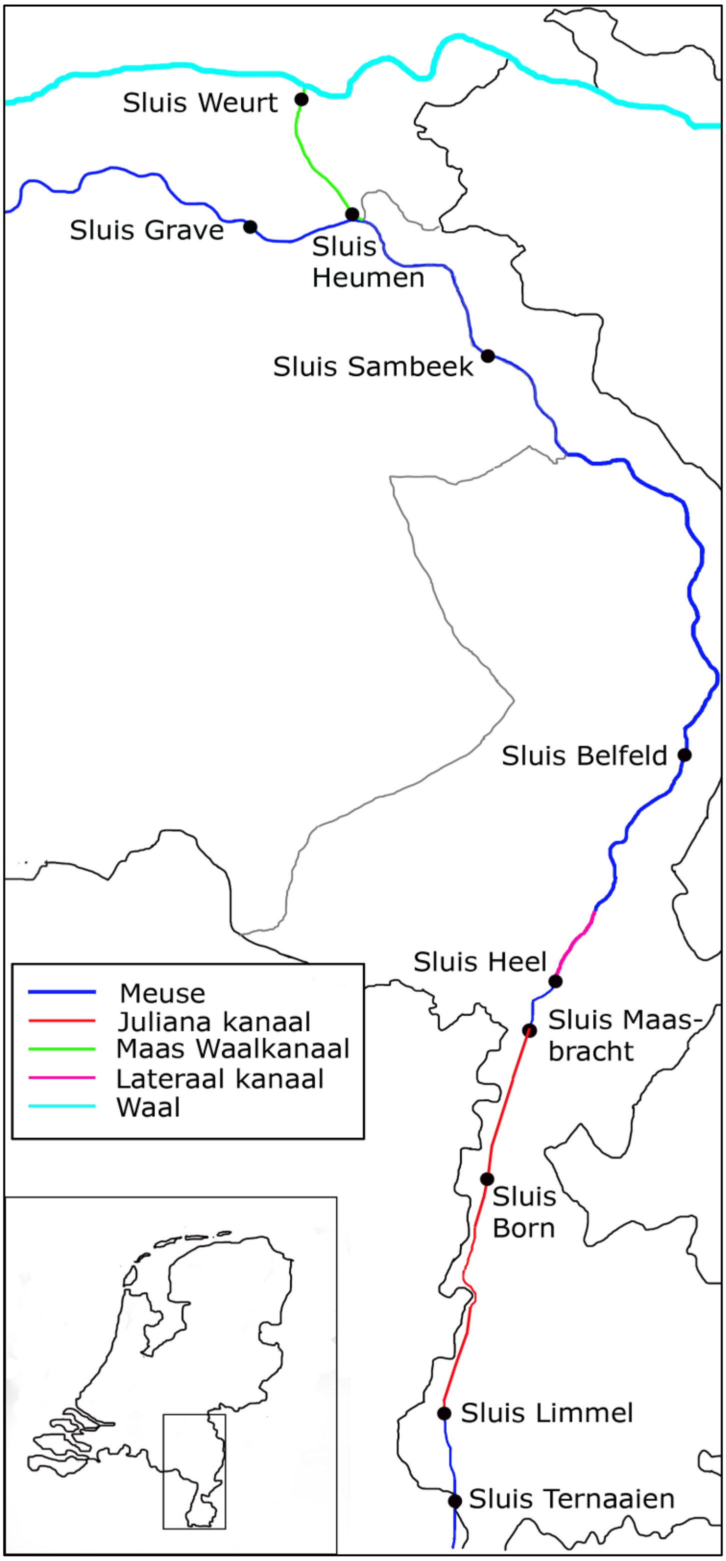

Figure 5.1: Map of the Meuse route

5.1. *The Meuse route*

The Meuse route provides good opportunities for a functional flexible lock location, because the locks in the Meuse route are aged and/or not big enough anymore to fulfil their duty. A capacity problem does arise at this moment while it is only 40 years after the latest reconstruction of the Meuse route has taken place. This is a good example of locks that are designed for a constructive lifetime of 100 years and that only have a functional lifetime of 40 years. Thus, the locks in the Meuse route have to be reconstructed or rebuild.

In contrast to a sea lock, an inland navigation lock also has the advantage that the ship dimensions are restricted to the maximum capacity of the waterway.

The Meuse route consists of parts of the river Meuse, the 'Juliana kanaal', the 'Lateraal kanaal' (Linne – Bruggenum) and the 'Maas Waalkanaal'. All these waterways are shown in Figure 5.1. In the 'Literature report' [Groot, 2008], a more complete description of the route and information about all the navigation locks in this route is given. The Meuse route is part of the 'Benelux Vaarwegruit' as can be seen in Figure 5.2. The 'Benelux Vaarwegruit' is a waterway network which connects Antwerp, Rotterdam, Germany and East – Belgium. A more detailed overview of this waterway network can be seen in Figure 6.3 in the next chapter.

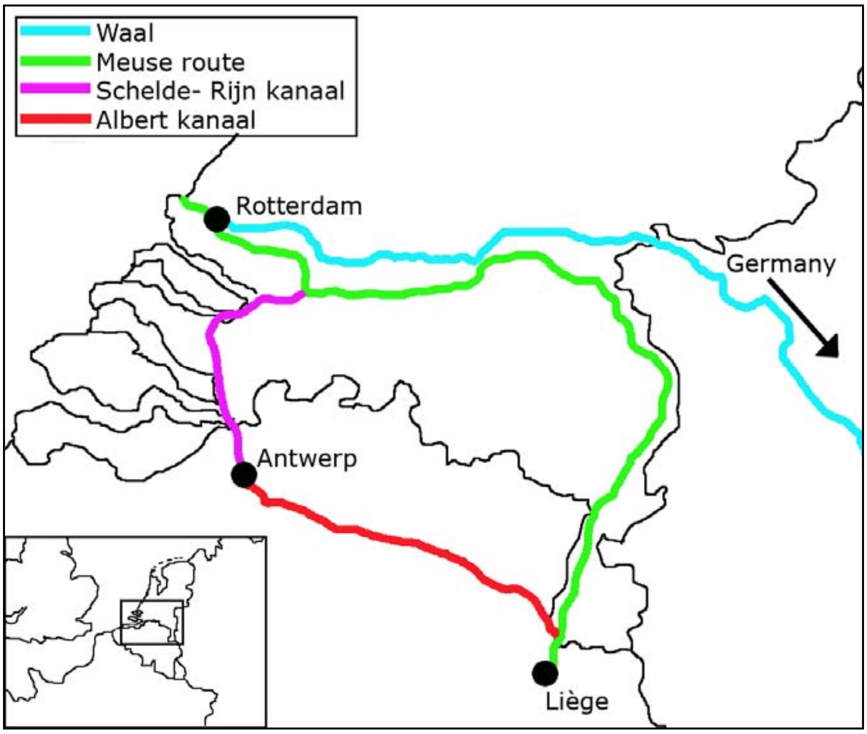

Figure 5.2: 'Benelux Vaarwegruit'

5.1.1. The Meuse route from 'Sluis Weurt' till 'Sluis Heel'

The minimal inland navigational cross section of the Meuse (from 'Sluis Weurt' till 'Sluis Heel') can be seen in Figure 5.3. This is the minimal cross section of the river and not of the (adjustable) civil structures that are in it. The depth of the water can be adjusted by dredging or by raising the water level.

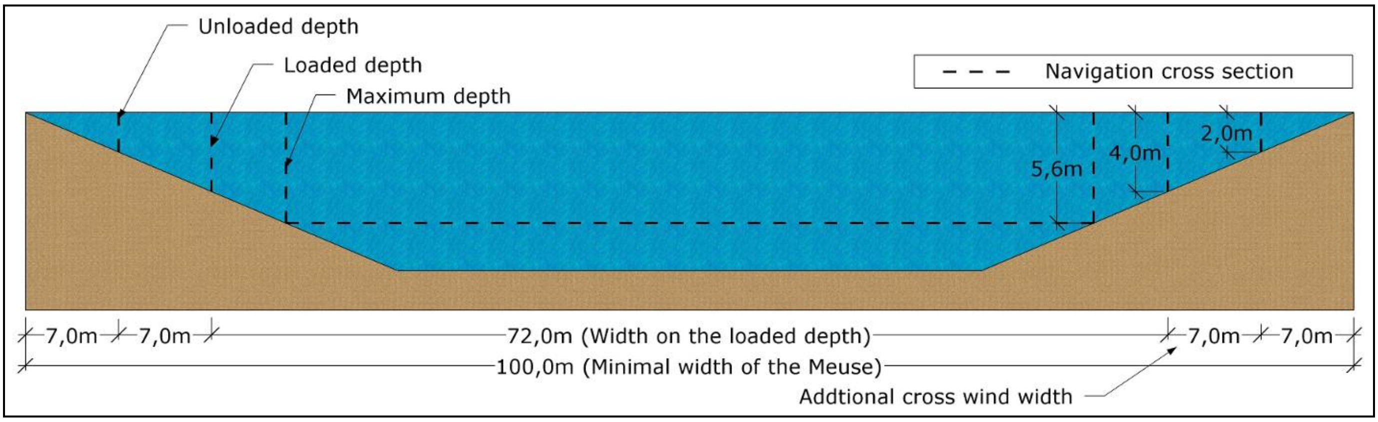

Figure 5.3: Navigation cross section of the Meuse from 'Sluis Weurt' till 'Sluis Heel' (m)

This is the maximum cross section that can be made on the basis of the measured minimal width of the Meuse (100 m), between 'Sluis Weurt' till 'Sluis Heel'. The soil is sandy around the Meuse and the maximum slope of sand combined with the water pressure and water flow in the Meuse leads to an assumed maximum slope of 16° (1:3,5). The values for the maximum depth and width that follows from this slope can be seen in Table 5.1. These dimensions are based upon the preferred cross section [Rijkswaterstaat AVV, 2005]. The unloaded width is larger than the loaded width, because unloaded ships needs a bigger drift angle to resist the cross winds.

Table 5.1: The minimal river cross section 'from 'Sluis Weurt' till 'Sluis Heel'

	Minimal river cross section		Maximal ship dimensions	
	Width (m)	Depth (m)	Width (m)	Depth (m)
Unloaded ship	86	2	18	2
Loaded ship	72	4	18	4

5.1.2. The Meuse route from 'Sluis Heel' till 'Sluis Ternaaien'

The Meuse route from 'Sluis Heel' till 'Sluis Ternaaien', has a smaller possible cross section. The 'Juliana kanaal' has the limiting width in this part. On the basis of the preferred cross section, the part from 'Sluis Heel' till 'Sluis Ternaaien' has a limiting ship dimension of 11,5 m width and 3,5 m depth [Rijkswaterwstaat Maaswerken, 2006]. However, it is possible that this part of the Meuse route is enlarged in the future, because at the moment, parts of the 'Juliana kanaal' are planned to be widened before 2020 (only 80 years after construction). Though, it will be more difficult to enlarge the part from 'Sluis Heel' till 'Sluis Ternaaien' than the part from 'Sluis Weurt' till 'Sluis Heel'. Therefore, the possibility that the upper part of the Meuse route will be enlarged is bigger.

5.2. 'Sluis Sambeek'

The navigation lock with the most passages each year in the Meuse route is 'Sluis Sambeek', because this is the first lock where the two transportation streams of the Meuse and the 'Maas Waalkanaal' come together. Therefore, this lock is selected as the location for the design of a functional flexible lock. In Figure 5.4 the layout of the lock complex 'Sluis Sambeek' can be seen and in Figure 5.5 an overview picture of the same lock complex is shown. In Table 5.2, the dimensions of the three locks that form the present lock complex 'Sluis Sambeek' are represented.

Table 5.2: The 'Sluis Sambeek' lock dimensions

	Lock chamber dimensions (m)			Lock head dimensions (m)		Year of
	Length	Width	Depth	Width	Depth	Construction
Lock 1	142	16	4.1	16	4.1	1970
Lock 2	142	16	4.1	16	4.1	1970
Lock 3	260	16	3.3	14	3.3	1929

By designing a new lock complex for 'Sluis Sambeek', also the design of a new lock complex for 'Sluis Belfeld' is made, because the 'Sluis Belfeld' lock complex has the same configuration as the 'Sluis Sambeek' lock complex.

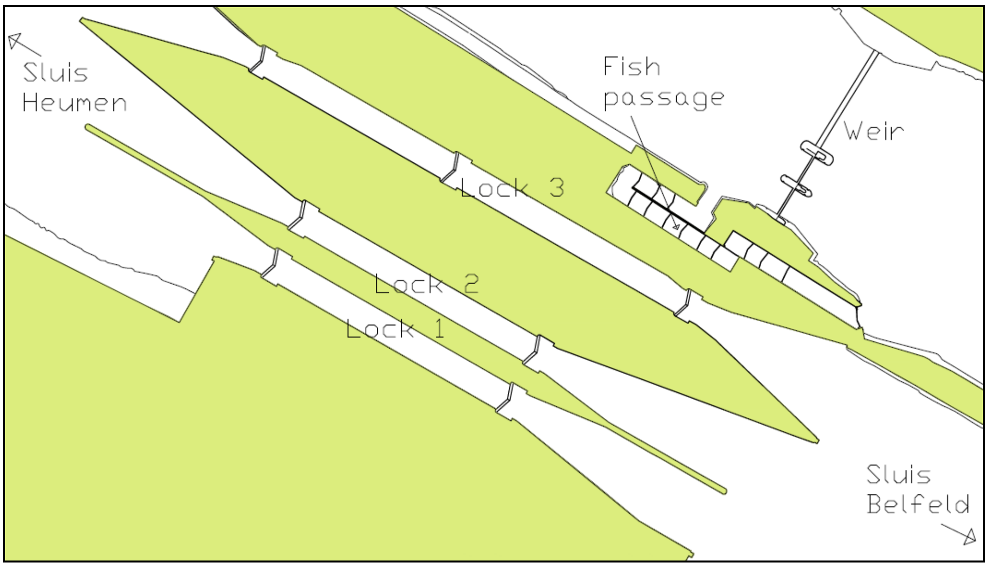

Figure 5.4: Layout of the lock complex 'Sluis Sambeek'

Figure 5.5: Overview picture of 'Sluis Sambeek'

Lock 1 and 2 were constructed in 1970 as a twin lock complex to support Lock 3. These locks are large enough to cope with CEMT-class Va vessels with a maximum depth of 3,5 m. The operating mechanisms of lock 1 and 2 are hydraulic and the lock chamber is filled and emptied by valves in the gate. The twin locks are in a good condition and with some minor reconstruction and maintenance, the locks still can be used for years. In Figure 5.6 a ship is navigating into one of the twin locks.

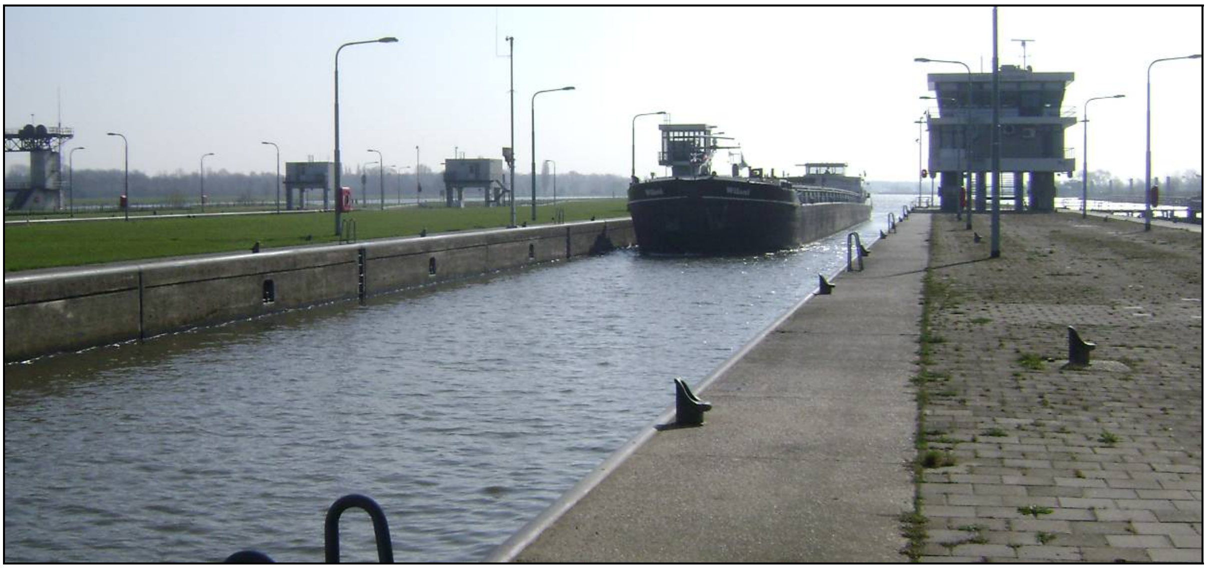

Figure 5.6: A ship navigates in one of the twin locks, from the upstream side

Lock 3 was already built in 1929 as a part of the Meuse normalisation to make the Meuse navigable. The lock consists of three pairs of mitre gates, as can be seen in Figure 5.4. Lock 3 has a lock chamber of 16 m wide, but the lock heads are only 14 m wide. Nowadays the lock chamber and the lock heads always have the same width, because the advantages of a bit more room in the lock chamber are not compensated by the disadvantages of the longer passing times, which result from the longer filling and emptying times. The depth of the lock is only 3,3 m, which means that ships can have a maximum depth of 2,8 m. The operating mechanisms of lock 3 are mechanical and needs to be greased every three weeks. In Figure 5.7 the panama wheel of the gate operation mechanism of lock 3 can be seen. The lock chamber is filled and emptied by longitudinal culverts, which are operated by vertical sliding gates. The intermediate lock head is not used anymore and the sliding gates of the culverts along this head are removed. Lock 3 is at the end of its lifetime and needs some major reconstructions to satisfy the present requirements.

Figure 5.7: The downstream Panama wheel of lock 3

In the reconstruction plans for lock complex 'Sluis Sambeek', lock 3 will be totally reconstructed. The lock chamber of lock 3 will be deepened and the 14 m wide lock heads will be replaced by two new 16 m wide lock heads. Furthermore, the walls of the lock chamber will be raised by 1,05 m.
Lock 1 and 2 will also be reconstructed, but on a smaller scale. Only the gates and the lock plateau between the twin locks will be raised by 0,3 m.
The heightening of the lock plateau and the walls is a consequence of the water level raising of 0,25 m in the head up section upstream 'Sluis Sambeek'. The water level is raised to create more depth and to protect the surroundings for drying up [Rijks-waterstaat maaswerken, 2006]. Another consequence of this water level rise is that the 80 year old weir will be equipped with new case panels, which still have to be placed by hand.

The reconstruction of lock 3 will be quite complex. Obstacles in the reconstruction are:

- The walls of the lock chamber have to be shored up after the chamber is deepened. This is a delicate job, because longitudinal culverts are integrated in the chamber wall.
- It has to be determined what has to be done with the intermediate head. This head has to be demolished and rebuild or replaced by a lock chamber wall. Also the longitudinal culverts have to be diverted along this new head or wall.
- The ends of the 80-year-old longitudinal culverts must be connected to the new lock heads to fill and empty the lock, or it has to be investigated whether the lock can be levelled up and down through the gates.
- Lock 3 is almost at the end of its structural lifetime. The normal lifetime of a hydraulic structure is 100 years and lock 3 is already 80 years old. Therefore, it is maybe cheaper to build a whole new lock.

In this report, this reconstruction plan of lock 3 (zero-alternative) is compared with two other alternatives. These alternatives are based upon the construction of a new functional flexible lock instead of restoring this 80-year-old lock.

6. Forecasting the trend in lock dimensions

To forecast the required lock dimensions for a lock in the Meuse route, two levels are distinguished, namely the international level and the regional level. The main focus in this forecast is on commercial shipping, because these vessels have a big economic value. Nevertheless, the recreational vessels are taken into account as well, because they also have to pass the locks in the Meuse route.
First the ship sizes and intensities are predicted at international and at regional level (paragraph 6.1 and 6.2). After that the ship intensities are checked (paragraph 6.3). Finally a prediction of the required lock dimensions for the next 100 years is performed (paragraph 6.4). The forecasted trend is supported by the tables that are shown in Appendix II.

6.1. International predictions in ship sizes and intensities

The development in size of the Dutch inland navigation fleet and the annual quantities of the Dutch water transport are used to produce an international inland navigation forecast. This is a representative sample to approach the total inland navigation fleet of Europe, because the Dutch inland navigation fleet represents 49% of the total European fleet [Groeneveld, 2002]. In the last decades, the total number of commercial inland navigation ships has decreased. Especially the number of small vessels has declined, as shown in Figure 6.1. In contrast to the decrease of small vessels (CEMT – classes 0, I, II and III), the number of large vessels (CEMT – classes V, VI and VII) has slowly increased. This scale enlargement is a result of a desired cost reduction in cargo transportation. Large vessels have a lower fuel consumption and less labour costs for each unit of cargo than smaller scale vessels and modalities. The different ship classes can be found in Appendix I.
Furthermore, the amount of cargo that is transported over the Dutch waterways also shows an increasing trend in ton-km, as can be seen in Figure 6.2. The logical conclusion of these two graphs is a growing number of large vessels and a more efficient utilisation of the available ship capacity in the future.

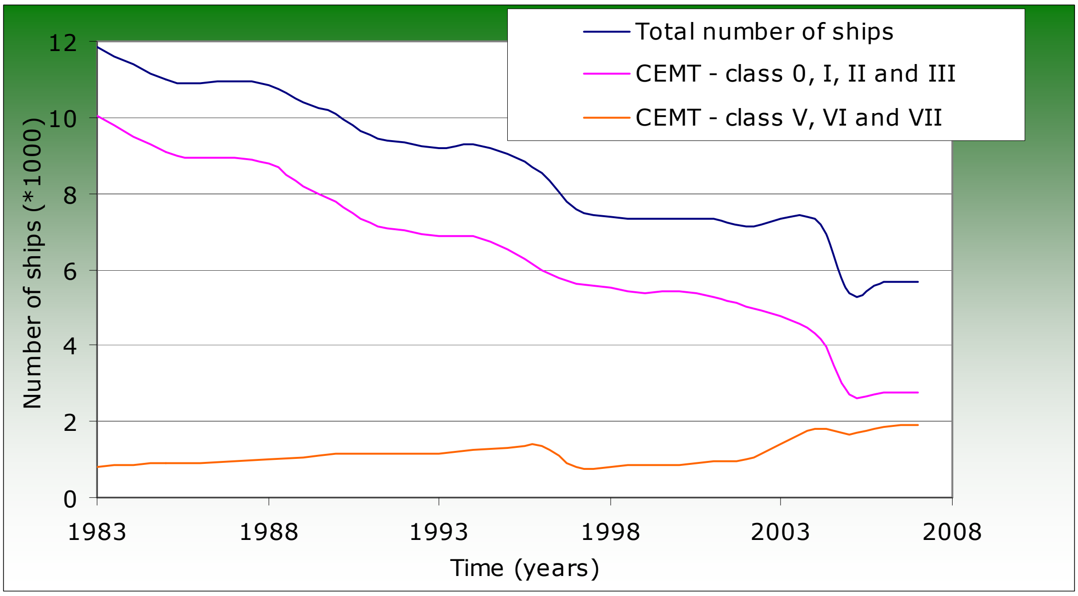

Figure 6.1: Ship size development in the Dutch inland navigation fleet [CBS/RWS]

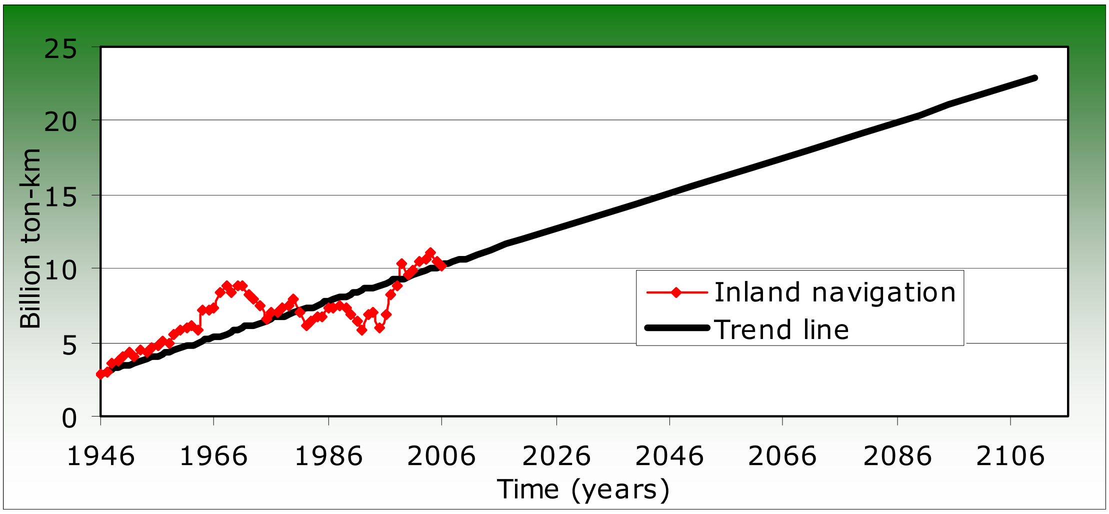

Figure 6.2: Forecast of ton-km for the next 100 years [Mom, 2008]

It is not possible to forecast the growth in the different types of large vessels, because statistics of these ships are only available from 2004 till now. Since the beginning of the 21^{e} century, a new CEMT-class VIa is introduced on the Rhine. This 'Rhine max ship' is, besides the push tow barges, the biggest ship on the Rhine. This new type of Rhine ship has the maximum size of 17,4 m wide, 146 m long and a maximum loaded depth of 4 m. Table 6.1 shows the number of new build CEMT – class VIa ships that are added to the Dutch fleet every year from 2004 till 2007. Though a good prediction is not possible with these data, the fact that these new large scale vessels are still built every year shows that the 'Rhine max ship' becomes more common in the future.

Table 6.1: New build VIa ships added each year [Expertise- en innovatiecentrum Binnenvaart, 2008]

Year	VIa
2004	20
2005	12
2006	14
2007	19

The pushed convoys with standardised barges are also a type of water transport that is used more and more. These barges (Europe II barges) have a standard width of 11,4 m and a length of 76,5 m. These barges can be coupled to form a unity of one, two or four barges. This kind of water transportation can be used for container or bulk transport.

Recreational vessels are not taken into account at international scale, because it is not known how much recreational vessel are active on the European waterways. Therefore, the influence of the recreational vessels on the lock dimensions will only be included at regional scale.

6.2. Regional ship sizes and intensities forecast ('Sluis Sambeek')

The prediction of the size and the number of ships that will pass the Meuse route for the next 100 years gives an uncertainty that is unavoidable, because this forecast depends on unpredictable factors and the prediction has to be made for a relatively long period. A few examples of unpredictable factors are the possible attraction that a larger lock will have, the development of the harbours in the region and the public pressure on environmental friendly transport.
To make a good prediction of the ship sizes and the intensities, the forecast is based on available transportation data from the past as well as on the visions and forecasts of the different involved governments and companies.

6.2.1. Forecast changing visions

Currently, the Meuse route is reconstructed to make it accessible for CEMT – Class Vb ships (two barges long). The reconstruction is planned to be finished in 2020. A reason for the modernisation of the Meuse route is improving its competitiveness and boost up the possibilities of durable transport in the Netherlands and its surrounding countries. When the Meuse route can handle larger scale vessels, it can play a bigger role in the 'Benelux Vaarwegruit' (Figure 6.3). The 'Benelux Vaarwegruit' is a waterway network which connects Antwerp, Rotterdam, Germany and East – Belgium.
To connect the extending harbour of Liège to the Meuse route, it is important to enlarge the locks of Ternaaien to facilitate a good ship passage from Maastricht to Liège for larger vessels. Thus, when the locks in Ternaaien are enlarged, inland navigation vessels are able to reach Liège and its hinterland by using the Meuse route instead of the 'Albert kanaal'. Moreover, it is assumed that the inland navigation harbours along the Meuse route will also be enlarged in the future.
All these reconstructions and new transport possibilities are assumed to give the Meuse route a new impulse and will help to maintain the growth of the annual transported tonnage through 'Sluis Sambeek'.

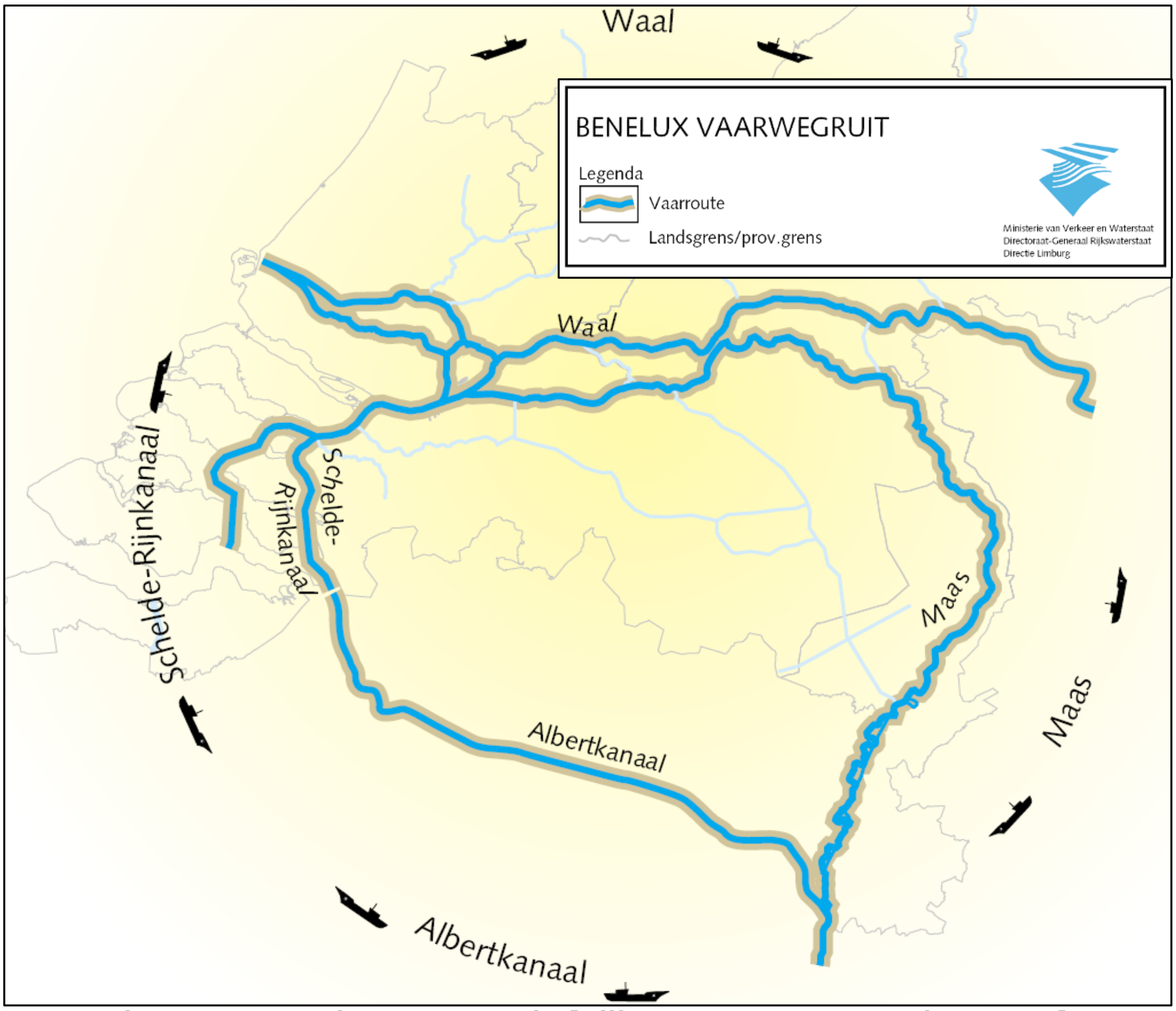

Figure 6.3: 'Benelux Vaarwegruit' [Rijkswaterstaat maaswerken, 2006]

In the future, it is possible that the dimensions of 'Sluis Sambeek' will be large enough to handle ships that are too wide to pass the 'Juliana kanaal'. It is not strange to consider a widening of the 'Juliana kanaal' in this case, because the this channel will already be widened in the next few years (80 years after construction) to facilitate the passage of CEMT – class Vb ships (paragraph 5.1). It is possible then that the 'Juliana kanaal' is widened again in 50 to 100 years and could have the same width as 'Sluis Sambeek' then. It is beyond the scope of this study to take this widening into account.

6.2.2. Number of ships and transported tonnages

The total number of commercial inland navigation ships that will pass the locks of 'Sluis Sambeek' is expected to decrease. This is the result of the scale enlargement in the European inland navigation fleet (paragraph 6.1). In contrast to the commercial ships, the number of passing recreational ships is expected to grow in the following years. This is expected because the number of wealthy people, that will have their retirement in the near future, is increasing and they are expected to spend a part of their leisure time on the water [Provincie Limburg, 2008].
The resulting forecasts for the next 100 years can be seen in Figure 6.4. These predictions are based on ship counts from 1983 till 2007 [CBS/RWS] and the expectations, based on the same ship counts, of the province of Limburg [Provincie Limburg, 2008].
The number of commercial ships is expected to decline rapidly. It is expected that the trend line will stabilise around the 24.000 commercial ships. This forecast is comparable with the prognosis of the province of Limburg. Stabilisation is expected for the increasing number of recreational vessels as well. The number of passing recreational vessels is also estimated to stabilize around the 24.000 ships. In paragraph 6.3 it will be shown that the existing lock facilities can handle this expected intensity for the next 100 years.

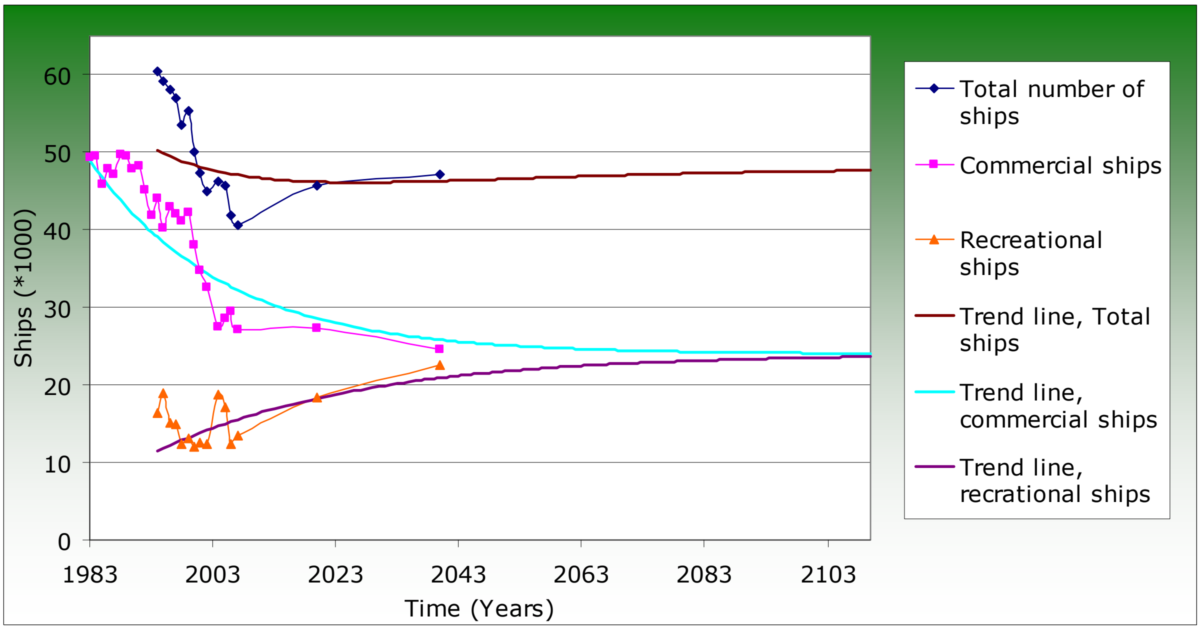

Figure 6.4: Forecast in annual number of ships that pass 'Sluis Sambeek'

In contrast to the declining number of commercial ships, it is expected that the annual amount of transported tonnage passing 'Sluis Sambeek' will grow. This expectation is based on the transport volumes passing 'Sluis Sambeek' in the past as well as on the expectations of the province of Limburg [Provincie Limburg, 2008]. The forecast, as can be seen in Figure 6.5, shows a trend that is related to the annual transported volume through the Netherlands. This gives a more reliable prediction, because the national transported volumes are known from 1946 till now [Mom, 2008] while the annual transport volumes passing 'Sluis Sambeek' are only known from 1995 till now.

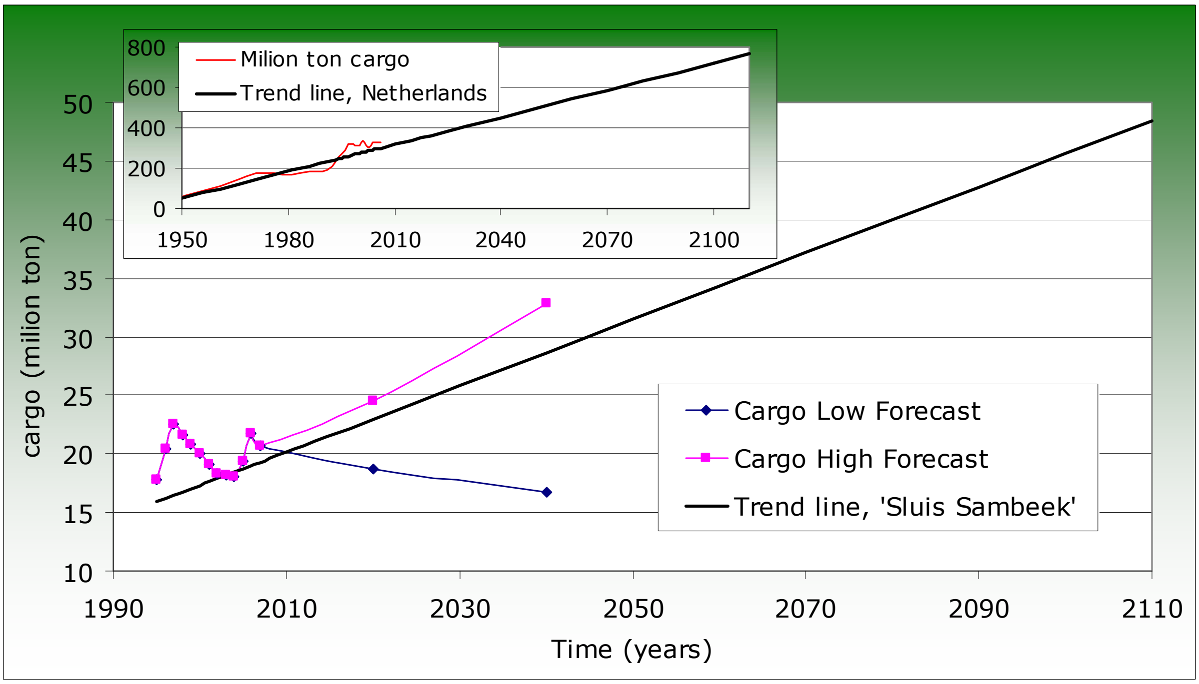

Figure 6.5: Forecast of the annual transported tonnage passing 'Sluis Sambeek' and the Netherlands

Following Figure 6.4 and Figure 6.5, it can be concluded that the maximum ship sizes are normative for the future lock dimensions.
Furthermore, an assumption is made about the growth in ship passages for different types of ships in Figure 6.6. It is difficult to predict what will happen in 100 years, because the different types of ships are only counted from 1998 till now on regional scale ('Sluis Sambeek'). However, in combination with the trend in the Dutch inland navigation fleet (Figure 6.1) and the total number of commercial ships, a forecast has been produced.

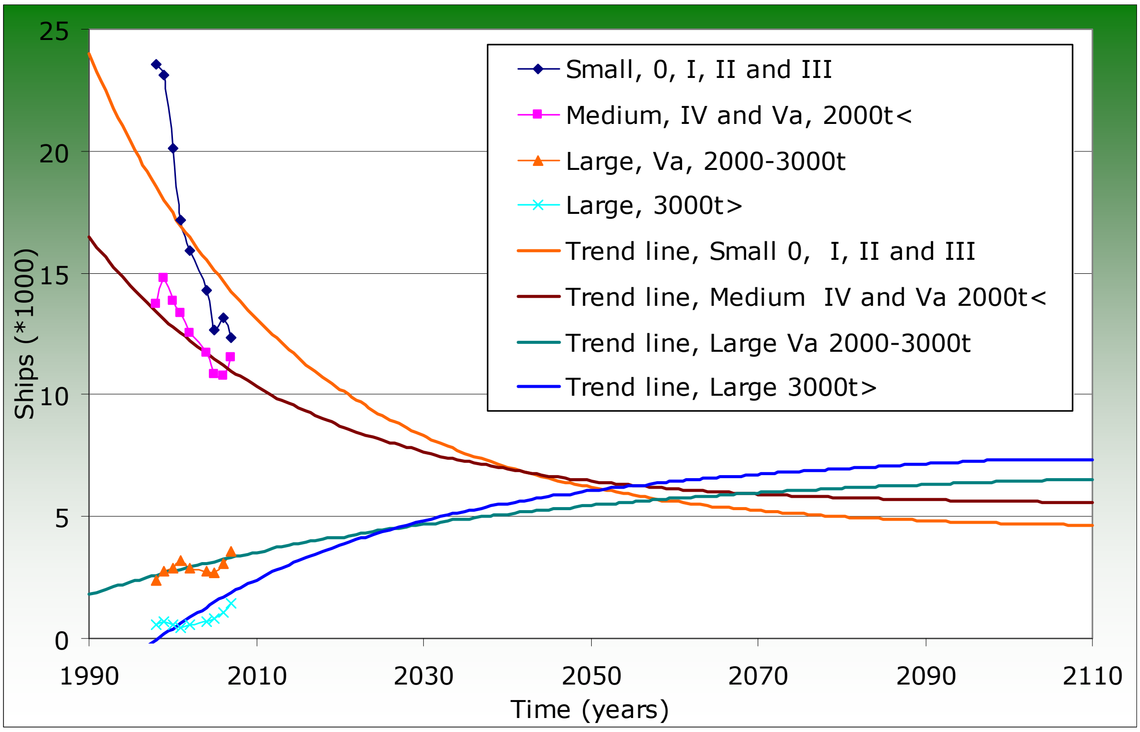

Figure 6.6: Forecast in ship passages through 'Sluis Sambeek' divided by CEMT class

It is not possible to make a good prediction of the number of passages of large vessels, because the different ship passages above the 3000 ton are not distinguished. The only reliable data to produce a forecast is the growth in containers and container ships. Therefore, a trend in container transport is forecasted in the next paragraph.

6.2.3. Container transport forecast

The container capacity of the inland navigation harbours along the Meuse shows a steady growth. It is planned to extend the harbours of Wanssum, Born and Stein and a new inland container terminal is opened in Venlo at the beginning of 2009. These harbour extensions, combined with the expected growth in export via the Meuse to Belgium, guarantee the growth in throughput of containers for the next years.
To forecast the container transport passing 'Sluis Sambeek', the expectation of the province of Limburg that is made in 2005 is used. However, this expectation is conservative, because in 2007 the number of transported containers through this lock was already higher than the low expectation for 2020 [Provincie Limburg, 2008]. Besides the expectations of the province, also the statistic data that was available for 'Sluis Sambeek' was used [CBS/RWS].
In Figure 6.7 the forecast in the number of Twenty feet Equivalent Unit (TEU) per ship passing 'Sluis Sambeek' can be seen. The black trend line is used to make a prognosis about this development. The uncertainty of the forecast is represented by the red triangle. The trend line in Figure 6.7 is formed by the forecast in the average number of TEU per ship divided by a load factor of 70%. This load factor is based on the assumption that container ships rarely navigate unloaded. Therefore, the load factor of container ships is higher than the average load factor of inland navigation ships of 60%.
The forecast of the container transport is based on the average ship size. This means that every smaller ship is compensated by a bigger ship and vice versa. Consequently, the maximum container vessel is bigger than assumed by the forecast based on the average ship size. As a consequence this forecast will be conservative, so it is assumed that the increase in ship size will even be larger.
Besides the forecast trend line, also the TEU capacities for a few ship types are drawn in the graph. When the trend line crosses a ship line in the graph, this type of ship is not sufficient anymore to provide enough transport capacity.

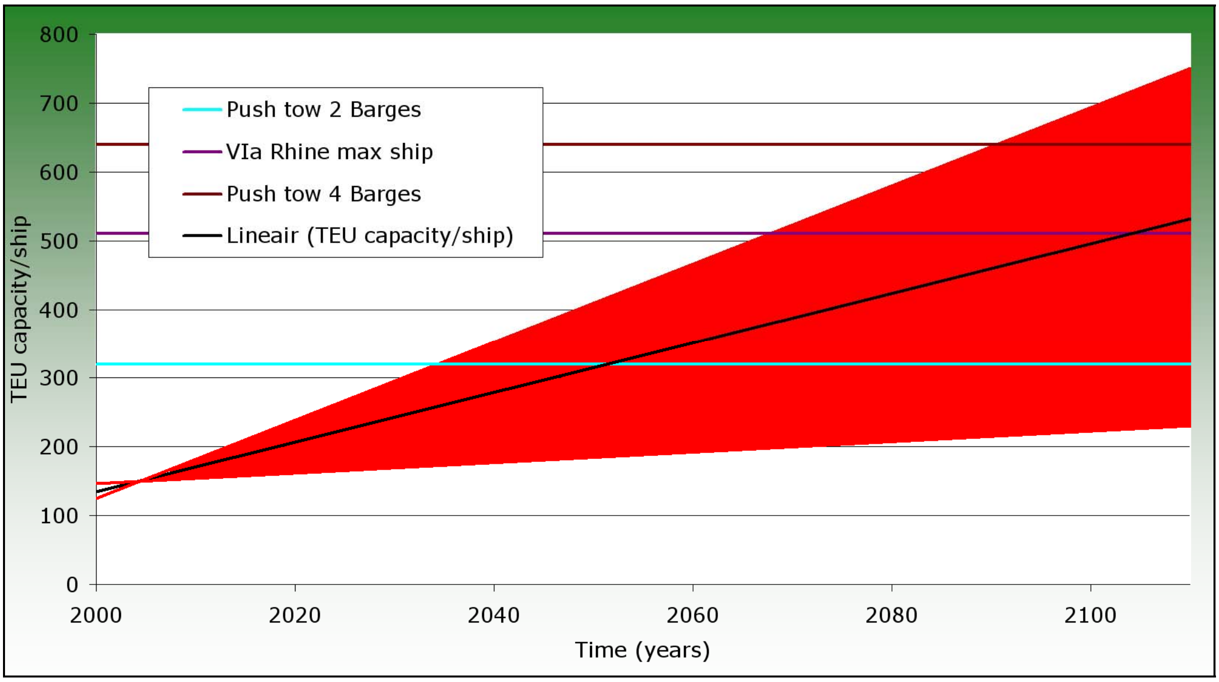

Figure 6.7: Forecast of the trend in the TEU capacity/ship

6.3. Intensity

In paragraph 6.1 and 6.2 it was concluded that the maximum ship sizes are decisive for the future lock dimensions of 'Sluis Sambeek' because the intensities will be at the same level or will even drop. In this paragraph it is checked whether the present number of locks of the lock complex is able to cope with the future intensities. This check is performed by using the 'Queuing theory' [Groeneveld, 2001]. This theory is explained and applied in Appendix II. The intensities are derived from the forecast in ship passages of Figure 6.6 and the ship frequency measurements of 2007 [Rijkswaterstaat DVS, 2008].
The resulting waiting times and the chance that an arriving ship has to wait are represented in Table 6.2. In 2007, the passing times of the whole complex are calculated with a locking time of 12 minutes [Burhenne, 2009] for the three locks together. In 2050 and 2110 the passing time of lock 1 and 2 are analysed separately from lock 3. Lock 1 and 2 will still have a locking time of 12 minutes. Lock 3 has an assumed locking time of 18 minutes.

Table 6.2: Expected average waiting times and the chance that a ship has to wait

	Average waiting times (min)	Locking time (min)	Chance that an arriving ship has to wait	Passing time (min)
2007 (Three locks)	1,3	12,0	14%	13,3
2050 (Two existing locks)	0,6	12,0	8%	12,6
2050 (Flexible lock)	4,7	18,0	21%	22,7
2110 (Two existing locks)	0,6	12,0	8%	12,6
2110 (Flexible lock)	6,0	18,0	25%	24,0

When the passing time is less than 30 minutes it is acceptable and when this time is between 30 and 45 minutes it is critical but acceptable [Glerum, 2000]. In Table 6.2 it can be seen that the passing time remains below the acceptable time of 30 minutes. Therefore, it can be concluded that the intensity is not an issue in this forecast.

6.4. Trend forecast in the Meuse route lock dimensions

In the previous paragraphs was concluded that the ship sizes are normative for the future lock dimensions. Therefore, only lock 3, the future functional flexible lock, of the lock complex 'Sluis Sambeek' is taken into account (paragraph 5.2).
The maximum dimensions of the ships in Figure 6.7 can be seen in Table 6.3. These dimensions are used to make a forecast of the expected required lock dimensions for the next 100 years. The maximum Rhine vessels (VIa) can load 5 layers of containers, but at this moment the Meuse route is only navigable for ships with 4 layers. However, it is assumed that in about 50-100 years the bridges are raised. Therefore, 5 layer vessels will be able to pass the Meuse then.

Table 6.3: Ship and lock dimension of large scaled vessels [Glerum, 2000]

CEMT class	Ship type	Capacity (TUE/ship)	Max ship size (m)			Lock size (m)			Normative year	Expected first year of arrival
			Width	Length	Depth	Width	Length	Depth		
Vb	Push tow 2 Barges	320	11,4	195	4	12,5	210,0	4,7	2051	2020
VIa	Rhine max ship	510	17,4	146	4	19,2	164,0	4,7	2104	2052
VIb	Push tow 4 Barges	640	22,8	195	4	24,0	220,0	4,7	2140	2105

Besides the maximum ship dimensions, the corresponding minimum lock dimensions are also represented in Table 6.3. These lock dimensions will have to be larger, because the decrease in ship intensity and the increase in annual transported tonnage will lead to an increase in ship size (see Figure 6.8). The normative year that is mentioned for each lock size in Table 6.3 shows the year that the lock usage will be on its peak.

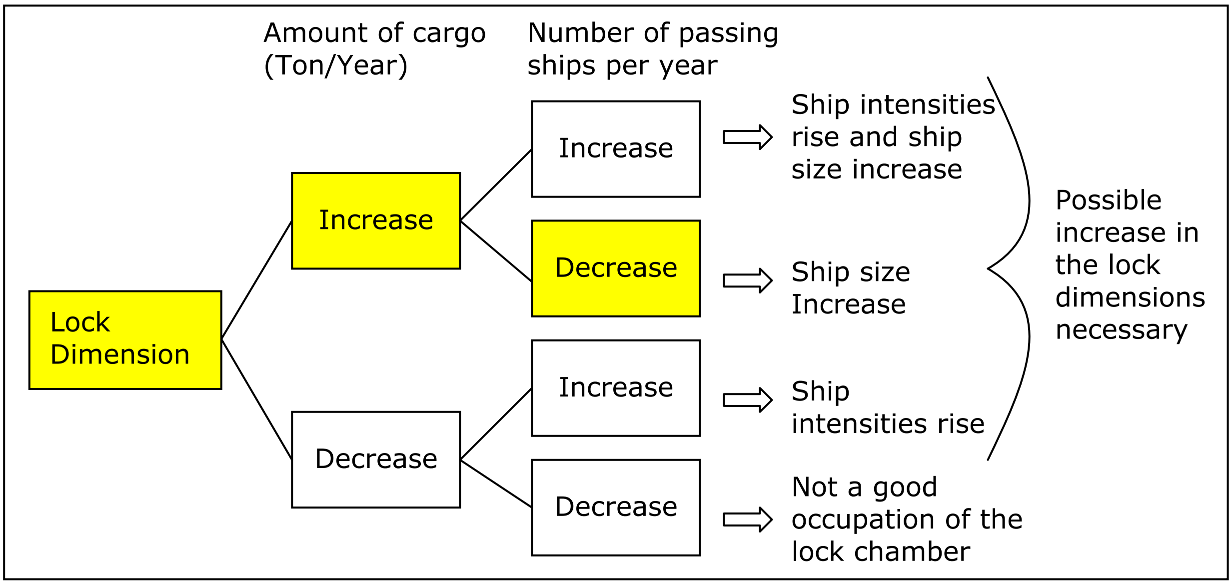

Figure 6.8: The effects of the ton/year and the number of commercial ships on the lock dimensions

The different required lock dimensions for each year are shown in Figure 6.9. The figure shows that the width will be the most logical flexible dimension, because the required width shows a steady increase and the other two dimensions do not fit in a predictable trend. Almost no increase in depth is necessary in the future, because the depth of inland navigation ships is limited by the river depth.
In the European inland navigation fleet, the longest ships are the CEMT - class Vb and VIb push tow barges. The maximal length of these vessels is 195 m. At the moment, it is planned that all the locks in the Meuse route will have a length of 225 m in 2020 to fit these vessels [Rijkswaterstaat Maaswerken, 2006]. The extra (225-195 =) 30 m is necessary to lock two CEMT – class Va ships at the same time. It is assumed that the length of the ships will not increase anymore in the next 100 years. Because, lock 3 of 'Sluis Sambeek' is already 260 m long, no flexible enlargement in length is necessary.

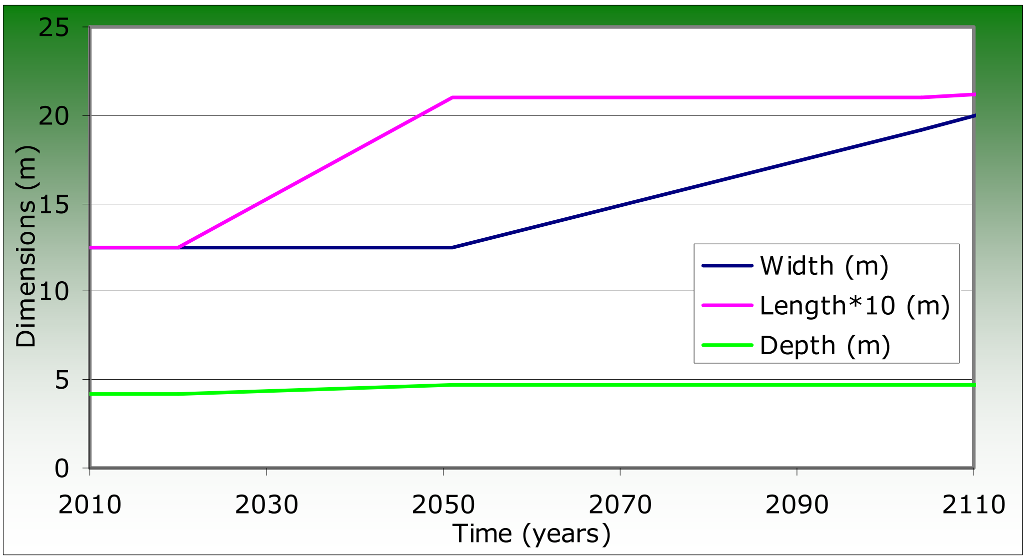

Figure 6.9: The expected required development in lock dimensions for 'Sluis Sambeek'

In Table 6.3 also an expected first year of arrival is given for the different types of ships. From that moment on, the required lock dimensions must be available to ensure a good

ship passage. According to the forecast, the reconstruction of the lock has to be completed at that moment.
In Figure 6.10, two types of lock width developments are combined with the trend forecast in the navigation lock width of 'Sluis Sambeek'.

- The lock size without rebuilding (the *once* path, see paragraph 4.1) will facilitate a good ship passage for the next 100 years. This lock will be build big enough for the large vessels that are expected to navigate on the Meuse in about 40 years.
- The lock size with rebuilding (the *twice* path, see paragraph 4.1) will facilitate a good ship passage for the next 40 years. After that, the lock can be reconstructed or rebuild.

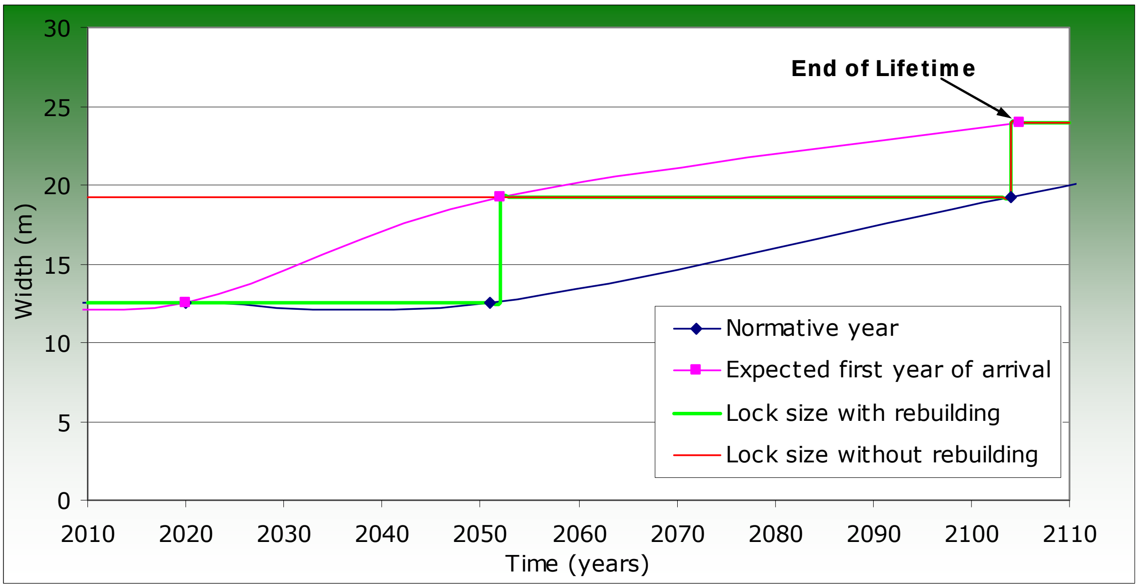

Figure 6.10: Trend forecast in the navigation lock width of 'Sluis Sambeek'

According to the forecast, another lock rebuilding is necessary around 2105 to be able to cope with CEMT-class VIb ships. However, it is not plausible that a lock with this CEMT -class VIb dimensions (24 m wide) will be build, because the Meuse is too small to cope with this type of ships. Therefore, the largest ships that will pass 'Sluis Sambeek', on the basis of the preferred cross section, have a width of 18 m (paragraph 5.1).

On the basis of this chapter, the final dimensions of the functional flexible lock are specified. In paragraph 5.1 it was shown that the maximum ship dimensions of the minimal Meuse cross section are 18 m wide and 4 m depth. The forecasted ship dimensions for 2052 are 146 x 17,4 x 4 m (Length x Width x Depth). To make use of the whole capacity of the Meuse, it is logical to assume a maximum ship dimensions of 146 x 18 x 4 m (Length x Width x Depth) in 2052. In combination with the 225 m length that is necessary to lock two CEMT-class Va ships at same time, this leads to the in Table 6.4 shown assumed lock dimensions for 2020 and 2052.

Table 6.4: The resulting functional flexible lock dimensions

Lock 3	Min. Lock dimensions (m)		
	Width	Length	Depth
2009 (Existing)	14	260	3,3
2020 (Flexible lock)	12,5	225	4,7
2052 (Flexible lock)	19,8	225	4,7

It has to be noticed that the forecasts in this chapter are uncertain, because a small change in the future can have a big effect on the future trend in shipping traffic. In this report, a forecast is done in a relatively short time. Nevertheless, it provides a good indication of the future developments in lock dimensions on the Meuse route.

7. 'Basis of Design'

In this chapter, the boundary conditions, the assumptions, and the basic data are determined and implemented into the program of requirements. These requirements will be the basis of the functional flexible lock designs in this report.

7.1. Boundary conditions

7.1.1. Location

In Chapter 5 an illustration and a description of the lock complex 'Sluis Sambeek' can be found. The complex consists of three locks of which only the oldest lock (lock 3) will be reconstructed or rebuild. In Figure 7.1, the system boundaries are represented by the red system border. The total length of the design location is about 470 m and the average width is 115 m. The smallest part is near the fish passage (85 m).

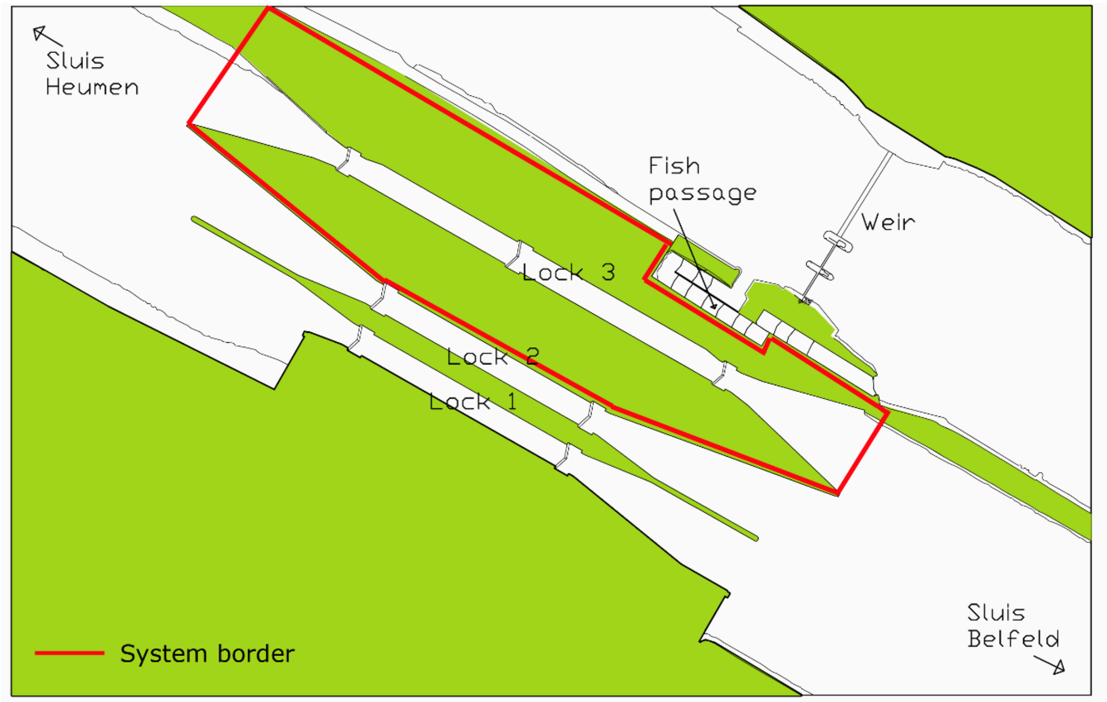

Figure 7.1: 'Sluis Sambeek' system boundaries for the functional flexible lock design

7.1.2. Hydraulic boundary conditions

The river profile
The minimum profile of the Meuse route can be seen in Figure 5.3. This profile allows a maximum ship size of 18 m wide and 4 m depth.

The river depth
The river depth that is required for a vessel depth of 4 m is 5,6 m. To determine the minimum bottom level, the different water levels at the head up sections upstream and downstream 'Sluis Sambeek' can be seen in Table 7.1.

Table 7.1: The average water levels in front of the upper and the lower head [Rijkswaterstaat Maaswerken, 2006]

	Water level (m + NAP)			
	Discharges lower than 600 m³/s		Discharges higher than 600 m³/s	
	Old	New	Old	New
Upstream 'Sluis Sambeek'	10,85	11,10	10,85	10,85
Downstream 'Sluis Sambeek'	7,70	8,10	7,70	7,70

The water height of 11,1 m + NAP upstream and 8,1 m + NAP downstream can be maintained most of the time, because of the standard discharge of 10 m^3/s [laag water in de Rijn en Maas, 2007]. Only when the discharge is higher than 600 m^3/s, the water height is lowered to 10,85 m + NAP upstream and to 7,7 m + NAP downstream. The new water levels are used as the representative water levels. Therefore, the corresponding river bottom levels are 5,5 m + NAP upstream and 2,5 m NAP + downstream (5,6 m river depth).
The annual average discharge can be seen in Figure 7.2. In this figure, it is shown that 88% of the time (321 days in a year) the water level can be kept on the new representative level. When the discharge is higher than 1000 m^3/s, the weir is lowered and the ships can navigate over the weir. On average this situation occurs only 3,5% of the time (13 days in a year).

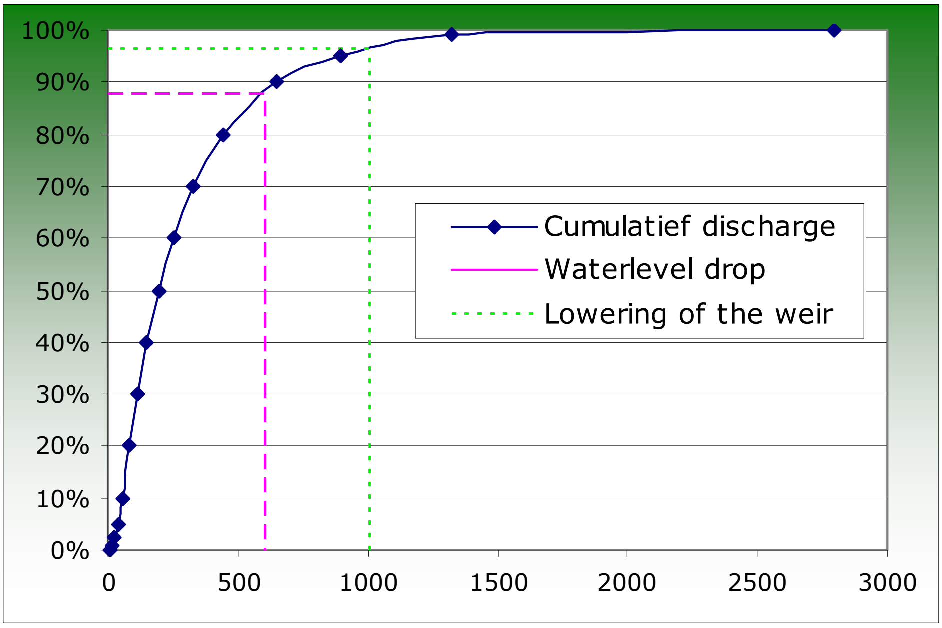

Figure 7.2: Annual average discharge of the Meuse, near 'Sluis Sambeek'

The water management function
The lock has no primary water retaining function, because the lock is not situated in a dike ring but in a river. However, the water retaining function has similarities with the water management function, which is needed to maintain a navigable depth in the head up section upstream of 'Sluis Sambeek'.
Less than 2,5% of the time too less water is available to level the ships up and down freely, so in this case the lock chambers have to be filled totally with ships, before the locking process can take place, to save water.
All lock complexes along the Meuse are expected to be flooded once in the 10 years. Therefore, these locks must be able to resist the currents that will occur during this flooding.

7.1.3. Soil conditions (including the groundwater table)

The new lock will be build at the location of the old lock. Therefore, the averages of two drillings and cone penetration tests near this lock are combined to form a soil profile. This soil investigation was performed by 'Mos Grondmechanica B.V.' [Thijsen, 1999]. The soil types and their corresponding properties can be found in Table 7.2. In Appendix III, the bottom profiles at the intermediate gates and at the up- and downstream head of the old lock are shown.

Table 7.2: Soil types and the corresponding properties

Soil types		γ_{dry}	γ_{wet}	φ'_{rep} [1]	c'_{rep} [1]	K_0	K_a	K_p
Soil nr	Soil type	kN/m^3	kN/m^3	°	kPa			
1	Fill sand[1]	17,0	19,0	30	0	0,50	0,33	3,00
2	Clay	13,9	18,0	22,5	10	0,62	0,45	2,24
3	Gravel[1]	18,0	20,0	35	0	0,43	0,27	3,69
4	Sand	16,6	20,1	32,5	0	0,46	0,30	3,32
1	Derived from [Molenaar, 2006]							

The maximum, minimum and average ground water tables resemble the upstream and downstream water levels, because the ground is relatively permeable.

7.2. Assumptions and basic data

7.2.1. Assumptions

The assumptions on which this report is based are listed below.

- On the basis of Chapter 6 it is assumed that the ship sizes are representative for the lock dimensions.
- The ship intensities do not have an impact on the lock dimensions (Chapter 6).
- Because the ship sizes are representative for the lock dimensions, only lock 3 is taken into account for the functional flexible lock design. The twin locks (lock 1 and 2) are only considered for a temporarily traffic diversion when lock 3 is obstructed for construction, reconstruction or maintenance.
- The only flexible dimension that is considered is the width. (Chapter 6).
- In the 'Tracébesluit Zandmaas/Maasroute' [Rijkswaterstaat Maaswerken, 2006] it is stated that lock 3 of 'Sluis Sambeek', must be made 16 m wide to handle the intensities. However, this graduation study shows that no extra capacity is needed and a new lock of 12,5 m wide is big enough to accommodate the ship passages for the next 40 years (Vb ships). Furthermore, it is shown that a width of 16 m is also not sufficient to make it possible to navigate VIa ships through the lock.
- It is assumed that it is not necessary to dewater the lock chamber. In contrast to the lock chamber, the gate chamber must have a dewatering possibility for the maintenance of the pivot, the gate and the sill.

7.2.2. Basic data

The materials used

In the next paragraph, three alternatives are elaborated. To be able to compare these alternatives, the same material properties are used as much as possible in the designs. The most significant materials and their corresponding properties are mentioned in Table 7.3. The concrete cover is assumed to be 40 mm.

Table 7.3: The used materials and their properties [ACL, 2004]

Material	Type of material	Weight (kN/m^3)	Stiffness (N/mm^2)	Tension strength (N/mm^2)	Pressure strength (N/mm^2)	Max Reinforcement
Concrete	B35	24	31000	1,4	21	1,94%
Underwater concrete	B35	23	31000	1,4	21	1,94%
Steel (Gates)	S235	78,5	210000	235	235	
Reinforcement steel	FEB500	78,5	210000	435	435	

The required lock dimensions
The ship sizes and their corresponding lock sizes are shown in Table 7.4. At this moment, Va and 2x Va ships are able to navigate on the Meuse. In 2020, Vb ships and in 2052 VIa ships will also navigate on the Meuse. VIb ships will not navigate on the Meuse, because the Meuse cannot cope with these ships. Therefore, VIb ships do not have an influence on the lock dimensions of 'Sluis Sambeek'.

Table 7.4: Ship sizes and the corresponding lock sizes

CEMT class	Ship type	Max ship size (m)			Lock size (m)		
		Width	Length	Depth	Width	Length	Depth
Va	Push tow 1 Barge	11,4	110	3,5	12,5	125,0	4,2
2 x Va	2x Push tow 1 Barge	11,4	220	3,5	12,5	225,0	4,2
Vb	Push tow 2 Barges	11,4	195	4	12,5	210,0	4,7
VIa	Rhine max ship	17,4	146	4	19,2	164,0	4,7
VIb	Push tow 4 Barges	22,8	195	4	24,0	220,0	4,7

Because, the ship intensities will become less in the future, the passing times will be within the norm (paragraph 6.3).

7.3. Program of requirements

The requirements are formulated on the basis of the 'Richtlijnen Vaarwegen RVW 2005' [Rijkswaterstaat AVV, 2005], 'Ontwerp van schutsluizen' part 1 [Glerum, 2000] 'Tracébesluit Zandmaas/Maasroute' [Rijkswaterstaat Maaswerken, 2006], the data that is provide by 'Rijkswaterstaat Limburg' [Burhenne, 2009] and the assumptions derived from the forecasted trend in Chapter 6.

7.3.1. Functional requirements

Inland navigation

Normative ship size and the corresponding lock size (Table 7.5)

Table 7.5: The normative ship sizes and the corresponding minimum lock sizes

Normative year	Max ship size (m)			Lock size (m)		
	Width	Length	Depth	Width	Length	Depth
2020	11,4	195	4	12,5	225,0	4,7
2052	17,4	146	4	19,2	225,0	4,7

Capacity
The capacity of the lock complex 'Sluis Sambeek' is big enough to handle the expected intensities. This was shown in Table 6.2.

Coping height above maximum water level
The height of the coping in the lock chamber must be 12,8 m + NAP according to the requirements for the lock 3 reconstructions.

The Halt line
The halt lines on the up- and downstream side are 3,5 m from the gate recess in case of a class V or a class VIa lock.

Water management (weir)

Handle the maximum river discharge
The maximum river discharge is 3650 m^3/s. This discharge will occur once in the 1250 years [Ministerie V&W, 1998].

Keep the water level navigable
The water level is always navigable during low water, because of a guaranteed discharge of 10 m^3/s. Only when the discharge is higher than 600 m^3/s, the water level is dropped to create a buffer (Table 7.1). When the water level at 'Sluis Belfeld is higher than 17,25 m + NAP no navigation is allowed anymore. The corresponding discharge of this water level is 2000 m^3/s [Ministerie V en W, 1998]. This discharge has a return period of once the in 10 years.

7.3.2. User requirements

Navigation lock

Water levels
The sill must have enough depth to let pass the representative ships 98,5% of the time. Ships with a depth of 4 m can pass in 91,5% of the time and ships with a depth of 3,5 m can always pass the lock or the weir. Only when the Meuse is not navigable, because the water level is higher than 17,25 m + NAP at 'Sluis Belfeld', 'Sluis Sambeek' cannot be passed. This situation occurs only once in the 10 years. In Table 7.6, the upstream and downstream lock levels that will be used to reconstruct 'Sluis Sambeek' can be seen.

Table 7.6: Lock levels of 'Sluis Sambeek' (m + NAP)

Lock levels	Upstream	Downstream
Minimum lock level	10,85	7,70
Average lock level	11,10	8,10
Maximum lock level	12,80	9,80

The size of the lock chamber
The lock width must be at least 1,1 times the maximum ship width. The length must be at least 1,12 times the maximum ship length. The depth of the lock must be 0,7 m deeper than the maximum ship depth in case of a CEMT – class V or VI ship.

The lock passing time
The lock passing time is required to be less than 30 minutes. In exceptional cases, for example during reconstruction, passing times of 45 minutes are allowed.

Service time
142 hours (till 2020).
168 hours (from 2020 on).

Availability
No specific requirements about the availability of the navigation lock are known. The availability of the lock complex is spread over three locks. Nevertheless, the times that one of the locks is not available must be restricted to a minimum.

Mooring facilities in the lock
In the longitudinal direction of the lock chamber, every 15 m a bollard (recess) has to be placed. The maximum height of a bollard (recess) is 1,75 m above the minimum lock level and the highest bollard (recess) must be just under the lock plateau. The maximum bollard forces are 280 KN [Molenaar, 2006].

The deformations of the lock wall and floor
The deformations of a permanent sheet pile wall must be smaller than 1/200 of the length of the wall. For temporarily sheet pile walls, a deformation of 1/100 of the length is allowed [CUR, 2005].
The deformation of the lock head floor must be restricted to 3 cm, otherwise the gates cannot open anymore. For the lock head walls, a standard deformation requirement of 1/300 of the length is used.

Lifespan
The normal lifespan of a hydraulic structure is 100 years. Therefore, the structural lifespan of the new lock is 100 years as well.

Lock approach

Free space
The free space that is needed to slow down and to moor is 2,5 times the ship length.

Standby berths
The length of the standby berths must be 1,3 times the lock chamber length.
The width of the standby berths is the same as the width of the lock chamber.

Waiting berths
When it is expected that ships have to wait longer than one locking time before they are locked, waiting berths are needed. These berths have the same width as the lock chamber.

Guide walls
The guide walls protect and lead the ships to the lock head and have an angle between 1:4 and 1:8.

7.3.3. Maintenance requirements

During the maintenance of the gates, it must be possible to dewater the lock chamber. The lock heads must be able to resist the uplift which is a consequence of the dewatering. In contrast to the lock heads, the lock chamber does not have to be dewatered, so the lock chamber does not have to be able to resist the uplift.

7.3.4. Environmental requirements

The construction and the usage of the navigation lock has to be designed in a way that the environmental footprint is as low as possible.

7.3.5. Construction requirements

Dewatering
No dewatering requirements are known. However, the dewatering of a not water sealed building pit is not advisable, because the surrounding soil is relatively permeable. Also, the stability of lock 1, lock 2 and the weir must be guaranteed.

Obstruction
During the (re)construction of lock, the hindrance of the passing ships must be restricted to a minimum. Though, it is not expected that the lock complex is obstructed totally, because the lock 1 and 2 are usable during the (re)construction of lock 3.

7.3.6. Structural requirements

Loads
Permanent Loads
Self weight
- Water pressure
- Ground pressure

Variable loads
- Water pressure differences, due to water level difference
- Wave loads
- Current
- Breasting loads
- Mooring loads
- Temperature loads

Exceptional loads
- Ship collision
- Explosion
- Ice loads
- Earthquakes

Load factors according to the TGB 1990 can be found in Table 7.7.

Table 7.7: Load factors [TGB, 1990]

	Load Combination	Permanent loads		Variable loads	Exceptional loads
		unfavourable	favourable		
Ultimate limit state	1	1,2	0,9	1,5	
	2	1,35	0,9		
	3	1,0	1,0	1,0	1,0
Serviceability limit state	4	1,0	1,0	1,0	

Load combinations
The 'Technische Adviescommissie voor de Waterkeringen' (TAW) has specified the load factors for water retaining civil structures in 1997 (Table 7.8). These specified load factors are used in combination with Table 7.7 to generate the load combinations of a lock.

Table 7.8: Specified load factors [TAW, 1997]

Load	Computation values	
	Dominant load	Combined load
Permanent		
Self weight	(1,35 or 1,2 or 1,0 or 0,9) F_{rep}	(1,2 or 1,0 or 0,9) F_{rep}
Ground pressure	(1,2 or 1,0 or 0,9) F_{rep}	(1,2 or 1,0 or 0,9) F_{rep}
Water pressure	(1,2 or 1,0 or 0,9) F_{rep}	(1,2 or 1,0 or 0,9) F_{rep}
Variable		
Pressure differences (water level)	1,25 F_{1250} - F_{10000}	1,25 F_{10}
Pressure differences (waves)	1,25 F_{1250} - F_{10000}	1,25 F_{10}
Current	1,3 F_{50}	1,3 F_{1}
Ship wave	1,3 F_{50}	1,3 F_{1}
Ship current	1,3 F_{50}	1,3 F_{1}
Mooring forces	1,3 F_{50}	1,3 F_{1}
Wind load	1,3 F_{50}	0,2*1,5* F_{50}
Temperature	1,3 F_{50}	1,3 F_{1}
Traffic load	1,3 F_{50}	1,3 F_{1}
Exceptional		
Ship collision	F_{1000}	0
Earthquake	F_{1000}	0
Explosion	F_{1000}	0
Ice	F_{1000}	0

F_{rep} = Representative of the characteristic load for permanent loads

F_n = Load that will be exceeded on average once in n year

7.3.7. Flexibility requirements

Obstruction time

The time that the lock is not usable must be minimised during reconstruction. Furthermore, the time that the lock is out of order must be compensated by the extra benefits that will be generated by the enlarged lock.

The year that the reconstruction has to be finished

The assumed years of reconstruction of lock 3 are represented in Table 7.9. Furthermore, the required lock dimensions are given according to Chapter 6.

Table 7.9: The required lock dimensions and the corresponding years of construction

Lock 3	Lock dimensions (m)		
	Length	Width	Depth
2020 (Build the flexible lock)	225	12,5	4,7
2052 (Reconstruct the flexible lock)	225	19,8	4,7

8. Lock development scenarios

8.1. Three alternatives

The two design methods that are mentioned in Chapter 4, the functional flexible lock and the structural flexible lock, are worked out in this chapter. Furthermore, a zero alternative is designed to compare the two design methods with the existing plans of the department of public works [Rijkswaterstaat Maaswerken, 2006].
Different design scenarios are possible within these two design methods, as can be seen in Figure 4.1. In paragraph 6.4 it was shown which lock dimensions are the most convenient in serving the ship traffic through 'Sluis Sambeek' for the next 100 years. When these lock dimensions are implemented in the two design methods, three alternatives can be generated. The different alternatives are listed below and worked out in the last three paragraphs of this chapter.

1. (Short-life) Standard lock: The lock is adjusted for the CEMT – class Vb ships, as has been planned in the 'Tracébesluit Zandmaas/Maasroute' [Rijkswaterstaat Maaswerken, 2006]. In this plan, the old lock is modernised and extra depth is created by raising the water level as well as lowering the lock chamber bottom. The lock heads will be reconstructed and will be made 16 m wide instead of the existing 14 m width (zero-alternative).
2. (long-life) Functional flexible lock: This is a standard lock design, which has a functional flexible function. This implies that the structure stays the same while the function is changing. This lock is built big enough for the CEMT – class VIa ships at point 0 (2020), so the functional life of the lock will be 100 years. This means that the lock does not need to be rebuilt or reconstructed.
3. (long-life) Structural flexible lock: This lock design has a functional and a structural function. This means that the lock is adjusted when that is required by the ship traffic. This lock is initially suitable for CEMT – class Vb ships. When it is required the lock can easily be reconstructed around 2052 for CEMT – class VIa ships.

The different lock dimensions that are expected to be required in 2020 and 2052 can be found in Table 8.1 for each alternative. The red coloured dimensions have to be changed in 2052 into the yellow coloured dimensions according to the trend forecasts (Chapter 6).

Table 8.1: Required dimensions for the different alternatives

Alternatives	Year	Required lock dimensions (m)		
		Length	Width	Depth
1. (Short-life) standard lock (Zero-alternative)	2020	260	16	4
	2052	225	19,8	4,7
2. (Long-life) functional flexible lock	2020	225	19,8	4,7
3. (Long-life) structural flexible lock	2020	225	12,5	4,7
	2052	225	19,8	4,7

In paragraph 6.4, it was explained that the maximum required lock length is 225 m. It is not favourable to have a longer lock chamber than necessary, because then the locking cycle (paragraph 3.2.2) takes more time, which results in a longer waiting time for the passing ships. That is why the length of alternative 1 will be reduced in 2052 (Table 8.1). The width and the depth of this alternative have to be extended in 2052.
In this chapter, alternative 3 is described more extensively than the other two alternatives, because a more innovative approach of lock design is used for alternative 3 than for alternative 1 and 2. Besides an analysis of the three alternatives, also a sensitivity analysis is performed to determine which elements and components of a lock have an effect on a Life Cycle Management (LCM) analysis of a functional flexible lock. After this analysis, the shared structural components will be discussed. The shared structural components are the parts of the lock that have an influence on the costs or on the design, although they are the same for each alternative.

8.2. Sensitivity analysis

In this paragraph, all the elements and components of the navigation lock (see paragraph 3.2.1) are considered to determine which of them have an influence on the alternative selection by means of a LCM analysis. The elements are separated as is represented in Figure 8.1. Each section of this paragraph will describe a different element. Within these elements it is determined which components are affected by the width as a flexible lock dimension. Furthermore, it is discussed how much influence an adjustment of the width has on each affected element.
Mechanical items are not taken into account for all the alternatives. Although the mechanical items represent almost 20% of the total initial construction costs, they only have a lifespan of about 30 years. This data is obtained from the design requirements for 'Sluizen 4, 5 en 6 in de Zuid Willemsvaart' [Withagen, 2009]. Consequently, the mechanical items are at the end of their lifespan when one of the lock alternatives must be reconstructed or rebuild, so the costs of the mechanical items are assumed to be the same for each alternative.

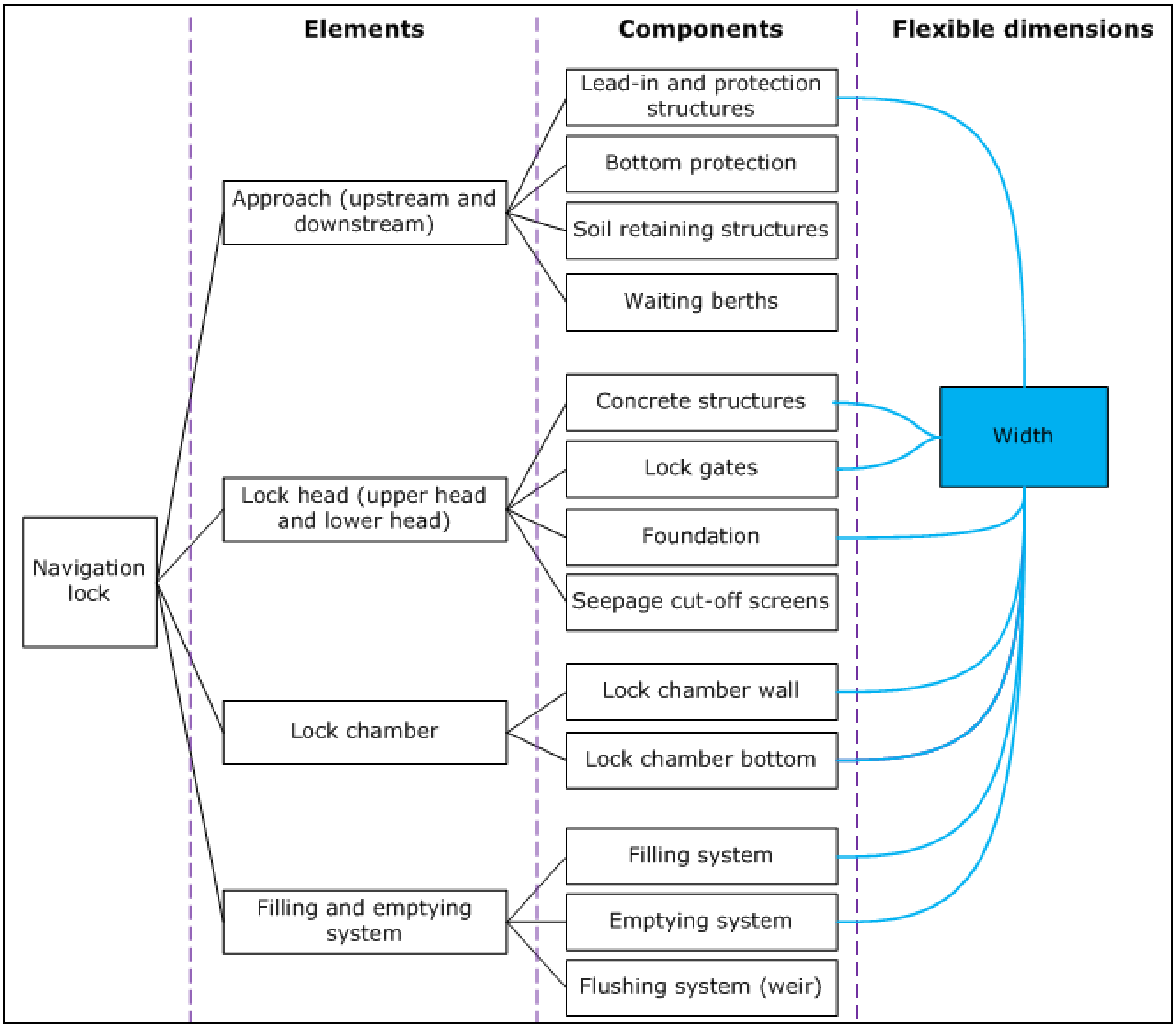

Figure 8.1: The system of a flexible lock with the width as a flexible dimension

8.2.1. Approach (upstream and downstream)

The flexible width has little or no effect on the approach of the lock. Only the lead-in and protection structures are affected by the flexibility of the width. A few small adjustments could be necessary, like widening the lead-in jetties. The associated costs will be low and the disturbance for the passing ships will be limited. Therefore, the lock approach is not considered in the alternative comparison.

8.2.2. Lock head (upper head and lower head)

In contrast to the lock approach, the flexible width has a large effect on the lock heads. Namely, three of the four components are influenced by the flexible width. Especially the concrete structure and the lock gates are influenced. The lock heads have to be replaced totally to be sure that they remain stiff and strong enough. This element (lock head) will be worked out extensively for all the alternatives. Though, the details like the pivot, the collar strap and the mitre sill will not be treated, because it has been assumed that these details do not make the difference in costs. The seepage cut-off screens differ for each alternative, because they are placed to prevent piping. In case of a permeable lock chamber bottom, the screens must be longer than in case of an impermeable lock chamber bottom. Thus, the seepage cut–off screens are treated separately for each alternative as well.

8.2.3. Lock chamber

The flexible width also has a large effect on the lock chamber, because the walls have to be replaced and the bottom protection has to be extended. Therefore, the lock chamber will also be treated separately for each alternative. The type of lock chamber bottom, permeable or impermeable, also has an effect on the seepage cut-off screens which have to be placed under the lock heads.

8.2.4. Filling and emptying system

In case of a longitudinal culvert system, the filling and emptying system would have a large influence on the construction costs. Though, in the next paragraph it can be seen that valves in the gate are sufficient to reach the required emptying and filling time. When the lock becomes wider, also the gates and the valves become wider. This means that a flexible width does not affect the filling and emptying time. As a consequence, the filling and emptying system will not be discussed in detail for each alternative. The small differences in passing time will be discussed for each alternative separately in Chapter 9.

8.3. Shared structural components

This paragraph deals with the parts of the lock design that are similar for each alternative. In the next three paragraphs, the lock design is discussed separately for each of the alternatives (paragraph 8.4 till 8.6).

8.3.1. The lock gates

The three alternatives all will be based upon a mitre gate, because this type of gates is widely used in inland navigation locks of this size. This can be seen in Appendix IV in Figure IV 1. The other options, like a roller gate or a vertical lift gate, result in thicker gates or wider lock heads which is not desirable in case of a flexible lock.
The thickness of the lock gates will be determined by using the standard rule of 1/18*the lock width. The height of the gates will be the same as the lock heads and the angle of the gates will be 1:3. The gate recess is related to the determined thickness of the lock gate. The length of the gate recess will be the gate length plus 0,8*the gate thickness. The width of the gate recess will be 1,4*the gate thickness [Glerum, 2000].
When the steel is covered with a good anti-corrosion layer, the lifetime of the lock gate is infinite. This can be done by painting the gate with intervals of 10 year or by applying a more durable layer of aluminium.

8.3.2. The filling and emptying system

It can be concluded that no longitudinal culvert systems are needed and a through-the-heads filling and emptying system satisfies the requirements. This follows from the first approximation of the filling and emptying system which is performed by an empirical selection procedure from the Chinese code [PIANC, 2009].

$$M = \frac{T}{\sqrt{H}}$$

In which, *H(m)* is the lift height of the lock and *T(min)* is the time to fill the chamber.
The following values were derived for Chinese inland waterways:

$3{,}5<M$	through heads system
$2{,}5<M<3{,}5$	through heads system or simple longitudinal culvert system
$2{,}4<M$	simple longitudinal culvert system
$1{,}8<M<2{,}4$	a more complex longitudinal culvert system
$M<1{,}8$	a rather/very complex (advanced) longitudinal culvert system

In case of 'Sluis Sambeek', a filling time of 8 minutes (T) is assumed and a representative water level difference of 3,4 m (H). From this data follows that M = 4,3. Thus, the new lock in the lock complex 'Sluis Sambeek' can be equipped with the through heads system.
Furthermore, the different maximal levelling times for a gate valve and a stilling chamber are calculated according to [Glerum, 2000]. These calculations can be seen in Appendix IV. For each alternative, the corresponding locking times are determined on the basis of the standard distribution of a locking cycle [PIANC, 2009]. From this calculation follows that a gate valve system is sufficient and that a stilling basin is not needed, because all the resulting passing times are within the norm of 30 minutes.

8.3.3. Demolishing the old lock

Before a new lock can be build, the old lock structure has to be removed partly. In Appendix IV, the demolishing volumes, of the parts that have to be removed are determined to quantify the removal of the old lock. In alternative 1, the lock is demolished in a different way than in the other two alternatives. In this alternative, only parts of the old lock are demolished and the old lock chamber wall is re-used.
In alternative 2 and 3 the old lock heads, including the intermediated head, are demolished because these heads obstruct the construction of the new lock heads. The new lock chamber wall will be 225 m long. Though, the old lock chamber wall has a length of about 260 m. So at least 65 m of the old lock wall, 35 m of the lock chamber wall, and 30 m of the intermediated head has to be demolished on both sides of the chamber. Also the concrete blocks of 0,5 m thick, that cover the bottom of the old lock, are removed. From the 195 m lock chamber that is left, the lock wall only has to be demolished at one side. The other wall can be left behind without causing hindrance during the construction. As a consequence the anchorages have to be drilled through the old lock wall. This can be seen in Figure 8.2.

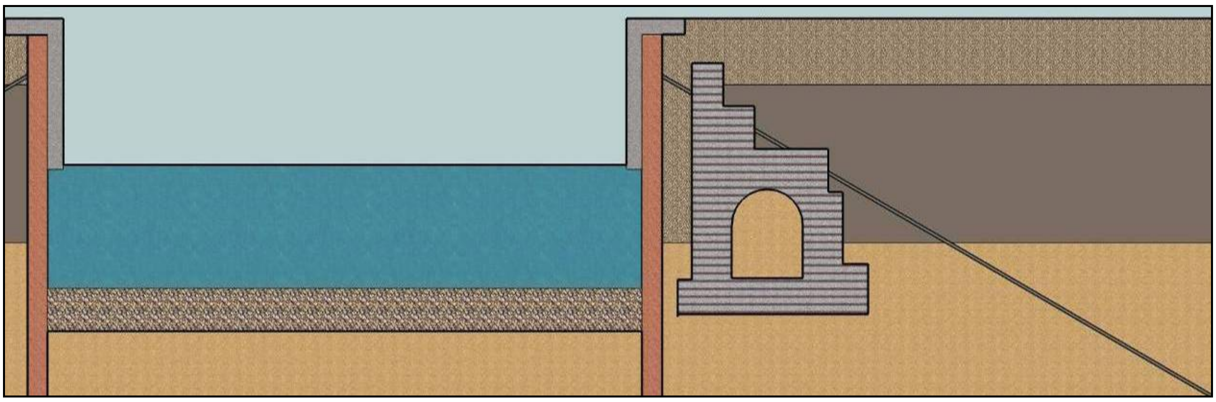

Figure 8.2: The remaining old lock chamber wall

8.4. Alternative 1: short-life lock (zero alternative)

This alternative represents the solutions that will be used by the department of public works [Rijkswaterstaat Maaswerken, 2006]. In this option, the old lock is renovated by deepening the lock and replacing the lock heads. Also the intermediate lock head is deepened or replaced. According to Chapter 6, it is expected that this lock will be too small in 2052. A new wider and deeper lock has to be constructed then to replace the 120 year old lock. In this alternative study it is assumed that this new lock will be exactly the same as the lock that is designed for alternative 2 (paragraph 8.5). So, in this paragraph only the reconstruction or restoration of the lock will be quantified. The calculations and the quantities can be found in Appendix V.

8.4.1. Lock head

The lock head will be 16 m wide and must have a minimum sill depth of 4 m. With this information, the lock heads are designed and can be seen in Figure 8.3. Before these heads are built, the old heads first have to be demolished. The new lock heads can be build upon the sand without piles, just as in the old situation. These heads will be build in building pits, which are created by using underwater concrete in combination with GEWI anchorages and sheet pile walls. The top level of the lock head will be 12,8 m + NAP and the bottom level will be 4,1 m + NAP (the top of the concrete floor).
According to Lane, the seepage cut–off screens have to be 5,75 m, to prevent piping underneath the lock heads.

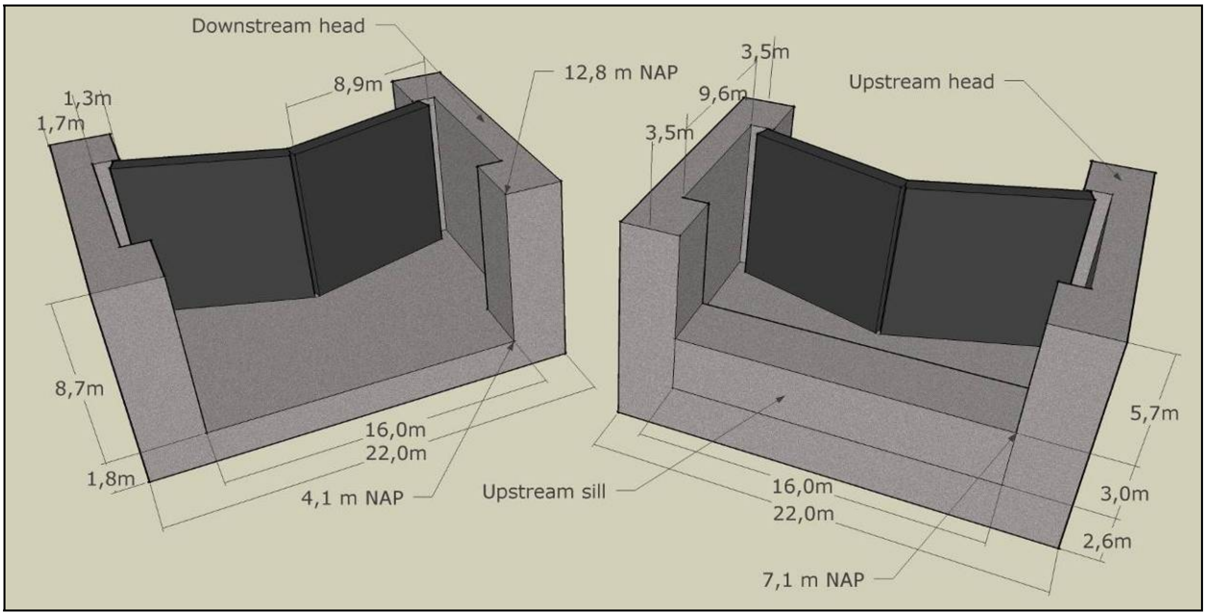

Figure 8.3: Upstream and downstream lock head of the alternative 1 renovation lock

8.4.2. Lock chamber

The old bottom consists of concrete blocks of 0,5 m thick. These blocks will be removed and the lock chamber will be deepened from a level of 3,7 m + NAP to a level of 2,9 m + NAP. Then, a 1 m thick filter layer, consisting of three different gravel layers, will be dumped to form the new permeable lock bottom.
Instead of the existing longitudinal filling and emptying system, the lock chamber will be filled and emptied by valves in the gates. The old culvert system will be filled with concrete to stabilise the lock chamber wall. Consequently, it is assumed that the filled culvert system gives enough stability to resist the lowering of the chamber bottom with 0,30 m. Besides the lowering of the bottom, also the lock chamber wall will be heightened with 1,05 m. It is assumed that a concrete wall of 0,8 m wide that is anchored upon the old lock wall is strong and stiff enough. A cross-section of the renovated old lock chamber can be seen in Figure 8.4.
The intermediated gates will be removed, and the sill of this head will be lowered from 4,25 m + NAP to a level of 3,75 m + NAP (the top of the existing intermediated head floor) to create enough depth.

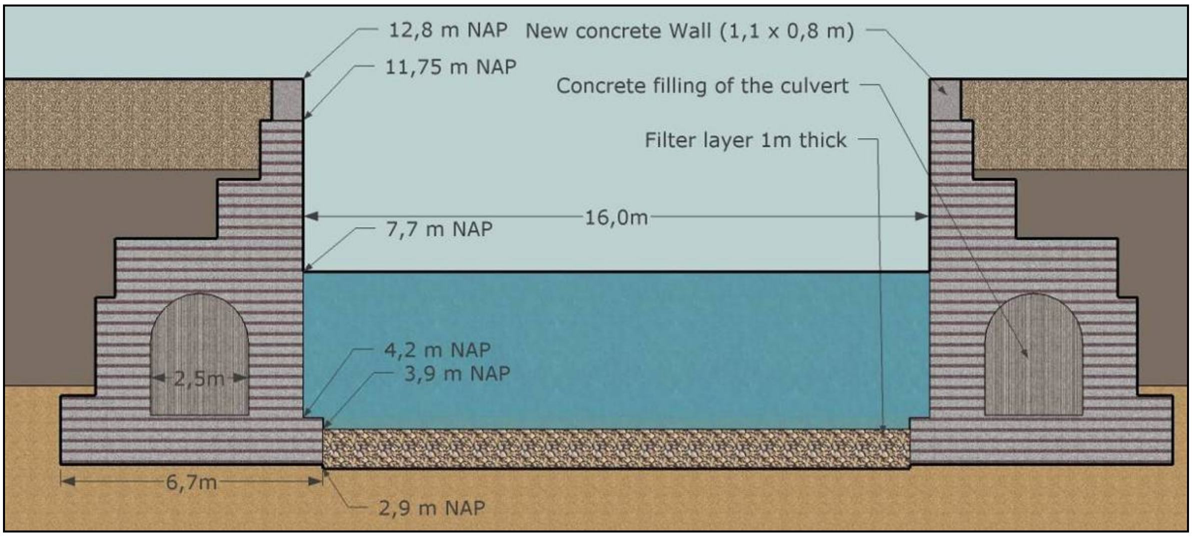

Figure 8.4: Renovation of the old lock chamber of alternative 1

8.5. Alternative 2: Long life functional flexible lock

The calculations for this paragraph can be found in Appendix VI. In this appendix also the final cost calculation quantities are determined to express the alternative in costs for Chapter 9.

8.5.1. Lock head

The lock heads will be constructed as a concrete U-shaped structure. This structure will be build in-situ. For the construction of the lock heads, the inland navigation lock of Lith [Glerum, 2000] is used as a reference. The walls of this reference project are 4 m thick and the floor is 2,5 m thick. These dimensions are sufficient for the downstream head. For the upstream head, these dimensions are too light to resist the uplift as a consequence of dewatering the lock head for maintenance. Therefore, the floor is constructed thicker to resist the uplifting forces. The floor of the upstream head will become 2,8 m thick and an extra sill of 3 m high will be made upon the front pier of the upstream lock head to increase the weight of the head. Both the lock heads and the corresponding measures can be seen in Figure 8.5. The top level of the lock will be at 12,8 m + Nap and the bottom of the lock head will be at 3,4 m + NAP (the top of the concrete floor).

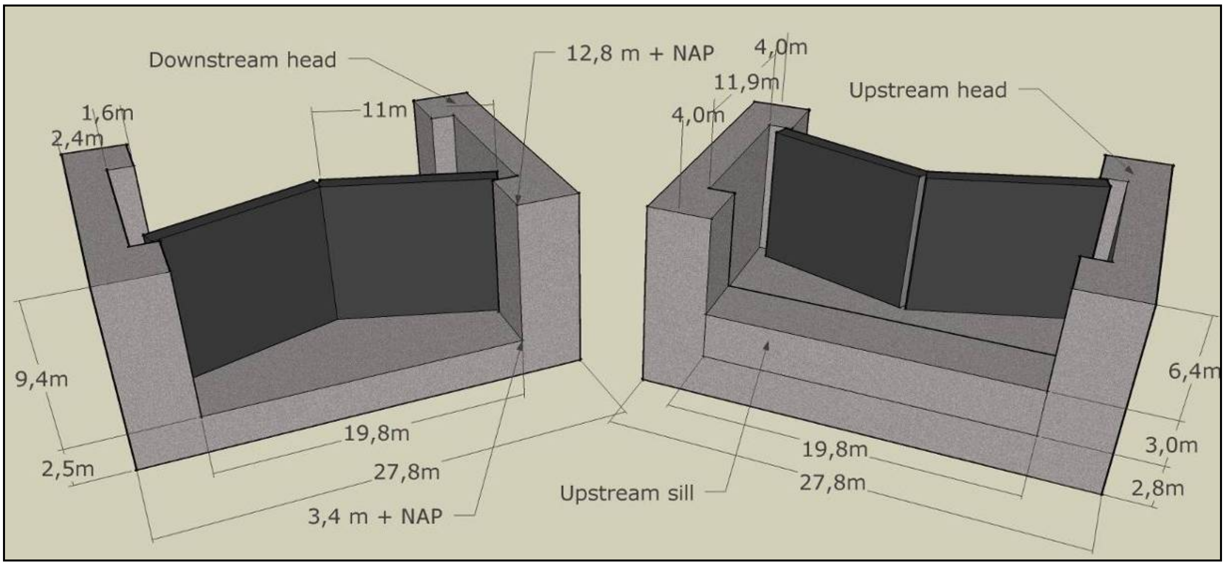

Figure 8.5: Upstream and downstream lock head of alternative 2

The heads are built upon the sand without piles, because the soil is strong enough to resist the load of the lock heads. The heads will be build in a building pit, in the same way as in alternative 1, using underwater concrete, GEWI Piles and sheet pile walls.
In the first design stage, no seepage cut–off screen is needed, because the closed length of the lock bottom is long enough to decrease the currents and to prevent piping. However, the cost calculation in Chapter 9 will show that a permeable bottom with seepage screens is a cheaper option than an impermeable bottom without these screens. Thus two seepage cut–off screens of 5,2 m depth are needed.

8.5.2. Lock chamber

For the lock chamber, three options were considered:

1. A U–shaped concrete structure.
2. A sheet pile wall combined with a permeable bottom of rubble mound.
3. A sheet pile wall combined with an impermeable bottom of underwater concrete.

In the first place, option 3 was chosen (Figure 8.6), because this solution seems to be less expensive and less complicated than the other two options. The U-shaped concrete was no option from the start, because this type of lock chamber needs expensive temporary structures. Option 2 was rejected in the first place, because heavier and longer sheet piles are needed when no underwater concrete is used.

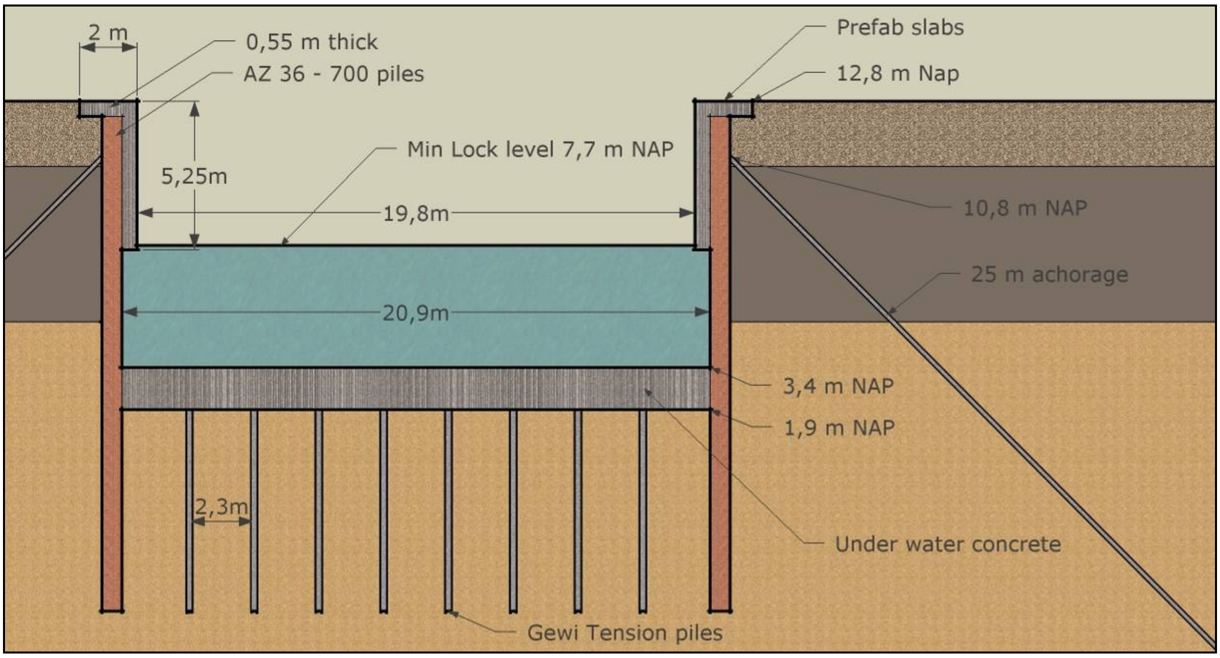

Figure 8.6: Lock chamber option 3 of alternative 2

During the cost calculation for Chapter 9, the permeable filter bottom of alternative 3 (paragraph 8.6.2) turned out to be cheaper than the lock chamber with the underwater concrete and the relatively expensive permanent GEWI-Tension piles. Therefore, the costs of options 2 and 3 of alternative 2 were reconsidered. Option 2 appeared to be the cheapest option and was finally chosen. A cross-section of the lock chamber of option 2 can be seen in Figure 8.7. The technical design of this type of lock chamber can be found in Appendix VII.
The steel sheet piles will be covered with concrete prefab slabs to protect the steel wall from corrosion. This method is also derived from the design of the new inland navigation lock of Lith [Glerum, 2000]. The steel walls will be anchored with grout anchors. It will be impossible to dewater the lock chamber. The lock chamber will have a minimum lock level of 7,7 m + NAP.

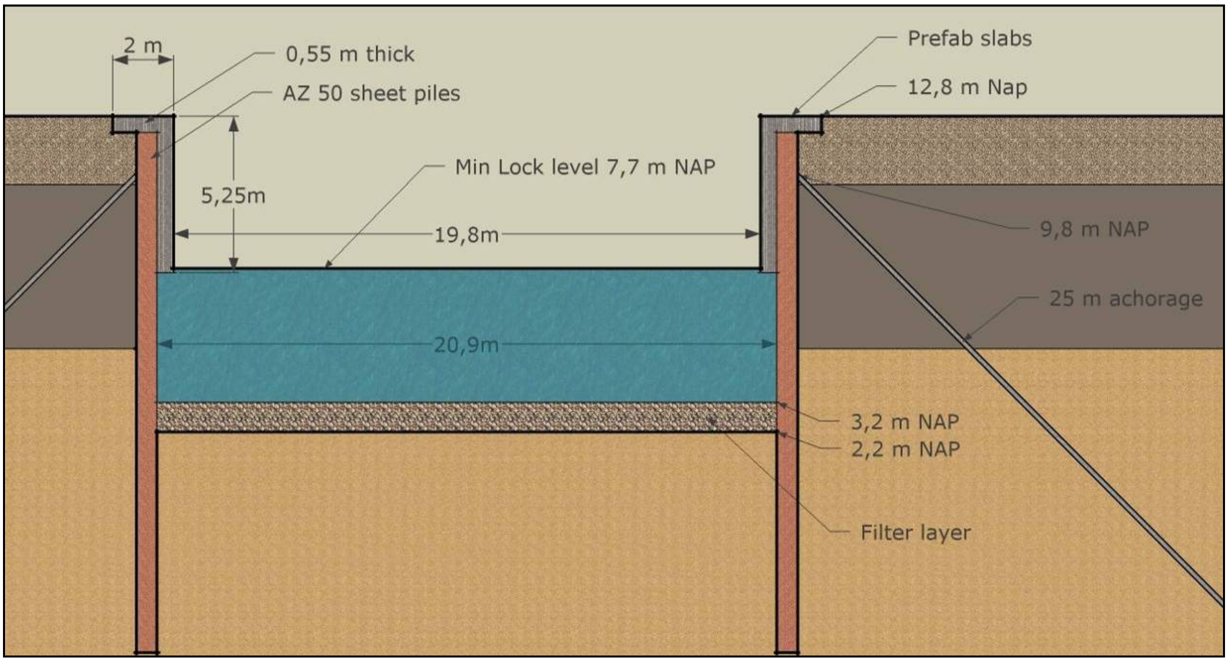

Figure 8.7: The selected lock chamber option 2 of alternative 2

8.6. Alternative 3: Long-life structural flexible lock

The construction of a long-life flexible lock consists of two phases. In both phases the lock will have a depth of 4,7 m and a length of 225 m. Only the width will be flexible as determined in Chapter 6. The first phase is the construction of the small flexible lock. This lock will have a width of 12,5 m and will be finished in 2020. This small lock can be changed into a larger lock in the second phase. The large lock will have a width of 19,8 m and will be ready in 2052. In the next two paragraphs, the structural design of both phases is worked out roughly. The calculations and the quantification of the components can be found in Appendix VII.

8.6.1. Lock head

The lock head will be the most difficult part to make flexible, because the head must be strong and stiff enough and it must be able to be replaced or reconstructed. In former studies only a solution for a flexible lock chamber was found and a solution for a flexible lock head was not given [Bonnes, 2005]. In this paragraph, four design possibilities are worked out for designing a flexible lock head (Figure 8.8). First, the requirements for a structural flexible lock head are given. Second, four design possibilities are mentioned. Finally, one design possibility will be selected and worked out.

Requirements
For the design of a flexible lock head a number of requirements are necessary. Two types of requirements are distinguished, namely the selection requirements and the detailed design requirements. The selection requirements are used to determine which of the design possibilities must be selected. The detailed design requirements will be applied on the selected design possibility.

Selection requirements
1. It must be possible to make the lock head wider in a relatively short time.
2. The lock head structure must be stable in both the construction phase and the users phase.
3. It must be relatively easy (low costs) to replace or reconstruct the lock heads after their functional lifetime.
4. The expected initial costs must be as low as possible to reduce the Whole Life Costing (WLC).

Detailed design requirements

- The lock head must be stiff and strong enough to resist the forces related to the gates, the water and the ground pressures.
- The lock head must be able to resist the uplifting force of the water pressure during maintenance (when the lock is dewatered).
- The small downstream lock head has a maximum floating depth of 4 m during the transport from the building pit to the lock location. (4,1 m depth of lock 1 and 2 of the lock complex).
- The large downstream lock head has a maximum floating depth of 4,6 m (4,7 m depth of the new large lock).
- The maximum length of the small lock head is 15,6 m (16 m wide of lock 1 and 2 of the lock complex).
- The maximum length of the large lock head is 19,4 m (19,8 m wide of the new large lock).

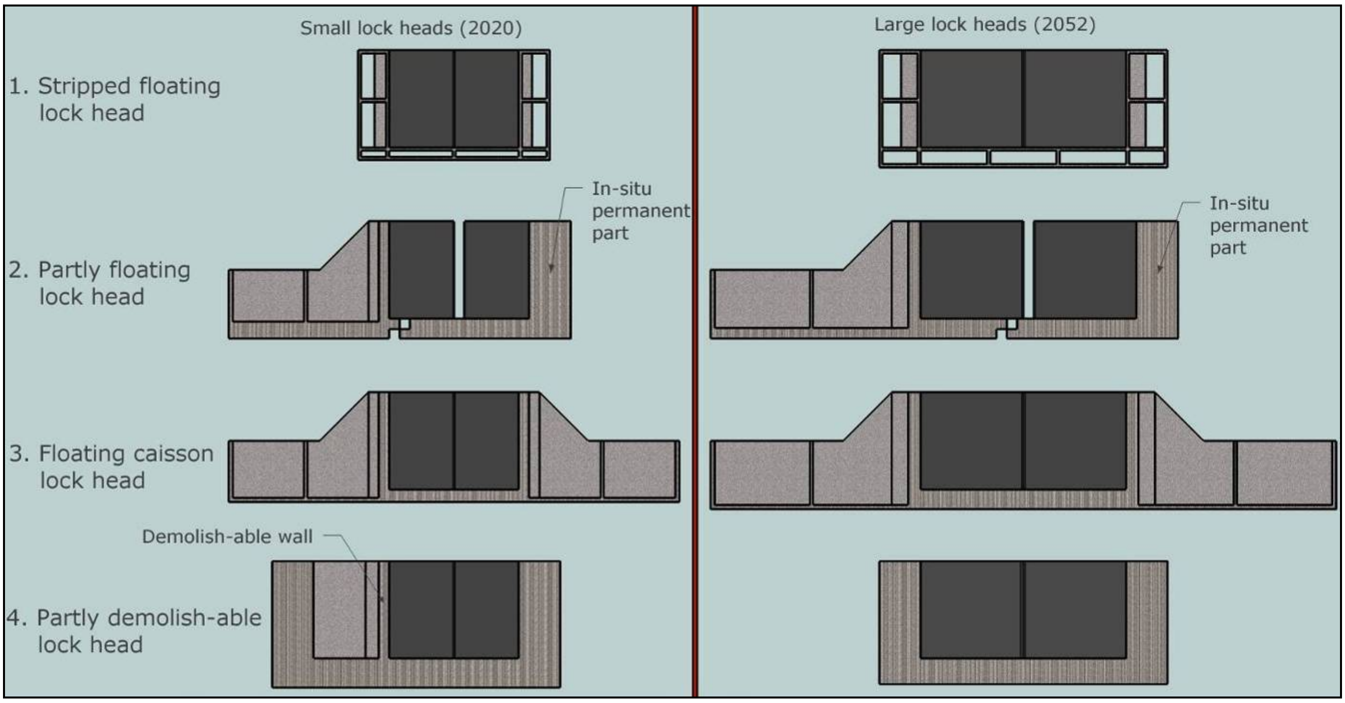

Figure 8.8: Cross-section of the design possibilities

Design possibilities

The four design possibilities are shown in Figure 8.8 and are represented below.

1. *Stripped floating lock head*: The heads will be constructed with thin prestressed concrete slabs and the space in between these slabs will be free of concrete. These air chambers will be used for the floating capacity during transport. After the head is immersed, the chambers can be filled with ballast concrete to stabilise the lock head and to resist the uplift pressures [Ravenstijn, 2001]. This is initial a relatively cheap option, because the floating capacity is created in a standard lock construction. As a consequence, no extra concrete is required. However, the lock head cannot be float up again, because it is filled with irremovable ballast concrete.
2. *Partly floating lock head*: In this case, one side of the head and a part of the floor will be poured in situ. The other side of the head will be floated in as a caisson and will be immersed on the construction joint. When the width has to be extended, a wider new part can be floated in. The advantage of this design is that only a part of the head has to be replaced. Though, this part will be relatively large, because the floor of the side caisson has to be thick enough to resist the moment that will act on the wall. In addition, the weight is not in balance during transportation, because the floor slab is at one side of the caisson and will not

have any floating capacity. It is assumed that this will cause instability during transportation.

3. *Floating caisson lock head*: The lock heads will be constructed as U-shaped concrete structures with at both sides a caisson to give the heads enough floating capacity for transportation. After the head is immersed, the caissons can be filled with sand to prevent the head from uplifting. The lock head can be made wider by removing the sand, floating out the small lock head, and floating in the wider new lock head. The disadvantages of this design are the large dimensions of the floating heads. Though, the amount of concrete is relatively small compared to the outside dimensions. This is a result of the relatively thin construction of the caisson walls. The advantage of this design is the relatively short construction time on location, because the lock head can be floated in and out quite easily.
4. *Partly demolish-able lock head:* For this design possibility a standard lock head, as can be seen in alternative 2 (paragraph 8.5.1), will be build in situ. This lock head will be constructed large enough for the assumed lock extension in 2052. In this lock head, a demolish-able wall will be build to reduce the width of the head. This wall can be demolished when the lock is extended. The initial amount of concrete for this possibility is relatively high and the obstruction time during the reconstruction will be relatively long. Though, an advantage is that no building pit upstream is needed.

The four design possibilities are compared by using a Multi Criteria Analysis (MCA), which can be seen in Table 8.2. The design possibilities are judged with scores from 1 to 5 for each selection requirement. A score of 1 means that the design possibility does not satisfy the requirement at all. A score of 5 means that the requirement is met totally.

Table 8.2: MCA analysis of the four design possibilities

Requirements	Alternatives			
	1.	2.	3.	4.
1. Width extension in a short time	1	5	5	2
2. A stable construction	5	1	5	5
3. Easy reconstruction	1	3	4	4
4. Low initial costs	5	3	4	2
Total score	12	12	18	13

In Table 8.2 it can be seen that design possibility 3 is the best option. Consequently, option 3 is selected for further elaboration in the next part of this paragraph. A more detailed design of this option can be found in Chapter 11 and Chapter 12.

Construction of the lock head

The lock heads have to be constructed in a building pit upstream of 'Sluis Sambeek' and will be floated to the lock location. The building pit will be located in the river forelands of the Meuse. This pit will be constructed by digging a hole with a slope. The soil along the Meuse is sandy and permeable. Therefore, the dredged building pit has to be protected from the seepage water. This can be done by placing a seepage screen in the slope around the building pit and by using a well pointing system. The screen must be high enough to resist piping. The seepage water that still flows into the pit has to be pumped away.

After the building pit is ready, the two lock heads will be built upon a gravel layer of 0,25 m to be able to create water pressure under the lock heads. It is assumed that the construction time of the lock heads is about half a year. The gate chambers (the concrete U in the middle) will be shut by bulk heads on both sides. As soon as the heads are ready, the dike between the river and the building pit is removed and the lock heads can be floated out and transported by tugs to the lock location. During this transportation, the floating lock heads will be equipped with bollards and protection materials. At the lock location, the heads will be immersed on a gravel layer of 0,5 m. This layer creates the possibility to float the lock head again and change it into a wider lock head. After the small lock head is removed, the gravel layer and the surrounding soil are dredged to make room for the large lock head. Before this head will be immersed, a new gravel layer of 0,5 m will be dumped.

Design of the lock head

The walls of the lock heads are thin in comparison to a standard lock design. These stripped walls are supported by triangle buttresses that transport the reaction force of the gates and an eventually high water level directly to the foundation. Furthermore, the outside walls and floors of the caissons at each side will have a thickness of 0,4 m. The inside walls of the caissons will be 0,3 m thick. The dimensions of the outside walls and the floors are checked in Appendix VII. The other dimensions of the small and the large floating caisson lock heads are depicted in Figure 8.9 and Figure 8.10.
A seepage screen has to be placed, because the piping length of 15,6 m (the length of the small lock head) is not enough to resist piping. The seepage cut-off screens that have to be placed under the downstream lock head are 5,9 m high, as determined in Appendix VII. The screen will consist of sheet piles and is constructed at the end of both sides of the lock chamber. The screens will be made wide enough to provide a good screen for the small and the large lock heads.
At the edge of the lock chamber and the seepage cut-off screen, a frame of concrete and steel will be constructed. A rubber profile that will be placed on the lock head will be pushed against this frame, so a watertight connection will be created. During the construction of the lock, the lock head with the rubber profiles will be floated against the frame and will be immersed. In paragraph 10.4, a more detailed description of this construction is given and a sketch of the frame is shown.

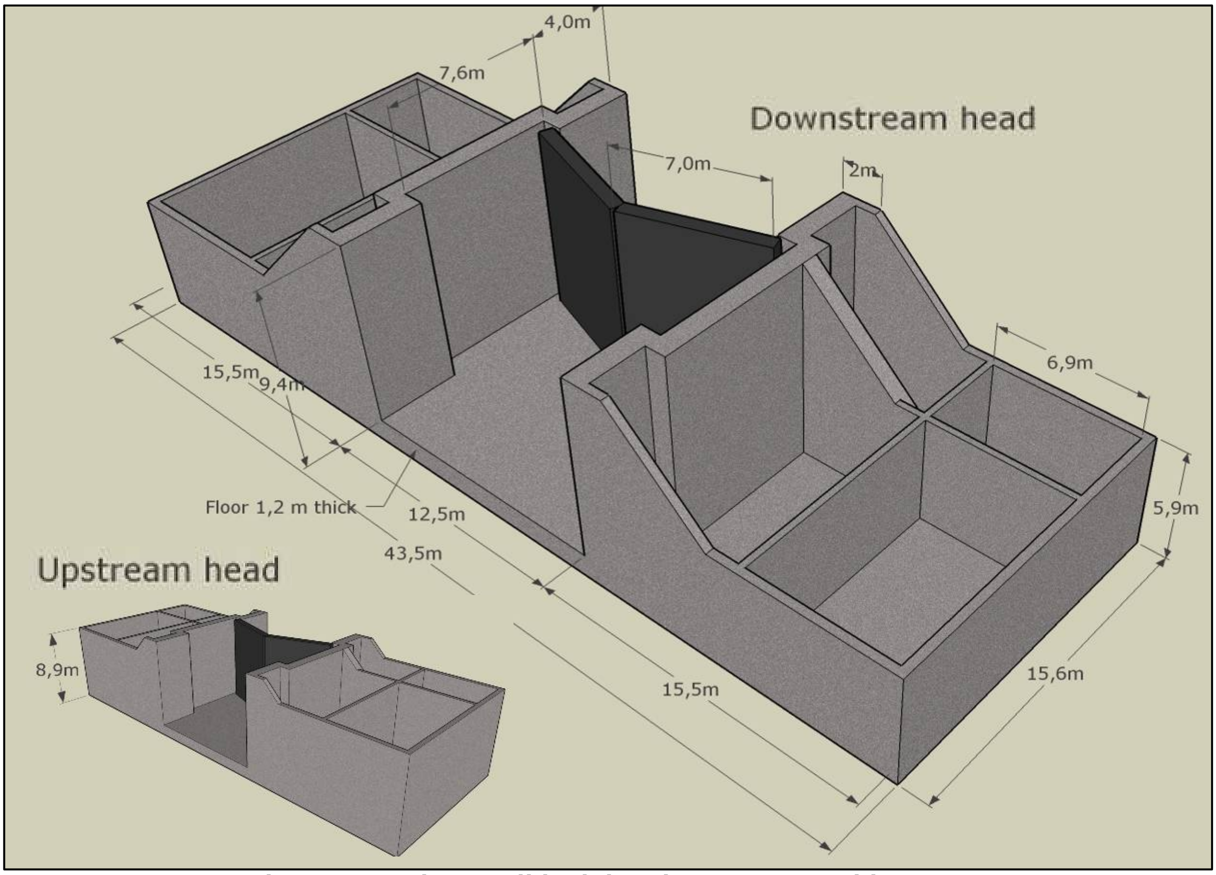

Figure 8.9: The small lock heads, constructed in 2020

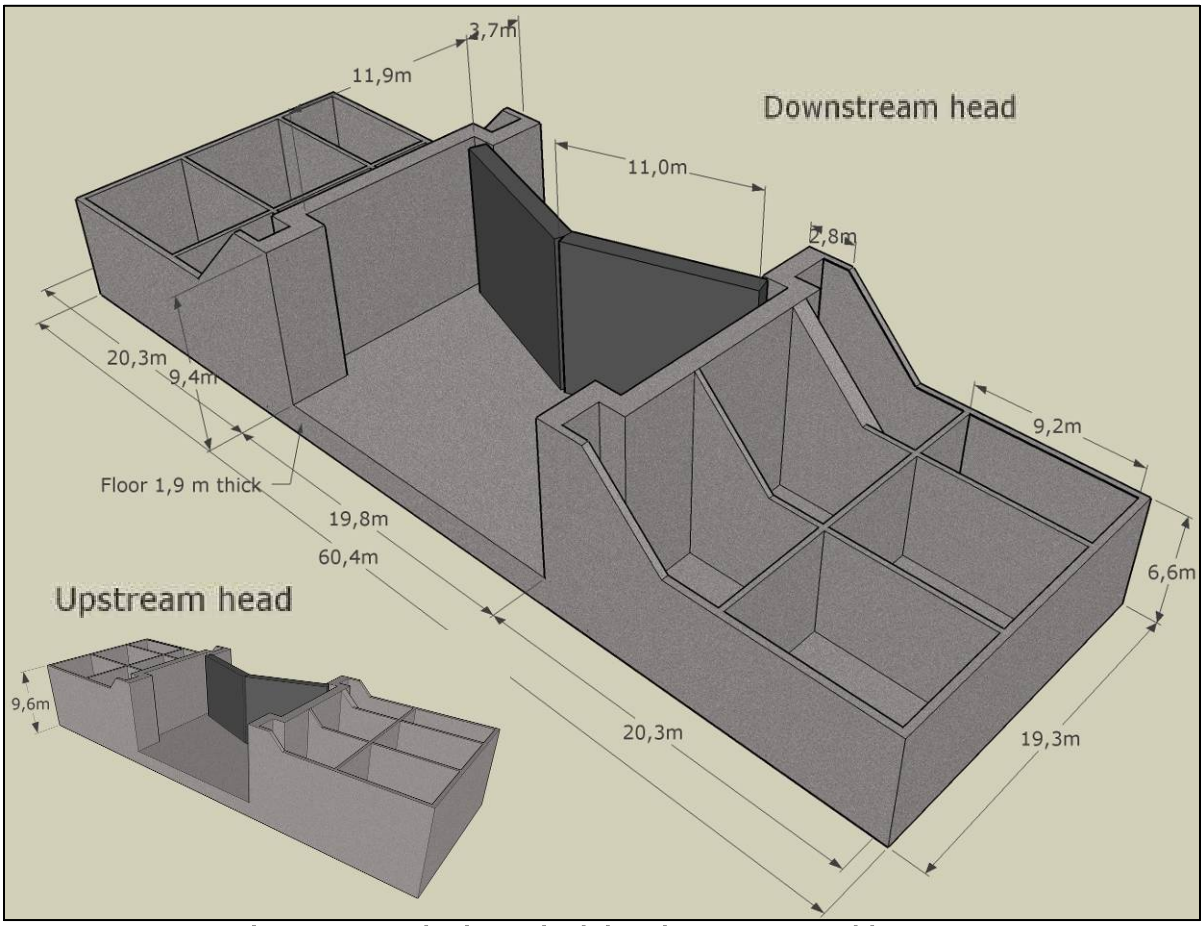

Figure 8.10: The large lock heads, constructed in 2052

8.6.2. Lock chamber

The lock chamber will have a permeable bottom. This is derived from the graduation report 'Flexibele zeesluis als nieuwe maritieme toegang kanaal Gent Terneuzen' [Bonnes, 2005]. According to this report, this is the most flexible solution. In that graduation report, the wall of the lock chamber consists of prefab floatable caissons to provide flexibility in the width direction. For lock 3 of 'Sluis Sambeek', steel sheet piles are chosen, because 'Sluis Sambeek' is an inland navigation lock with 3 times less depth than the sea lock in Terneuzen. Moreover, less corrosion is expected than for a sea lock, because the environment of the Meuse is not salt. The wall will be constructed of steel sheet piles that are heavy enough to maintain a wall displacement within the allowed tolerances.
The side that is extended in the width direction was in the first consideration equipped with a cofferdam (Figure 8.11). This saves anchorages and the sheet pile wall for the second phase is already placed then. When the lock is extended, the front sheet pile row of the cofferdam will be removed and the other one will be anchored. The steady, not flexible side of the lock chamber will be constructed with only one row of sheet piles that are directly anchored. However, after comparing the initial costs of the lock chamber with a cofferdam with the costs of the lock chamber without a cofferdam in Chapter 9, the option without a cofferdam is chosen.
The permeable bottom consists of a 1 m thick gravel filter layer. It is assumed that this filter will consist of 3 layers of gravel to create a stable bottom. The small lock chamber of the first phase will be constructed in the same way as is represented in Figure 8.7, only this chamber will be smaller (12,5 m). The second phase (after 2052) will have the same layout as in Figure 8.7. When further research shows that a permeable bottom is impossible, also an underwater concrete floor with tension piles, as can be seen in Figure

8.6, could be considered. Though, this option is more expensive. The steel sheet piles in the lock chamber wall will be protected by the same prefab slabs as used in alternative 2.

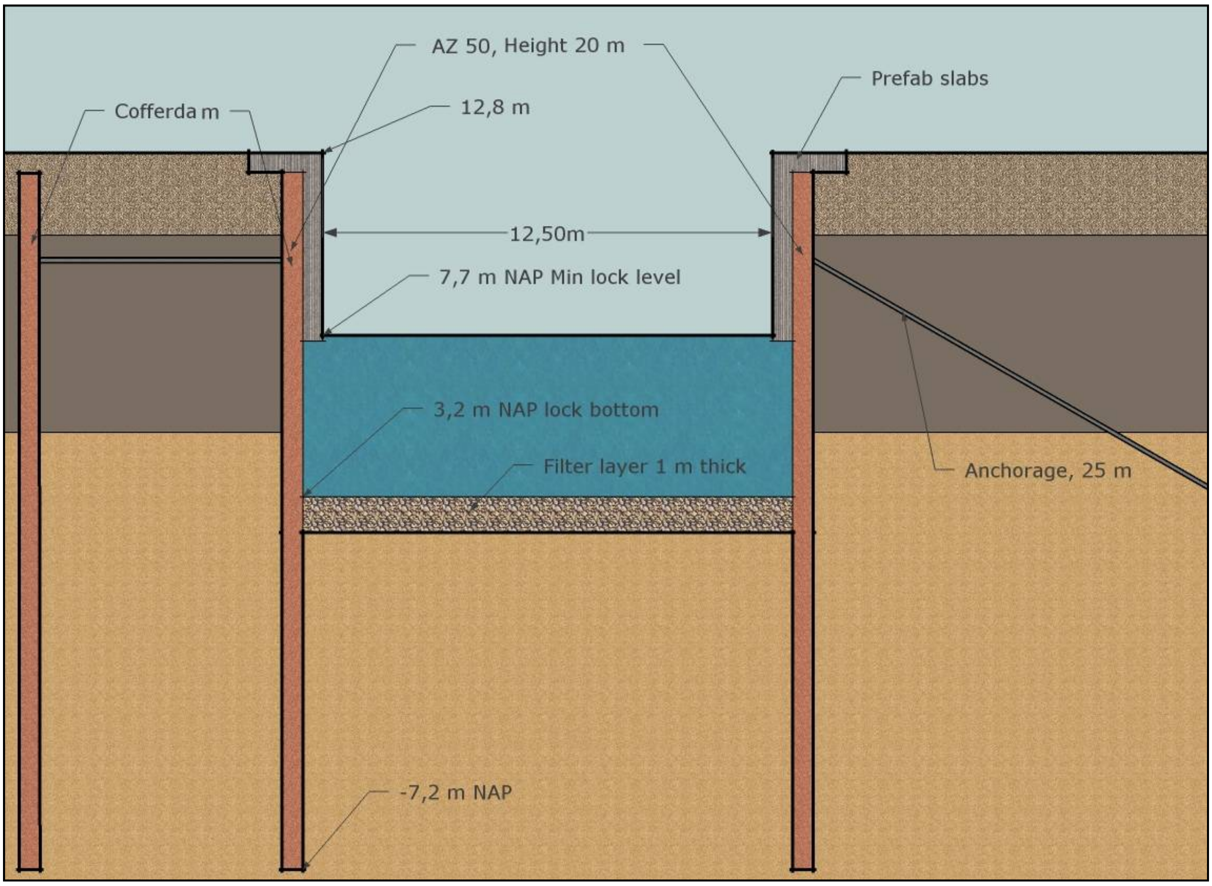

Figure 8.11: The small flexible lock chamber of alternative 3

When the lock is extended, the slabs at one side of the lock chamber wall have to be demolished. During this operation, a new row of sheet piles have to be drilled and anchored beyond the wall that is demolished. A new row of concrete slabs has to be constructed on the new sheet pile wall after the old row of sheet piles is pulled, the soil is dredged and the additional filter layer is dumped.

9. Whole Life Costing (WLC) analysis

In this WLC analysis it is determined which of the three alternatives from Chapter 8 is the best (re)construction option for lock 3 of the lock complex 'Sluis Sambeek'. The WLC analysis is a part of the Life Cycle Management (LCM) approach (see Chapter 2) and is used to compare alternatives. To determine the WLC, all the costs that will occur in the lifetime of the lock will be taken into account, including the lifetime costs and benefits. For this purpose a Net Present Value (NPV) calculation is used to determine the WLC for the current (2009) price level.
It was assumed that the new lock design must be sufficient for the next 100 years, so a period from now till 2110 was analysed. In this WLC analysis, the end of the functional lifetime of the lock will be 2105. This is a consequence of the trend forecast in the lock dimensions (paragraph 6.4), because the lock size which is needed in 2052 will be able to cope with the ship sizes till 2105 (see Figure 6.10).

9.1. Cost variables

To determine the WLC, three types of variables are gathered in this paragraph. These variables are real interest and inflation, the initial cost variables and the lifetime cost variables.

9.1.1. Interest and inflation

In a Net Present Value (NPV) calculation, the initial investments need to be multiplied by the real interest. The real interest is the interest corrected by the inflation. For this purpose the average interest over government loans from 1900 till 2002 [CBS, 2003] is compared to the average inflation of the last 100 years [CBS, 2009]. This leads to a real interest percentage of 1,9%. By using this real interest, all the costs that are calculated in this chapter are representative for the current (2009) price level.
The same initial construction costs can result in a different WLC value. A relatively high real interest results in a higher WLC value than a relatively low real interest. Thus, the lower the real interest percentage the smaller is the effect of high initial costs on the WLC value.

9.1.2. Initial cost variables

In Appendix VIII the initial cost variables that are used, can be seen. These variables are obtained from BAM Civiel [Hogendonk, 2009] and are mainly obtained from the tender 'De reconstructie van de sluizen van Born, Maasbracht en Heel'.
To determine the initial costs, the all-in unit prices of the construction materials are used. Only for alternative 3 a lot of extra costs are taken into account, because the construction of prefab units on a location upstream results in additional operations. For instance the construction of a building pit, the transport over water and the floating in of the lock heads. The other two alternatives are based on standard lock designs. Thus for these two alternatives the unit prices are sufficient to determine a reliable price.
Furthermore, a difference is made in the way the same materials are used. Concrete, for example, is split into wall volumes, floor volumes, thin wall volumes, small concrete work volumes and prefab slab lengths. For each concrete volume also the different reinforcement tonnages are included in the price.
By using these rough cost variables, the possibility that a few expenses are forgotten or are estimated too low is present. These expenses are therefore covered by a 10% raise of the initial construction cost calculations.

9.1.3. Lifetime cost variables

The lifetime cost variables are obtained from the literature and are further specified in the benefits and costs over the lifetime (paragraph 9.3).

9.2. Initial construction costs

The initial costs for each alternative are determined by combining the conceptual design from Chapter 8 with the initial cost variables of paragraph 9.1.2. The determined material amounts are multiplied by the costs per unit. The final initial costs can be seen in Table 9.1 and the detailed initial cost calculations are shown in Appendix IX. The initial cost calculations are assumed to be 10% higher than estimated (real option initial costs). This correction of the costs indicates that an uncertainty of about 10% can be expected. The uncertainty is represented in Table 9.1 by giving the positive, the negative and the real option initial costs. The positive and the negative initial costs are respectively 10% lower and higher than the real option costs. It is assumed that the final initial costs will be between the positive and the negative value.
The construction time of the floatable prefab lock heads of alternative 3 is supposed to be 26 weeks. During these 26 weeks, the building pit that is located upstream of 'Sluis Sambeek' must be kept dry by using dewatering pumps.
The lock gates of alternative 1 and 3 are changed in 2052, because the lock heads will be constructed wider in that year. Only for alternative 2 the same lock gates are used for the whole lifetime. It has been assumed that when the lock gates are maintained well, the gates will last a structural life of about 100 years.

Table 9.1: The real option initial costs and its uncertainty

Alternative		Initial costs (Millions)		
		Positive	Real option	Negative
1.		€ 32,1	€ 35,3	€ 38,6
2.	Permeable lock chamber	€ 22,4	€ 24,6	€ 26,8
2.	Impermeable lock chamber	€ 23,8	€ 26,2	€ 28,6
3.	With cofferdam	€ 32,3	€ 35,5	€ 38,8
3.	Without cofferdam	€ 32,2	€ 35,5	€ 38,7

As can be seen in Table 9.1, two options are considered for alternative 2. Firstly, the impermeable lock chamber bottom option, which is constructed by underwater concrete and GEWI piles. Secondly, the permeable lock chamber bottom option with a filter layer, seepage cut-off screens and a heavier sheet pile wall, which is the same as the sheet pile wall in alternative 3. The option with the permeable bottom is chosen, because this option has the lowest initial costs.
For alternative 3 also two options are considered for the lock chamber, namely the option with a cofferdam wall at one side and a sheet pile wall on the other side, or the option without a cofferdam wall and a sheet pile wall at both sides. In Table 9.1 is shown that the total initial costs are the same. However, in Table 9.2 can be seen that the real option initial costs in 2020 are much lower for the option without a cofferdam. This option is chosen, because this is favourable for the WLC analysis (paragraph 9.4).

Table 9.2: The real option initial costs for each year

Alternative		Real option initial costs (Millions)		
		2020	2052	Total
1.		€ 10,5	€ 24,8	€ 35,3
2.	Permeable lock chamber	€ 24,6	€ 0,0	€ 24,6
3.	With a cofferdam	€ 20,6	€ 14,9	€ 35,5
3.	Without a cofferdam	€ 18,5	€ 17,0	€ 35,5

On the basis of the real option initial costs in 2020, alternative 1 is the best option. This alternative will be the cheapest in the short term. Because the initial costs are just a part of the WLC analysis, the final conclusion for the best alternative in the long term is determined in the last paragraph of this chapter. In the last paragraph, the initial costs are combined with the costs over the lifetime in the Net Present Value calculation (NPV).

9.3. Benefits and costs over the lifetime

The benefits and the lifetime costs are only determined for the new or renovated lock. In Chapter 6 it was demonstrated that the twin locks (lock 1 and 2) are able to handle the intensities of the small ships in the future. So, the new lock 3 is just important for the larger ships that cannot use the twin locks. As a consequence, the lifetime cost calculations are based on the waiting time, the diversion routes and the diversion possibilities for the large ships. The lifetime costs are formed by the following parts:

- Passing time
- Obstruction time
- Lock depth
- Maintenance
- Loss of water

The benefits and costs are expressed in the lifetime costs, because these costs can be quantified. Nevertheless, every reduction of the costs can be seen as a benefit. The benefits and costs over the lifetime can be affected by more factors, but it is difficult to quantify all these factors in costs or even to determine these factors.
The calculated lifetime costs are fed into the initial cost calculation in Appendix IX. The combination of the initial and the lifetime costs is used to determine the NPV calculation in paragraph 9.4.

9.3.1. Passing time

The passing time of an inland navigation lock consists of two main parts, namely the locking time and the waiting time (Figure 9.1).

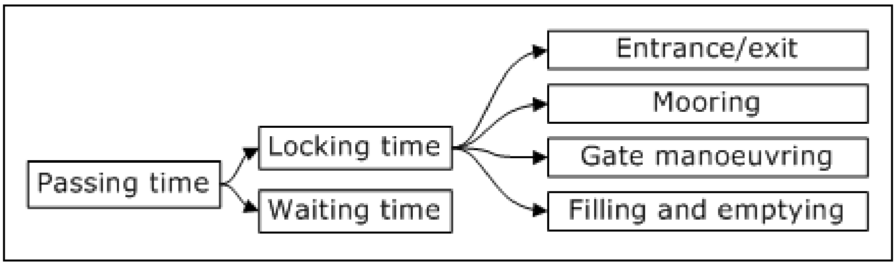

Figure 9.1: Passing time of an inland navigation lock

The locking time is subdivided in four parts. These parts are shown in Table 9.3 with their corresponding percentage of the locking time and the resulting minutes that are required for each part. The passing time is a direct effect of the in Chapter 3 mentioned system capacity. In this chapter, also a detailed scheme of the locking time (½ locking cycle) is shown (Figure 3.4).
The used time indications are relatively short, because it is assumed that all the alternatives have new lock gates which will be equipped with the latest techniques. It is also expected that the equipment of the large ships becomes better in the future, so the entrance/exit and the mooring time will be shorter. The filling and emptying time is calculated separately for each alternative by the method from Appendix IV. The combination of these four parts results in the locking time for each alternative.

Table 9.3: Locking time distribution [PIANC, 2009]

Lock time distribution parts	Percentage	Assumed time (min)
Entrance/exit	18%	3
Mooring	18%	3
Gate manoeuvring	11%	2
Filling/Emptying (Average)	53%	8
Total	100%	16

The waiting time can be calculated with the queuing theory, which was already used in paragraph 6.3. For each alternative, the waiting times are based on the locking time.

The resulting passing times are multiplied by the annual number of large ships (Figure 6.6, 'large 3000t>') that has been forecasted in Chapter 6. This will lead to an annual number of passing hours for the large ships that have to pass 'Sluis Sambeek'. The annual passing times can be multiplied by the relative level of operational costs. These costs are based on the price levels of 2005 and are derived from the report 'Innovations in Navigation Lock Design' [PIANC, 2009]. It is assumed that half of the largest ships will be container vessels. This results in the costs of one waiting hour of around €175, which is corrected for the price level of 2009. The annual passing hours and the corresponding costs are shown for each alternative in Table 9.4.

Table 9.4: The composition of the average annual passing time and resulting costs

Type of lock	Year	Average time (min)			Number of ships	Average passing time (hours)	Annual costs
		Queuing	Locking	Passing			
Alternative 1							
Old lock	2020	2,8	16,9	19,6	3.824	1728	€ 303.107
	2051	4,8	16,9	21,7	6.100		
New large lock	2053	4,2	17,0	21,2	6.182	2454	€ 430.456
	2105	5,3	17,0	22,3	7.321		
Alternative 2 (functional flexible lock)							
New large lock	2020	1,7	14,4	16,0	3.824	1386	€ 243.127
	2051	2,9	14,4	17,2	6.100		
	2053	4,2	17,0	21,2	6.182	2454	€ 430.456
	2105	5,3	17,0	22,3	7.321		
Alternative 3 (structural flexible lock)							
Small lock	2020	2,6	17,5	20,1	3.824	1755	€ 307.955
	2051	4,5	17,5	22,0	6.100		
Large lock	2053	4,2	17,0	21,2	6.182	2454	€ 430.456
	2105	5,3	17,0	22,3	7.321		

9.3.2. Obstruction time

During the rebuilding or the reconstruction of lock 3, only the ships that are too large for the twin locks are obstructed. It has been assumed that large ships that have to pass 'Sluis Sambeek' (paragraph 9.3.1) are not able to pass anymore during the obstruction time. Furthermore, the twin locks can handle the possible increased intensities, during the obstruction of lock 3 (paragraph 6.3). These increased intensities can be handled with a negligible amount of waiting time for the smaller ships (< 'large 3000t>').
The only ships that cannot pass during the rebuilding or the reconstruction of lock 3, are the push tow units with two barges (CEMT - class Vb). In this case the barges must be transported separately through the smaller twin locks. The obstruction costs consist of the extra passing costs of the barges and the separating and connection costs. It has been assumed that the separation and connection time will be 30 min extra for each passing CEMT – class Vb ship. Also the tug boat has to pass the twin locks two times extra to collect the second barge. The obstruction costs and the resulting weekly costs are shown in Table 9.5.

Table 9.5: The obstruction time and the resulting weekly costs

Type of lock	Additional waiting time		Number of ships	Duration (weeks)	Weekly		Total obstruction cost
	2* twin-locks (min)	Separating (min)			Hours	Costs	
Alternative 1 (2052)							
Old lock => New large lock	24	40	6.141	156	106	€ 22.100	€ 3.447.599
Alternative 2 (functional flexible)							
No obstruction costs							
Alternative 3 (structural flexible) (2052)							
Small lock => Large lock	24	40	6.141	16	106	€ 22.100	€ 353.600

There are no obstructions during the construction of the first stage of the lock in 2020 (for all alternatives), because lock 1 and 2 (twin locks) are able to cope with the passing ships. The obstruction time during the lifetime of the structure from 2020-2105 is determined separately for each alternative.

Alternative 1
In case of alternative 1, a new lock has to be constructed around 2052 at the lock location, so the renovated lock will be obstructed during the complete construction time. The recently constructed 'Maxima Sluis' near Lith had a construction time of three years [Glerum,2000]. This lock is comparable with the new large lock of alternative 2. Therefore, it is assumed that the obstruction time will be around 156 weeks (three years)

Alternative 2
Alternative 2 has no obstruction costs at all, because this lock will last for the functional and structural life of 100 years.

Alternative 3
The obstruction time that is expected for alternative 3 is only 16 weeks, namely 8 weeks to replace the upstream lock head and 8 weeks to replace the downstream lock head. For each head, the replacing consists of five phases of 1,5 week. Also half a week is taken for reserve. The five phases are:

- Removing the sand from the caissons
- Let the head float up and remove it
- Create a new foundation bed for the new large head
- Float in the new large head
- Ballast the new large head

This relatively short time is the result of the flexibility of the lock design. Most activities, like the widening of the lock chamber and the widening of the lock head pit can be done during operation of the lock. The large flexible lock heads are built at another location in a building pit upstream 'Sluis Sambeek'. During the construction of the lock heads, the lock can operate as usual. The heads have to be immersed separately to guarantee the water management function of 'Sluis Sambeek'.

9.3.3. Lock depth

The depth of the renovated old lock in alternative 1 is 4 m instead of the 4,7 m of the lock 3 in alternative 2 and 3. This means that from 2020 till 2052, only CEMT-class Vb ships (push tow unit with two barges), with a maximum depth of 3,5 m instead of the normal 4 m, can pass 'Sluis Sambeek' in alternative 1. Therefore, the tonnage that can be transported by these push tow units through the renovated old lock (alternative 1) is less than in case of alternative 2 and 3. This is a disadvantage of alternative 1. It is not possible to quantify the costs of this loss of tonnage in this stage, because an economic analysis of the Meuse route is necessary for this. Thus in this analysis the loss of tonnage in alternative 1 will be taken into account as a negative point of this alternative.

9.3.4. Maintenance

Maintenance is needed for all the alternatives. Lock 3 is in the same condition for each alternative, because the old lock of alternative 1 will be renovated and alternative 2 and 3 are new. Thus in this cost analysis the maintenance costs are not included for the reason that no large differences are expected in the costs for the different alternatives. Nevertheless, in an ideal LCM analysis these costs must be taken into account as well.

9.3.5. Loss of water

The Meuse has a standard flow of 10 m^3/s near 'Sluis Sambeek', thus it is always possible to lock ships from the upstream side to the downstream side and vice versa. In the ideal situation the loss of water is not an issue. Because the twin locks and the largest lock together (new large lock), can handle more than 100 lockings a day, with this standard flow, this should be enough to cope with the expected intensities. However, in practice the loss of water is an issue during dry periods [Burhenne, 2009], because a lot of water is spilled over the weir to maintain the water levels in the downstream head up sections. Therefore, the waiting times become longer and only full lock chambers are levelled up and down. Alternative 3 has the advantage in comparison with alternative 1 and 2 that it has a smaller lock chamber (2020 - 2052). This results in less loss of water, so the locking process is more flexible and results in shorter waiting times during the dry periods.

9.4. Whole Life Costing (WLC) calculation

The WLC of the three alternatives are calculated by using a Net Present Value (NPV) calculation. For each alternative, the final costs in 2105 are determined on the basis of the positive, the negative and the real option initial costs. This in combination with the assumed real interest rate of 1,9% and the real option lifetime costs leads to the WLC as represented in Table 9.6. As an example, the NPV calculation of alternative 3 can be seen in Appendix X.

Table 9.6: Whole Life Costing (WLC) for the year 2105

	WLC (Millions)		
	Positive	Real option	Negative
Alternative 1	€ 147,0	€ 158,1	€ 169,1
Alternative 2	€ 143,9	€ 155,1	€ 166,4
Alternative 3	€ 160,6	€ 173,3	€ 186,0

The initial costs that were calculated for 2020 in paragraph 9.2 are spread over three years in the NPV for each alternative, because this is the normal construction time for an inland navigation lock ('Maxima Sluis'). The initial costs for 2052 are spread for each alternative differently. The construction of the new large lock of alternative 1 will be spread over three years. This was already determined in the obstruction time calculation (paragraph 9.3.2). The construction of the large flexible lock of alternative 3 will last only two years, because the lock chamber only needs some reconstruction and the locks heads can be floated in, in a relatively short time. Alternative 2 has no reconstruction costs in 2052.
On the basis of the WLC it can be determined that alternative 3 is not a good option, because the price is much higher than the price of the other two alternatives. The other two alternatives almost have the same real option whole life costs.

9.5. Risk analysis

In this LCM analysis, a lot of risks are involved in the WLC calculations, because the calculated values are real option values. This means that all these values have an uncertainty. This uncertainty can be expressed in different risk factors. These risk factors are:

- The expected real interest percentage is 1,9%. A small change in this percentage can have a big effect on the alternative selection.
- The predicted year when reconstruction is needed can be change by a different trend in the ship passages than was forecasted in Chapter 6. It is even possible that no reconstruction appears to be needed.
- The initial costs of each alternative can become higher or lower than is estimated in this report.

Besides the uncertainty in the calculated values, also external factors are involved that can change the indirect effects and costs. For instance, it is possible that when the total Meuse route from 'Sluis Weurt' till 'Sluis Ternaaien' is extended, it will attract more ship traffic than was predicted in Chapter 6. It is not within the scope of this research to take these factors into account as well.
In Figure 9.2, the selection of an alternative is represented in a decision tree [CUR, 1997]. The different real option values with their uncertainties are represented in this decision tree. However, it is beyond the scope of this research to connect a chance to the different paths that can be followed in the decision tree. It can also not be estimated within which real interest or within which range of reconstruction years the uncertainty must be located.

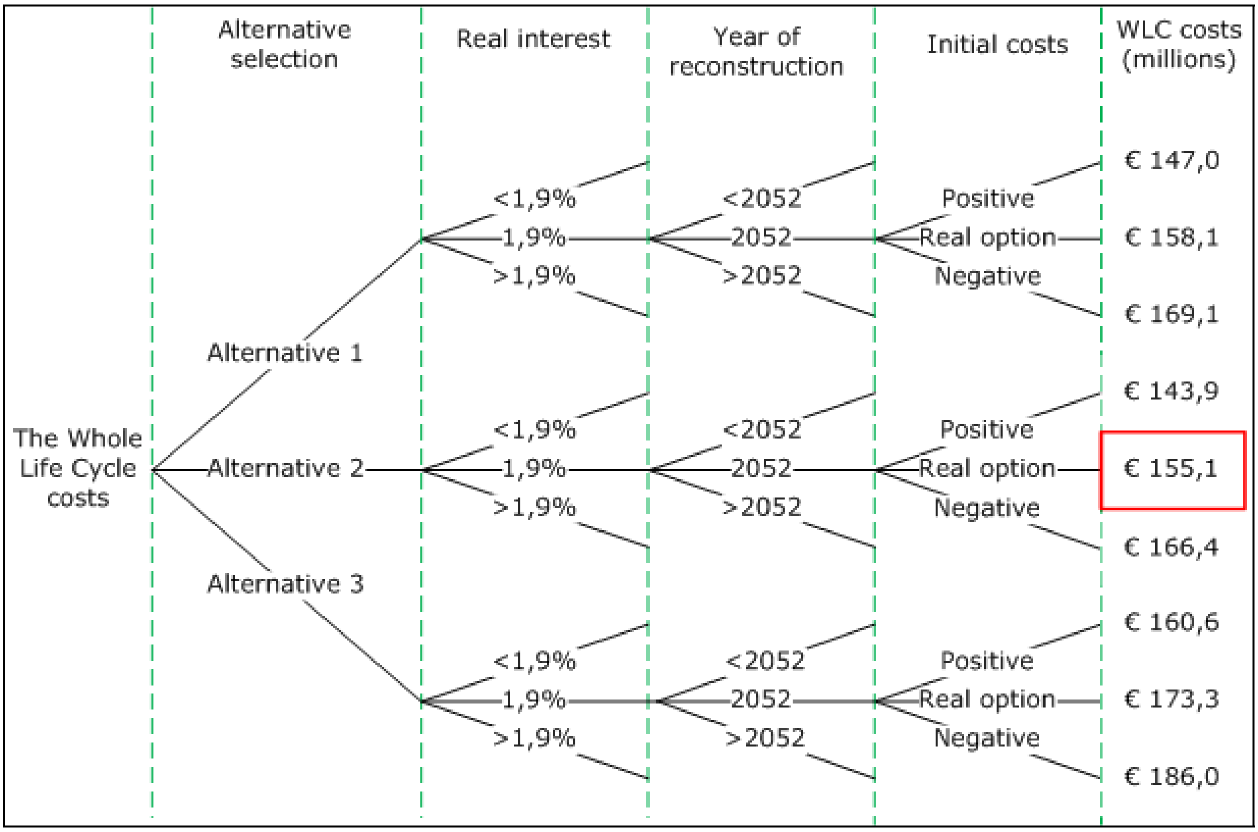

Figure 9.2: Alternative selection, decision tree

Assumed that the real interest is 1,9% and the year of reconstruction is 2052, as is determined in this report, the costs of the real option for each alternative are determined (see Figure 9.2). Alternative 2 appears to have the best whole life real option costs.
The best way to decrease the uncertainty in this LCM analysis is refining the initial cost determination. The initial costs can easily be made more reliable. For instance by making a more detailed design than the rough designs from Chapter 8. It will be more difficult to refine the real interest or the year of reconstruction, because these data are based upon a forecasted trend that may change completely due to unexpected events.

9.6. The resulting alternative

The Whole Life Costing (WLC) is used to determine which of the three alternatives will be the best solution for the lock complex reconstruction of 'Sluis Sambeek'. From this cost calculation, it should be concluded that alternative 3 is too expensive. Though, the water saving capacity of alternative 3 is better than the water saving capacity of the other two alternatives. Alternatives 1 and 2 are compared in the risk analysis. Alternative 2 appears to be the best solution. So it is better to construct a new large lock with an expected functional lifetime of 100 years at once, than to renovate the existing old lock 3 and built the new large lock in the assumed year 2052.
An additional motivation for the selection of alternative 2 instead of alternative 1 is the sill depth. Alternative 1 will have a reduced water depth above the sill in the period from 2020 till 2052, which will result in a loss of transported tonnage for each passing ship.

In case of 'Sluis Sambeek', the structural flexible lock of alternative 3 is not an option, because the WLC are too high compared to the other two alternatives. The weekly obstruction costs, during the reconstruction of lock 3, are relatively low for 'Sluis Sambeek', because lock 1 and 2 of the complex can handle the traffic intensity without almost no additional waiting time. Thus the structural flexible lock of alternative 3 could be a better option when the ships that pass the lock depend more on the lock availability than in case of 'Sluis Sambeek'. A bigger dependency on the lock availability will result in higher weekly obstruction costs, which lead to a shorter desired obstruction and construction time.
Also the relatively short periods with a too small discharge in case of 'Sluis Sambeek', are disadvantageous for alternative 3. Because alternative 3 saves a lot of water compared to the other two alternatives, alternative 3 is a better option when the discharge is often small.
Furthermore, alternative 3 is less expensive compared to alternative 2, when no reconstruction appears to be necessary, because the initial costs of alternative 3 in 2020 are less than the initial costs of alternative 2.

Finally, it can be concluded that alternative 2 is the best solution for the lock 3 (re)construction of 'Sluis Sambeek'. However, alternative 3 is the most innovative structural option of the three alternatives and could be a better solution for another lock reconstruction where the boundary conditions are more favourable for alternative 3. Therefore, alternative 3 is worked out in the next chapters.

10. Detailed structural flexible lock design

The structural design of alternative 3 contains standard parts and innovative parts. In this graduation report, only the innovative parts will be discussed. In this structural flexible lock design, four different innovative parts are distinguished:

1. The float up of the small lock heads after 40 or 50 years (paragraph 10.1).
2. The construction planning, which is different than in case of a standard lock reconstruction (paragraph 10.2).
3. The structural design of the floating lock heads that have to be checked and optimised (paragraph 10.3).
4. The water tight connections that have to be created between the lock chamber and the lock heads (paragraph 10.4).

Due to the limited time, it is not possible to work out all four innovative parts. That is why only the structural design of the floating lock heads (part 3) will be elaborated. Nonetheless, the construction planning (part 2) is worked out partly to identify possible loads and load combinations that are relevant for the lock heads. However, in this chapter a short explanation of all four innovate parts is given, which can be used to support further research into these parts and to give insight in the problems that can occur in a structural flexible lock design.

10.1. Floating up the lock heads

After 40 or 50 years, the small lock heads need to be floated up again. To fulfil this requirement, water must be able to flow under the lock head. This must be a sufficient amount of water to create enough lift to float the lock head. This principle is shown in Figure 10.1. When the lock head is founded on a gravel layer, it is possible that after 40 or 50 years the layer is not permeable enough anymore to provide the needed water flow. This reduced permeability could be caused by a silt layer against the sides of the gravel layer or by a clogged gravel layer. Nevertheless, there is no guarantee that this float up will really cause problems. Further studies have to be performed about this issue, to make this sure.

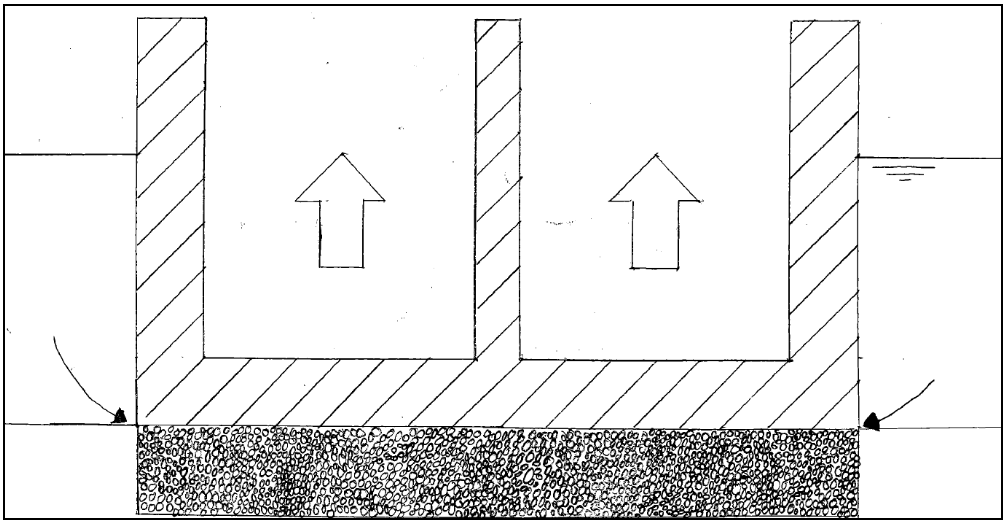

Figure 10.1: The float up of the lock head and the water flow under it

When it is concluded that this floating up after 40 or 50 years really is a problem, three solutions for this problem are listed below.

- Sink the heads down upon a geometrically closed filter, which is designed according to the following design rules [Schiereck, 2001].

  Stability: $\frac{d_{15F}}{d_{85B}} < 5$, internal stability: $\frac{d_{60}}{d_{10}} < 10$ and permeability $\frac{d_{15F}}{d_{15B}} > 5$.

This will result in a gravel layer without movement of the particles and a guaranteed permeability, but this solution is still vulnerable for silt layers that will close of the sides of the filter gravel layer.
- Use horizontal plastic tubes that will be imbedded in the gravel layer to clean the gravel layer with high water pressure when the heads have to be lifted up.
- Use vertical tubes through the concrete, which can be used to clean the gravel layer with high water pressure.

10.2. Construction planning

The construction planning of a structural flexible lock is relatively complex compared to a standard lock. This is because two construction plans are needed: one for the construction of the new structural flexible lock in 2020, and one for the width extension of the lock around 2052. Moreover, the lock will be (re)constructed at two places, namely the lock location and the building pit for the floating heads. This is the case for both construction plans.
The reconstruction planning of the lock (2052), will be worked out in broad outlines in Chapter 11. The construction of the initial structural flexible lock (2020) is not considered in this report. Only the construction planning around the large upstream lock head is determined in detail, because this lock head will be analysed further in Chapter 12.

10.3. Structural floating lock head design

The floating lock heads of alternative 3, which are designed in Chapter 8, are a new way of combining a lock head with a floating caisson. The concrete construction of the floating lock heads needs to fulfil the displacement requirements and strength requirements that are stated in the program of requirements (paragraph 7.3). These requirements have to be satisfied for all the governing load combinations in the construction phase and users phase. For instance, the heads must resist the force from the water, the soil and the gates onto the lock head wall during the users phase. On the other hand, the lock head wall also has to transfer the forces from the lifting side caissons to the gate chamber during the construction phase when the lock heads float up.
This innovative part is worked out in detail for one lock head, because the load combinations and the design of the lock heads are almost the same for each lock head. In Chapter 12, the large upstream lock head will be checked and optimised, because it is assumed that this lock head has to bear the heaviest loads.

10.4. The connection of the lock head with the lock chamber

Because the lock chamber has a permeable bottom, measures have to be taken to prevent piping under or beside the lock heads. In case of a standard lock with a permeable bottom, the seepage cut-off screen is poured into the concrete at the top and the walls are connected watertight to the lock chamber. When the lock heads are floated in, the water tightness has to be created in another way. The solution for this connection problem comes from the construction of immersed tunnels. These tunnels consist of parts that are immersed on a river or channel bottom. These parts are connected water tight by using rubber profiles that are shown in Figure 10.2.

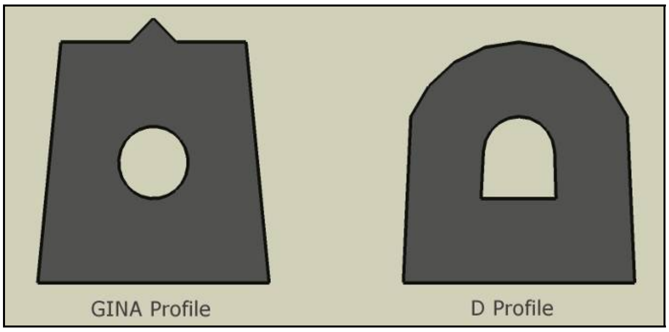

Figure 10.2: Connection profiles

These profiles are squeezed by the water pressure, which pushes the new immersed tunnel part against the one that was already immersed. This principle can be seen in Figure 10.3. In this way a watertight connection is created.

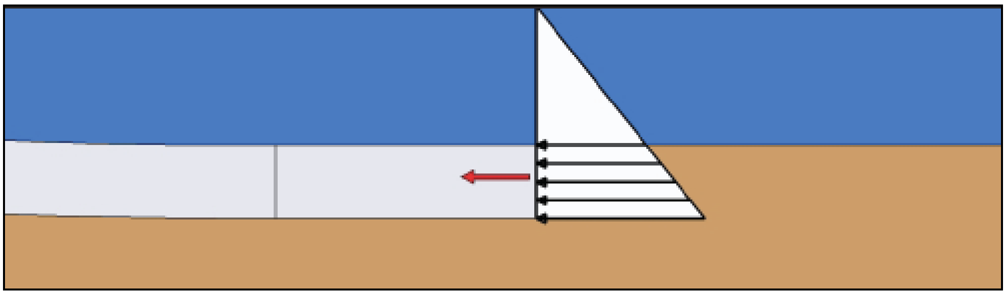

Figure 10.3: Immersion force [Jong, 2009]

To use one of these connection profiles, a frame of concrete and steel have to be placed at the end of the lock chamber, so a flat surface is created. The frame can be seen in Figure 10.4.

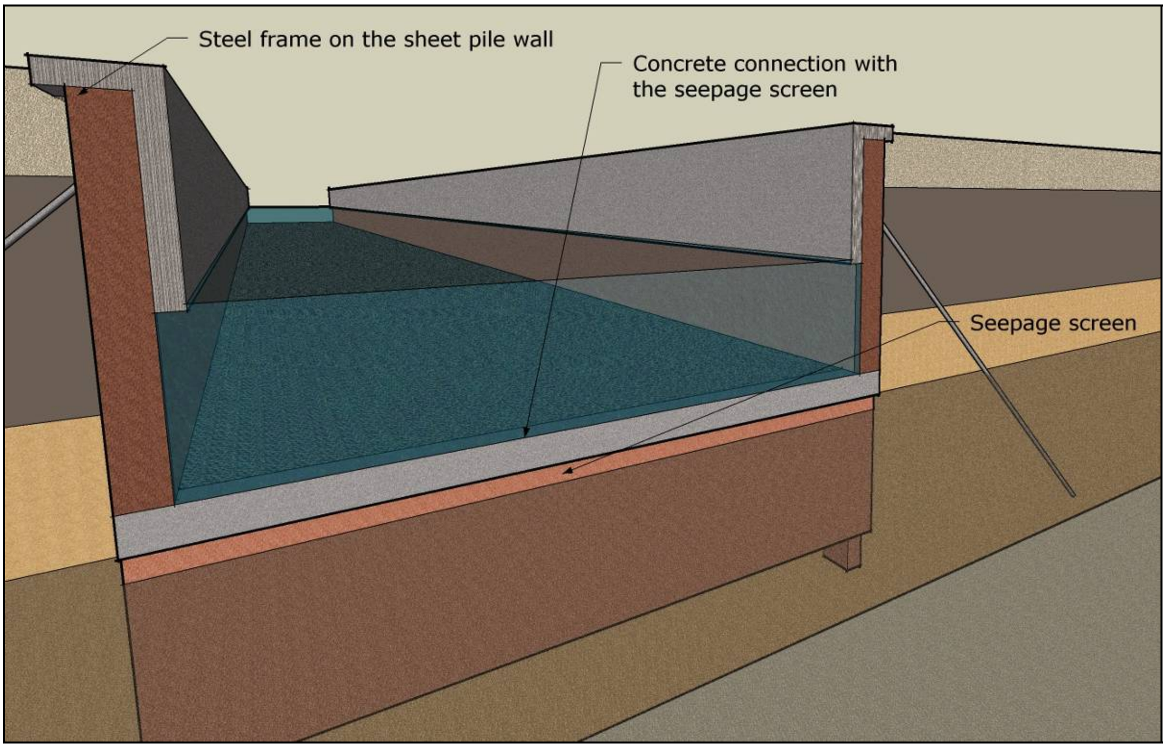

Figure 10.4: Frame on the lock chamber for the watertight connection

The rubber profile is placed onto the floating lock head. Consequently, a force comparable to the shown immersion force is needed to create the water tight connection between the lock chamber and the lock head. This force has to be created by different water levels on both sides of the lock head, so the head is pushed against the frame. This water level difference can be created by placing a screen on the concrete connection sill and pump the water away between the head and the frame, which is shown in Figure 10.5 on the next page. After enough force is created, the side caissons of the lock heads have to be filled with sand to maintain the needed immersion force. This method is certainly needed for the downstream lock head, because no favourable water level difference is available. Research is needed to determine whether the same method is needed for the upstream lock head. Furthermore the detailed design of the watertight connection would be an interesting study object.

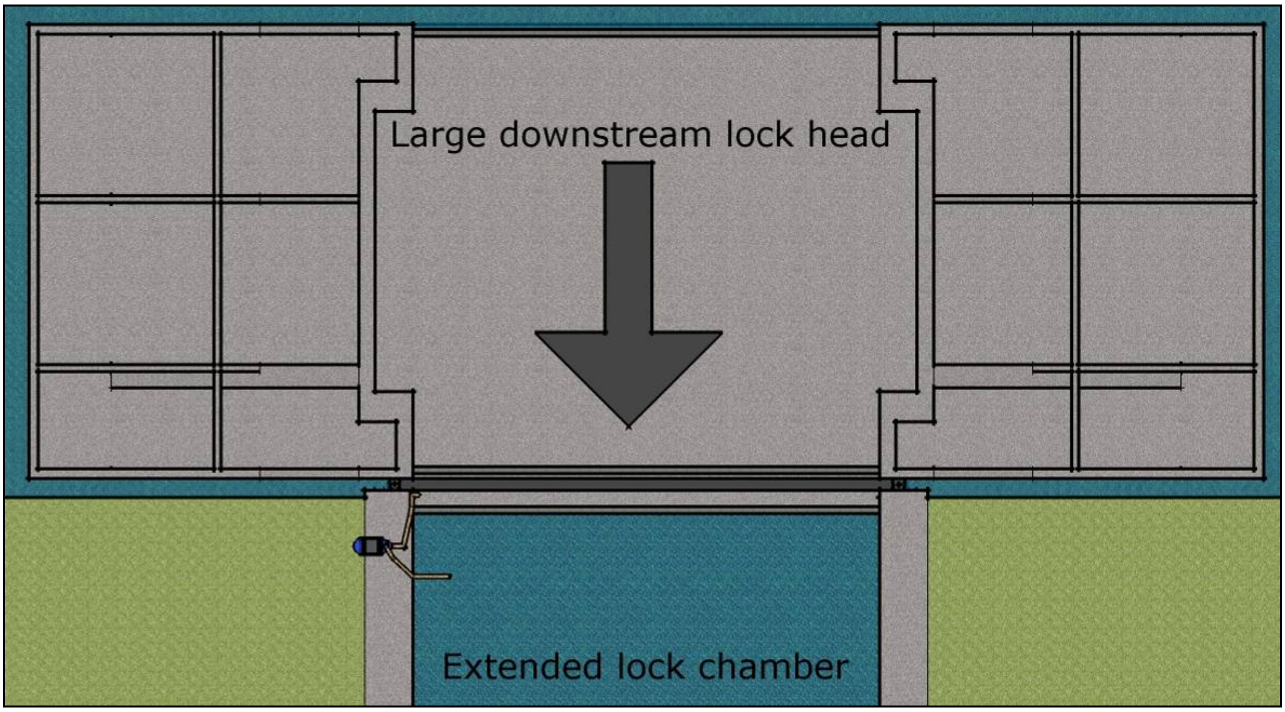

Figure 10.5: Immersion force creation for the watertight connection

11. Construction planning (2052)

11.1. Construction planning of the reconstruction

Only the construction planning of the reconstruction, which is expected to take place in 2052, of a structural flexible lock is considered in this report. This will lead to the detailed construction planning of the large upstream lock head, including the lock head construction in the building pit, the transportation of the floating head, and the floating in and immersion of the lock head. This information is used to determine the different load combinations for the lock head design (Chapter 12). In the next three paragraphs, the construction planning is worked out. First, the layout of the structural flexible lock that is planned to be build in 2020 is sketched. This will be the starting point for the reconstruction in 2052. Second, the construction planning at the lock location is drawn up. Finally, the construction planning at the building pit location is discussed. These (re)construction plans are put together in a detailed situation sketch for every load combination in Chapter 12.

11.2. Existing situation before the reconstruction (2020-2052)

In Figure 11.1 the existing situation before the reconstruction is depicted. The way that this structural flexible lock is built is not worked out in this graduation report.

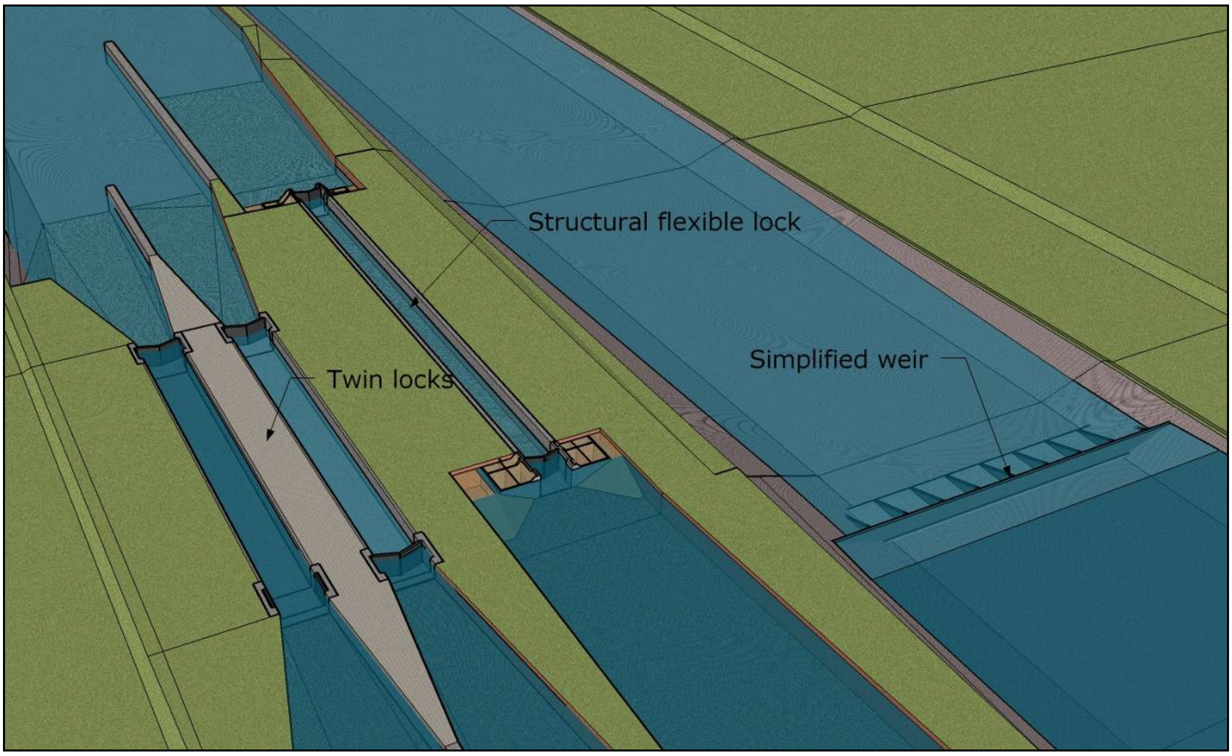

Figure 11.1: Existing situation before the reconstruction

This lock is built according to the conceptual design of paragraph 8.6. As can be seen in Figure 11.1, the structural flexible lock is located at the place of the old lock and fits in the lock complex 'Sluis Sambeek'.
The lock heads are founded on a gravel layer and are watertight connected to the lock chamber with rubber profiles as shown in paragraph 10.4. The seepage cut-off screens that are part of this watertight connection are already constructed wide enough to fit the large lock heads that will be placed in 2052. The bays in which the small lock heads are situated are assumed to be wide enough to float in the large lock heads. These bays are created by sheet pile walls or by another not further specified soil retaining structure. As a result, the small lock heads and the new large lock heads only have to resist the soil loads of the filling sand.

The lock chamber is constructed, as was specified in paragraph 8.6.2, with at both sides a single sheet pile wall with grout anchorages.

11.3. Construction planning at the lock location

To illustrate the reconstruction of the structural flexible lock, the order of execution at the lock location is divided in 15 phases. The planning of this reconstruction is not expressed in time, because this has no effect on the load combinations, which is the focus of this chapter.
The floating lock heads are controlled during floating. The lock heads will be float up and immersed by using the caissons as ballast tanks.
The dimensions of the upstream and the downstream lock head can be found in Appendix VII. The draft of the large upstream lock head is 5,11 m and the draft of the large downstream lock head is 4,6 m.

Phase 1 (Figure 11.2):

- Remove the prefab wall slabs of the lock chamber side that is extended
- Place the new sheet pile wall at the side of the lock that is extended
- Anchor the new sheet pile wall
- Dredge the sand between the old and the new sheet pile wall

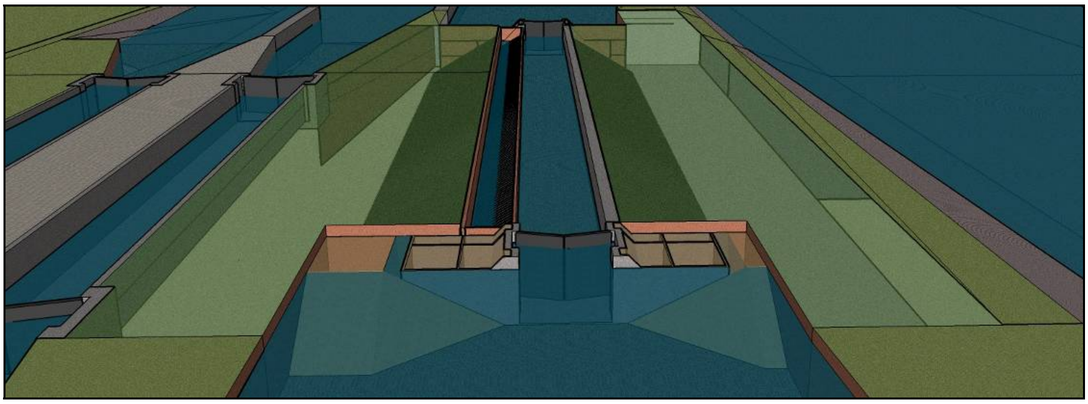

Figure 11.2: Lock location reconstruction phase 1

Phase 2 (Figure 11.3):

- Remove the anchorages of the old wall
- Remove the old sheet pile wall
- Dump the additional filter layer
- Place the new prefab wall slabs

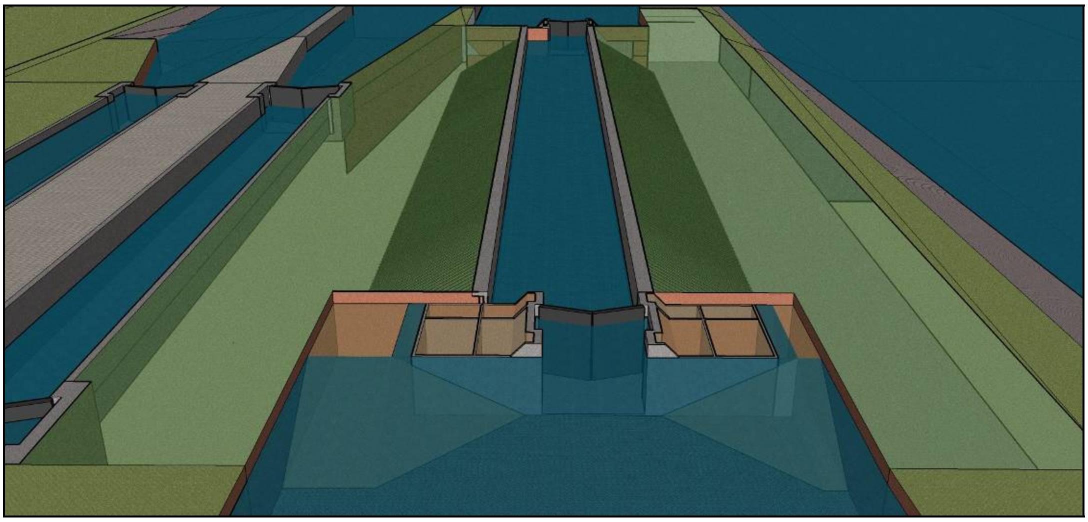

Figure 11.3: Lock location reconstruction phase 2

Phase 3 (Figure 11.4):
- Place the bulk heads in the small upstream lock head
- Remove the gates of this head

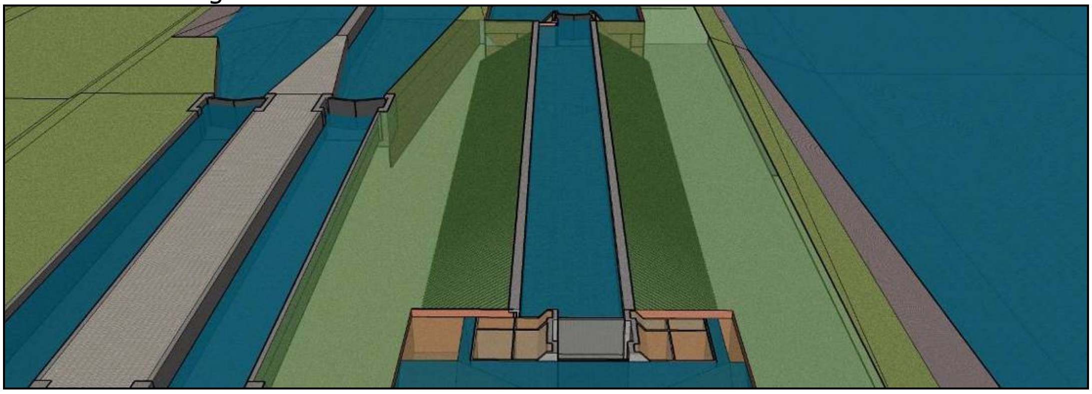

Figure 11.4: Lock location reconstruction phase 3

Phase 4 (Figure 11.5):
- Remove the ballast sand of the small upstream lock head
- Excavate the soil around this head

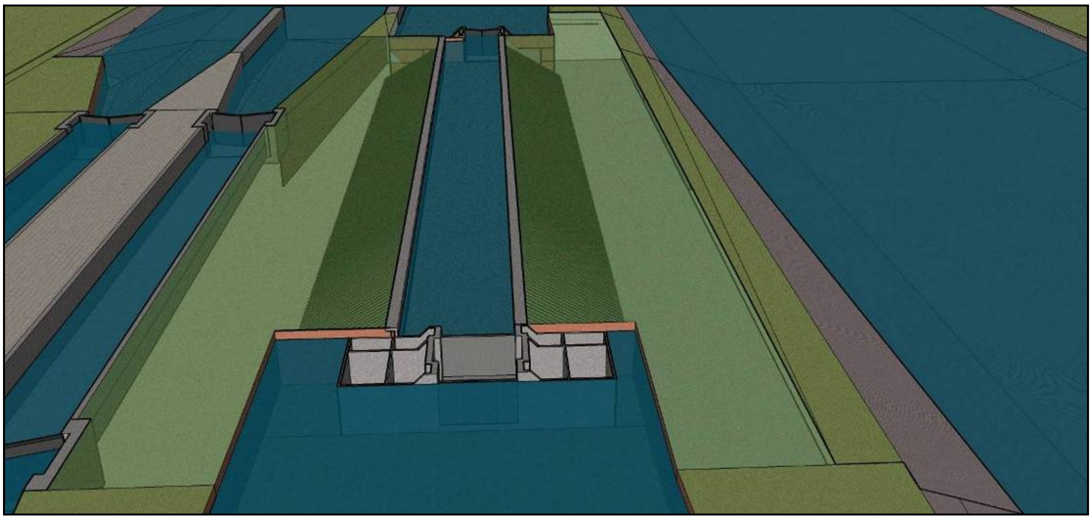

Figure 11.5: Lock location reconstruction phase 4

Phase 5 (Figure 11.6):
- Float out the small upstream lock head

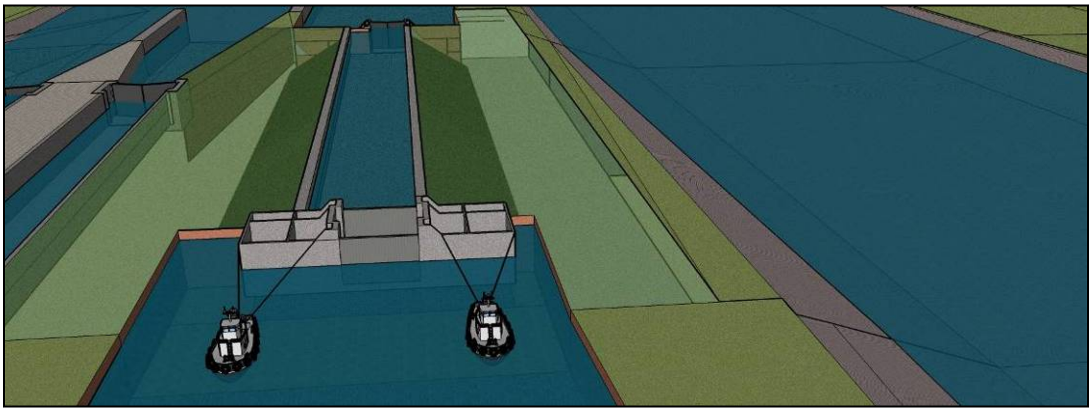

Figure 11.6: Lock location reconstruction phase 5

Phase 6 (Figure 11.7):

- Create place for the large upstream lock head by excavating the remaining soil
- Prepare the gravel bed for this head
- Prepare the connection for this head with the lock chamber

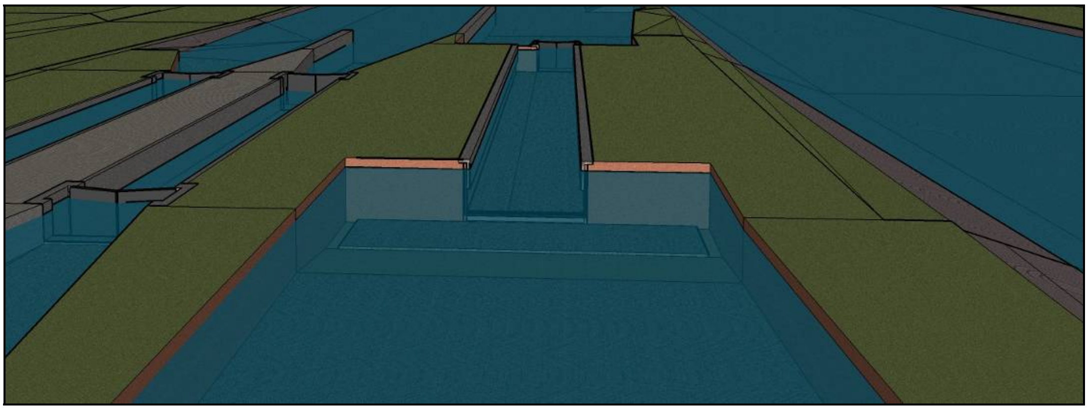

Figure 11.7: Lock location reconstruction phase 6

Phase 7 (Figure 11.8):

- Float in the large upstream lock head
- Immerse this head on the gravel bed

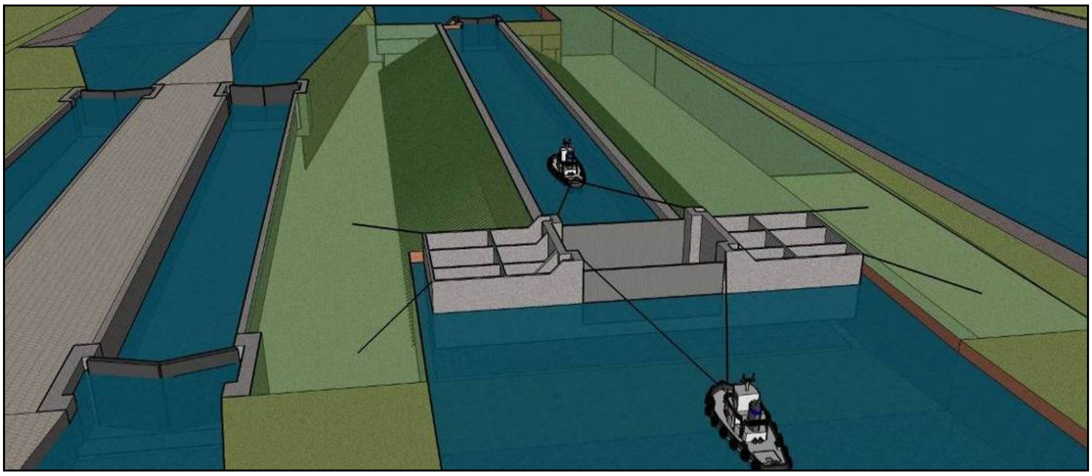

Figure 11.8: Lock location reconstruction phase 7

Phase 8 (Figure 11.9):

- Ballast the large upstream lock head
- Place the gates in this head

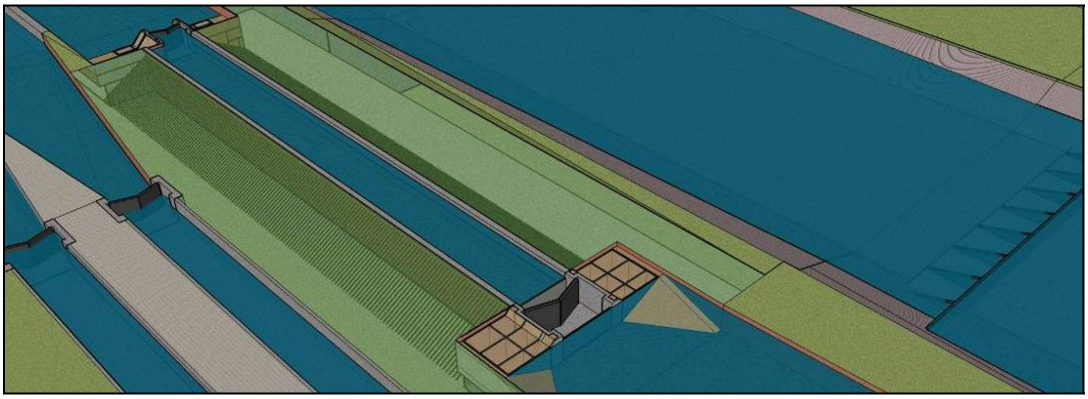

Figure 11.9: Lock location reconstruction phase 8

Phase 9 (Figure 11.10):
- Remove the bulk heads of the large upstream lock head
- Float the large downstream lock head into the extended lock chamber

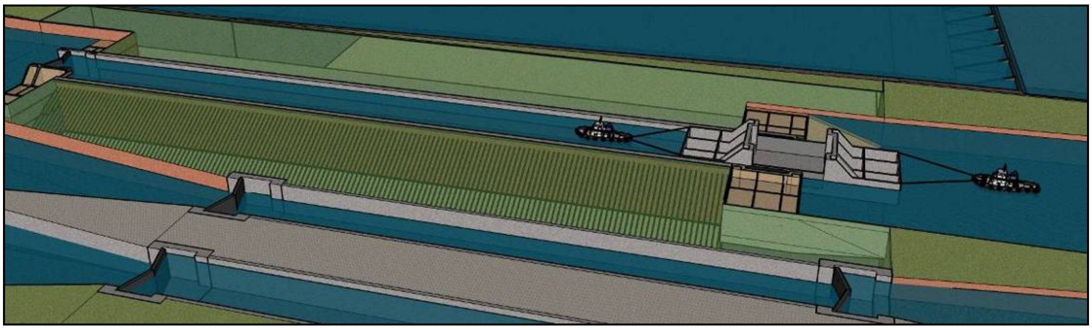

Figure 11.10: Lock location reconstruction phase 9

Phase 10 (Figure 11.11):
- Lower the lock level to the downstream water height
- Place the bulk heads in the small downstream lock head
- Remove the gates of this head

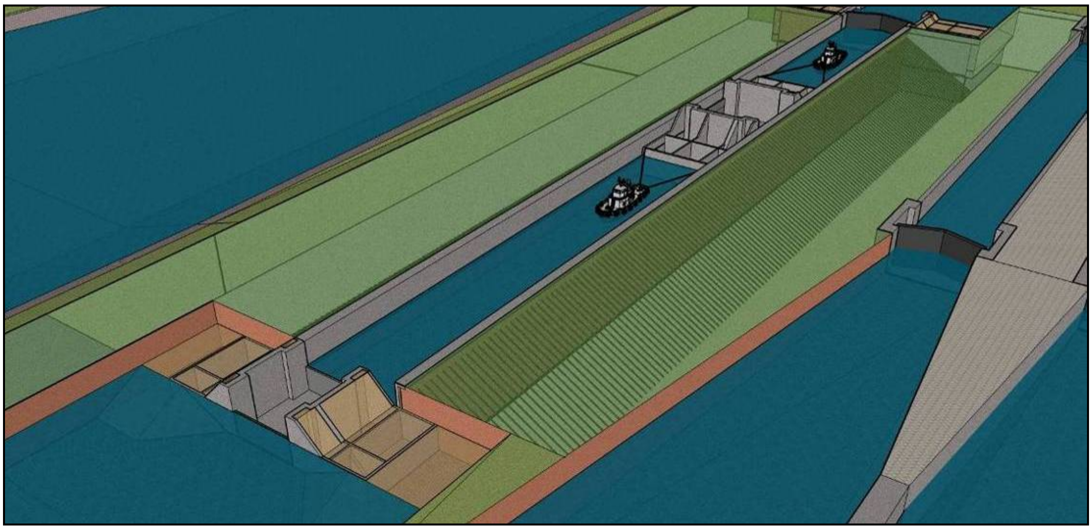

Figure 11.11: Lock location reconstruction phase 10

Phase 11 (Figure 11.12):
- Remove the ballast sand of the small downstream lock head
- Excavate the soil around this head

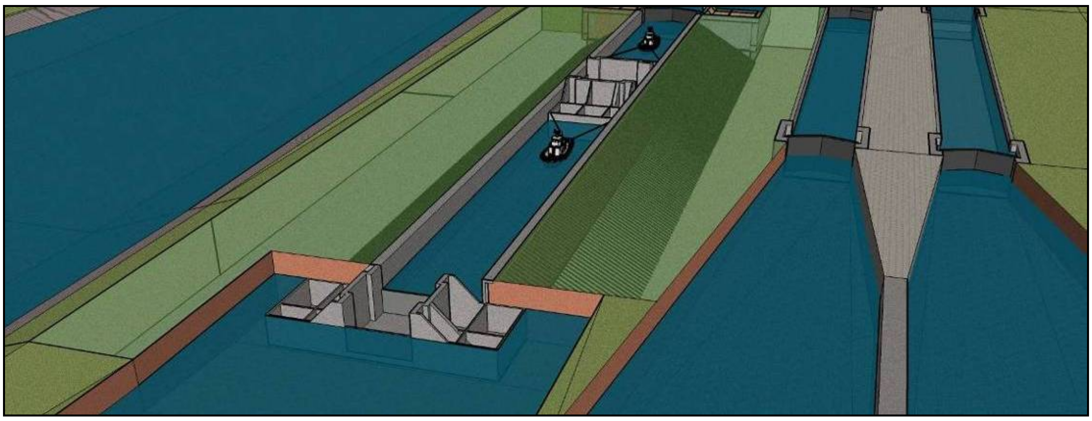

Figure 11.12: Lock location reconstruction phase 11

Phase 12 (Figure 11.13):

- Float out the small downstream lock head

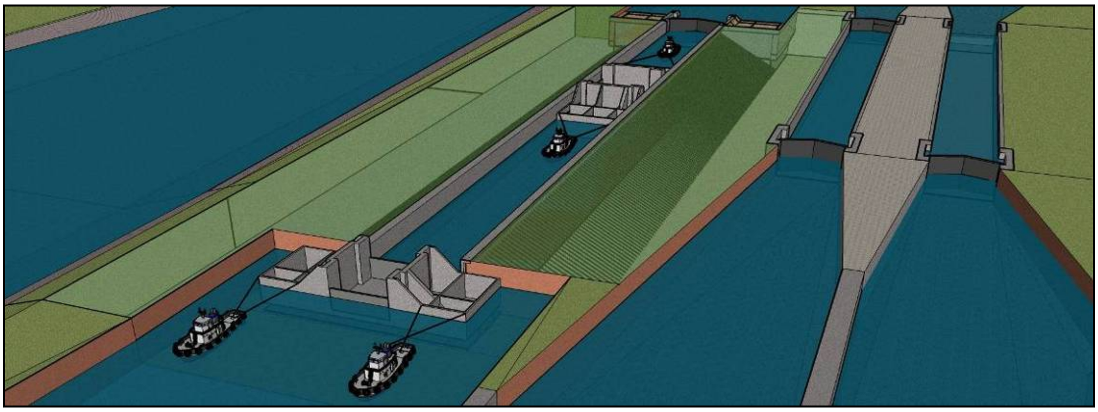

Figure 11.13: Lock location reconstruction phase 12

Phase 13 (Figure 11.14):

- Create place for the large downstream lock head by excavating the remaining soil
- Prepare the gravel bed for this head
- Prepare the connection for this head with the lock chamber

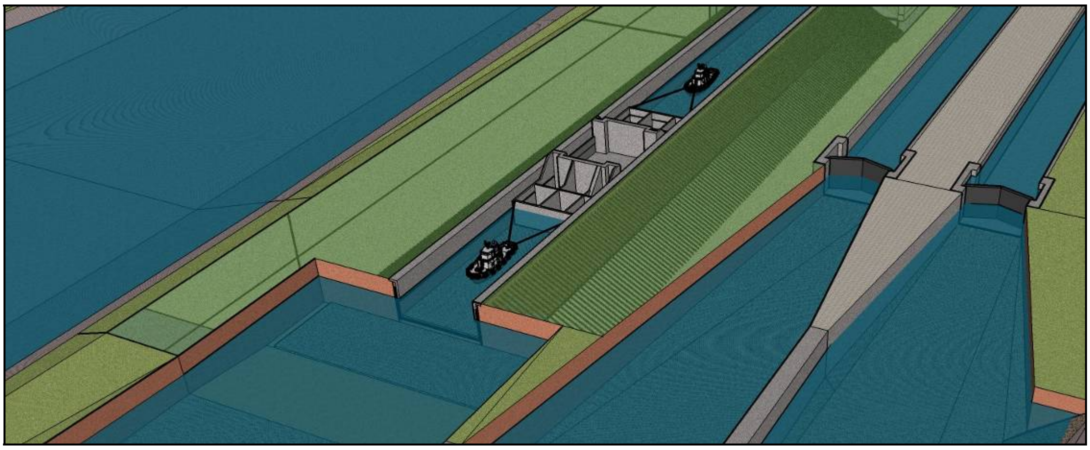

Figure 11.14: Lock location reconstruction phase 13

Phase 14 (Figure 11.15):

- Float in the large downstream lock head
- Immerse this head on the gravel bed

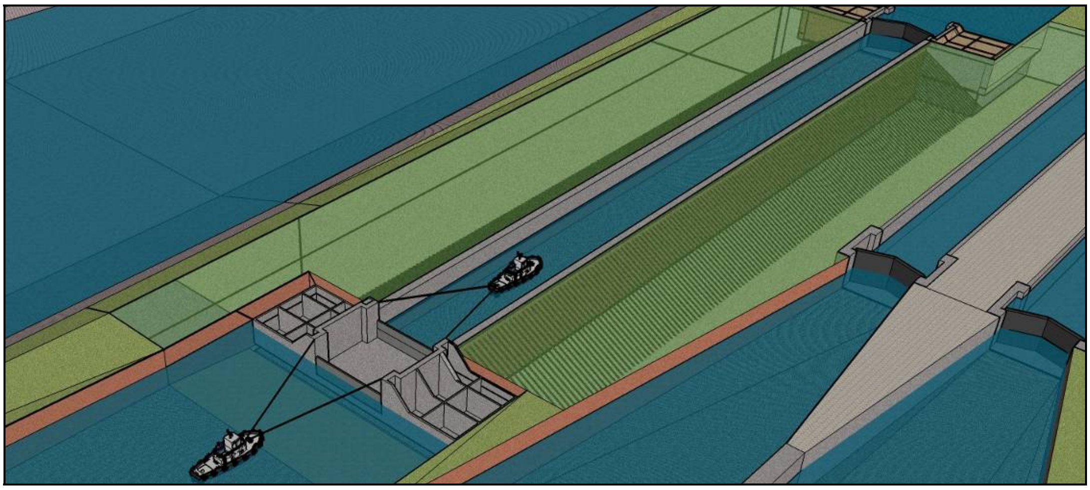

Figure 11.15: Lock location reconstruction phase 14

Phase 15 (Figure 11.16):

- Ballast the large downstream lock head
- Place the gates in this head
- Remove the bulk heads of this head

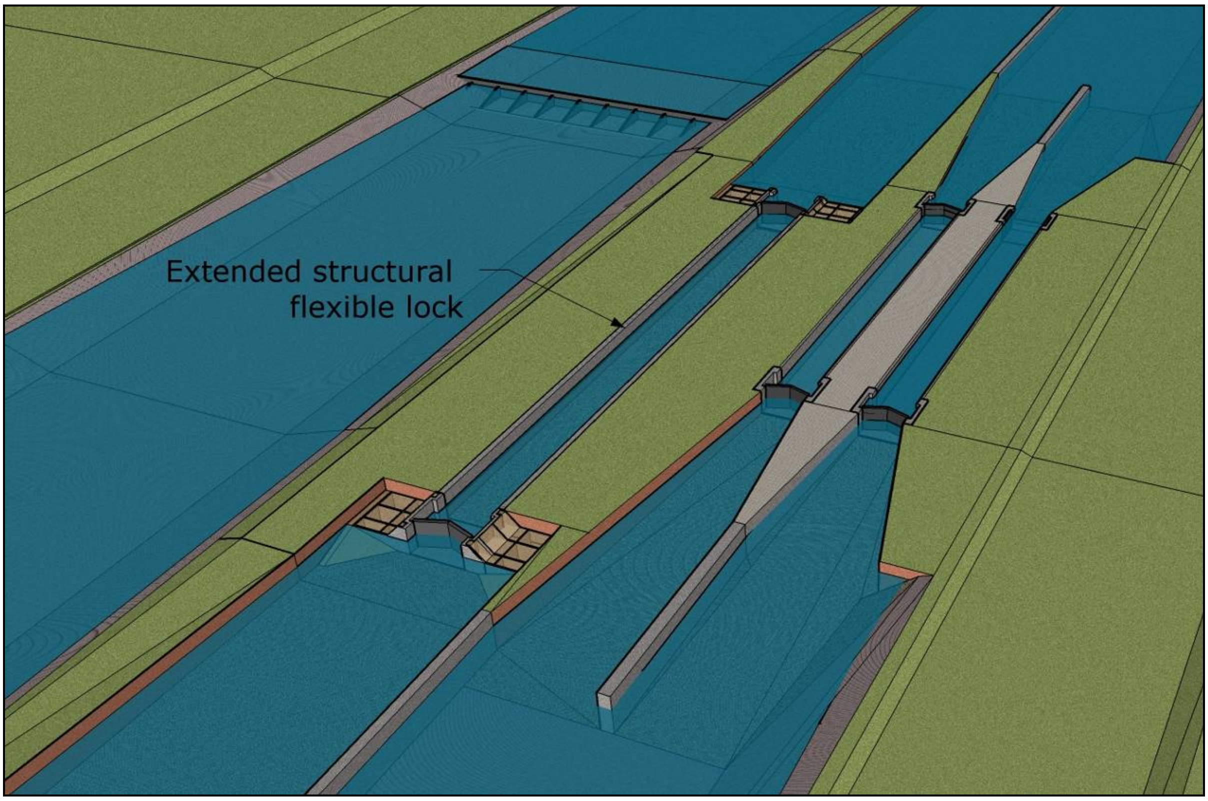

Figure 11.16: Lock location reconstruction phase 15

After phase 15, the extended large inland navigation lock is ready to be used. The time from phase 3 till phase 15 is assumed to be 16 weeks. This will be the time that the lock is obstructed.

11.4. Construction planning in the building pit

During the activities from phase 1 and phase 2 at the lock location, the floating lock heads are constructed in a building pit upstream of 'Sluis Sambeek'. This building pit will be located in the river foreland of the Meuse. The construction method of the large lock heads is sketched shortly, because this is not important for the different loading combinations that can occur. It is assumed that phase 1 till phase 5 at the building pit location will take about 26 weeks. After the heads are ready, they are transported to the lock location where they will be immersed.

Phase 1 (Figure 11.17):

- Excavate the building pit in the river foreland
- Place the seepage screens in the dikes
- Install the well-pointing system and dewater the building pit
- Prepare a working floor on the bottom of the pit
- Dump the gravel that will be situated under the floor of the lock heads

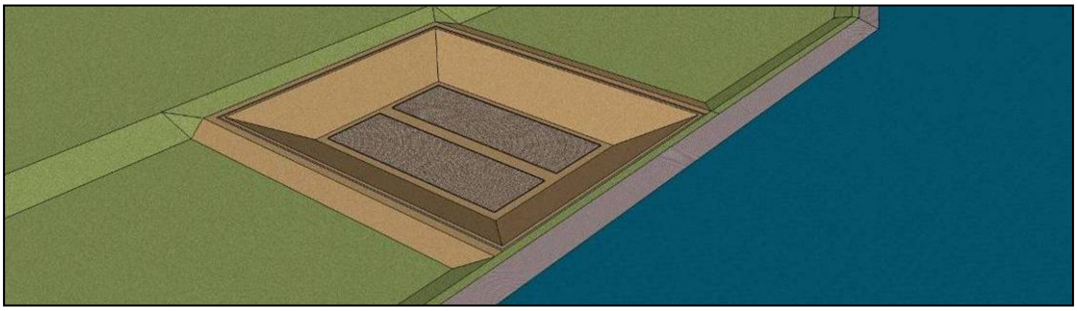

Figure 11.17: Building pit construction planning phase 1

Phase 2 (Figure 11.18):

- Place the form works for the floor
- Place the floor reinforcement steel
- Place the pivot
- Pour the concrete floor
- Place the wall formworks in series after the concrete is cured
- Place the wall reinforcement in series
- Place the collar strap
- Pour the wall concrete in series

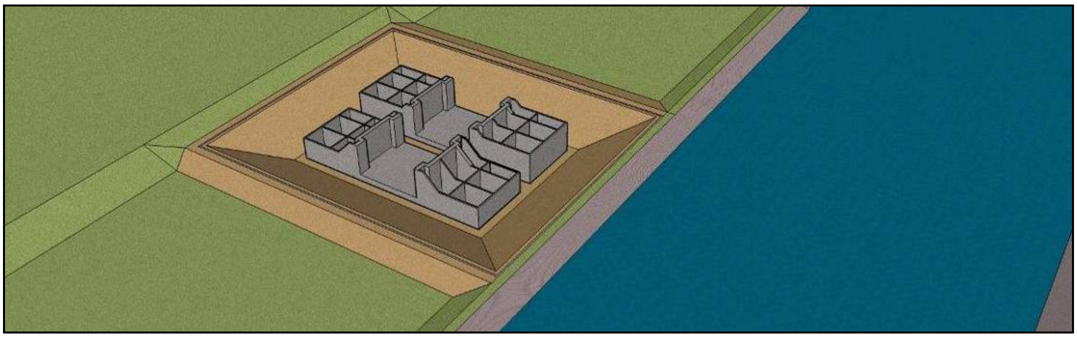

Figure 11.18: Building pit construction planning phase 2

Phase 3 (Figure 11.19):

- Place the bulk heads
- Place the tow bollards

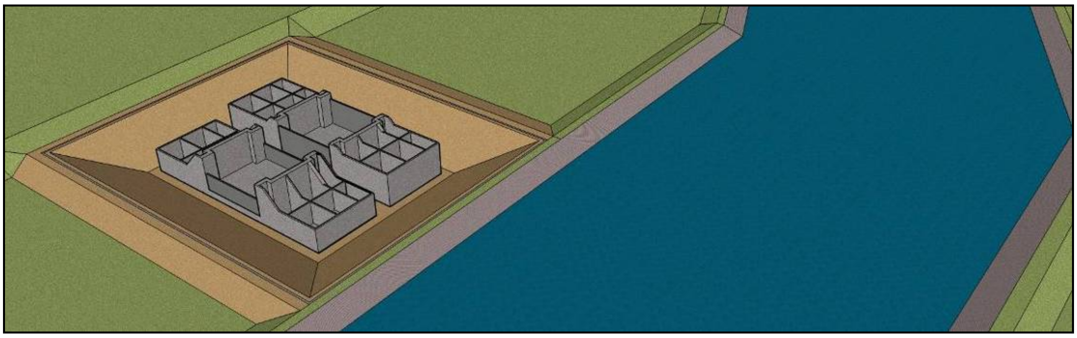

Figure 11.19: Building pit construction planning phase 3

Phase 4 (Figure 11.20):
- Breach the dike to the river
- Float up the heads

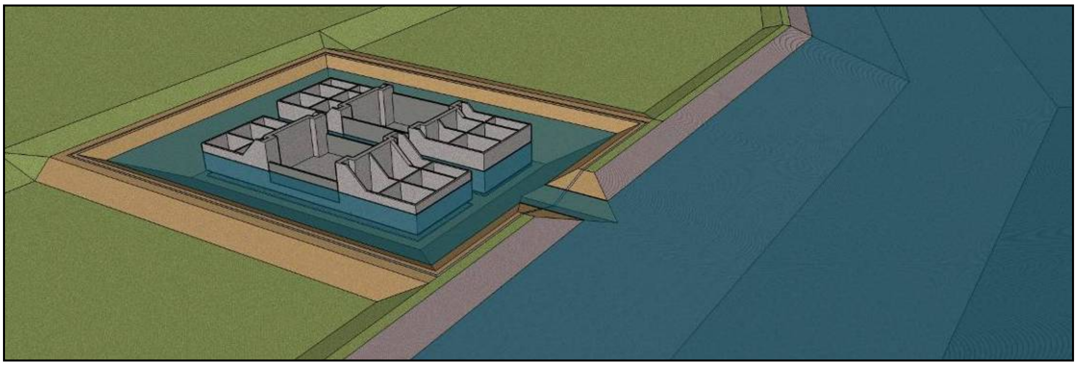

Figure 11.20: Building pit construction planning phase 4

Phase 5 (Figure 11.21):
- Excavate the whole river dike
- Float out the heads and transport them to the lock location
- Fill up the building pit

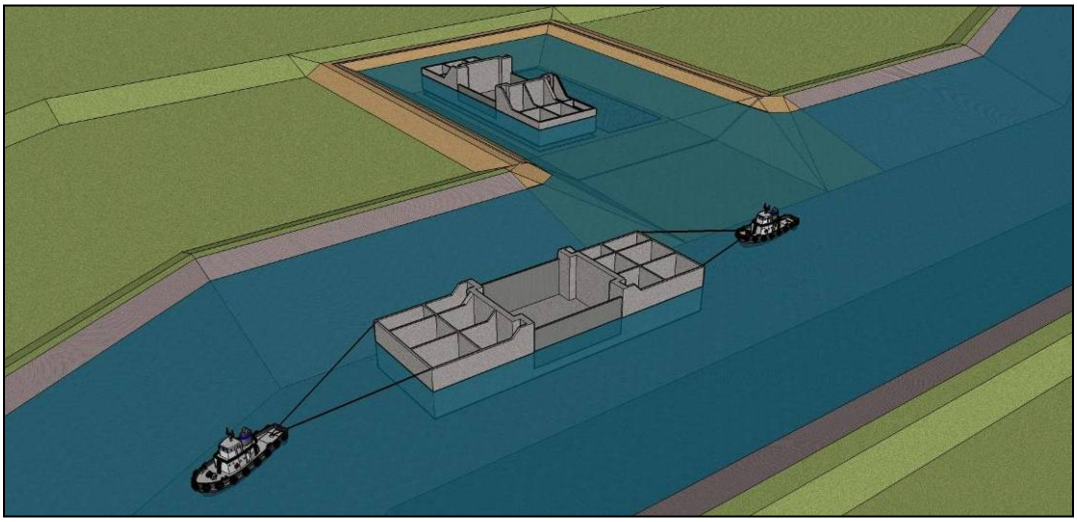

Figure 11.21: Building pit construction planning phase 5

12. Structural floated lock head design

In this chapter, the large upstream lock head of alternative 3 will be checked and optimised. This is done for the most critical cross-sections. These checks and optimisations will be performed using the 'Basis of Design' (Chapter 7) in combination with the conceptual design of alternative 3 (Chapter 8) and the reconstruction planning (Chapter 11).

12.1. Load combinations

Before the construction can be checked and optimised, the leading load combinations have to be determined. These load combinations can occur in two phases, namely in the construction phase and in the users phase. In the next two paragraphs, the different load combinations are explained. To give inside into the loads that can occur, the different load combinations are visualised in Figure 12.1 till Figure 12.9.

12.1.1. Construction phase

C. 1. Transport: During transport, floating and the pulling of the tug boats, three different load situations are possible, namely a horizontal balanced loading (Figure 12.1), a rotation over the short axis (Figure 12.2) and a rotation over the long axis (Figure 12.3).

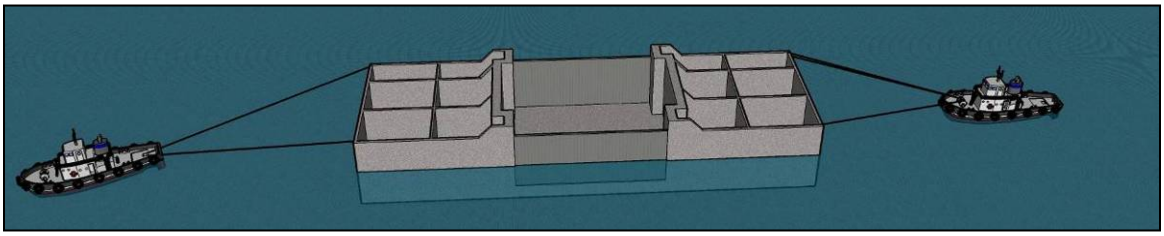

Figure 12.1: Load combination C.1a, horizontally loaded

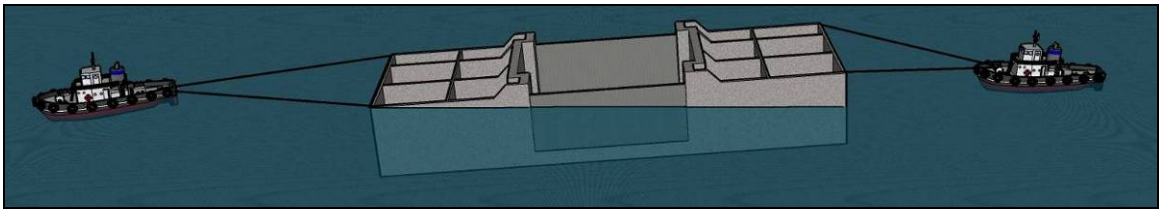

Figure 12.2: Load combination C.1b, rotated over the short axis

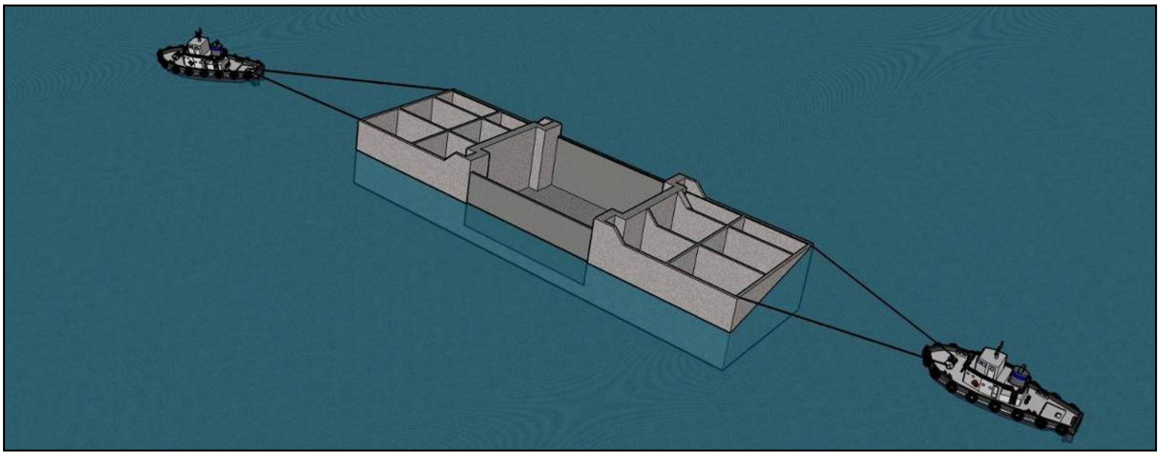

Figure 12.3: Load combination C.1c, rotated over the long axis

C. 2. Immersion: During floating in and immersion, the highest water pressures will occur without ballast (Figure 12.4).

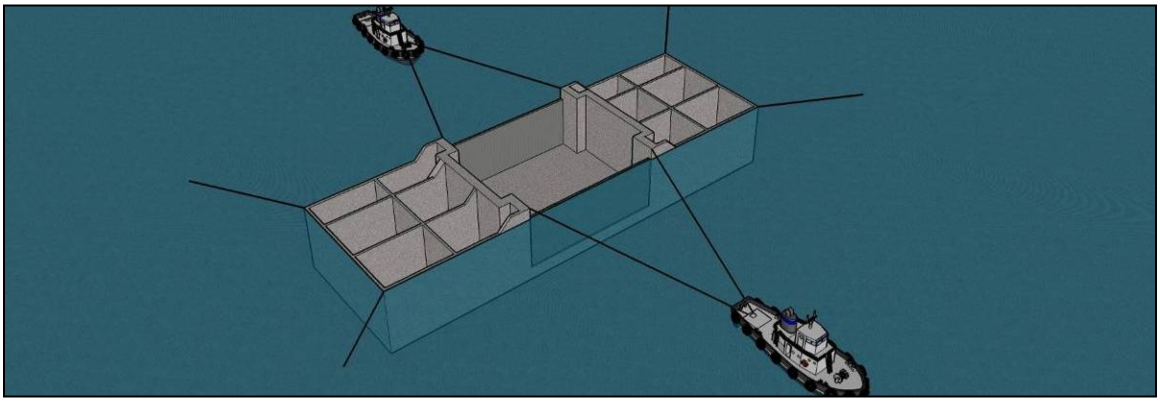

Figure 12.4: Load combination C.2

C. 3. Ballast Filling: During ballasting, the soil pressure to the inside (Figure 12.5).

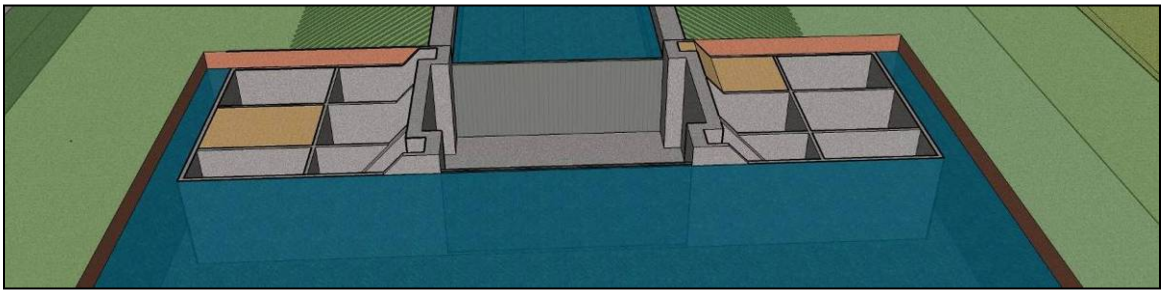

Figure 12.5: Load combination C.3

12.1.2. Users phase

U. 1. Empty head: Dewatered lock head and a low ground water level (Figure 12.6).

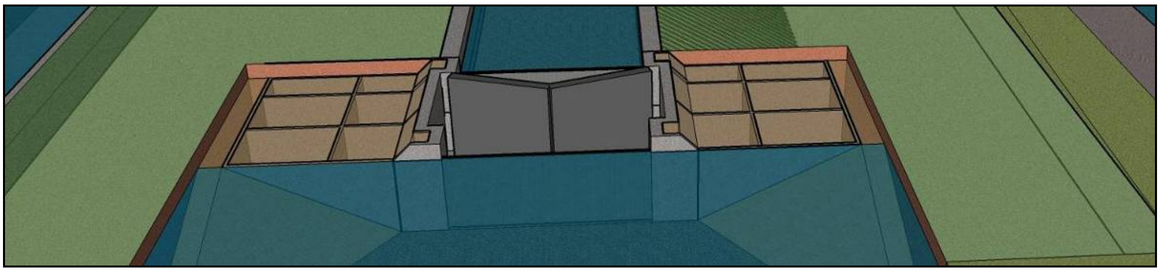

Figure 12.6: Load combination U.1

U. 2. Just opened gates: Low water level in the lock and a low ground water level outside the lock (Figure 12.7).

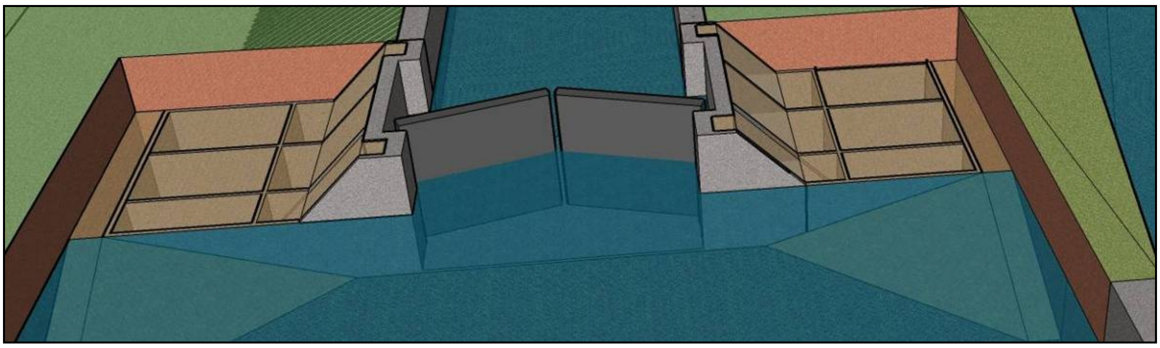

Figure 12.7: Load combination U.2

U. 3. Water retaining gates 1: Highest water level outside and lowest water level inside in combination with a high ground water level (Figure 12.8).
U. 4. Water retaining gates 2: Highest water level outside and lowest water level inside in combination with a low ground water level (Figure 12.8).

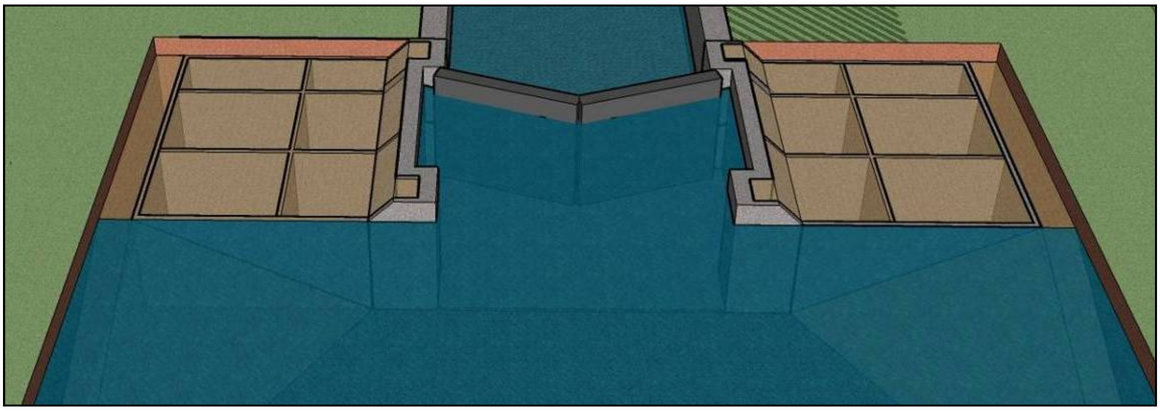

Figure 12.8: Load combination U.3 and U.4

U. 5. Water filled lock head: Highest water level inside the lock head in combination with a low ground water level (Figure 12.9).

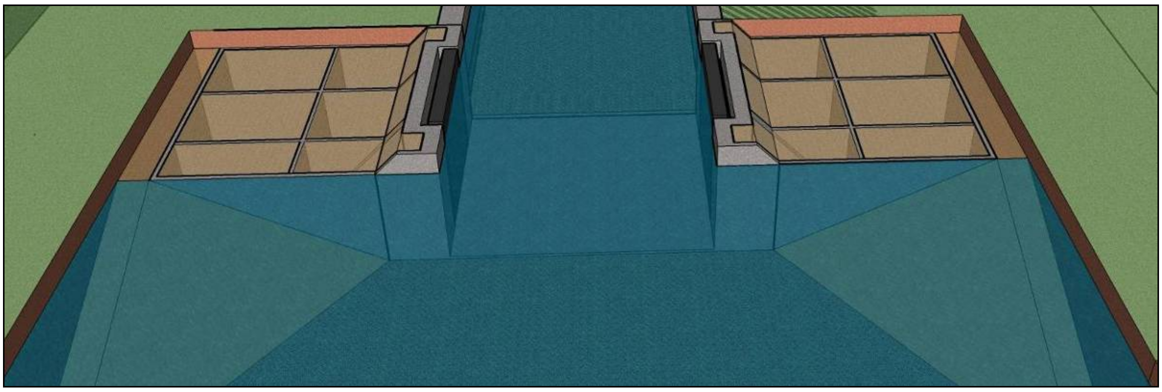

Figure 12.9: Load combination U.5

12.2. The boundary conditions

The different boundary conditions which are necessary to check and optimise the large lock heads are presented in this paragraph. First, the load factors and the representative loads are given. After that, the different failure possibilities are presented and explained.

12.2.1. The load factors

To determine the design values, the representative loads have to be multiplied by the load factors. These load factors are described in Table 12.1 for the construction phase and in Table 12.2 for the users phase. The load factors are derived from Table 7.7 and Table 7.8.

Table 12.1: Load combinations during the construction phase (C)

Load combination		Loads on the structure	Load factor
C.1.	Transport (water depth 5,1 m)	• Water pressure outside • Self weight	1,25 0,9
C.2.	Immersion (water depth 9,6 m)	• Water pressure outside • Self weight	1,25 0,9
C.3.	Ballast filling (low ground water level, 6,2 m)	• Water pressure outside • Soil pressure (caisson sand filling) • Self weight	0,9 1,2 1,2

Table 12.2: Load combinations during the users phase (U)

Load combination		Loads on the structure	Load factor
U.1.	Empty head (low ground water level)	• Water pressure outside • Soil pressure • Ground load • Self weight • Weight of the gates	0,9 1,2 1,2 1,2 1,2
U.2.	Just opened gates (low ground water level)	• Water pressure outside • Water pressure inside • Self weight • Weight of the gates	0,9 1,25 1,2 1,2
U.3	Water retaining gates 1 (high ground water level 9,6 m)	• Water pressure outside • Water pressure inside floor • Water pressure inside walls • Water load on the gates => wall • Weight of the gates • Self weight	1,25 0,9 1,25 1,2 0,9 0,9
U.4.	Water retaining gates 2 (low ground water)	• Water pressure outside • Water pressure inside floor • Water pressure inside walls • Water load on the gates => wall • Weight of the gates (wet) • Self weight	0,9 1,25 0,9 0,9 1,5 1,2
U.5.	Water filled lock head (low ground water)	• Water pressure outside • Water pressure inside floor • Water pressure inside walls • Weight of the gates • Soil pressure wall • Soil pressure floor side caisson • Ground load • Self weight	0,9 1,25 1,25 1,2 1,2 0,9 1,2 1,2

It is common to take a load factor of 1,0 for the water pressure and use the maximum water level (Table 7.6). However, in Table 12.2 a load factor of 1,25 is used in combination with the average water level.

12.2.2. The representative loads

The loads that are working on the construction are displayed in Table 12.3. Some of these loads are explained in Appendix XI. The other loads are obtained from the rough design (Chapter 8) or the 'Basis of Design' (Chapter 7). It is assumed that the foundation will give enough support to the structure in the users phase.

Table 12.3: Representative loads

Loads	Value	Unit
Water pressure	10	kN/m^2
Dry soil weight	17	kN/m^3
Wet soil weight	19	kN/m^3
Concrete weight	24	kN/m^3
Ground Load	20	kN/m^3
Vertical pivot force (U.1) (Spread over 2 m)	893	kN
Vertical pivot force (U.2, U.3, U.4 and U5) (Spread over 2 m)	841	kN
Collar strap force (U.2)	984	kN
Horizontal pivot force (U.2)	984	kN
Distributed load gate loads against the wall (U.3, U.4) (triangle)	224	kN/m
Distributed gate loads against the mitre sill (U.3, U.4)	68	kN/m

12.2.3. Failure possibilities in the limit states

Before the failure possibilities are worked out, the effective height of the concrete cross-section has to be determined. This effective height is presented by the letter d and is formed by the total height minus the concrete cover and half the reinforcement bar thickness.

$$d = h - c - \tfrac{1}{2} * Ø = h - 50$$

d = Effective concrete height (mm)
h = Concrete height (mm)
c = Concrete cover = 40 (mm)
Ø = Longitudinal reinforcement is assumed to be 20 mm

Ultimate limit state (ULS)

The formulas to check different failure mechanics in ULS are obtained from 'Gewapend Beton' [Walraven, 2004].

Fracture due to bending and/or normal force
The longitudinal reinforcement must take up the bending moment and the normal force in a concrete cross-section. The economic reinforcement percentages are between 0,5% and 0,75% for one direction. This will represent a longitudinal reinforcement in between Ø16-100 mm (0,5%) and Ø20-100mm (0,75%) in a 40 cm thick floor. In combination with the transversal direction, this will lead to a total economic reinforcement percentage in between the 1% and 1,5% per side.
The longitudinal reinforcement percentages in a concrete cross-section can be calculated with the next formulas.

$$N_s = \frac{M_d}{z} \pm N_d$$

N_s = Normal force in the reinforcement (kN)
M_d = Resulting (ULS) moment (kNm)
z = Arm (m) is assumed to be $0{,}9*(d/10^3)$
N_d = Resulting (ULS) normal force (kN)

$$A_s = \frac{N_s * 10^3}{f_s}$$

A_s = Reinforcement steel surface (mm^2)
f_s = Reinforcement steel strength (N/mm^2)

$$\omega = \frac{A_s}{A_c} * 100\%$$

A_c = Concrete surface (mm^2)
ω = Reinforcement percentage (%)

Fracture due to shear force
It must be checked whether or not a concrete cross-section can resist the maximum shear force, with or without stirrups.

$$\tau_d = \frac{V_d * 10^3}{b * d}$$

τ_d = Shear stress (N/mm^2)
V_d = Shear force (kN)
b = Width of the cross-section (mm)

$\tau_d < \tau_1$ No shear reinforcement required (τ_1 = 0,56 N/mm^2)
$\tau_1 < \tau_d < \tau_2$ Shear reinforcement required (τ_2 = 4,2 N/mm^2)

Fracture due to torsion
Torsion may occur in the total structure, but not in a 2D cross sections. So 3D modelling must be performed before the torsion is known in detail. Not much torsion is expected, because the floating lock heads are supported at all sides by the water in the construction phase. In the users phase also not much torsion is expected, because the head is assumed to be equally supported by the gravel foundation.

Serviceability limit state (SLS)

Unacceptable deformations
The only deformations that are unacceptable are those of the walls and the floor of the gate chamber. These parts of the structure must stay within the deformation limits, to prevent the lock gates from getting stuck.

- The walls are allowed to have a maximum deformation of l/300
- The floor is allowed to have a maximum deformation of 30 mm

The deformations of the side caissons are not important, because they only have to provide floating capacity and space for the ballast sand.
The deformations can be calculated for different standard load situations, which can be found in [ACL, 2004].

Unacceptable cracking
It has been assumed that the construction will be in the stabilised cracking stage. The cross-section must satisfy one of the two formulas that are represented below. These formulas are obtained from the Dutch concrete codes [NEN 6720].

1. $\text{Ø} \leq \dfrac{k_1 * \xi}{\sigma_s}$
2. $s \leq 100 * (\dfrac{k_2 * \xi}{\sigma_s} - 1{,}3)$

s = Reinforcement bar distance
ξ = Bond factor (1)
σ_s = Steel stress (N_s/A_s) (N/mm^2)
k_1 = 3750 (environmental class 2)
K_2 = 750 (environmental class 2)

12.2.4. Assumptions

In this paragraph a few assumptions are represented to exclude phenomena that could occur in the structure during the construction or the users phase. These phenomena are not checked in this report.

- When the lock head caisson is floating, the bigger floating capacity of the side caissons could push the top of the gate chamber walls to each other. This is prevented by constructing the bulk heads as stamps that support the gate chamber walls during the transport of the heads.
- The lock head caisson needs to have enough friction with the gravel bed to resist the water level differences in the users phase. It is assumed that enough friction is available, because the connection surface and the weight of the ballasted lock head are bigger than a standard lock design.
- In the cross-section schematisations the deformation of the gate chamber walls is not calculated. However, it assumed that the deformations stay with in the limits (paragraph 0) for two reasons. First, the subsoil is assumed stable, so the deformation of the whole structure is relative small. Second, the gate chamber wall is supported by the caisson walls.

12.3. Critical cross-sections

In Figure 12.10, the critical cross-sections, which will be checked and optimised, are shown in an overview of the large upstream lock head. Besides these cross-sections, the total structure has to be calculated to make an optimal design. Though, this report only considers the most important cross–sections.

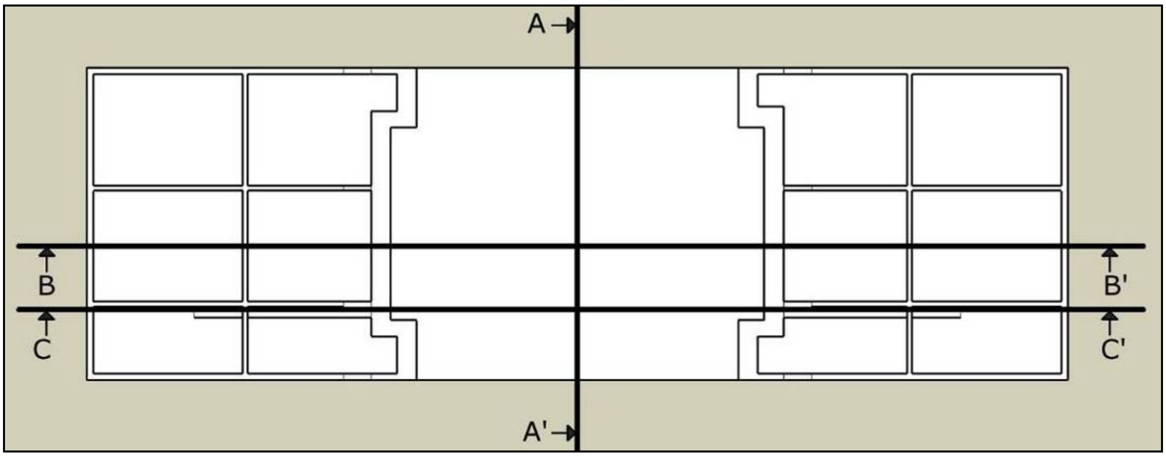

Figure 12.10: Critical cross sections overview

In the following three paragraphs, the cross-sections are represented and schematised. These schematisations and the load combinations are used to determine the envelope moments, shear forces, normal forces and deformations for each cross-section. The cross-sections are worked out in Appendix XI, including the resulting envelope moments, shear forces, normal forces and deformations.

12.3.1. Cross-section AA'

Cross–section AA' (Figure 12.11) is selected because the floor of the gate chamber must be as thin as possible, because this will improve the floatability of the caisson. Though, the floor also must be able to resist the loads that are acting on it.

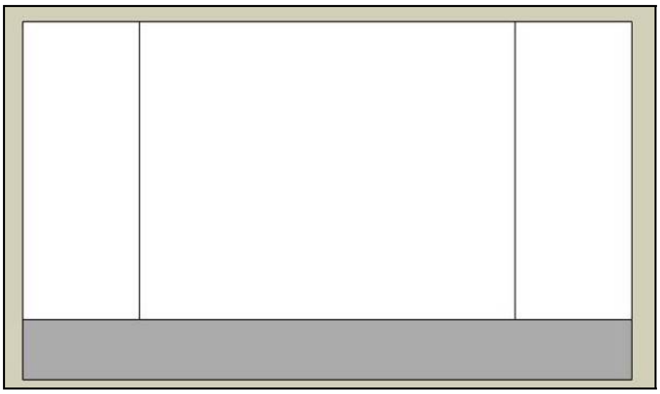

Figure 12.11: Cross-section AA'

Cross-section AA' is schematised by a beam on two hinge supports, as can be seen in Figure 12.12.

Figure 12.12: Schematisation of cross-section AA'

The governing load combinations for this cross section are C.1a, C.1c, C.2, U.3, U.4 and U5 (See paragraph 12.1).

12.3.2. Cross-section BB'

The connection between the side caissons and the gate chamber, as can be seen in Figure 12.13, must be strong enough to resist the uplifting forces during the users phase. Though, this connection must also be able to resist the sinking force in the construction phase. These two load cases can be checked in cross-section BB'.

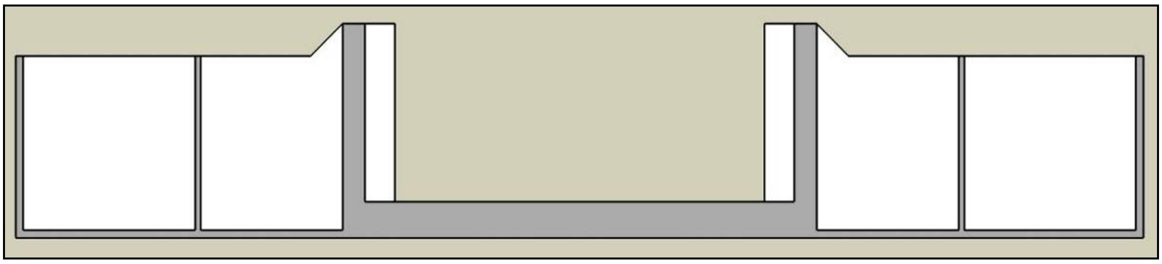

Figure 12.13: Cross-section BB'

The floor of cross-section BB' is schematised by a beam on six hinge supports, as can be seen in Figure 12.14. The walls are schematised as supports. The resulting moments from these walls that will work on the floor are implemented at the supports. These moments are calculated for different standard load situations [Wippel, 1983] on plates and can be found in Appendix XI.

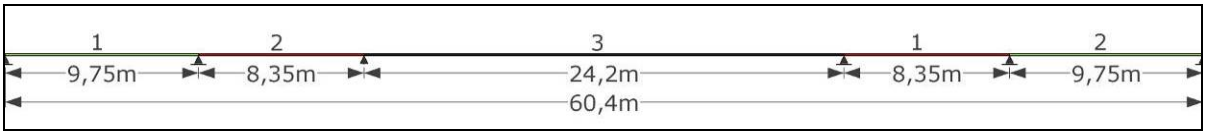

Figure 12.14: Schematisation of cross-section BB'

The governing load combinations for this cross section are C.1a, C.1b, C.2, C.3, U.1 and U.5 (See paragraph 12.1).

12.3.3. Cross-section CC'

Cross section CC' is situated at the point where the resulting loads from the gates are working. In this cross-section it is checked whether the floor can be constructed thinner. Also it is checked whether the triangle buttresses are sufficient enough to resist the resulting loads from the gates.

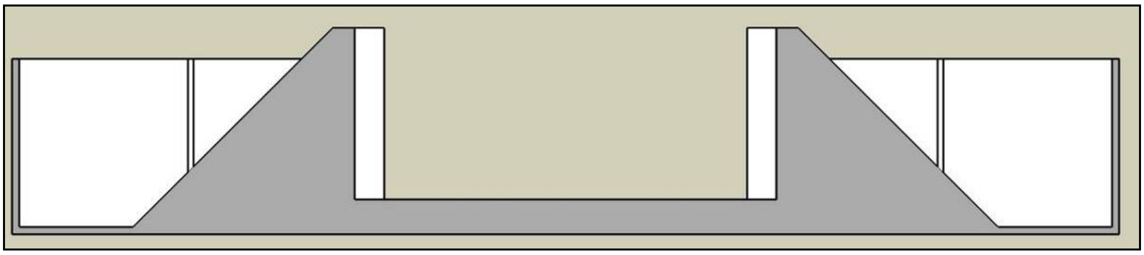

Figure 12.15: Cross-section CC'

The floor of this cross-section is schematised as a beam that is clamped at two sides. This can be seen in Figure 12.16.

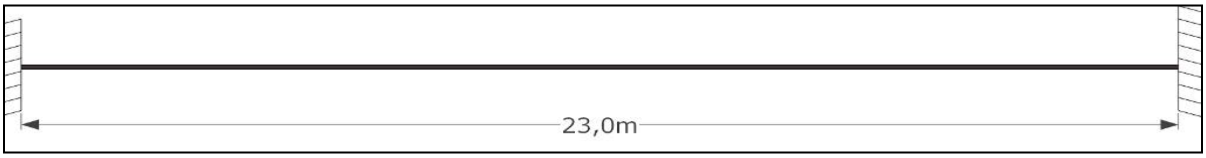

Figure 12.16: Schematisation of cross-section CC'

The governing load combinations are C.2, U.1, U.2, U.3, U.4 and U5 (See paragraph 12.1).

In the examination of this cross-section, it is assumed that no moment is transferred from the lock wall to the floor, because the triangle buttress takes up the resulting moments. The triangle buttress is 6^3 = 216 times stiffer than the floor (Figure 12.17). This results in combination with a stiff reacting subsoil, in a moment transfer to the lock head floor that is negligible.

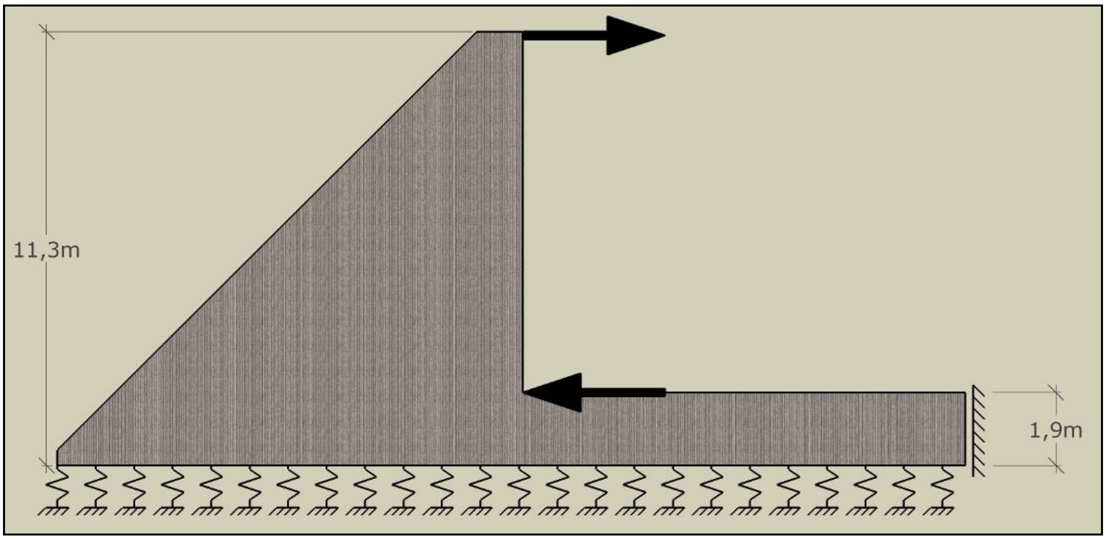

Figure 12.17: The cross section CC' stiffness distribution

12.4. Structural checks and optimisation

The structure, as designed in Chapter 8, is checked in Appendix XI on all the failure possibilities from paragraph 12.2.3. From these checks it can be concluded that the structure of the large upstream lock head is able to cope with all the load combinations. However, the reinforcement of the floor and the walls of the lock head are not economical, because less reinforcement steel is needed than the economic value. This means that the floor and the walls of the gate chamber (the concrete U in the middle) can be designed thinner.

The walls and the floor of the side caisson are already checked in Chapter 8 and are sufficient. They cannot be lightened, because in that case the structure will not be constructible anymore. Besides the moments that can occur in the side caissons, the walls and floor of the side caissons, which are connected to the gate chamber, also have to resist shear force. The shear force is directed down during the construction phase when the heads are floating. In contrast, the shear force is also directed up during the users phase due to the uplifting water pressure. These shear forces are also checked in Appendix XI. The side caissons turn out to be able to resist these shear forces without stirrups.

Although only the upstream large lock head is checked, also the downstream large lock head can be optimised, because the structures are almost similar. In Appendix XII the large lock heads are optimised both on the basis of the checks from Appendix XI. The result of this optimisation can be seen in Figure 12.18. Besides the optimised gate chamber, also the outer dimensions of the lock head are smaller because the floatability of the lock head is increased. Furthermore, the walls of the side caissons are constructed half a meter higher than the maximum locking level, to reduce the danger of losing ballast sand. The walls have a thickness of 0,7 m instead of the 1,2 m of the first design. The floor also become thinner, namely 1,5m instead of 1,9 m.
The deformations are expected to be within the limits, but they are not checked for the optimised lock heads.

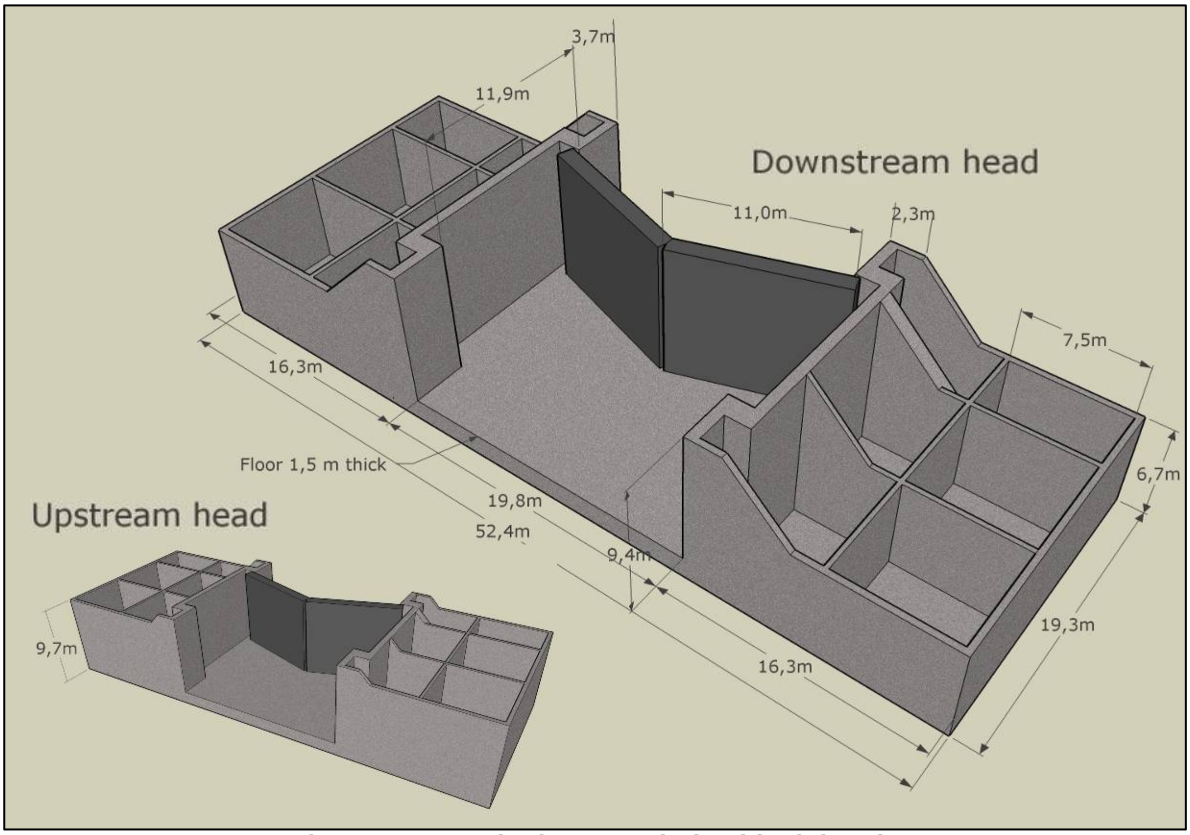

Figure 12.18: The large optimised lock heads

To check and optimise the lock head structure entirely, the floating lock head has to be modelled completely (3D model) to provide an insight into the interaction between the different cross-sections. When a model like this is made, it is possible that the gate chamber can be constructed even thinner than the optimised structure in Figure 12.18.

13. Conclusions and recommendations

13.1. Conclusions

In this study, the feasibility and constructability of a functional flexible lock design was studied. This was done for the inland navigation lock complex 'Sluis Sambeek' in the Meuse route. This report gave an impression of the possibilities to design and construct a functional flexible lock. A functional flexible lock design makes it possible to extend the functional (economic) lifespan of a lock, so it equals its structural (technical) lifespan.
In this conclusion, the three different problem definitions that were formulated in Chapter 4 are discussed separately. This paragraph ends with a main conclusion which is based on the objective that was formulated in Chapter 4.

13.1.1. Meuse route trend forecast

According to the forecasted trend in Chapter 6, the number of passing ships will decrease in the future, while the amount of transported tonnages will rise. Therefore, larger but less ships will pass 'Sluis Sambeek'.
The minimum needed lock dimensions following from this forecast can be seen in Table 13.1. The (re)construction of the (flexible) lock is expected to be required in 2020 to fit CEMT-class Vb vessels and in 2052 to fit CEMT-class VIa vessels.

Table 13.1: The minimum lock dimensions according to the forecast

Lock 3	Min. Lock dimensions (m)		
	Width	Length	Depth
2020	12,5	225	4,7
2052	19,8	225	4,7

Because it was expected that the ships become wider, the flexible lock only have to be extended in the width direction in the future.

13.1.2. Alternative selection

In Chapter 8, three alternatives were considered to fulfil the required lock dimensions. These alternatives were compared using the Life Cycle Management (LCM) approach. The three alternatives were:

1. Zero-alternative: renovation of the lock as planned by the department of public works (paragraph 8.4).
2. Functional flexible lock: standard lock that is built large enough to facilitate passages for the maximum expected ships in the next 100 years (paragraph 8.5).
3. Structural flexible lock: a relatively easy extendable navigation lock, which can be enlarged when it is required. Thus, the maximum dimensions are reached step-wise (paragraph 8.6).

While often only the initial costs are taken into account, this study used a Whole Life Costing (WLC) analysis with a risk inventory to compare the alternatives (Chapter 9). From this analysis followed that a functional flexible lock (alternative 2) appeared to be the least expensive option for 'Sluis Sambeek'. It was assumed that the real interest will be 1,9%, the year of reconstruction will be in 2052 and the real option costs are determined correctly.

The zero-alternative appeared to be the alternative with the lowest initial construction costs. However, this study showed that another alternative is the most inexpensive option when the whole life costs are taken into account.

The whole life costs of alternative 3 are too high in case of 'Sluis Sambeek'. However, in another lock reconstruction alternative 3 could be a better solution. This alternative could be more beneficial in the following situations:

- Compared to both alternatives, less water is lost for each locking cycle in alternative 3. So when the loss of water is more important, alternative 3 will be a better option.
- When no lock reconstruction appears to be necessary in the future, alternative 3 will be less expensive than alternative 2. Thus, when there is a high uncertainty in the ship dimension growth or in the passing intensities, alternative 3 will be more favourable than alternative 2.
- The reconstruction of the structural flexible lock (alternative 3) is expected to cause only 16 weeks of obstruction. The construction of a new lock, as replacement for the renovated old lock (alternative 1), will have an obstruction time of about three years. When the lock obstruction is more far-reaching, the obstruction costs will rise. For instance, when the lock complex contains only one or two locks, the effect of an obstruction of one of these locks is bigger than in case of 'Sluis Sambeek'. In this case alternative 3 will be a better option than alternative 1, because the obstruction time has to be low when the obstruction costs are high.

Although alternative 3 was the most expensive option, this option was elaborated further in the report (Chapter 10 till 12) because this alternative is the most innovate one. The feasibility of this structural flexible lock design is discussed in the next paragraph.

13.1.3. Structural feasibility of a structural flexible lock

Chapter 10 till 12 showed that it is possible to construct a structural flexible lock. Some new innovative parts have to be designed or considered, namely:

- The float up of the small lock heads after 40 or 50 years (paragraph 10.1).
- The construction planning (paragraph 10.2).
- The structural lock head design (paragraph 10.3).
- The watertight lock chamber and the lock heads connection (paragraph 10.4)

In this report, only the reconstruction planning that will take place in 2052 was worked out (Chapter 11). Furthermore, the structural lock head design of this reconstruction was worked out in detail (Chapter 12). In this structural lock head design, the in Chapter 8 designed lock heads of alternative 3 appeared to be able to cope with the expected loads. They could even be optimised and the floor and the walls of the gate chamber could be designed more slender. With this information, the WLC could be updated in a further study.

13.1.4. Main conclusion

This main conclusion is based on the objective of this study. '*Design and study the possibilities of a functional and structural flexible inland navigation lock for the Meuse route ('Sluis Sambeek') that is able to cope with the ship sizes and intensities that are expected to occur in the next 100 years'.*

This report showed that it is possible to construct a functional inland navigation lock that is able to cope with the changing ship sizes and intensities for the next 100 years. There are two options, namely constructing a standard lock that is big enough to serve the ship traffic for the next 100 years (functional flexible lock) or constructing an adjustable lock which can be adjusted when it is required by the ship traffic (structural flexible lock).

Furthermore, it was shown in Chapter 9 that a functional inland navigation lock is a cheaper option on the basis of Whole Life Costing (WLC) than the construction or renovation of a standard lock. Thus, it is useful to consider the possibilities of a functional or structural flexible lock design for 'Sluis Sambeek'. The initial construction costs of these alternatives are higher, but in most cases the whole life costs will be lower.

Generally, it is good to consider a functional flexible lock design when an inland navigation lock will be built or renovated, because a flexible navigation lock will deliver a ship passage with a negligible amount of obstruction time for its whole lifetime. This is beneficial for the most durable transportation modality of the Netherlands, the transportation per ship.

13.2. Recommendations

Besides the conclusions, also a few recommendations are formulated.

- A more extensive trend forecast will make the alternative comparisons more reliable (Chapter 6).

- Further research is needed to determine whether it is possible for 'Sluis Sambeek' to combine a permeable lock chamber bottom with a seepage cut-off screen (Chapter 8).

- In case of 'Sluis Sambeek', the subsoil is stiff and strong enough to found the floating lock heads of alternative 3 upon a gravel layer that is supported by the sandy subsoil (Chapter 8). Further research has to be done to determine the possibilities of a foundation in case of a soft subsoil, for instance a pile foundation.

- The structural flexible lock head option with a demolish-able wall (option 4) for alternative 3 (paragraph 8.6.1) have to be reconsidered in further studies, because this option could be a cheaper option then the selected option with a floating lock head (option 3).

- Besides the fact that demolishing brings costs, demolishing also could bring in money when the steel of the gates and the concrete rubble are sold. In this report only the selling of the pulled sheet pile walls was taken into account. When the selling of steel and concrete rubble will also be taking into account, the Whole Life Costing (WLC) analysis may have another result.

- The loss of economic value due to the lower sill depth of alternative 1 (Chapter 9) has to be studied in the future to determine its effect on the WLC analysis.

- Further research has to be performed about the possibility to float the lock heads out after a period of 40-50 years. When it is concluded that this will cause problems, the solution of paragraph 10.1 could be used or a new solution has to be elaborated.

- The watertight connection between the lock head and the lock chamber has to be designed in detail. Paragraph 10.4 already brought up a solution, but this solution has to be elaborated in detail.

- A new design step can be made with the new optimised lock head design of Chapter 12. This optimised design will change the outcome of the WLC analysis.

- By using a 3D model, the spreading of the forces over the whole structure can be taken into account. The floating lock heads of alternative 3 can be checked better and can be optimised further then.

- In this design, no prestressing is considered. When further research in this subject is performed, prestressing could result in a more slender design of the floating lock head floors.

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Appendix I. Ship classes

CEMT-klasse	Standaardschepen waarop de classificatie is gebaseerd			Duwstel		
	Type	Lengte	Breedte	Lengte	Breedte	Hoogte
		m	m	m	m	m
0	Kleinere vaartuigen en recreatievaart	variërend	variërend			variërend
I	Spits	38,50	5,05			4,00
II	Kempenaar	50 - 55	6,60			4,00 - 5,00
III	Dortmund-Eemskanaal-schip	67 - 80	8,20			4,00 - 5,00
IV	Rijn-Hernekanaalschip	80 - 85	9,50	85	9,50	5,25 of 7,00
Va	Groot Rijnschip	95 - 110	11,40	95 - 110	11,40	5,25 of 7,00 of 9,10
Vb				172 - 185	11,40	
VIa				95 - 110	22,80	7,00 of 9,10
VIb		140	15,00	185 - 195	22,80	7,00 of 9,10
VIc				270 - 280 193 - 200	22,80 33,00 - 34,20	9,10
VII				285 195	33,00 34,20	9,10

Figure I 1: Classification of inland waterways CEMT 1992 [Rijkswaterstaat DVS,2008]

Motorvrachtschepen (Motorvessels)								Doorvaart-hoogte incl. 30 cm schrikhoogte
CEMT-klasse	RWS/CBS-klasse	Karakteristieken maatgevend schip*				Classificatie		
		Naam	Breedte	Lengte	Diepgang (geladen)	Laadvermogen	Breedte en lengte	
			m	m	m	t	m	m
0	M0	Overig				1-250	B < = 5,00 of L < = 38,00	
I	M1	Spits	5,05	38,5	2,5	251-400	B = 5,01-5,10 en L > = 38,01	5,25**
II	M2	Kempenaar	6,6	50-55	2,6	401-650	B = 5,11-6,70 en L > = 38,01	6,1
III	M3	Hagenaar	7,2	55-70	2,6	651-800	B = 6,71-7,30 en L > = 38,01	6,4
	M4	Dortmund Eems (L < = 74 m	8,2	67	2,7	801-1050	B = 7,31-8,30 en L = 38,01-74,00	6,6
	M5	Verl. Dortmund (L > 74 m)	8,2	80-85	2,7	1051-1250	B = 7,31-8,30 en L > = 74,01	6,4
IV	M6	Rijn-Herne Schip (L < = 86 m)	9,5	80-85	2,9	1251-1750	B = 8,31-9,60 en L = 38,01-86,00	7,0**
	M7	Verl. Rijn-Herne (L > 86 m)	9,5	105	3,0	1751-2050	B = 8,31-9,60 en L > = 86,01	7,0**
IVb								7,0**
Va	M8	Groot Rijnschip (L < 111 m)	11,4	95-110	3,5		B > 9,60-11,50 en L > = 38,01-<111	9,1**
	M9	Verlengd Groot Rijnschip (L > 111 m)	11,4	135	3,5		B > 9,60-11,50 en L > = 38,01-<111	9,1**
Vb								9,1**
VIa		Rijnmax Schip	17,0	135	4,0		B > 11,51 en L > = 38,01	7,0**

Figure I 2: Inland navigation fleet Classification RWS/CBS-2008, Motorvessels [Rijkswaterstaat DVS,2008]

Duwstellen (Barges)									Doorvaart-hoogte incl. 30 cm schrikhoogte
CEMT-klasse	RWS/CBS-klasse	Karakteristieken maatgevend duwstel*					Classificatie		
		Combinatie	Breedte	Lengte	Diepgang (geladen)		Laadvermogen	Breedte en lengte	
			m	m	m		t	m	m
I	BO1		5,2	55	1,9		0-400	B < = 5,20 en L = alle	5,25**
II	BO2		6,6	60-70	2,6		401-600	B = 5,21-6,70 en L = alle	6,1
III	BO3		7,5	80	2,6		601-800	B = 6,71-7,60 en L = alle	6,4
	BO4		8,2	85	2,7		801-1250	B = 7,61-8,40 en L = alle	6,6
IV	BI	Europa I duwstel	9,5	85-105	3,0		1251-1800	B = 8,41-9,60 en L = alle	7,0**
Va	BII-1	Europa II duwstel	11,4	95-110	3,5		1801-2450	B = 9,61-15,10 en L < = 111,00	9,1**
	BIIa-1	Europa IIa duwstel	11,4	92-110	4,0		2451-3200	B = 9,61-15,10 en L < = 111,00	9,1**
	BIIL-1	Europa II Lang	11,4	125-135	4,0		3201-3950	B = 9,61-15,10 en L = 111,01-146,00	9,1**
Vb	BII-2l	2-baksduwstel lang	11,4	170-190	3,5-4,0		3951-7050	B = 9,61-15,10 en L > = 146,01	9,1**
VIa	BII-2b	2-baksduwstel breed	22,8	95-145	3,5-4,0		3951-7050	B = 15,11-24,00 en L < = 146,00	7,0**
VIb	BII-4	4-baksduwstel (incl. 3-baks lang)	22,8	185-195	3,5-4,0		7051-12000 (7051-9000)	B = 15,11-24,00 en L = 146,01-200	9,1**
VIc	BII-6l	6-baksduwstel lang (incl. 5-baks lang)	22,8	270	3,5-4,0		12001-18000 (12001-15000)	B = 15,11-24,00 en L > = 200,01	9,1**
VIIa	BII-6b	6-baksduwstel breed (incl. 5-baks breed)	34,2	195	3,5-4,0		12001-18000 (12001-15000)	B > = 24,01 en L = alle	9,1**

Figure I 3: Inland navigation fleet Classification RWS/CBS-2008, Barges [Rijkswaterstaat DVS,2008]

Koppelverbanden (Convoys)								Doorvaart-hoogte incl. 30 cm schrikhoogte
CEMT-klasse	RWS/CBS-klasse	Karakteristieken maatgevend koppelverband*				Classificatie		
		Combinatie	Breedte	Lengte	Diepgang (geladen)	Laadvermogen	Breedte en lengte	
			m	m	m	t	m	m
I	C1l	2 spitsen lang	5,05	77-80	2,5	< = 900	B < = 5,1 en L = alle	5,25**
	C1b	2 spitsen breed	10,1	38,5	2,5	< = 900	B = 9,61-12,60 en L < = 80,00	5,25**
IVb	C2l	Klasse IV + Europa I lang	9,5	170-185	3,0	901-3350	B = 5,11-9,60 en L = alle	7,0**
Vb	C3l	Klasse Va + Europa II lang	11,4	170-190	3,5-4,0	3351-7250	B = 9,61-12,60 en L > = 80,01	9,1**
VIa	C2b	Klasse IV + Europa I breed	19,0	85-105	3,0	901-3350	B = 12,61-19,10 en L < = 136,00	7,0**
	C3b	Klasse Va + Europa II breed	22,8	95-110	3,5-4,0	3351-7250	B > 19,10 en L < = 136	9,1**
VIb	C4	Klasse Va + 3 Europa II	22,8	185	3,5-4,0	> = 7251	B > 12,60 en L > = 136,01	9,1**

Figure I 4: Inland navigation fleet Classification RWS/CBS-2008, Convoys [Rijkswaterstaat DVS,2008]

Appendix II. Trend data

The forecasted trend data (from Table II 1 till Table II 5)

Table II 1: Dutch inland navigation fleet [CBS/RWS]

CEMT - class	Time in years											
	1983	**1985**	**1986**	**1987**	**1988**	**1989**	**1990**	**1991**	**1992**	**1993**	**1994**	**1995**
0	4583	3915	3817	3753	3622	3332	2933	2701	2714	2769	2839	2684
I	1823	1678	1657	1750	1740	1574	1521	1376	1221	1108	1053	987
II	1987	1891	1862	1869	1849	1779	1823	1715	1684	1596	1562	1465
III	1628	1600	1584	1560	1556	1517	1492	1454	1432	1408	1413	1396
IV	988	1035	1048	1072	1090	1137	1183	1179	1171	1138	1181	1193
Va	747	800	839	860	874	934	991	990	995	1020	1096	1155
Vb	72	86	89	101	111	130	143	140	141	149	162	161
VIa												
VIb												
VIc												

CEMT - class	Time in years										
	1996	**1997**	**1998**	**1999**	**2000**	**2001**	**2002**	**2004**	**2005**	**2006**	**2007**
0	2418	2294	2278	2208	2251	2342	2163	1487	363	391	419
I	915	736	664	645	620	601	576	291	181	183	173
II	1312	1155	1052	1007	940	944	913	858	712	728	723
III	1332	1429	1526	1535	1620	1398	1366	1675	1454	1466	1448
IV	1213	1118	1075	1102	1068	1111	1105	1212	1018	1053	1034
Va	1170	680	652	678	696	774	810	988	978	1086	1091
Vb	164	86	85	89	86	83	81	211	183	183	194
VIa		59	57	62	61	88	97	503	412	485	492
VIb								84	76	84	87
VIc								9	10	10	11
VIIa								11	8	11	12
	Adjusted, it does not fit in the column (2149)										

Table II 2: Annual number of ships passing "Sluis Sambeek" [CBS/RWS] [Provincie Limburg, 2008]

	Number of ships				Average (t) loading capacity Commercial	percentage load capacity used
	year	Recreational	Commercial	Total		
	1994	16400	44000	60400		
	1995	18933	40300	59233		
	1996	15167	42900	58067		
	1997	14877	42023	56900		
	1998	12455	41146	53601		
	1999	13039	42246	55285		
	2000	11995	38096	50091		
	2001	12571	34813	47384		
	2002	12353	32533	44886	1127	51%
	2004	18774	27488	46262	1203	56%
	2005	17150	28598	45748	1222	56%
	2006	12402	29533	41935	1282	59%
	2007	13508	27147	40655	1325	59%
Forecast low (2005)	2020	18453	26462	44915	1427	50%
	2040	22517	19462	41979	1717	50%
Forecast high (2005)	2020	18453	28140	46593	1757	50%
	2040	22517	29804	52321	2223	50%
Forecast Trend	2110	23632	24072	47704	3355	60%

Table II 3: Annual transport amount of cargo transported through "Sluis Sambeek"[CBS/RWS] [Provincie Limburg, 2008]

Total transport cargo		
Year	Million ton	With the used tonnage trend and a percentage of load capacity used of 60%. The corresponding needed load capacity in 2110 (100 year) is 81,8 million ton cargo.
1995	17,8	
1996	20,5	
1997	22,5	
1998	21,7	
1999	20,8	
2000	20,0	
2001	19,1	
2002	18,3	
2003	18,2	
2004	18,1	
2005	19,4	
2006	21,7	
2007	20,7	
2008	18,7	
2020	18,7	Forecast low (2005)
2040	16,7	
2020	24,5	Forecast High (2005)
2040	32,9	
2110	48,5	Forecast trend

Table II 4: Annual container transport through "Sluis Sambeek" [CBS/RWS] [Provincie Limburg, 2008]

	Load factor		70%		
	Total container transport				TEU capacity/ship
	year	Ships	TEU	TEU/ship	
	2000	512	60168	118	168
	2002	819	81753	100	143
	2004	1147	103286	90	129
	2005	1202	128523	107	153
	2006	1243	135165	109	155
	2007	1696	186164	110	157
Average forecast	2020	1783	220238	124	176
	2040	1945	402842	207	296
	2020	1695	177325	105	149
Forecast Low	2040	1616	208270	129	184
	2020	1871	263151	141	201
Forecast High	2040	2273	597414	263	375

Table II 5: Annual harbour container capacity [Provincie Limburg, 2008]

Maximum container transport Capacity (TEU)				
Harbour	2006	2007	Future	Max
Wanssum	70000	95000	100000	100000
Venlo	0	0	40000	60000
Born	-	100000	200000	250000
Stein	-	55000	100000	200000
Total			440000	610000

Queuing theory [Groeneveld, 2001]
With the queuing theory the average waiting time can be calculated roughly.

Average waiting time:

$$W = \frac{1}{n * \mu} * \frac{\rho^n}{n!} * \frac{P(0)}{(1 - \rho/n)^2} \text{ (hours)}$$

λ = average arrival rate (ships/hour)
μ = average service rate (locking rate)(½ cycle/hour)
n = number of servers (number of locks)
$\rho = \frac{\lambda}{\mu}$ = the cumulative utilisation of all the locks together (%)
$\psi = \frac{\rho}{n}$ = utilisation per lock (%)

P(0) = Change that the system is empty = $(1 + \rho + \frac{\rho^2}{2!} + \frac{\rho^{(n-1)}}{(n-1)!} + \frac{\rho^n}{n(1-\rho/n)!})^{-1}$ (%)

Probability that an arriving ship has to wait (only available for a number of locks)

$$st = P(0) * \frac{\rho^n}{n!} * \frac{1}{n - \rho} \text{ (\%)}$$

This calculation is performed by an M/M/n operation. This means that the computation is done with an average service rate and average arrivals rate both as negative exponential distributions in combination with a variable number of servers (locks).
For this calculation is assumed that minimal 4 recreational vessels or 3 small professional ships can fill one lock, thus the corresponding proportions are 0,25 and 0,33 (ship/lock).
The intensities from the forecast of Table II 2 are used to carry out this calculation.

In 2007 the passing times of the whole complex are calculated with a locking time of 12 minutes (0,2 hours) [Burhenne, 2009] for all three the locks. In 2050 and 2110 the passing time of lock 1 and 2 are analysed separated from lock 3. Lock 1 and 2 still have a locking time of 12 minutes and lock 3 has an assumed locking time of 18 minutes (0,3 hours). The total hours of service is at this moment (2007) still 142 hours a week, but when the Meuse route improvements are ready they will be extend to 168 hours a week (24/7). The queuing time calculation for the year 2007, 2050 and 2110 are shown in Table II 6, Table II 7 and Table II 8.

Table II 6: Queuing theory calculation of the year 2007

Three locks			
Vessel type / CEMT - class	Number of ships in real	Size (Ship/Lock)	Relative number of ships
Recreational vessels	13.508	0,25	3.377
Small, 0, I, II, III	12.351	0,33	4.117
Medium, IV, Va (2000t>)	11.545	1,0	11.545
Big, Va (2000t<)	5.019	1,0	5.019
Total	42.423		24.058

Total hours of service per year	7384		ρ	65%
Average arrival rate, λ (ship/hour)	3,26		ψ (utilisation)	22%
Passing time (hours)	0,20	=>	P(0) (system empty)	49%
Average service rate, μ (½ cycles/ hour)	5,00		W, average waiting time (hours)	0,00
Number of service points (locks)	3			
			st (waiting probability)	6%

Table II 7: Queuing theory calculation of the forecasted year 2050

Two locks of (142*16)					
Vessel type / CEMT - class	Number of ships in real	Size (Ship/Lock)	Relative number of ships		
Recreational vessels	21.709	0,25	5.427		
Small, 0, I, II, III	6.184	0,33	2.061		
Medium, IV, Va (2000t>)	6.450	1,0	6.450		
Big, Va (2000-3000t)	5.453	1,0	5.453		
Total	39.796		19.392		
Total hours of service per year		8760		ρ	44%
Average arrival rate, λ (ship/hour)		2,21		ψ (utilisation)	22%
Lock time (hours)		0,20	=>	P(0) (system empty)	64%
Average service rate, μ (½cycles/hour)		5,00		W, average waiting time (hours)	0,01
Number of service points (locks)		2			
				st (waiting probability)	8%
One flexible lock					
Vessel type / CEMT - class	Number of ships in real	Size (Ship/Lock)	Relative number of ships		
Very Big, 3000t>	6.057	1,0	6.057		
Total hours of service per year		8760		ρ (utilisation)	21%
Average arrival rate, λ (ship/hour)		0,69	=>	W, average waiting time(hours)	0,08
Lock time (hours)		0,30			
Average service rate, μ (½cycles/hour)		3,33			

Table II 8: Queuing theory calculation of the forecasted year 2110

Two locks of (142*16)					
Vessel type / CEMT - class	Number of ships in real	Size (Ship/Lock)	Relative number of ships		
Recreational vessels	23.632	0,25	5.908		
Small, 0, I, II, III	4.645	0,33	1.548		
Medium, IV, Va (2000t>)	5.582	1,0	5.582		
Big, Va (2000-3000t)	6.540	1,0	6.540		
Total	40.399		19.578		
Total hours of service per year		8760		ρ	45%
Average arrival rate, λ (ship/hour)		2,23		ψ (utilisation)	22%
Passing time (hours)		0,20	=>	P(0) (system empty)	63%
Average service rate, μ (½cycles/hour)		5,00		W, average waiting time (hours)	0,01
Number of service points (locks)		2			
				st (waiting probability)	8%
One flexible lock					
Vessel type / CEMT - class	Number of ships in real	Size (Ship/Lock)	Relative number of ships		
Very Big, 3000t>	7.305	1,0	7.305		
Total hours of service per year		8760		ρ (utilisation)	25%
Average arrival rate, λ (ship/hour)		0,83	=>	W, average waiting time(hours)	0,10
Passing time (hours)		0,30			
Average service rate, μ (½cycles/hour)		3,33			

Appendix III. Soil profile

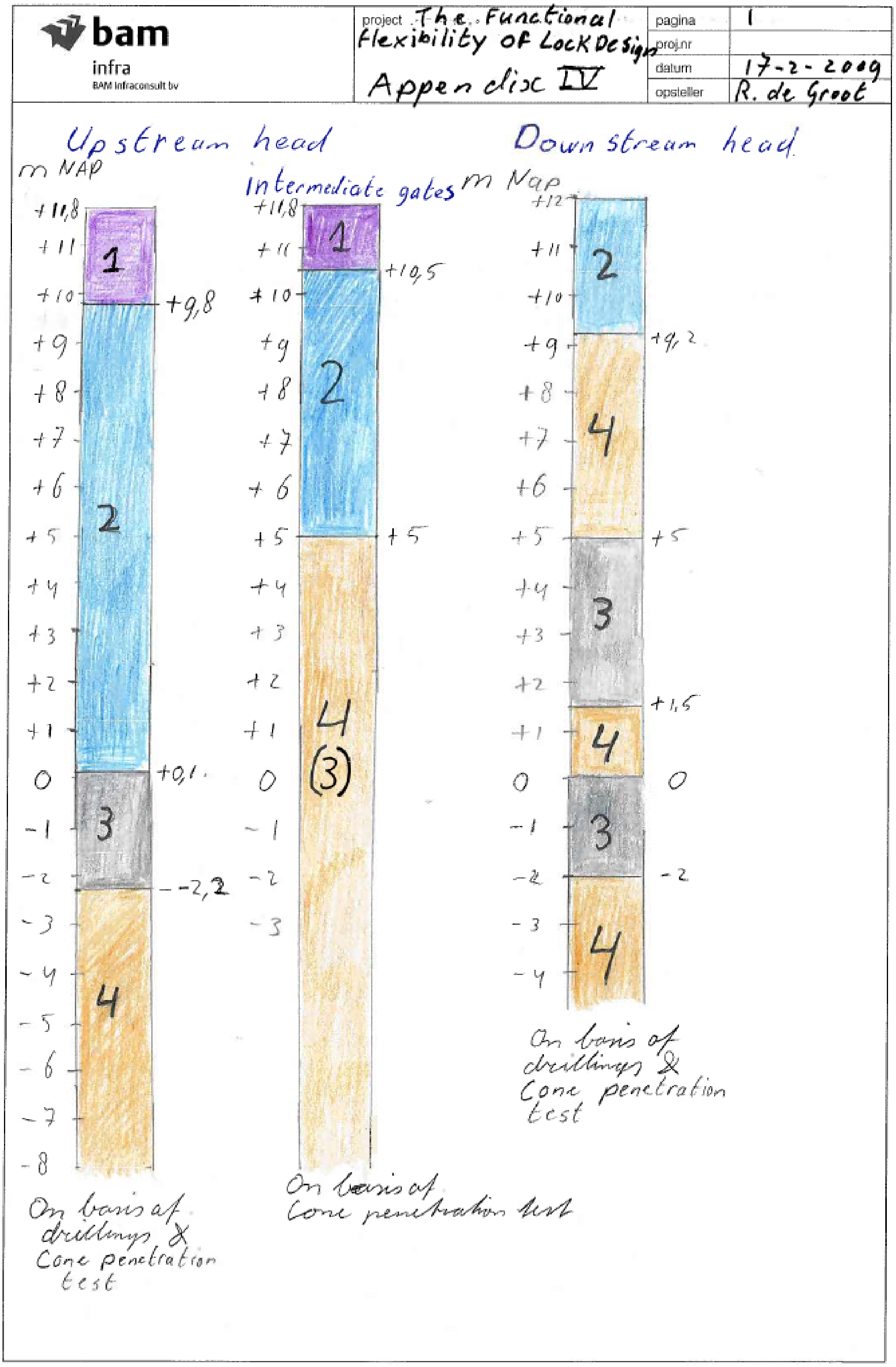


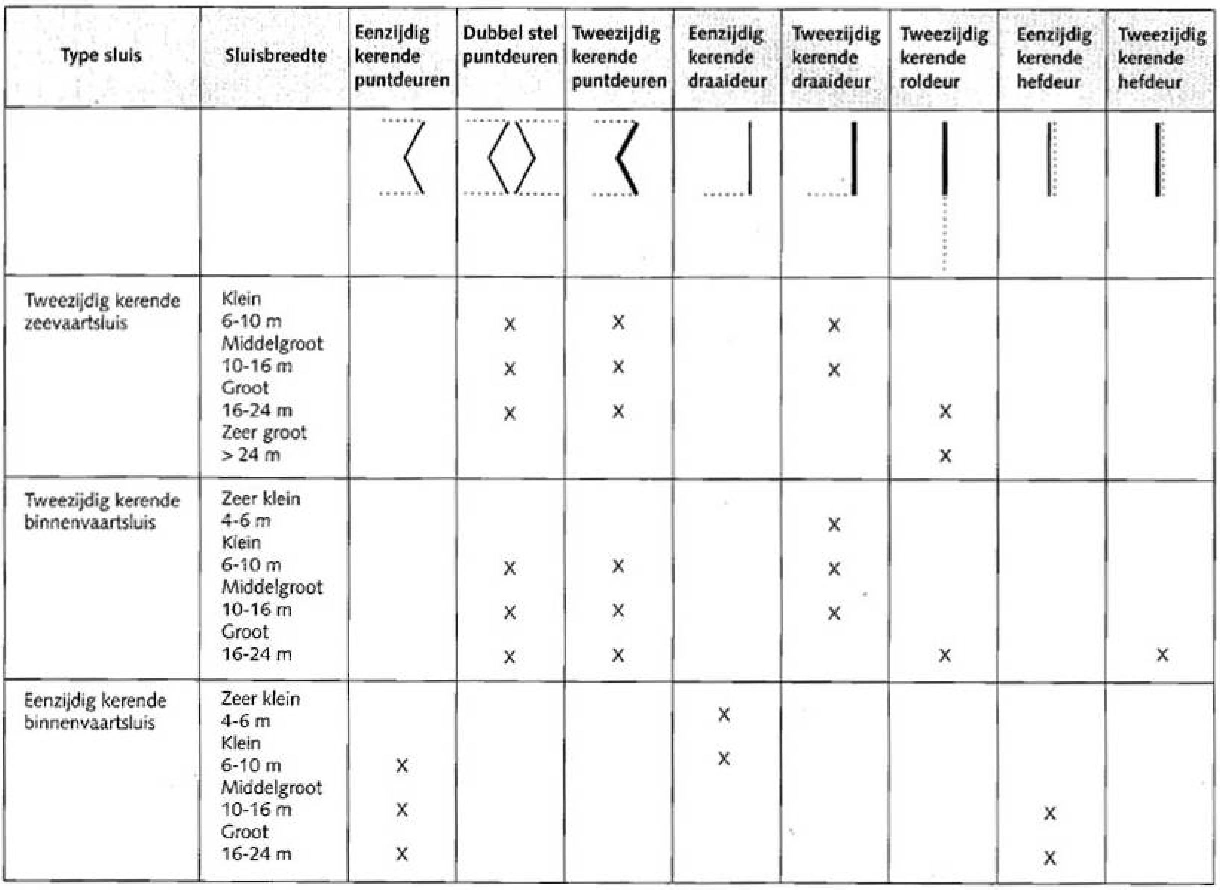
Figure III 1: Soil profile of 'Sluis Sambeek'

Table III 1: Soil profile of 'Sluis Sambeek'

Soil types		γ_{dry}	γ_{wet}	φ'_{rep} [1]	c'_{rep} [1]	K_0	K_a	K_p
Soil nr	Soil type	kN/m³	kN/m³	°	kPa			
1	Fill sand[1]	17,0	19,0	30	0	0,50	0,33	3,00
2	Clay	13,9	18,0	22,5	10	0,62	0,45	2,24
3	Gravel[1]	18,0	20,0	35	0	0,43	0,27	3,69
4	Sand	16,1	20,1	32,5	0	0,46	0,30	3,32
[1]	Assumed on basis of [Molenaar, 2006]							

Appendix IV. Shared structural components

Determination of the lock gate type



Type sluis	Sluisbreedte	Eenzijdig kerende puntdeuren	Dubbel stel puntdeuren	Tweezijdig kerende puntdeuren	Eenzijdig kerende draaideur	Tweezijdig kerende draaideur	Tweezijdig kerende roldeur	Eenzijdig kerende hefdeur	Tweezijdig kerende hefdeur
Tweezijdig kerende zeevaartsluis	Klein 6-10 m		x	x		x			
	Middelgroot 10-16 m		x	x		x			
	Groot 16-24 m		x	x			x		
	Zeer groot > 24 m						x		
Tweezijdig kerende binnenvaartsluis	Zeer klein 4-6 m					x			
	Klein 6-10 m		x	x		x			
	Middelgroot 10-16 m		x	x		x			
	Groot 16-24 m		x	x			x		x
Eenzijdig kerende binnenvaartsluis	Zeer klein 4-6 m				x				
	Klein 6-10 m	x			x				
	Middelgroot 10-16 m	x						x	
	Groot 16-24 m	x						x	

Figure IV 1: The required type of gate for each situation [Glerum, 2000]

Approximation of the filling and emptying system

It is checked if the levelling must take place through gate valves or by using a stilling basin. Therefore, it is calculated what the filling and the emptying time is for both systems. In Table IV 1 the different filling and emptying times for different ship types and lock types are given. The average lock cycle distribution for inland navigation locks can be seen in Table IV 2, according to the report 'Innovations in Navigation Lock Design' [PIANC, 2009]

These two tables are combined in Table IV 3 and this leads to the average maximum locking times of a lock and this time must be under the 30 minutes. Furthermore, a small amount of waiting time has to be added to define the final average passing time. According to Table 6.2 the maximum passing time is 24 minutes, thus the maximum average passing time is less than 30 min.

Table IV 1: Filling and emptying times comparison

Sill depth lower head (m NAP)	Dimensions of the lock	Ship type	Ship depth	Resulting filling or emptying time (min)			
				Emptying		Filling	
				Gate	Stilling chamber	Gate	Stilling chamber
3,4	Large lock (19,8*225m)	Va	3,5	5,8	5,8	8,3	7,0
		Vb	4	6,1	6,1	6,6	6,5
		VIa	4	8,7	8,7	9,3	9,3
3,4	Small lock (12,5*225m)	Va	3,5	7,7	7,7	9,5	8,2
		Vb	4	9,2	9,2	9,8	9,8
4,1	old lock (16*260m)	Va	3,5	8,3	8,3	12	9,2
		Vb	3,5	8,3	8,3	9,4	8,8

Table IV 2: Lock cycle distribution for inland navigation locks [PIANC, 2009]

Action	Percentage of time
Entrance/exit	18%
Mooring	18%
Gate manoeuvring	11%
Filling/ Emptying time	53%
Total	100%

Table IV 3: Average passing time different lock and ship types

Sill depth lower head (m NAP)	Dimensions of the lock	Ship type	Ship depth	Resulting average lock cycle time (min)	
				Gate	Stilling basin
3,4	Large lock (19,8*225m)	Va	3,5	13,3	12,1
		Vb	4	12,0	11,9
		VIa	4	17,0	17,0
3,4	Small lock (12,5*225m)	Va	3,5	16,2	15,0
		Vb	4	17,9	17,9
4,1	old lock (16*260m)	Va	3,5	19,2	16,5
		Vb	3,5	16,7	16,1

An example of the levelling time calculation for a gate valve is represented in Table IV 4 for the emptying time and in Table IV 5 for the filling time. The corresponding formula codes from [Glerum, 2000] are given in between the brackets.

Table IV 4: Emptying time calculation (large lock, gate, VIa) [Glerum, 2000]

1. Maximum surface of the empty hole		
$A_h = \sqrt{\dfrac{F'_p * A_{kse} * b_k * l_k}{1000 * \mu^2}}$		[6.7]
A_h	= surface of the empty hole	13,1
F'_p	= positive (force along the ship) (permillage)	0,8
A_{kse}	= wet cross section = $((h_{ben}-z_k)b_k - A_s)$ (m^2)	23,5
h_{ben}	= water level at the downstream side (m NAP)	8,1
z_k	= level lock bottom (m NAP)	3,4
b_k	= width of the lock (m NAP)	19,8
A_s	= Cross – section of the ship = $b_s * d_s$ (m^2)	69,6
b_s	= ship width (m)	17,4
d_s	= ship depth (m)	4,0
l_k	= lock length	225,0
μ	= discharge coefficient	0,7
b_h	= width of the valves (m)	13,3
2. Maximum lifting speed of the valve, due to translation waves		
$v_{h0} = \dfrac{F'_n * g * A_{kso}}{1000 * \mu * b_h * v_0}$		[6.8]
v_{hm}	= lifting speed of the valves, due toe translation waves (m/s)	0,009
F'_n	= negative (force along the ship) (permillage)	0,8
A_{ks0}	= wet cross section = $((h_{bov}-z_k)b_k - A_s)$ (m^2)	82,9
h_{bov}	= water level at the downstream side (m)	11,1
b_h	= width of the valves (m)	0,59
v_0	= $\sqrt{(2*g*\Delta h_0)}$ (m/s)	7,7
Δh_0	= starting level difference = h_{bov} - h_{ben} (m)	3,0
3. Maximum lifting speed of the valve, in combination with a smooth discharge		
$v_{hh} = \dfrac{3 * \mu * A_h^2 * v_0}{4 * d_3^2 * b_h * V}$		[6.9]
v_{hh}	= lifting speed of the valves, (smooth discharge) (s) [6.10]	0,004
d_3	= relation end lifting time and max discharge	1,10
V	= $b_k*l_k*\Delta h_0$ (m^3)	13365,0
v_h	= the biggest of both lifting speeds (m/s)	0,004
t_h	= Lifting time of the valves = $A_h/(b_h* v_h)$ (s) [6.11]	279,4
h_h	= Lifting height of the valves = v_h*t_h (m)	1,0
4. Fill time of the chamber		
$t_{ev} = \frac{1}{2} * t_h + \dfrac{b_k * l_k * v_0}{g * \mu * A_h}$		[6.12]
t_{ev}	= fill time of the chamber (s)	8,7
5. Negative (force along the ship) check		
$F'_m = \dfrac{16000/27 * \mu * b_h * v_h * v_0 * V * (c_{l1} / A_{ksm} - c_{l2} / A_{km})}{A_{km} * c_b * g * l_s}$		[6.13]
F'_m	= negative (force along the ship) (permillage) (max 0,8)	0,2
A_{ksm}	= wet cross–section besides the ship=$(h_{bov}$-5/9 $*\Delta h_0$- $z_k)*b_k$-A_s m^2	49,9
A_{km}	= wet cross–section behind the ship = $(h_{bov}$-5/9 $*\Delta h_0$- $z_k)*b_k$ (m^2)	119
C_b	= block coefficient	0,9
l_s	= ship length (m)	146
c_{l1}	= $(l_k-x_b)/ l_k)^2$ 0,9 ; c_{l2} = $(l_k-l_s-x_b)/ l_k)^2$	0,1
x_b	= distance bow - lock head (m)	10

Table IV 5: Filling time calculation (Large lock, gate, VIa) [Glerum, 2000]

1. Maximum lifting speed of the valve, due to translation waves		
$v_{h0} = \frac{F'_p * g * A_{kso}}{1000 * \mu * b_h * v_0}$		[6.1]
v_{h0}	= lifting speed, due to translation waves (m/s)	0,003
F'_p	= positive (force along the ship) (permillage)	0,8
A_{kso}	= wet cross section = $((h_{ben}-z_k)b_k - A_s)$ (m^2)	23,5
h_{ben}	= water level at the downstream side (m NAP)	8,1
z_k	= level lock bottom (m NAP)	3,4
b_k	= width of the lock (m NAP)	19,8
A_s	= Cross – section of the ship = $b_s * d_s$ (m^2)	69,6
b_s	= ship width (m)	17,4
d_s	= ship depth (m)	4,0
μ	= discharge coefficient	0,7
b_h	= width of the valves (m)	13,3
v_0	= $\sqrt{(2*g*\Delta h_0)}$ (m/s)	7,7
Δh_0	= starting level difference = $h_{bov} - h_{ben}$ (m)	3,0
h_{bov}	= water level at the downstream side (m)	11,1
g	= Gravity (m/s^2)	9,81
2. Maximum lifting speed of the valve, due to (vulstralen)		
$v_{hm} = \frac{-F'_n * A_{ksm} * c_b * g * l_s}{16000/27 * \mu * b_h * v_0 * V * (c_{l1} / A_{str} - c_{l2} / A_{ksm})}$		[6.2]
v_{hm}	= lifting speed (vulstralen) (m/s)	0,003
F'_n	= negative (force along the ship) (permillage)	0,8
A_{ksm}	= wet cross – section, during max discharge = $(h_{ben}+5/9 * \Delta h_0 - z_k) * b_k - A_s$ (m^2)	56,5
C_b	= block coefficient	0,9
l_s	= ship length (m)	146,0
V	= $b_k * l_k * \Delta h_0$ (m^3)	13365,0
l_k	= lock length	225,0
A_{str}	= $1{,}5 * b_h * d_2 * (h_{ben}-z_k)$ (m^2)	23,4
d_2	= cross section coefficient of the jet	0,3
c_{l1}	= $((l_k-x_b)/ l_k)^2$	0,9
c_{l2}	= $((l_k-l_s-x_b)/ l_k)^2$	0,1
x_b	= distance bow - lock head (m)	10,0
v_h	= the biggest of both lifting speeds (m/s) [6.3]	0,003
3. Lifting time of the valves		
$t_h = d_3 * \sqrt{\frac{2 * b_k * l_k * v_0}{3 * g * \mu * v_h * b_h}}$		[6.4]
t_h	= lifting time of the valves (s)	420,0
d_3	= relation end lifting time and max discharge	1,4
A_h	= surface of the fill hole = $b_h * v_h * t_h$ (m^2) [6.5]	14,4
h_h	= Lifting height of the valves = $v_h * t_h$ (m)	1,1
4. Fill time of the chamber		
$t_{ev} = \frac{1}{2} * t_h + \frac{b_k * l_k * v_0}{g * \mu * A_h}$		[6.6]
t_{ev}	= fill time of the chamber (min)	9,26

Demolishing the old lock

In Table IV 6 the dry and the wet demolishing volumes are calculated, to use these volumes in the cost calculations.

Table IV 6: Calculation of the demolishing volumes of the old lock

Upstream head dimensions		
Average water level	11,1	m NAP
Top level wall	11,75	m NAP
Bottom level (top floor)	5,2	m NAP
Bottom level floor	3	m NAP
Length Wall	30	m
Length Floor	15,5	m
Inside width	14	m
Inside height	6,55	m
Wall thickness	7	m
Reduction surface culvert	7	m^2
Dry surface	9,1	m^2
Wet surface wall	68,6	m^2
Wet surface floor	61,6	m^2
Volume of concrete	3286	m^3

Downstream head dimensions		
Average water level	8,1	m NAP
Top level wall	11,75	m NAP
Bottom level (top floor)	4,25	m NAP
Bottom level floor	2,25	m NAP
Length Wall	30	m
Length Floor	15,5	m
Inside width	14	m
Inside height	7,5	m
Wall thickness	7	m
Reduction surface culvert	7	m^2
Dry surface	51,1	m^2
Wet surface wall	39,9	m^2
Wet surface floor	56	m^2
Volume of concrete	3598	m^3

Lock chamber dimensions		
Min water level	7,7	m NAP
Max water level	11,1	m NAP
Average water level	9,4	m NAP
Top level wall	11,75	m NAP
Bottom level (top floor)	4,2	m NAP
Length	230	m
Width	16	m
Height	7,55	m

Removal concrete blocks		
Volume	3680	m^3

Wall removal		
Dry surface of the Wall	3,5	m^2
Wet surface of the Wall	26,3	m^2
Length double removal	35	m
Length single removal	195	m
Volume of concrete	7897	m^3

Intermediate head dimensions		
Min water level	7,7	m NAP
Max water level	11,1	m NAP
Average water level	9,4	m NAP
Top level wall	11,75	m NAP
Bottom level (top floor)	4,25	m NAP
Bottom level floor	2,25	m NAP
Length Wall	30	m
Length Floor	15,5	m
Inside width	16	m
Inside height	7,5	m
Wall thickness	7	m
Reduction surface culvert	7	m^2
Dry surface	32,9	m^2
Wet surface wall	58,1	m^2
Wet surface floor	60	m^2
Volume of concrete	3660	m^3

Total demolishing volume		
Volume concrete	18441	m^3
Removal of concrete blocks	3680	m^2

Appendix V. Alternative 1

Lock Head

The lock head is built in a building pit, the used volumes and outside dimensions can be seen in Table V 1. The lock heads are designed on the same basic calculation as shown in Appendix VI. So the design is checked for maximum reinforcement percentages, displacements and resistance against uplift. A sketch of the final design is shown in Figure 8.3.

Table V 1: Volumes and dimensions of the renovation lock heads of alternative 1

Concrete upstream head		
Type of concrete	B35	
Length	16,6	m
Width	22,0	m
Height	11,3	m
Sill length	3,5	m
Sill Width	16,0	m
Sill height	3,0	m
Floor volume	1118	m^3
Wall volume	649	m^3

Concrete downstream head		
Type of concrete	B35	
Length	16,6	m
Width	22	m
Height	10,5	m
Floor Volume	657	m^3
Wall Volume	649	m^3

Gates		
Type of steel	S235	
Number of gates	4	
Length	8,9	m
Thickness	0,9	m
Height	9,4	m
percentage steel	20%	m
Volume steel	60,24	m^3
Weight of the steel	473	ton

Seepage cut-off screen		
Type of screen	AZ12	
Cross section	0,0126	m^2/m
Top level screen	4	m NAP
Bottom level	-2	m NAP
Height	6	m
Width	18	m
Number	2	
Surface	207	m^2
Volume	3	m^3
Weight	20	ton

Temporarily sheet piles (total) 2 pits		
Type	AZ36-700	
Steel type	S235	
Margin	3	m
Length	19,6	m
Width	25,0	m
Height	18,00	m
Length	178,4	m
Thickness	0,499	
Cross section	0,0216	m^2/m
Sheet pile surface	3211,2	m^2
Steal volume	69,4	m^3
Steal weight	544	ton

Underwater concrete upstream		
Type	B35	
Top level	1,5	m NAP
Bottom level	0,0	m NAP
Thickness	1,5	m
Length	19,6	m
Width	25,0	m
Volume	735	m^3

Underwater concrete downstream		
Type	B35	
Top level	2,3	m NAP
Bottom level	0,8	m NAP
Thickness	1,5	m
Length	19,6	m
Width	25,0	m
Volume	735	m^3

Wall anchors temporarily (Van Leeuwen)		
Type	800 Ø70/M74/Ø200 mm	
Sheet pile length	78,4	m
Spreading	1,5	m
Number	52	

Excavation downstream head		
Top level	12,8	m NAP
Water level	7,7	m NAP
Bottom dredge level	0,8	m NAP
Excavation	4382	m^3

Excavation upstream head		
Top level	12,8	m NAP
Water level	11,1	m NAP
Bottom dredge level	0,0	m NAP
Excavation	4675	m^3

Fill up after construction		
Upstream head	664,44	m^3
Downstream head	617,4	m^3

GEWI Anchors temporarily		
Type	Ø 25T DSI (9m)	
Length	39,20	m
Number per width	7	
Spreading length	2,5	m
Number	110	

Also a seepage cut-off screen is needed, the height of this screen will be 5,75 m. This Height is calculated by Lane:

$$\Delta H_{crit} = (L_v + (1/3) * L_h) / C_L = (x + (1/3) * 16{,}6) / 5 = 3{,}4m => L_v = 11{,}5$$

ΔH_{crit} = Max water level difference >11,1-7,7=3,4 m

C_L = 5 (coarse sand)

L_v = vertical length = x=> 11,5 => 5,75 m screen height.

L_h = horizontal length =16,6 m

Lock chamber

In Table V 2 the dimensions and the volumes of the renovation of lock 3 can be seen. A cross-section of this renovation is shown in Figure 8.4.

Table V 2: Volumes and dimensions of the renovated lock chamber

Old lock chamber		
Length	260	m
Width	16	m
Top level	11,75	m NAP
Bottom level	4,25	m NAP

Concrete top heightening		
Type	B35	
New top level	12,8	m NAP
Thickness (m)	0,8	m
Length (m)	520	m
Volume (m)	437	m^3

Sand filling (fluid)		
Culvert height	2,7	m
Culvert width	2,5	m
Volume	3510	m^3

Bottom protection		
Layers	3	
Top level	3,9	m NAP
Bottom level	2,9	m NAP
Thickness	1	m
Length	225	m
Volume filter layer	4160	m^3

Excavation		
Top level	3,7	m NAP
Bottom dredge level	3,1	m NAP
Deep dredging	2496	m^3

Appendix VI. Alternative 2

Lock heads

The dimensions and the quantities of the lock heads are shown in Table VI 1.

Table VI 1: Dimensions and quantities for the alternative 2 lock heads

Concrete upstream head		
Type of concrete	B35	
Length	19,9	m
Width	27,8	m
Height	12,2	m
Sill length	4,0	m
Sill Width	19,8	m
Sill height	3,0	m
Floor Volume	1787	m^3
Wall Volume	1139	m^3

Concrete downstream head		
Type of concrete	B35	
Length	19,9	m
Width	27,8	m
Height	11,9	m
Floor volume	1383	m^3
Wall volume	1139	m^3

Gates		
Type of steel	S235	
Number of gates	4	
Length	11,0	m
Thickness	1,1	m
Height	9,4	m
Percentage steel	20%	m
Volume steel	91	m^3
Weight of the steel	714	ton

Excavation upstream head		
Top level	12,8	m NAP
Water level	11,1	m NAP
Bottom dredge level	-0,9	m NAP
Excavation	7579	m^3

Excavation downstream head		
Top level	12,8	m NAP
Water level	7,7	m NAP
Bottom dredge level	-0,6	m NAP
Excavation	7413	m^3

GEWI Anchors temporarily		
Type	Ø 25T DSI	
Length	45,80	m
Number per width	11	
Spreading length	2,5	m
Number	202	

Temporarily sheet piles (total) 2 pits		
Type	AZ36-700	
Steel type	S235	
Margin	3	m
Length	22,9	m
Width	30,8	m
Height	18,00	m
Total Length	214,8	m
Length for one	0,7	m
Thickness	0,499	m
Cross section	0,0216	m^2/m
Sheet pile surface	3866,4	m
Steal volume	83,5	m^3
Steal weight	656	ton

Underwater concrete upstream		
Type	B35	
Top level	0,6	m NAP
Bottom level	-0,9	m NAP
Thickness	1,5	m
Length	22,9	m
Width	30,8	m
Volume	1058	m^3

Underwater concrete downstream		
Type	B35	
Top level	0,9	m NAP
Bottom level	-0,6	m NAP
Thickness	1,5	m
Length	22,9	m
Width	30,8	m
Volume	1058	m^3

Seepage cut-off screen		
Type of screen	AZ12	
Cross section	0,0126	m^2/m
Top level screen	3	m NAP
Bottom level	-2	m NAP
Height	5	m
Width	30	m
Number	2	
Surface	310	m^2
Volume	4	m^3
Weight	31	ton

Fill up after construction		
Upstream head	838,14	m^3
Downstream head	817,53	m^3

Wall anchors temporarily (Van Leeuwen)		
Type	800 Ø70/M74/Ø200 mm	
Sheet pile length	91,6	m
spreading	1,5	m
Number	61	

The design is based on the maximum reinforcement percentage, the maximum displacement and the resistance against uplift. In Table VI 2 the parameters and starting points for these design calculations are presented. The weight of the gate and the resulting forces can be found in Table VI 3. In Table VI 4 these forces combined with the water pressure and the ground pressure are used to determine the maximum displacement and the reinforcement percentage of the walls. Then the resulting moments from the walls and the other forces that act on the floor are checked in Table VI 5. Finally the design is optimised in Table VI 6, by taking into account the uplift of the heads. The different forces that act on the lock heads are displayed in Figure VI 1.

Table VI 2: Parameters and starting point for the design calculations

Soil types		γ_{dry}	γ_{wet}	φ'_{rep} [1]	c'_{rep} [1]	K_0	K_a	K_p
Soil nr	Soil type	kN/m^3	kN/m^3	°	kPa			
1	Fill sand[1]	17,0	19,0	30	0	0,50	0,33	3,00
2	Clay	13,9	18,0	22,5	10	0,62	0,45	2,24
3	Gravel[1]	18,0	20,0	35	0	0,43	0,27	3,69
4	Sand	16,1	20,1	32,5	0	0,46	0,30	3,32
[1]	Assumed on basis of [Molenaar, 2006]							

Properties of the materials		
Steel	78,5	kN/m^3
Reinforcement steel	435	N/mm^2
Concrete (weight)	24	kN/m^3
Concrete (E)	31000	N/mm^2
Underwater concrete	23	kN/m^3
Water pressure	10	kN/m

Load factors	
Favourable	0,9
Permanent	1,2
Variable water loads	1,25
variable mitre gate	1,5

Loads		
Ground load	20	kN/m^2

Lock levels		
Min lock level downstream	7,7	m NAP
Max lock level downstream	8,1	m NAP
Min lock level upstream	10,85	m NAP
Max lock level upstream	11,1	m NAP

Main levels		
Top level	12,8	m NAP
Bottom level	3,4	m NAP

Load spreading over the wall	
2	m

Maximum displacements		
Lock head wall	l/300	
Lock head floor	30	mm

Table VI 3: Gate weight and corresponding resulting forces

Height	9,4	m
Length	11,0	m
Thickness	1,1	m
Volume	113,7	m^3
Percentage steel	20%	
Steel weight	78,5	kN/m^3
Weight of the gate	1785,7	kN
Uplift of the water	104,1	kN
Effective weight	1681,7	kN
Horizontal weight component at the top of the lock head	983,9	kN

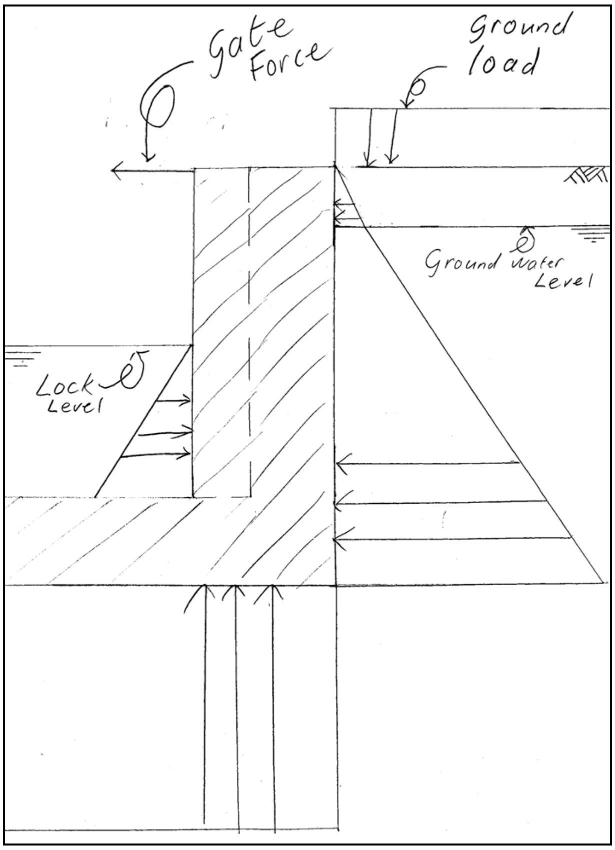


Figure VI 1: Distribution of the loads on the lock head

Table VI 4: Reinforcement check, alternative 2, lock head Wall

Maximum moment and displacement from the outside of the lock to the inside										
Soil type	Soil weight	Height from the bottom		K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)		Point of impact	
		high	low				Triangle	Rectangle		
1 dry	17,0	9,4	7,7	0,50	14,5	17,3	14,7	133,5	8,3	3,9
1 wet	19,0	7,7	7,1	0,50	5,7	7,1	2,1	50,6	7,3	3,6
2 wet	18,0	7,1	1,6	0,62	61,1	91,7	252,1	146,7	3,4	0,8
4 wet	20,1	1,6	0	0,46	14,9	17,9	14,3	0,0	0,5	0,0
Triangle q - water load inside the lock					-43,0	-38,70	-83,2		1,4	
Rectangular q - load (ground load)					9,3	14	130,5		4,7	
					SLS	ULS				
Reaction force on the head (Gate) (kN/m)					492,0	738,0			9,4	
Ground load			20	kN/m^2	Moment (kNm/m) (ULS)			12227		
					Displacement at the top (SLS)			6,2	<mm	31
Ground water height			11,1	m NAP						
Water height inside			7,7	m NAP						
Bottom height			3,4	m NAP						

Maximum moment and displacement from the inside of the lock to the outside										
Soil type	Soil weight	Height from the bottom		K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)		Point of impact	
		high	low				Triangle	Rectangle		
1 dry	17,0	9,4	7,1	0,50	19,6	23,5	27,0	166,6	7,9	3,6
2 dry	13,9	7,1	4,3	0,62	24,0	30,0	42,0	129,1	5,2	2,2
2 wet	18,0	4,3	1,6	0,62	30,0	45,0	60,8	72,0	2,5	0,8
4 wet	20,1	1,6	0	0,46	14,9	17,9	14,3	0,0	0,5	0,0

Triangle q - water load inside the lock	-77,0	-96,25	-370,6		2,6
Triangle q - water force trough the gate	354,9	443,6	953,7	Outside	1,4
Triangle q - water force trough the gate	-635,4	-794,3	-3058,0	Inside	2,6

					Limit	
Ground water height	7,7	m NAP	Moment (kNm/m) (ULS)	-5915		
Water height inside	11,1	m NAP	Displacement at the top (SLS)	9,9	< mm	31
Bottom height	3,4	m NAP				

Moment reinforcement combined wall		
f_s	435	N/mm^2
d	2,4	m
z	2,1	m
N	8689	kN
A_s	19974	mm^2
Reinforcement percentage	0,83%	<1,94%
I	1,152	m^4

Concrete Volume of the Wall		
Concrete recess thickness	1,6	m
Length of the recess	11,9	m
Wall thickness	2,4	m
Total thickness of the Wall	4	m
Length of the two piers	4	m
Total length	19,9	m
Volume of one Wall	569	m^3
Top surface of the wall	60,6	m^2

Maximum moment and displacement from the outside of the lock to the inside										
Soil type	Soil weight	Height from the bottom		K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)		Point of impact	
		high	low				Triangle	Rectangle		
1 dry	17,0	9,4	7,7	0,50	14,5	17,3	14,7	133,5	8,3	3,9
1 wet	19,0	7,7	7,1	0,50	5,7	7,1	2,1	50,6	7,3	3,6
2 wet	18,0	7,1	1,6	0,62	61,1	91,7	252,1	146,7	3,4	0,8
4 wet	20,1	1,6	0	0,46	14,9	17,9	14,3	0,0	0,5	0,0

Rectangular q - load (ground load)	9,3	14	130,5		4,7

Ground load	20	kN/m^2	Moment (kNm/m) (ULS)	2434,8		
Ground water height	11,1	m NAP	Displacement at the top (SLS)	0,5	< mm	31
Water height inside	7,7	m NAP				
Bottom height	3,4	m NAP				

Table VI 5: Reinforcement check, alternative 2, lock head floor

Forces from outside to the inside		
Wall moment	-5914	kNm/m
Ground water level	11,1	m NAP
Water level in the lock	7,7	m NAP
Top concrete floor	3,4	m NAP
Bottom concrete floor	0,90	m NAP
Rep. lock width	19,8	m
Max floor width	23,0	m
Water pressure moment	-5871	kNm/m
Moment concrete weight	3570	kNm/m
Resulting moment	-8215	kNm/m

Forces from the inside to the outside		
Lock gates moment	12226	kNm/m
Ground water level	7,7	m NAP
Water level in the lock	11,1	m NAP
Top concrete floor	3,4	m NAP
Bottom concrete floor	0,9	m NAP
Max floor width	23	m
Water pressure moment	2318	kNm/m
Moment concrete weight	4761	kNm/m
Resulting moment	19306	kNm/m

Upstream sill on the upstream head		
Length	4	m
Width	19,8	m
Top level	6,4	m NAP
Volume	237,6	m^3

Dewatered lock		
Wall moment	2434	kNm/m
Ground water level	11,1	m NAP
Water level in the lock	3,4	m NAP
Top concrete floor	3,4	m NAP
Bottom concrete floor	0,9	m NAP
Max floor width	23,0	m
Water pressure moment	-8430	kNm/m
Total moment	-5996	

Maximum displacement (SLS)			
q max ground water level	-102	kN/m^2	
q min ground water level	-43	kN/m^2	
q load, max lock water level	77	kN/m^2	
q min lock water level	43	kN/m^2	
q no water in the lock head	0	kN/m^2	
The outside to the inside	-3	mm <	30
The inside to the outside	4	mm <	30
Dewatered lock	-1	mm <	30

Moment reinforcement		
f_s	435	N/mm^2
d	2500	mm
z	2175	mm
N (kN)	12654	kN
A_s	29089	mm^2
Reinforcement	1,16%	<1,94%
I	1,302	m^4

Table VI 6 : Uplift check, alternative 2, lock head

Uplift control Downstream head		
Highest water level	8,1	m NAP
Width	27,8	m
Length	19,9	m
Bottom of the floor	0,9	m NAP
Total uplift force (SLS)	39832	kN
Volume of the floor	1383	m^3
Volume two walls	1139	m^3
Weight of the structure (SLS)	60518	kN
Uplift limit must higher than 0	12720	kN

Uplift control Upstream head		
Highest water level	11,1	m NAP
Width	27,8	m
Length	19,9	m
Bottom of the floor	0,6	m NAP
Total uplift force (SLS)	58088	kN
Upstream floor thickness	2,8	m
Volume of the floor	1549	m^3
Volume two walls	1139	m^3
Weight of the structure (SLS)	64501	kN
Weight of the extra sill (SLS)	5702	kN
Uplift limit must higher than 0	498	kN

Seepage cut off screen

No seepage cut off screen was needed according to Lane in case of an impermeable lock bottom. But, after the cost calculation of Chapter 9 the permeable lock bottom appeared to be cheaper, than the concrete impermeable bottom. So finally a seepage screen of 10,4/2= 5,2 m height is needed.

$$\Delta H_{crit} = (L_v + (1/3) * L_h) / C_L = (x + (1/3) * (19,9)) / 5 = 3,4m => L_v = 10,4m$$

ΔH_{crit} = Max water level difference >11,1-7,7=3,4 m

C_L = 5 (coarse sand)

L_v = vertical length = x

L_h = horizontal length =264,8 m

Lock chamber (Not used anymore)
In Table VI 7 all the construction parts of the lock chamber and the quantities of these parts can be seen. These parts were needed for the original lock chamber design with an impermeable bottom, but finally the same lock chamber as the large flexible lock from alternative 3 is used, which has a permeable filter bottom and heavier sheet piles.

Table VI 7: Dimensions and quantities of the alternative 2 lock chamber

Underwater concrete		
Type	B35	
Top level	3,4	m NAP
Bottom level	1,9	m NAP
Thickness	1,5	m
Length	225	m
Width	20,9	m
Volume	7054	m^3

Prefab concrete slabs		
Type	B35	
Thickness	0,55	m
Height	5,25	m
Slab	1,50	m
Length	450,00	m

GEWI Anchors		
Type	Ø 25T DSI	
Length	8,60	m
Number per width	8	
Spreading length	2,5	m
Total number	720	

Excavation		
Top level	12,8	m NAP
Water level	7,7	m NAP
Bottom dredge level	1,9	m NAP
Excavation	51257	m^3

Wall anchors (Van Leeuwen)		
Type	800 Ø70/M74/Ø200 mm	
Length	25	m
Spreading	1,5	m
Total number	300	

Sheet piles		
Type	AZ36-700	
Steel type	S235	
Top level	12,80	m NAP
Bottom level	-5,20	m NAP
Height	18,00	m
Length	450	m
Thickness	0,499	m
Cross section	0,0216	m^2/m
Sheet pile surface	8100	m^2
Steal volume	175,0	m^3
Steal weight	1373,4	ton

In Figure 8.6 the cross section of the lock chamber is sketched. In this sketch all the obtained materials and dimensions for the lock chamber can be seen. In Figure VI 2 and Figure VI 3 the moments the shear forces and deflections that are acting on the sheet pile wall can be seen. The sheet pile walls are calculated in two phases, namely the construction and the users phase. The pile length is after some iteration in M-sheet determined. The ground level of + 12,8 m Nap is in these two figures represented as the 0 level.
The deflections are within the norms of the CUR 166 (1/100 construction phase and 1/200 for the using phase). Also the moments and the shear forces can be handled by the sheet pile profiles. The anchors must be able to cope with a horizontal force of 528 kN. When the anchors are place under an angle of 30º this means that the anchors must be able to handle a force of 528/cos(30)= 587 kN/m. The anchors are place every 1,5 m, so this means that the maximum force this anchor has to handle is 880,5 kN.
The underwater concrete floor is determined on 1,5 m thickness in the next computation is determined which force each of the tension piles have to handle when they are situated on 2,5 m from each other (6,25 m^2 of water pressure)

Max groundwater level:	+11,1 m Nap
Min lock water level:	+7,7 m Nap
Top floor level:	+3,4 m Nap
Under floor level:	+2,4 m Nap
Floor surface:	6,25 m^2

$$6{,}25 * ((3{,}4 - 1{,}9) * 23 + (7{,}7 - 3{,}4) * 10) + F_{tension} = 6{,}25 * 1{,}2 * (11{,}1 - 1{,}9) * 10$$
$$484{,}4 + F_{tension} = 690$$
$$F_{tension} = 205.6kN$$

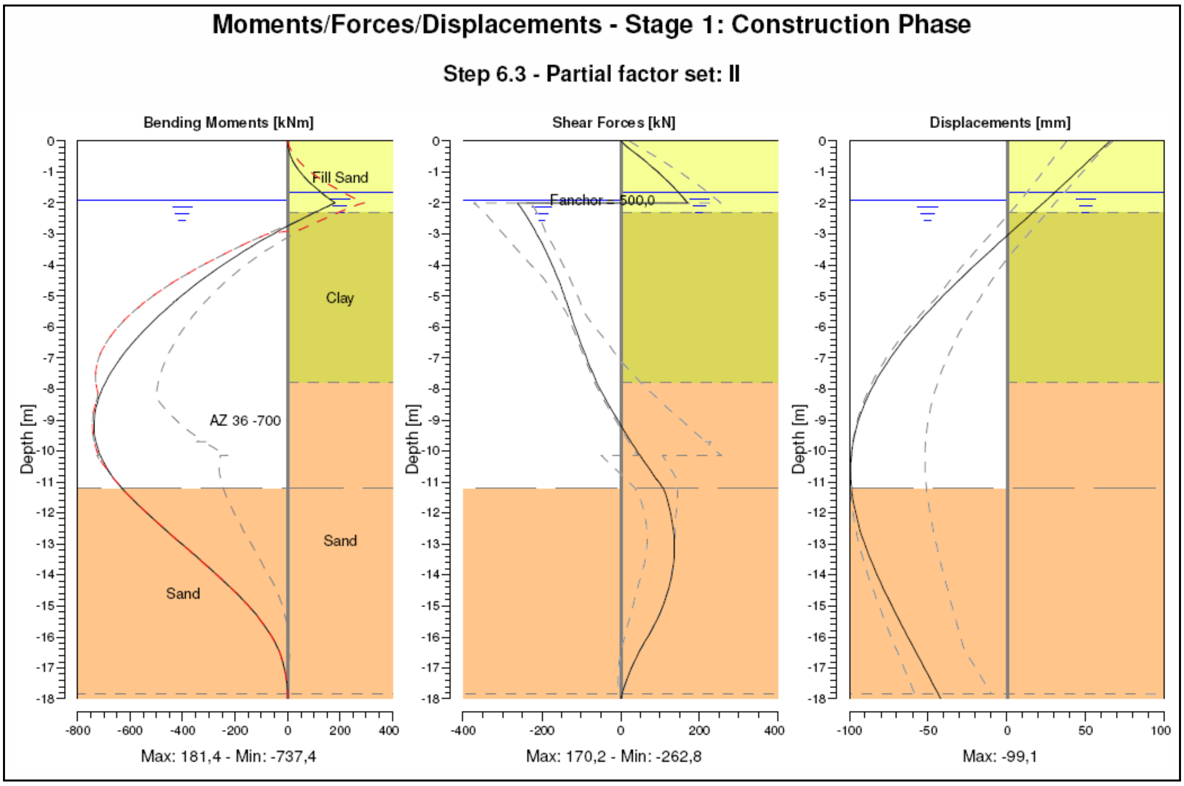


Figure VI 2: Moments/forces/displacements, construction phase, alternative 2

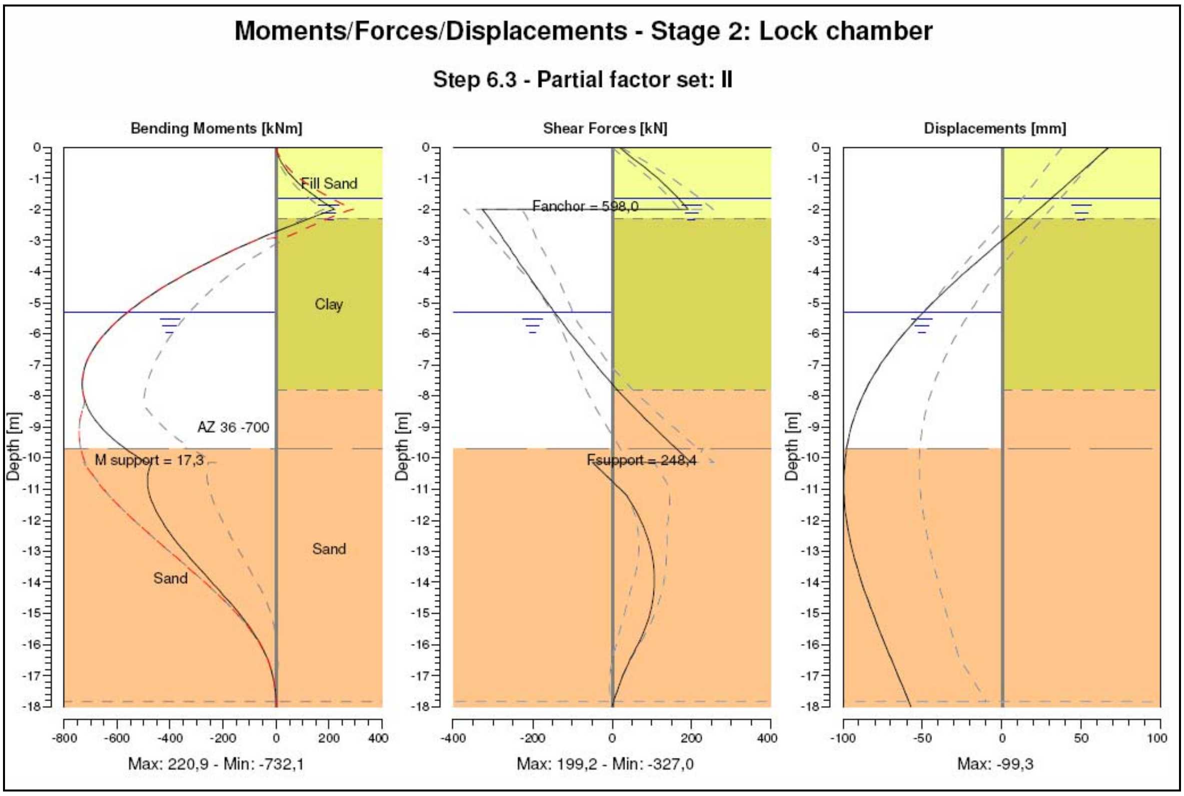


Figure VI 3: Moments/forces/displacements, users phase, alternative 2

Appendix VII. Alternative 3

Lock Head

Volumes

In Table VII 1 the initial volumes and dimensions for the flexible lock heads are determined and in Table VII 2 the volumes of the upstream building pit can be seen. The additional volumes and dimensions in case of an extension of the flexible lock heads are presented in Table VII 3 and the corresponding building pit volumes are shown in Table VII 4.

Table VII 1: Initial volumes and dimensions of the small lock heads (2020)

Concrete upstream head		
Type of concrete	B35	
Length	15,6	m
Width	43,5	m
Height	10,6	m
Floor Volume	309	m^3
Wall Volume	363	m^3
Caisson Volume	652	m^3

Concrete downstream head		
Type of concrete	B35	
Length	15,6	m
Width	43,5	m
Height	10,6	m
Floor Volume	308,9	m^3
Wall Volume	363,4	m^3
Caisson Volume	522	m^3

Gates		
Type of steel	S235	
Number of gates	4	
Length	7,0	m
Thickness	0,7	m
Height	9,4	m
Percentage steel	20%	m
Volume steel	37	m^3
Weight steel	289	ton

Filling Sand volume		
Sand Filling	4465	m^3

Foundation layer (Gravel)		
Thickness	0,5	m
Volume	678,6	m^3

Excavation upstream head		
Slope	1	2
Top width	87,9	m
Top level	12,8	m NAP
Water level	11,1	m NAP
Bottom dredge level	1,7	m NAP
Excavation	11377	m^3

Excavation downstream head		
Slope	1	2
Top width	87,9	m
Top level	12,8	m NAP
Water level	7,7	m NAP
Bottom dredge level	1,7	m NAP
Excavation	11377	m^3

Seepage cut-off screen		
Type of screen	AZ12	
Cross section	0,0126	m^2/m
Top level screen	3,2	m NAP
Bottom level	-2,7	m NAP
Height	5,9	m
Width	23	m
Number	2	
Surface	271,4	m^2
Volume	3	m^3
Weight	27	ton

Table VII 2: Building pit volumes for the construction of the small heads (2020)

Building pit upstream		
Spacing	2,5	m
Slope	1	2
Width pit	41,2	m
Length pit	48,5	m
Floating depth	4,5	m
Pit depth	5,5	m
Gravel layer	0,25	m
Gravel volume	339,3	m^3
Excavation	17236	m^3
Filling up	17236	m^3
Working floor	1998	m^2
Seepage screen		
Type of screen	AZ 12	
Depth	21	m
Length	224	m
Surface	4697	m^2
Construction time		
Number of weeks	26	weeks
Transport and immersion of the lock heads		
Number of heads (pieces)	2	
Surface of the bulk heads	310	m^2

Table VII 3: Additional volumes and dimensions of the large lock heads (2052)

Concrete upstream head		
Type of concrete	B35	
Length	19,3	m
Width	60,4	m
Height	11,3	m
Floor Volume	931	m^3
Wall Volume	379	m^3
Caisson Volume	1046	m^3
Concrete downstream head		
Type of concrete	B35	
Length	19,3	m
Width	60,4	m
Height	11,3	m
Floor Volume	931	m^3
Wall Volume	379	m^3
Caisson Volume	820	m^3
Gates		
Type of steel	S235	
Number of gates	4	
Length	11,0	m
Thickness	1,1	m
Height	9,4	m
Percentage steel	20%	m
Volume steel	91	m^3
Weight steel	714	ton

Foundation layer		
Thickness	0,5	m
Volume	1165,72	m^3
Excavation upstream head		
Slope	1	2
Top width	107,6	m
Top level	12,8	m NAP
Water level	11,1	m NAP
Bottom dredge level	1,0	m NAP
Excavation	19130	m^3
Excavation downstream head		
Slope	1	2
Top width	107,6	m
Top level	12,8	m NAP
Water level	7,7	m NAP
Bottom dredge level	1,0	m NAP
Excavation	19130	m^3
Fluid sand volume		
Sand Filling	7633	m^3
Sand removing	4465	m^3

Table VII 4: Building pit volumes for the construction of the large heads (2052)

Building pit upstream		
Spacing	2,5	m
Slope	1	2
Width pit	48,6	m
Length pit	65,4	m
Floating depth	5,1	m
Pit depth	6,1	m
Gravel layer	0,25	m
Gravel volume	582,9	m^3
Excavation	28864	m^3
Filling up	28864	m^3
Working floor	3178	m^2
Seepage screen		
Type of screen	AZ 12	
depth	23	m
Length	277	m
Surface	6369	m^2
Construction time		
Number of weeks	26	weeks
Transport and immersion of the lock heads		
Number of heads (pieces)	2	
Surface of the bulkheads	491	m^2

Dimensions

The dimensions of the small upstream lock head (2020) are represented in Figure VII 1, Figure VII 2 and Figure VII 3. The dimensions of the large upstream lock head (2052) can be seen in Figure VII 4, Figure VII 5 and Figure VII 6. The dimensions of the downstream heads are almost the same, only the height of the caisson walls is different, this can be seen in Figure 8.9 and Figure 8.10.

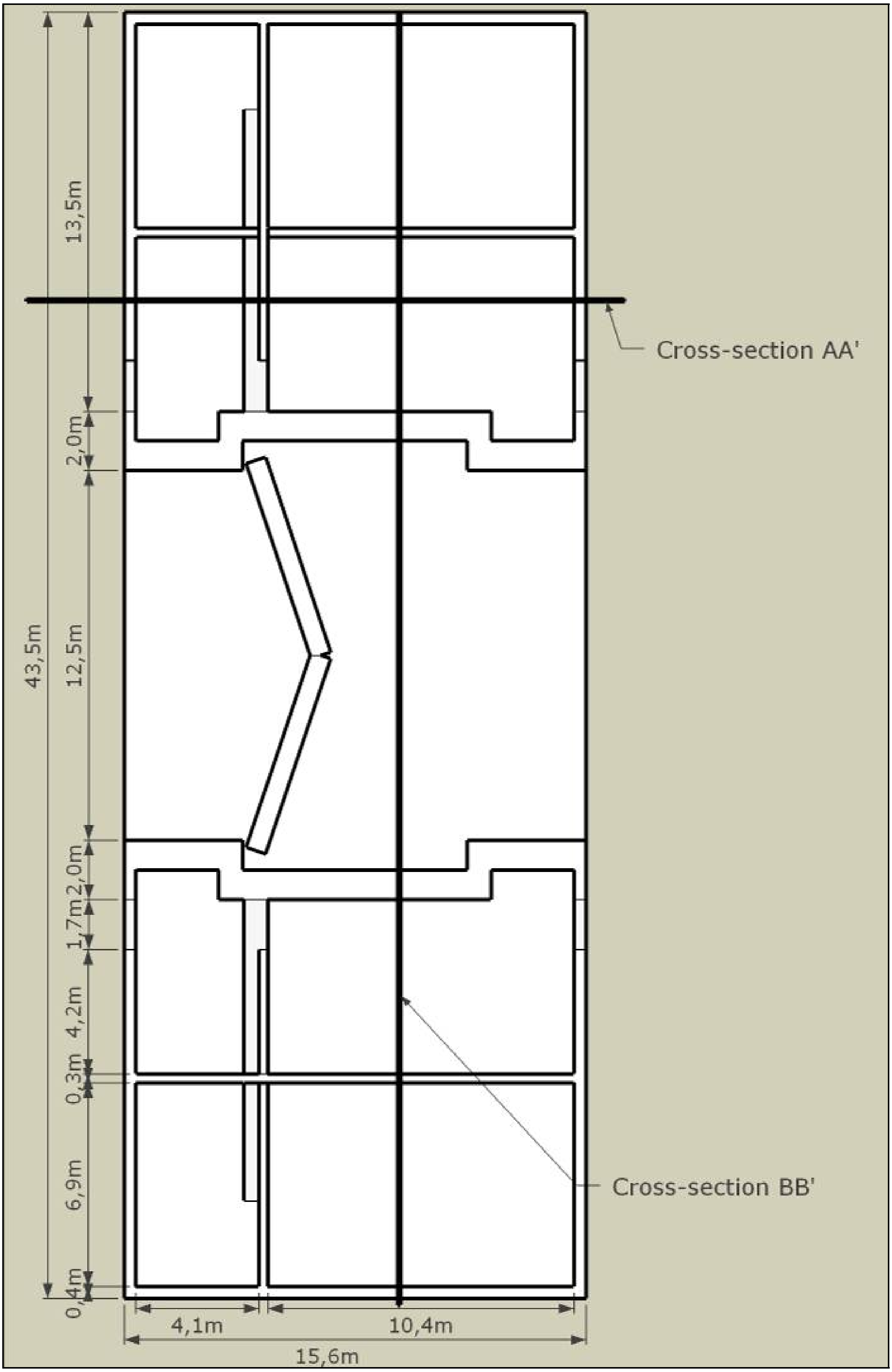


Figure VII 1: Upstream small lock head overview (2020)

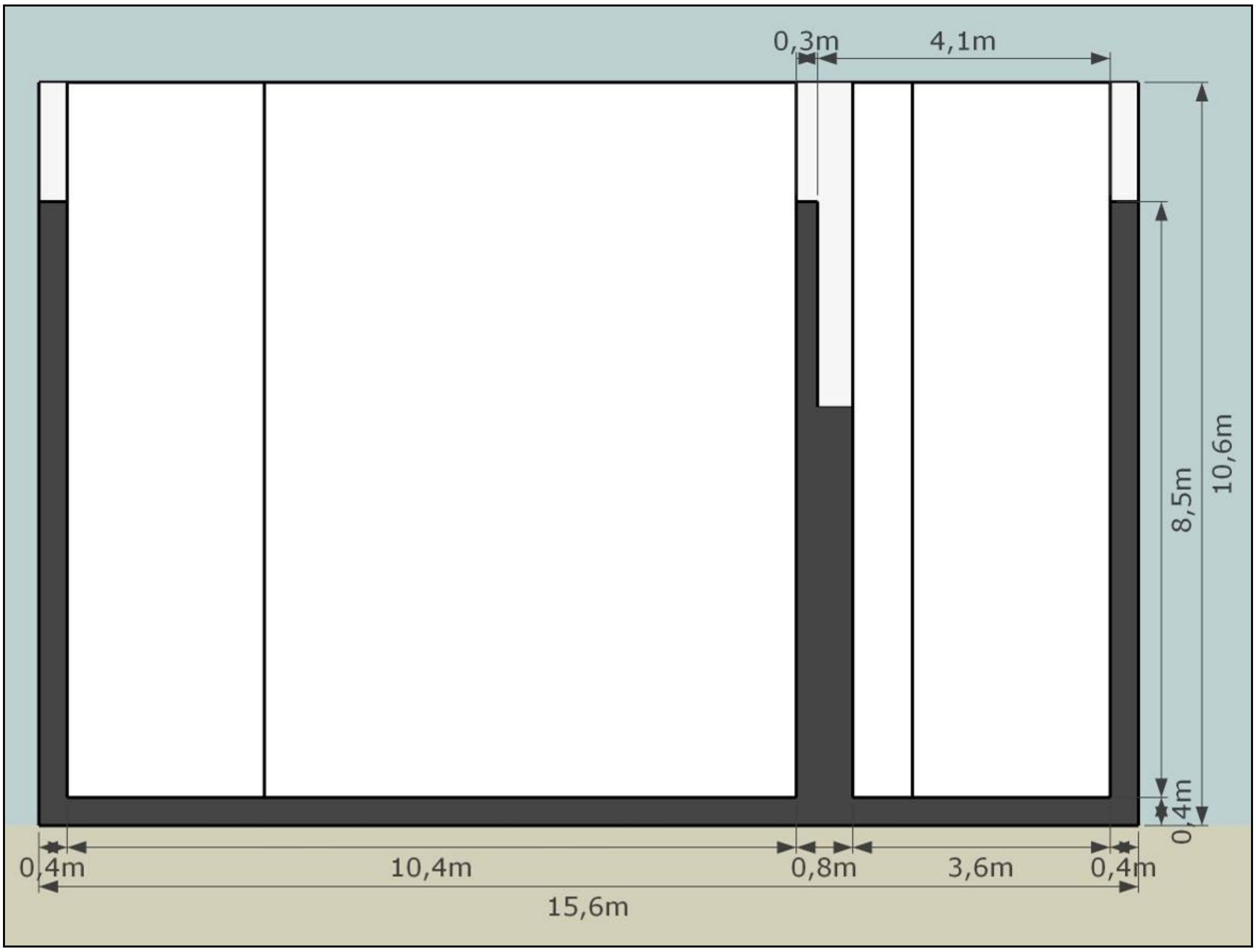


Figure VII 2: Upstream small lock head Cross section AA' (2020)

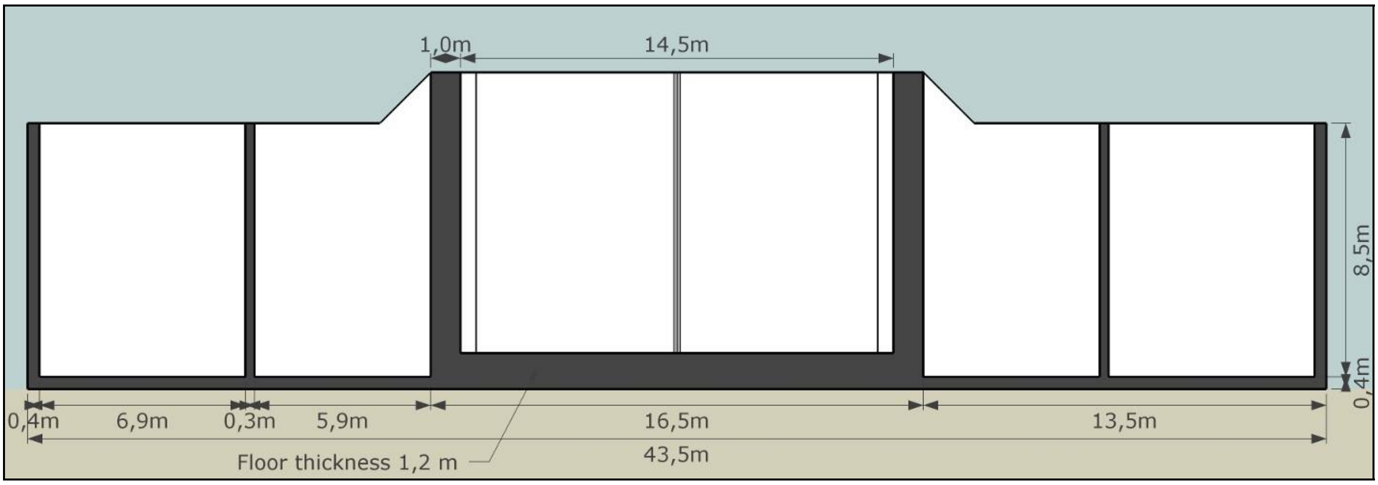


Figure VII 3: Upstream small lock head long section BB' (2020)

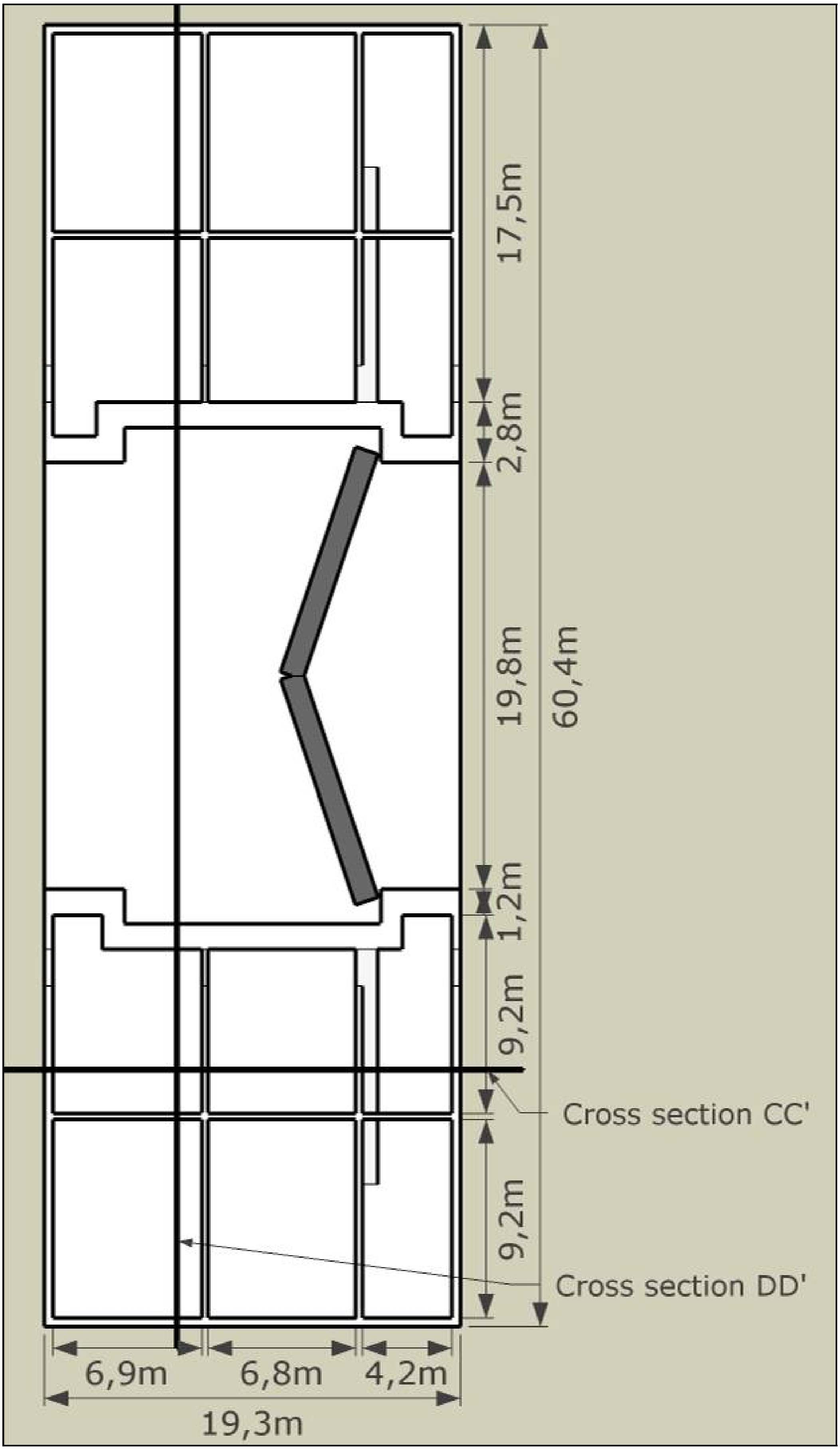


Figure VII 4: Upstream large lock head overview (2052)

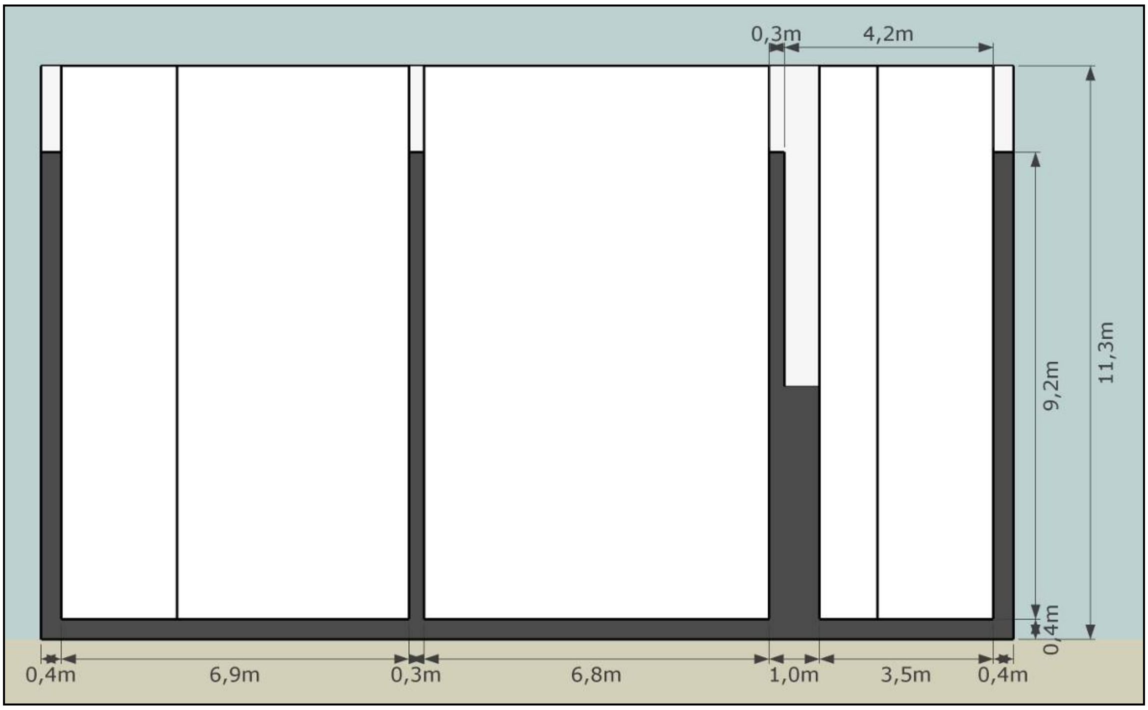


Figure VII 5: Upstream large lock head Cross section CC' (2052)

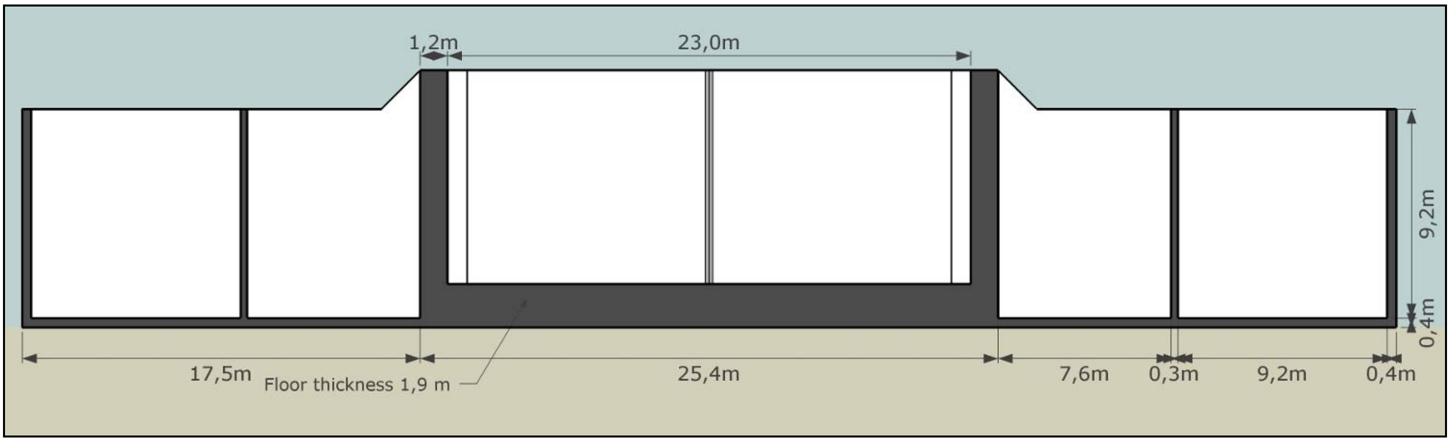


Figure VII 6: Upstream large lock head long section DD' (2052)

Design Calculations

The design is based on the maximum reinforcement percentage, the maximum displacement and the requirements that are represented in paragraph 8.6.1 and the parameters and starting points from Appendix VI (Table VI 2) are used. Also Figure VI 1 is applicable on these calculations. In this appendix only the calculations for the small lock head (2020) are represented, the large lock head calculation has the same basis, only the parameters differ.

The lock head wall thickness is determined in Table VII 5, the forces of the gate have no influence on the lock head wall, because the wall is supported on its reaction point on the lock wall. The weight of the gate and the resulting forces can be found in Table VII 6. In Table VII 7 these forces combined with the water pressure and the ground pressure are used to determine the maximum reaction moment. Then the resulting moments from the walls and the other forces that act on the floor are checked in Table VII 8. The floor is made as thin as possible to reduce the floating depth. The depth during transportation and the resistance against uplift are determined in Table VII 9. The last check is the stability check in Table VII 10.

Table VII 5: Wall reinforcement and displacement check, small lock

Maximum moment and displacement from the outside of the lock to the inside

Soil type	Soil weight	Height from the bottom: high	Height from the bottom: low	K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)	Point of impact
4 dry	16,1	9,4	7,7	0,46	12,7	15,2		
4 wet	20,10	7,7	0	0,46	71,6	85,9		
Triangle q - water load inside the lock					-43,0	-38,70	-83,2	1,4
Triangle q - soil load outside the lock (4dry)					12,7	15,2	12,9	8,3
Rectangular q - load (4 dry)					12,7	15,2	117,0	3,9
Triangle q- soil load (4 wet)					71,6	85,9	330,8	2,6
Rectangular q - load (ground load)					9,3	14	130,5	4,7

					Limit
Ground load	20	kN/m^2	Moment (kNm/m) (ULS)	1900	
Ground water height	11,1	m NAP	Displacement at the top (mm)	8,7	31
Water height in the head	7,7	m NAP			
Bottom height	3,4	m NAP			

Maximum moment and displacement from the inside of the lock to the outside

Soil type	Soil weight	Height from the bottom: high	Height from the bottom: low	K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)	Point of impact
4 dry	16,1	9,4	4,3	0,46	38,0	45,6		
4 wet	20,10	4,3	0	0,46	40,0	48,0		
Triangle q - water load inside the lock					-77,0	-96,25	-370,6	2,6
Triangle q - soil load outside the lock (4 dry)					38,0	45,6	116,3	6,0
Rectangular q - load (4 dry)					38,0	45,6	196,0	2,2
Triangle q- soil load (4 wet)					40,0	48,0	103,2	1,4

					Limit
Ground water height	7,7	m NAP	Moment (kNm/m) (ULS)	316	
Water height in the head	11,1	m NAP	Displacement at the top (mm)	-2,4	31
Bottom height	3,4	m NAP			

Moment reinforcement combined wall		
f_s	435	N/mm^2
d	1	m
z	0,9	m
N	1821	kN
A_s	4187,3	mm^2
Reinforcement percentage	0,42%	<1,94%
I	0,083	m^4

Concrete Volume of the Wall		
Concrete recess thickness	1	m
Length of the recess	7,6	m
Wall thickness	1	m
Total thickness of the Wall	2	m
Length of the two piers	4	m
Total length	15,6	m
Volume of one Wall	182	m^3
Top surface of the wall	18,0	m^2

Table VII 6: Gate weight and corresponding resulting forces, small lock

Height	9,4	m
Length	7,0	m
Thickness	0,7	m
Volume	46,1	m^3
Percentage steel	20%	
Steel weight	78,5	kN/m^3
Weight of the gate	1084,7	kN
Uplift of the water	63,2	kN
Effective weight	1021,5	kN
Horizontal weight component at the top of the lock head	380,3	kN

Table VII 7: Wall moment check, small lock

Maximum moment and displacement from the outside of the lock to the inside								
Soil type	Soil weight	Height from the bottom high	Height from the bottom low	K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)	Point of impact
4 dry	16,1	9,4	7,7	0,46	12,7	15,2		
4 wet	20,10	7,7	0	0,46	71,6	85,9		
Triangle q - water load inside the lock					-43,0	-38,70	-83,2	1,4
Triangle q - soil load outside the lock (4dry)					12,7	15,2	12,9	8,3
Rectangular q - load (4dry)					12,7	15,2	117,0	3,9
Triangle q- soil load (4wet)					71,6	85,9	330,8	2,6
Rectangular q - load (ground load)					9,3	14	130,5	4,7
					SLS(kN/m)	ULS(kN/m)		
Reaction force on the head (Gate)					190,2	285,3	285,3	9,4
Ground load		20	kN/m^2		Moment (kNm/m) (ULS)		4582	
Ground water height		11,1	m NAP					
Water height inside		7,7	m NAP					
Bottom height		3,4	m NAP					

Maximum moment and displacement from the inside of the lock to the outside								
Soil type	Soil weight	Height from the bottom high	Height from the bottom low	K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)	Point of impact
4 dry	16,1	9,4	4,3	0,46	38,0	45,6		
4 wet	20,10	4,3	0	0,46	40,0	48,0		
Triangle q - water load inside the lock					-77,0	-96,25	-370,6	2,6
Triangle q - soil load outside the lock (4dry)					38,0	45,6	116,3	6,0
Rectangular q - load (4 dry)					38,0	45,6	196,0	2,2
Triangle q- soil load (4 wet)					40,0	48,0	103,2	1,4
Triangle q - water force trough the gate outside					225,8	282,3	606,9	1,4
Triangle q - water force trough the gate inside					-404,4	-505,5	-1946,0	2,6
Ground water height		7,7	m NAP		Moment (kNm/m)		-3809	(ULS)
Water height inside		11,1	m NAP					
Bottom height		3,4	m NAP					

Maximum moment and displacement from the outside of the lock to the inside								
Soil type	Soil weight	Height from the bottom: high	Height from the bottom: low	K_0	SLS (kN/m^2)	ULS (kN/m^2)	ULS resulting force (kN/m)	Point of impact
4 dry	16,1	9,4	7,7	0,46	12,7	15,2		
4 wet	20,10	7,7	0	0,46	71,6	85,9		
Triangle q - soil load outside the lock (4dry)					12,7	15,2	12,9	8,3
Rectangular q - load (4 dry)					12,7	15,2	117,0	3,9
Triangle q- soil load (4 wet)					71,6	85,9	330,8	2,6
Rectangular q - load (ground load)					9,3	14	130,5	4,7

Ground load	20	kN/m^2	Moment (kNm/m) (ULS)	2020
Ground water height	11,1	m NAP		
Water height inside	3,4	m NAP		
Bottom height	3,4	m NAP		

Table VII 8: Reinforcement check, small lock, lock head floor

Forces from outside to the inside		
Wall moment	-3809	kNm/m
Ground water level	11,1	m NAP
Water level in the lock	7,7	m NAP
Top concrete floor	3,4	m NAP
Bottom concrete floor	2,20	m NAP
Rep. lock width	12,5	m
Max floor width	14,5	m
Water pressure moment	-1906	kNm/m
Moment concrete weight	681	kNm/m
Resulting moment	-5034	kNm/m

Forces from the inside to the outside		
Lock gates moment	2681	kNm/m
Ground water level	7,7	m NAP
Water level in the lock	11,1	m NAP
Top concrete floor	3,4	m NAP
Bottom concrete floor	2,2	m NAP
Max floor width	14,5	m
Water pressure moment	1229	kNm/m
Moment concrete weight	908	kNm/m
Resulting moment	4818	kNm/m

Dewatered lock		
Wall moment	2019	kNm/m
Ground water level	11,1	m NAP
Water level in the lock	3,4	m NAP
Top concrete floor	3,4	m NAP
Bottom concrete floor	2,2	m NAP
Max floor width	14,5	m
Water pressure moment	-2923	kNm/m
Total moment	-904,1	

Maximum displacement (SLS)			
q max ground water level	-89	kN/m^2	
q min ground water level	-43	kN/m^2	
q max lock water level	77	kN/m^2	
q min lock water level	43	kN/m^2	
q no water in the lock head	0	kN/m^2	
From the outside to the inside	-7	mm<	30
From the inside to the outside	3	mm<	30
Dewatered lock	1	mm<	30

Moment reinforcement		
f_s	435	N/mm^2
d	1200	mm
z	1044	mm
N (kN)	9438	kN
A_s	21696	mm^2
Reinforcement	1,81%	<1,94%
I	0,144	m^4

Table VII 9: The floating depth and the uplift resistance, small heads (2020)

Parameters		
Width	43,5	m
Length	15,6	m
Top level	12,8	m NAP
Top level floor	3,4	m NAP
Top level bottom	2,2	m NAP
Volume of the floor	308,9	m^3
Volume of the walls	363,4	m^3
Volume of the side slabs	334,9	m^3
Volume of the slab walls upper head	317,3	m^3
Volume of the slab walls lower head	187,2	m^3
Reduction volume of the walls	114,2	m^3
Total concrete volume upper head	1210,3	m^2
Total concrete volume lower head	1080,2	m^3
Total weight of the concrete upper head	29047,9	kN
Total weight of the concrete lower head	25924,5	
Extra weight of the sand	54685,3	kN
Extra weight of the water barrages upper head	1925,0	kN
Extra weight of the water barrages lower head	1175,0	kN

Uplift control Downstream head		
Highest water level	8,1	m NAP
Total uplift force (SLS)	40037	kN
Uplift limit must higher than 0	35688	kN

Uplift control Upstream head		
Highest water level	11,1	m NAP
Total uplift force (SLS)	60395	kN
Uplift limit must be higher than 0	11259	kN

Floating depth		
Floating depth upper head	4,56	< 5,2 m
Floating depth lower head	3,99	< 4,0 m

Table VII 10: Stability check, for the small heads, during floating

The construction is stable when G < M		
Upper head stability		
Half of the volume	605	m^3
Cross-section above the floor	86	m^3
Floor volume (1,2m)	546	m^3
I (of the smallest cross section)	13762	m^4
V (water displacement)	3097	m^3
B (Centre of pressure)	2,28	m
G (Centre of gravity)	1,88	m
M (meta centre)	6,73	m

Lower head stability		
Half of the volume	540	m^3
Cross-section above the floor	86	m^3
Floor volume (1,2m)	546	m^3
I (of the smallest cross section)	13762	m^4
V (water displacement)	2710	m^3
B (Centre of pressure)	2,00	m
G (Centre of gravity)	1,20	m
M (meta centre)	7,08	m

Seepage cut-off under the heads

A seepage cut-off screen is needed with a length of 11,8/2 =5,9 m under the upstream head and the downstream head to prevent piping. This is calculated with the method of Lane:

$$\Delta H_{crit} = (L_v + (1/3) * L_h) / C_L = (x + (1/3) * 15{,}6) / 5 = 3{,}4m => L_v = 11{,}8$$

ΔH_{crit} = Max water level difference >11,1-7,7=3,4 m

C_L = 5 (coarse sand)

L_v = vertical length = x=> 11,8 => 5,9 m screen height.

L_h = horizontal length =15,6 m

Seepage cut-off around the building pit

Two lengths have to be determined for the building pit of the small heads and for the building pit of the large heads. No demonstrable horizontal length is available. So only the vertical length is determined. This will be the pit depth + needed screen depth. The water level is taken 0,4 m deeper than the building pit bottom, because the floor must be stable.

$$\Delta H_{crit} = (L_v + (1/3) * L_h) / C_L = (L_v + (1/3) * 0) / 5 = L_v / 5$$

C_L = 5 (coarse sand)

L_h = horizontal length = 0 m

Small heads

Pit depth = 5,6 m

ΔH_{crit} = Max water level difference = 6,0 m

L_v = vertical length = 30 => 15 m

Screen height = 15+6 = 21 m

Large heads

Pit depth = 6,1

ΔH_{crit} = Max water level difference = 6,5 m

L_v = vertical length = 32,5 => 16,5 m

Screen height = 16,5+6,5 = 23 m

Side caissons

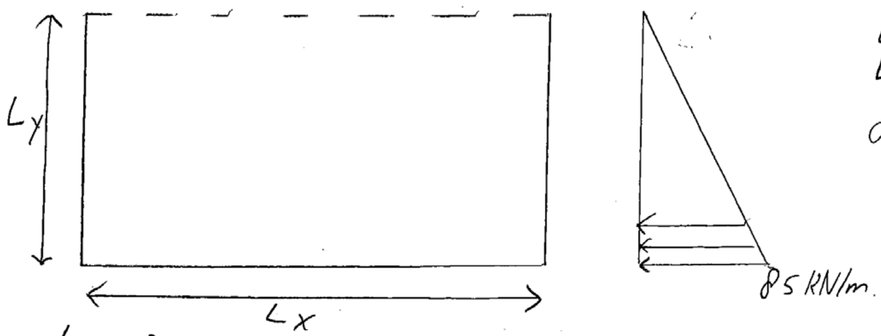
The moments that act on the outside walls and floors of the side caissons are controlled on the maximum reinforcement requirements. This is check is performed for the small and the large lock heads in respectively Figure VII 7 and Figure VII 8.

Side Caisson control Small Lock heads

[Wippel, 1983]

Platte nr. III/1/b.

Largest side Wall



$L_y = 8{,}5\ m.$
$L_x = 10{,}35\ m$
$d = 0{,}4\ m.$

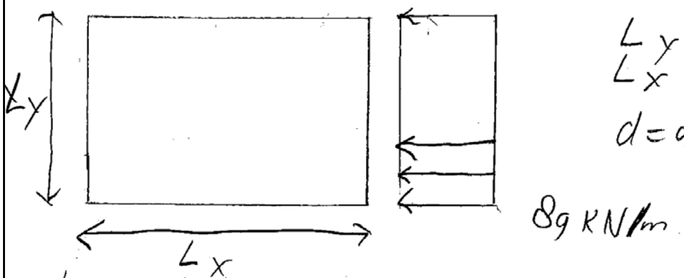
$\frac{L_y}{L_x} = \frac{8{,}5}{10{,}35} = 0{,}82 \Rightarrow \quad m_y = -13{,}5 \quad m_x = -16{,}7.$

$K = \frac{\gamma_w \cdot q \cdot L_x \cdot L_y}{2} = \frac{1{,}25 \cdot 85 \cdot 8{,}5 \cdot 10{,}35}{2} = 4673{,}67\ KN$

$M_{y,max} = \frac{K}{m_y} = 346{,}2\ KNm. \Rightarrow 2285\ mm^2 \Rightarrow Ø20-135mm.\ (d=0{,}4)$

$M_{x,max} = \frac{K}{m_x} = 279{,}8\ KNm. \Rightarrow \underbrace{1848\ mm^2 \Rightarrow Ø20-170mm}_{oke\ \rho_0\ 1\%}\ (d=0{,}4)$

Largest floor slab. (Platte nr. IV/6/a)



$L_y = 6{,}9\ m$
$L_x = 10{,}35\ m.$
$d = 0{,}4\ m.$

$\frac{L_y}{L_x} = \frac{6{,}9}{10{,}35} = 0{,}67 \quad m_y = -25{,}0 \quad m_x = -18{,}9$

$k = \gamma_w \cdot q \cdot L_x \cdot L_y = 1{,}25 \cdot 89 \cdot 6{,}9 \cdot 10{,}35 = 7995\ KN.$

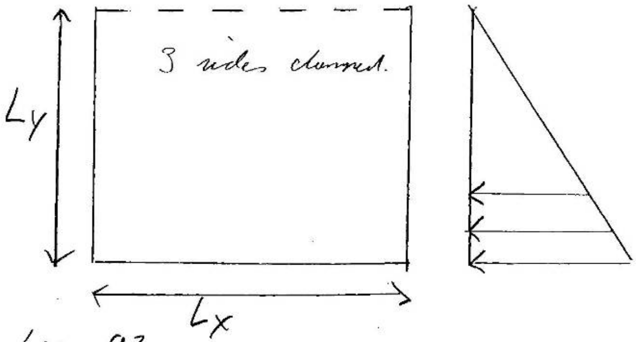
$M_{y\,max} = \frac{K}{m_y} = 317{,}8\ KNm \Rightarrow 2099\ mm^2 \Rightarrow Ø20-150\ mm\ (d=0{,}4)$

$M_{x\,max} = \frac{K}{m_x} = 420{,}4\ KNm \Rightarrow 2777\ mm^2 \Rightarrow \underbrace{Ø20-110\ mm}_{oke\ \rho_0\ 1{,}2\%}\ (d=0{,}4)$

Figure VII 7: Side caisson control, small lock heads [Wippel, 1983]

Side Caisson control   Large lock heads

Largest side Wall (Platte nr. III/1/6)



$L_y = 9{,}2\,m.$

$L_x = 9{,}2\,m.$

$d = 0{,}4\,m.$

$N = \frac{d^3}{12} \cdot E = \frac{0{,}4^3}{12} \cdot 31 \cdot 10^6 = 16{,}5 \cdot 10^4\,kN/m^2$

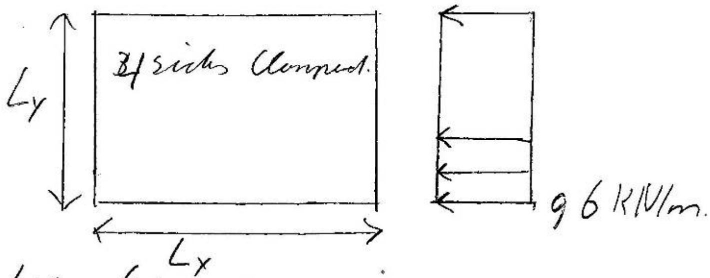
$\frac{L_y}{L_x} = \frac{9{,}2}{9{,}2} = 1 \Rightarrow m_y = 14{,}2 \quad m_x = 16{,}6 \quad K_w = 850$

$K = \frac{\gamma_w \cdot q \cdot L_x \cdot L_y}{2} = \frac{1{,}25 \cdot 92 \cdot 9{,}2 \cdot 9{,}2}{2} = 4867\,kN \;/\; w = \frac{K \cdot L_x^2}{K_w \cdot N} = 2{,}9\,mm$ oke

$M_{y\,max} = \frac{K}{m_y} = 343\,kNm \Rightarrow 2266\,mm^2 \Rightarrow \varnothing 20-135\,mm \; (0{,}4d)$

$M_{x\,max} = \frac{K}{m_x} = 293\,kNm \Rightarrow \underbrace{1936\,mm^2 \Rightarrow \varnothing 20-160\,mm}_{\text{oke } 1{,}1\%} \; (0{,}4d)$

Largest floor slab (Platte nr. IV/6/a)



$L_y = 6{,}9.$

$L_x = 9{,}2.$

$\frac{L_y}{L_x} = \frac{6{,}9}{9{,}2} = 0{,}75 \Rightarrow m_y = -18{,}9 \quad m_x = -23{,}4$

$K = \gamma_w \cdot q \cdot L_x \cdot L_y = 1{,}25 \cdot 96 \cdot 6{,}9 \cdot 9{,}2 = 7618$

$M_{y\,max} = \frac{K}{m_y} = 403\,kNm \Rightarrow 2649\,mm^2 \Rightarrow \varnothing 20-115 \; (0{,}4d)$

$M_{x\,max} = \frac{K}{m_x} = 325\,kNm \Rightarrow \underbrace{2147\,mm^2 \Rightarrow \varnothing 20-145}_{\text{oke } 1{,}2\%} \; (0{,}4d).$

$w = \frac{1}{384} \cdot \frac{q \cdot L_x^4}{\frac{1}{12} b \cdot d^3 \cdot E} = \frac{1}{384} \cdot \frac{96 \cdot 9{,}2^4}{\frac{1}{12} \cdot 1 \cdot 0{,}4^3 \cdot 31 \cdot 10^6} = 0{,}0108\,m \Rightarrow 10{,}8\,mm$ oke.

Figure VII 8: Side caisson control, large lock heads (2052) [Wippel, 1983]

Lock chamber

In Table VII 11 the initial volumes and dimensions for the flexible lock chamber are determined. The additional volumes and dimensions in case of an extension of the flexible chamber are presented in Table VII 12. For the construction of the alternative 2 lock chamber (Appendix VI) the Table VII 11 properties are used.

Table VII 11: Initial flexible lock chamber dimensions and quantities (2020)

Bottom protection		
layers	3	
Top level	3,2	m NAP
Bottom level	2,2	m NAP
Thickness	1	m
Length	225	m
Width	13,6	m
Volume filter	3060	m^3

Excavation		
Top level	12,8	m NAP
Water level	7,7	m NAP
Bottom dredge level	2,2	m NAP
Excavation	32436	m^3

Grout anchors (Van Leeuwen)		
type	850 Ø101,6/M107/Ø300 mm	
Length	25	m
spreading	1,5	m
Total number	300	

Prefab concrete slabs		
type	B35	
thickness	0,55	m
Height (m)	5,25	m
Slab (m)	1,50	m
Length (m)	450,00	m

Sheet piles		
type	AZ 50	
Steel type	S235	
Top level	12,80	m NAP
Bottom level	-7,20	m NAP
Height	20,00	m
Wet Length	450,00	m
Thickness	0,483	m
Sheet pile wet surface	9000	m^2
cross section	0,0322	m^2/m
Steal volume	290	m^3
Steal weight	2275	ton

Table VII 12: Additional flexible lock chamber dimensions and quantities (2052)

Bottom protection		
layers	3	
Top level	3,2	m NAP
Bottom level	2,2	m NAP
Thickness	1	m
Length	225	m
Width	7,3	m
Volume extra filter	1643	m^3

Removal of the sheet piles		
type	AZ 50	
Steel type	S235	
Height	20	m
Length	225	m
Sheetpile surface wet	4500	m^2
Steal Volume	145	m^3
Steal Weight	1137	ton

Grout anchors (Van Leeuwen)		
Type	850 Ø101,6/M107/Ø300 mm	
Length	25	m
spreading	1,5	m
Total number	150	

Excavation		
Top level	12,8	m NAP
Water level	7,7	m NAP
Bottom dredge level	2,2	m NAP
Excavation	17411	m^3

Prefab concrete slabs		
Type	B35	
thickness	0,55	m
Height (m)	5,25	m
Slab (m)	1,50	m
Length (m)	225,00	m

Sheet piles					
type	AZ 50		Thickness	0,483	m
Steel type	S235		Sheet pile wet surface	4500	m^2
Top level	12,80	m NAP	cross section	0,0322	m^2/m
Bottom level	-7,20	m NAP	Steal volume	145	m^3
Height	20,00	m	Steal weight	1137	ton
Wet Length	225,00	m			

In this calculation the cofferdam is also calculated, but finally the walls of the lock chamber consists of two single sheet pile walls as can be seen in Table VII 11 and Table VII 12.

The bottom protection must be able to resist the basin filling flows, the propeller jet flows and the initial water pressure differences that can occur during a locking cycle. In this comparison is assumed that the filter layer is 1 m thick. The bottom of the lock chamber is 20 cm lower than the top of the floor of the heads, because, the gravel must stay in the chamber. For the soil properties the gravel from Appendix III is assumed as representative.
One side of the sheet pile wall is anchored with grout anchorages and the other side of the small lock is made of a cofferdam. When the lock is enlarged in the future, the second sheet pile wall, of the cofferdam, also gets grout anchors. This construction can act as a cofferdam because the relation between the retaining height and the width of the cofferdam is within the norm of CUR 166: $0,7H < B < 1,5\ H$ ($H= 9,6$ and $B = 6,75$) => $6,72< 6,75 < 14,4$.
The deflections are within the norms of the CUR 166 (1/100 construction phase and 1/200 for the using phase). These deflections are tested with a high and a low water level in the lock and a high and a low ground water table. That is why a heavy sheet pile wall is used (AZ 50). The checks can be seen in Figure VII 9, Figure VII 10 and Figure VII 11. The anchorages are prestressed in the construction phase with a force of 550 kN/m. The maximum force will occur in stage 2 and gives a maximal force of 783 kN/m. The anchors are place under an angle of 30º this results in a force of 783/cos(30)= 904 kN/m. The anchors are place every 1,5 m, so this means that the maximum force this anchor has to handle is 1356kN. This will result in a grout anchor of 850 Ø101.6/M107/Ø300 mm as can be seen in Table VII 11. The cofferdam anchorage will have to resist a force of 738*1,5=1107 kN, so every 1,5 m an anchorage of 800 Ø70/M74/Ø200mm is placed [Gebr van Leeuwen Harmelen bv, 2007].

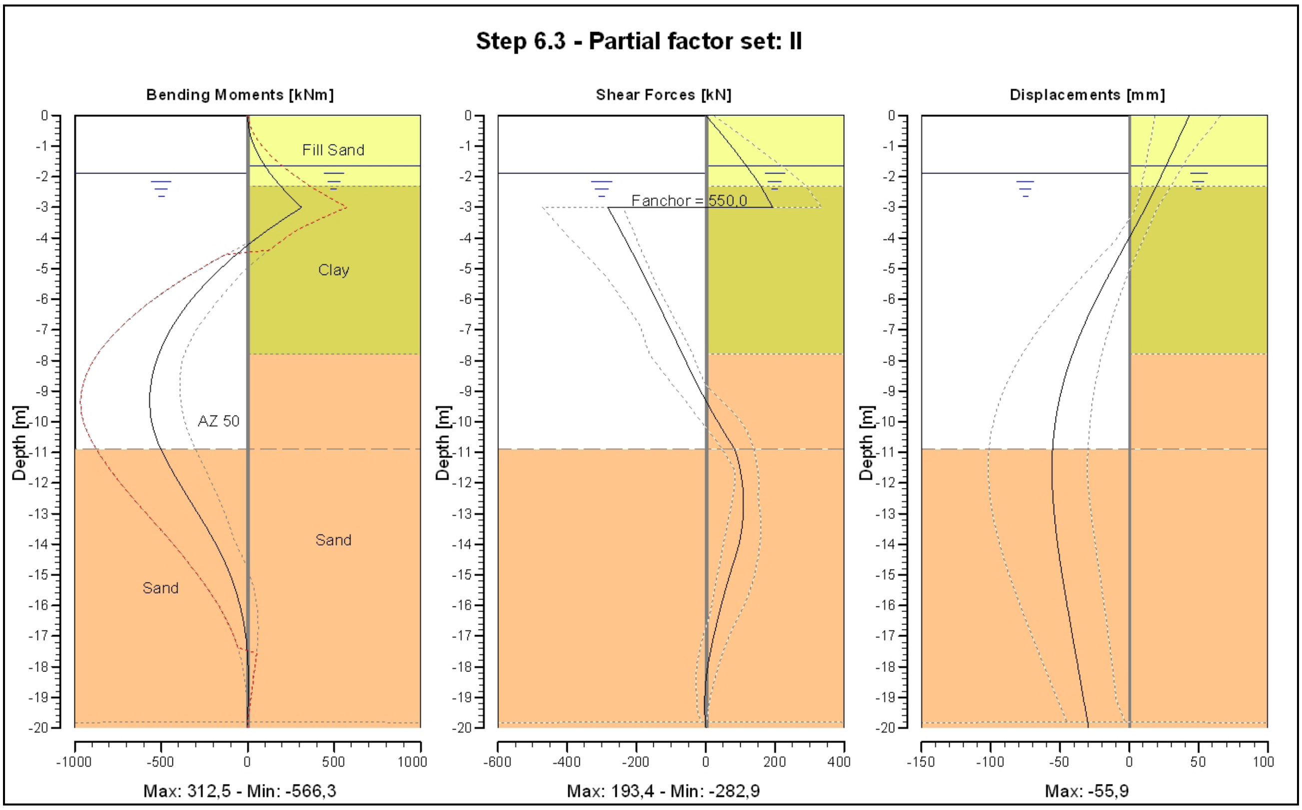


Figure VII 9: Moments/forces/displacements, construction phase, alternative 3

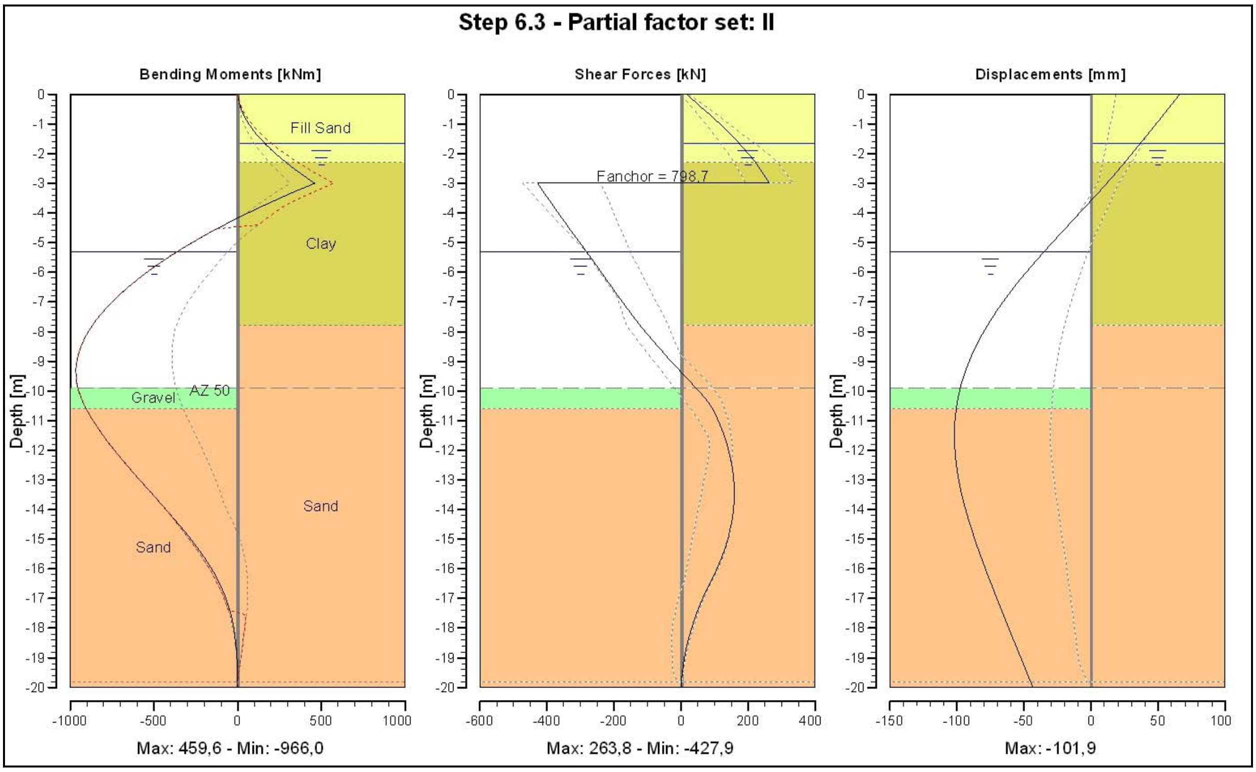


Figure VII 10: Moments/forces/displacements, lock chamber min water level, users phase, alternative 3

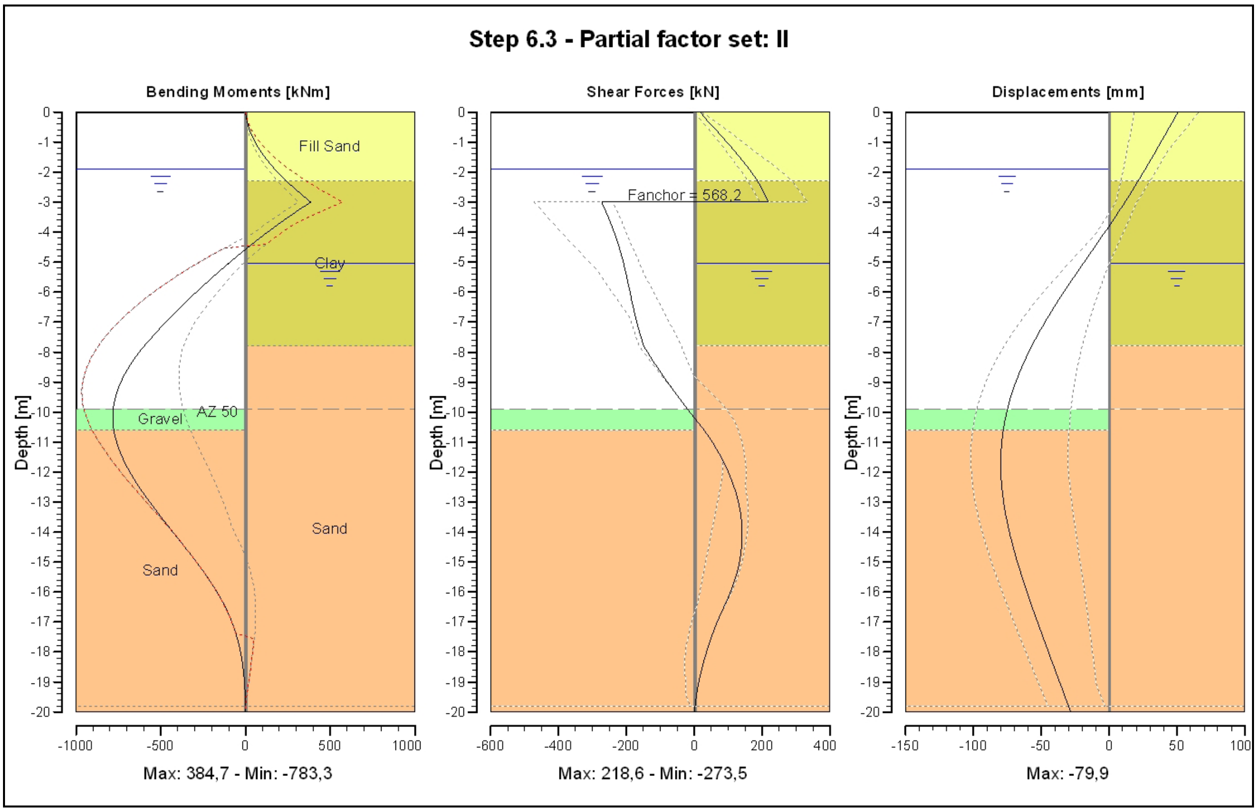


Figure VII 11: Moments/forces/displacements, lock chamber max water level, users phase, alternative 3

Appendix VIII.Cost variables

Table VIII 1: Cost variables [Hogendonk, 2009]

Price level	**2005**	2009	
Waiting time	€164	€ 175	hour
Demolishing			
Demolishing concrete structure (above and underwater)		€ 150	m^3
Removal concrete blocks		€ 15	m^2
Removal of the filter layer		€ 10	m^2
Sheet piling			
Sheet piling (including delivery)		€ 1.300	ton
Value after removal		-€ 550	ton
Apply/remove sheet piles		€ 80	m^2
Apply/remove dry sheet pile wall		€ 25	m^2
Anchorage			
GEWI temporarily (concrete anchorage)		€ 1.200	a piece
GEWI permanent (concrete anchorage)		€ 3.000	a piece
Grout anchorage (25 m), including waler		€ 8.000	a piece
Cofferdam anchorage (8m)		€ 10.000	a piece
Excavation			
Excavation		€ 8	m^3
Deep excavation		€ 10	m^3
Sand filling			
Fill up after construction (including compaction)		€ 15	m^3
Fluid concrete filling (alternative 1)		€ 75	m^3
Caisson filling (alternative 3)		€ 3	m^3
Caisson emptying (alternative 3)		€ 5	m^3
Filter layer			
Filter layer		€ 50	m^3
Concrete			
Floor lock head (alternative 1 and 2)		€ 260	m^3
Walls lock head (alternative 1 and 2)		€ 450	m^3
Thin walls (alternative 3)		€ 400	m^3
Underwater concrete		€ 120	m^3
Small concrete works (alternative 1)		€ 400	m^3
Prefab slabs (alternative 2 and 3)		€ 2.600	m
Gates			
Gate construction steel (including coating)		€ 10.500	ton

Water tight connection between the heads and the chamber (alternative 3)		
D profile	€ 500	m
The steel and concrete frame on the lock chamber	€ 500	m
Building pit prefab heads (alternative 3)		
Excavation	€ 3	m^3
Filling up	€ 2	m^3
Seepage screen AZ 12 (hired)	€ 65	m^2
Gravel layer	€ 30	m^3
Working floor	€ 10	m^2
Dewatering of the building pit	€ 3.000	week
Opening the building pit	€ 350.000	for each time
Transport and the immersion of the lock heads		
Partition wall	€ 1.000	m^2
bollard	€ 5.000	piece
Protection	€ 10.000	piece
Ballast tank	€ 40.000	piece
Transport and uplift	€ 75.000	piece
Immersion	€ 250.000	piece
Immersion gravel bed	€ 50	m^3
Additional engineering and surveys	€ 65.000	whole project

Appendix IX. Initial costs and lifetime costs

Alternative 1

Table IX 1: Initial costs of the renovated old lock (2020), alternative 1

Demolishing	Volume (m^3)	Unit cost (€/m^3)	Costs
Concrete structure	6912	€ 150	€ 1.036.770
	Surface (m^2)	Unit cost (€/m^2)	
Removal concrete blocks	3680	€ 15	€ 55.200
Sheet piles (including seepage cut off)	Tonnage	Unit cost (€/ton)	
Sheet piling (including delivery)	565	€ 1.300	€ 734.455
Value after removal	544	-€ 550	-€ 299.470
	Surface (m^2)	Unit cost (€/m^2)	
Applying and removing sheet piling	6629	€ 80	€ 530.352
Anchorages	Pieces	Unit cost (€/unit)	
Temporarily GEWI anchorage (Ø 25T) (DSI)	110	€ 1.200	€ 131.712
Wall anchorage	52	€ 8.000	€ 418.133
Excavation	Volume (m^3)	Unit cost (€/m^3)	
Excavation	9057	€ 8	€ 72.456
Deep dredging	2496	€ 10	€ 24.960
Concrete filling of the lock chamber wall	Volume (m^3)	Unit cost (€/m^3)	
Fill up after construction	1282	€ 15	€ 19.228
Fluid concrete filling	3510	€ 75	€ 263.250
Concrete	Volume (m^3)	Unit cost (€/m^3)	
Concrete Floor	1775	€ 260	€ 461.469
Concrete Walls	1299	€ 450	€ 584.431
Underwater concrete	1470	€ 120	€ 176.400
Small Concrete works	437	€ 400	€ 174.720
Gates	Weight (ton)		
Construction steel (gate)	473	€ 10.500	€ 4.964.886
Filter layers	Volume (m^3)	Unit cost (€/m^3)	
Filter layer (1m)	4160	€ 50	€ 208.000
Total initial construction costs			€ 9.556.952

Table IX 2: Initial costs of the new large lock (2052), alternative 1

Demolishing	Volume (m^3)	Unit cost (€/m^3)	Costs
Concrete structure Filter layer	20019 4160	€ 150 € 10	€ 3.002.912 € 41.600
Sheet piles (including seepage cut off)	Tonnage	Unit cost (€/ton)	
Sheet piling (including delivery) Value after removal	2961 656	€ 1.300 -€ 550	€ 3.849.522 -€ 360.573
	Surface (m^2)	Unit cost (€/m^2)	
Applying and removing sheet piling	16733	€ 80	€ 1.338.624
Anchorages	Pieces	Unit cost (€/unit)	
Temporarily GEWI anchorage (Ø 25T) (DSI) GEWI anchorage (Ø 25T) (DSI) Wall anchorage 800 Ø70/M74/Ø200 mm	202 0 361	€ 1.200 € 3.000 € 8.000	€ 241.824 € 0 € 2.888.533
Excavation	Volume (m^3)	Unit cost (€/m^3)	
Excavation Deep excavation	64839 0	€ 8 € 10	€ 518.710 € 0
Concrete filling of the lock chamber wall	Volume (m^3)	Unit cost (€/m^3)	
Fill up after construction	1656	€ 15	€ 24.835
Concrete	Volume (m^3)	Unit cost (€/m^3)	
Concrete Floor Concrete Walls Underwater concrete	3170 2277 2116	€ 260 € 450 € 120	€ 824.113 € 1.024.675 € 253.915
	Length (m)	Unit cost (€/m)	
Prefab slabs	450	€ 2.600	€ 1.170.000
Gates	Weight (ton)		
Construction steel gate	714	€ 10.500	€ 7.500.016
Filter layers	Volume (m^3)	Unit cost (€/m^3)	
Filter layer (1m)	4703	€ 50	€ 235.125
Total initial construction costs			€ 22.553.833

Table IX 3: Total initial cost, alternative 1

	Year	Costs
Renovated old lock New large lock	2020 2052	€ 9.556.952 € 22.553.833
Total Initial costs (positive)		€ 32.110.785
Total initial costs +10% (real option)		€ 35.321.863
Total initial costs +20% (negative)		€ 38.532.941

Table IX 4: Lifetime costs, alternative 1

	Annual time (hours)	**Annual costs**
Passing time (2020-2052) **Passing time (2052-2110)**	1728 2454	€ 303.107 € 430.456
	Obstruction time	**Total costs**
Obstruction time (2052)	156	€ 3.447.599

Alternative 2

Table IX 5: Initial costs of the new large lock (2020), alternative 2

Demolishing	Volume (m^3)	Unit cost (€/m^3)	Costs
Concrete structure	18441	€ 150	€ 2.766.120
	Surface (m^2)	Unit cost (€/m^2)	
Removal concrete blocks	3680	€ 15	€ 55.200
Sheet piles (+ seepage cut off)	Tonnage	Unit cost (€/ton)	
Sheet piling (including delivery)	2961	€ 1.300	€ 3.849.522
Value after removal	656	-€ 550	-€ 360.573
	Surface (m^2)	Unit cost (€/m^2)	
Applying and removing sheet piling	17043	€ 80	€ 1.363.418
Anchorages	Pieces	Unit cost (€/unit)	
Temporarily GEWI anchorage (Ø 25T)	202	€ 1.200	€ 241.824
GEWI anchorage (Ø 25T) (DSI)	0	€ 3.000	€ 0
Anchorage 800 Ø70/M74/Ø200 mm	361	€ 8.000	€ 2.888.533
Excavation	Volume (m^3)	Unit cost (€/m^3)	
Excavation	64839	€ 8	€ 518.710
Deep excavation	0	€ 0	€ 0
Sand filling	Volume (m^3)	Unit cost (€/m^3)	
Fill up after construction	1656	€ 15	€ 24.835
Concrete	Volume (m^3)	Unit cost (€/m^3)	
Concrete Floor	3170	€ 260	€ 824.113
Concrete Walls	2277	€ 450	€ 1.024.675
Underwater concrete	2116	€ 120	€ 253.915
	Length (m)	Unit cost (€/m)	
Prefab slabs	450	€ 2.600	€ 1.170.000
Gates	Weight (ton)		
Construction steel gate	714	€ 10.500	€ 7.500.016
Filter layers	Volume (m^3)	Unit cost (€/m^3)	
Filter layer (1m)	4703	€ 50	€ 235.125
Total initial construction costs	**Positive costs**		€ 22.355.434
	Real option initial costs +10%		€ 24.590.977
	Negative initial costs +20%		€ 26.826.521

Table IX 6: Lifetime costs, alternative 2

Lifetime costs		
	Annual time (hours)	**Annual costs**
Passing time (2020-2052)	1386	€ 243.127
Passing time (2052-2110)	2454	€ 430.456
	Obstruction time	**Total costs**
Obstruction time (2052)	No obstruction costs, due to reconstruction	€ 0

Alternative 3

Table IX 7: Small flexible lock initial costs (2020), alternative 3

Demolishing	Volume (m^3)	Unit cost (€/m^3)	costs
Concrete structure	18441	€ 150	€ 2.766.120
	Surface (m^2)	Unit cost (€/m^2)	
Removal concrete blocks	3680	€ 15	€ 55.200
Sheet piles (+ seepage cut off)	Tonnage	Unit cost (€/ton)	
Sheet piling (including delivery)	2302	€ 1.300	€ 2.992.306
Value after removal	0	-€ 550	€ 0
	Surface (m^2)	Unit cost (€/m^2)	
Applying and removing sheet piling	9271	€ 80	€ 741.712
Anchorages	Pieces	Unit cost (€/unit)	
Wall anchorage 800 Ø70/M74/Ø200 mm	300	€ 8.000	€ 2.400.000
Excavation	Volume (m^3)	Unit cost (€/m^3)	
Excavation	55189	€ 8	€ 441.514
Deep excavation	0	€ 10	€ 0
Sand filling of the caissons	Volume (m^3)	Unit cost (€/m^3)	
Sand filling of the caissons	4465	€ 3	€ 13.394
Concrete	Volume (m^3)	Unit cost (€/m^3)	
Concrete Floor	618	€ 260	€ 160.618
Concrete Walls	727	€ 450	€ 327.096
Concrete side caissons (thin walls)	1174	€ 400	€ 469.743
	Length (m)	Unit cost (€/m)	
Prefab slabs	450	€ 2.600	€ 1.170.000
Gates	Weight (ton)	Unit cost (€/ton)	
Construction steel (gate)	289	€ 10.500	€ 3.037.196
Filter layers	Volume (m^3)	Unit cost (€/m^3)	
Filter layer (1m)	3060	€ 50	€ 153.000
Water tight connection	Length (m)	Unit cost (€/m)	
Gina or D profile	77,4	€ 500,00	€ 38.700
The steel and concrete frame	77,4	€ 500,00	€ 38.700
Building pit	Volume (m^3)	Unit cost (€/m^3)	
Excavation	17236	€ 3	€ 51.708
Filling up	17236	€ 2	€ 34.472
Gravel layer	339	€ 30	€ 10.179
	Surface (m^2)	Unit cost (€/m^2)	
Working floor	1998	€ 10	€ 19.982
Seepage screen AZ 12 (hired)	4697	€ 65	€ 305.327
	weeks	Unit cost (€/week)	
Dewatering the building pit	26	€ 3.000	€ 78.000
Opening the building pit	€ 350.000		€ 350.000
Transportation and immersion	Surface (m^2)	Unit cost (€/m^2)	
Partition wall	310	€ 1.000	€ 310.000
	Volume (m^3)	Unit cost (€/m^3)	
Immersion gravel bed	679	€ 50	€ 33.930
	Pieces	Unit cost (€/piece)	
bollard	2	€ 5.000	€ 10.000
Protection	2	€ 10.000	€ 20.000
Ballast tank	2	€ 40.000	€ 80.000
Transport and uplift	2	€ 75.000	€ 150.000
Immersion	2	€ 250.000	€ 500.000
Extra engineering whole project	€ 65.000		€ 65.000
Total initial construction costs			€ 16.823.898

Table IX 8: Large flexible lock additional costs (2052), alternative 3

Demolishing	Volume (m^3)	Unit cost (€/m^3)	costs
Concrete structure	0	€ 150	€ 0
	Surface (m^2)	Unit cost (€/m^2)	
Removal concrete blocks	0	€ 15	€ 0
Sheet piles (+ seepage cut off)	Tonnage	Unit cost (€/ton)	
Sheet piling (including delivery)	1137	€ 1.300	€ 1.478.705
Value after removal	1137	-€ 550	-€ 625.606
	Surface (m^2)	Unit cost (€/m^2)	
Applying and removing sheet piling	9000	€ 80	€ 720.000
Anchorages	Pieces	Unit cost (€/unit)	
Wall anchorage 800 Ø70/M74/Ø200 mm	150	€ 8.000	€ 1.200.000
Excavation	Volume (m^3)	Unit cost (€/m^3)	
Excavation	55671	€ 8	€ 445.367
Deep excavation	0	€ 10	€ 0
Sand filling of the caissons	Volume (m^3)	Unit cost (€/m^3)	
Sand filling of the caissons	7633	€ 3	€ 22.899
Sand removal of the caissons	4465	€ 5	€ 22.324
Concrete	Volume (m^3)	Unit cost (€/m^3)	
Concrete Floor	1863	€ 260	€ 484.337
Concrete Walls	757	€ 450	€ 340.697
Concrete side caissons (thin walls)	1867	€ 400	€ 746.758
	Length (m)	Unit cost (€/m)	
Prefab slabs	225	€ 2.600	€ 585.000
Gates	Weight (ton)	Unit cost (€/ton)	
Construction steel (gate)	714	€ 10.500	€ 7.500.016
Filter layers	Volume (m^3)	Unit cost (€/m^3)	
Filter layer (1m)	1643	€ 50	€ 82.125
Water tight connection	Length (m)	Unit cost (€/m)	
Gina or D profile	8,9	€ 500,00	€ 4.450
The steel and concrete frame	8,9	€ 500,00	€ 4.450
Building pit	Volume (m^3)	Unit cost (€/m^3)	
Excavation	28864	€ 3	€ 86.593
Filling up	28864	€ 2	€ 57.728
Gravel layer	583	€ 30	€ 17.486
	Surface (m^2)	Unit cost (€/m^2)	
Working floor	3178	€ 10	€ 31.784
Seepage screen AZ 12 (hired)	6369	€ 65	€ 413.973
	weeks	Unit cost (€/week)	
Dewatering the building pit	26	€ 3.000	€ 78.000
Opening the building pit	€ 350.000		€ 350.000
Transportation and immersion	Surface (m^2)	Unit cost (€/m^2)	
Partition wall	491	€ 1.000	€ 491.040
	Volume (m^3)	Unit cost (€/m^3)	
Immersion gravel bed	1166	€ 50	€ 58.286
	Pieces	Unit cost (€/piece)	
bollard	2	€ 5.000	€ 10.000
Protection	2	€ 10.000	€ 20.000
Ballast tank	2	€ 40.000	€ 80.000
Transport and uplift	2	€ 75.000	€ 150.000
Immersion	2	€ 250.000	€ 500.000
Extra engineering whole project	€ 65.000		€ 65.000
Total initial construction costs			€ 15.421.410

Table IX 9: Total initial costs, alternative 3

	Year	Costs
Small flexible lock	2020	€ 16.823.898
Adjusted large flexible lock	2052	€ 15.421.410
Positive costs		€ 32.245.309
Real option costs +10%		€ 35.469.840
Negative costs +20%		€ 38.694.371

Table IX 10: Lifetime costs, alternative 3

	Annual time (hours)	**Annual costs**
Passing time (2020-2052)	1755	€ 307.955
Passing time (2052-2110)	2454	€ 430.456
	Obstruction time	**Total costs**
Obstruction time (2052)	16	€ 353.600

Appendix X. Whole Life Costing (WLC) calculation

Table X 1: WLC real option calculation of alternative 3

			Price level of the year		2009	
Real interest		1,90%	Adjustment of the initial costs			10%
Year	Annual initial costs	Cumulative initial costs	Annual life time costs		Cumulative life time costs	Cumulative life time and initial costs
			passing	obstruction		
2018	€ 6.168.763	€ 6.168.763			€ 0	€ 6.168.763
2019	€ 6.168.763	€ 12.454.732			€ 0	€ 12.454.732
2020	€ 6.168.763	€ 18.860.135	€ 307.955		€ 307.955	€ 19.168.090
2021		€ 19.218.477	€ 307.955		€ 615.911	€ 19.834.388
2022		€ 19.583.628	€ 307.955		€ 923.866	€ 20.507.494
2023		€ 19.955.717	€ 307.955		€ 1.231.821	€ 21.187.538
2024		€ 20.334.876	€ 307.955		€ 1.539.777	€ 21.874.652
2025		€ 20.721.238	€ 307.955		€ 1.847.732	€ 22.568.970
2026		€ 21.114.942	€ 307.955		€ 2.155.687	€ 23.270.629
2027		€ 21.516.126	€ 307.955		€ 2.463.642	€ 23.979.768
2028		€ 21.924.932	€ 307.955		€ 2.771.598	€ 24.696.530
2029		€ 22.341.506	€ 307.955		€ 3.079.553	€ 25.421.059
2030		€ 22.765.995	€ 307.955		€ 3.387.508	€ 26.153.503
2031		€ 23.198.548	€ 307.955		€ 3.695.464	€ 26.894.012
2032		€ 23.639.321	€ 307.955		€ 4.003.419	€ 27.642.740
2033		€ 24.088.468	€ 307.955		€ 4.311.374	€ 28.399.842
2034		€ 24.546.149	€ 307.955		€ 4.619.330	€ 29.165.478
2035		€ 25.012.526	€ 307.955		€ 4.927.285	€ 29.939.811
2036		€ 25.487.764	€ 307.955		€ 5.235.240	€ 30.723.004
2037		€ 25.972.031	€ 307.955		€ 5.543.195	€ 31.515.227
2038		€ 26.465.500	€ 307.955		€ 5.851.151	€ 32.316.651
2039		€ 26.968.344	€ 307.955		€ 6.159.106	€ 33.127.450
2040		€ 27.480.743	€ 307.955		€ 6.467.061	€ 33.947.804
2041		€ 28.002.877	€ 307.955		€ 6.775.017	€ 34.777.894
2042		€ 28.534.932	€ 307.955		€ 7.082.972	€ 35.617.904
2043		€ 29.077.095	€ 307.955		€ 7.390.927	€ 36.468.023
2044		€ 29.629.560	€ 307.955		€ 7.698.883	€ 37.328.443
2045		€ 30.192.522	€ 307.955		€ 8.006.838	€ 38.199.360
2046		€ 30.766.180	€ 307.955		€ 8.314.793	€ 39.080.973
2047		€ 31.350.737	€ 307.955		€ 8.622.749	€ 39.973.486
2048		€ 31.946.401	€ 307.955		€ 8.930.704	€ 40.877.105
2049		€ 32.553.383	€ 307.955		€ 9.238.659	€ 41.792.042
2050		€ 33.171.897	€ 307.955		€ 9.546.614	€ 42.718.511
2051	€ 8.481.776	€ 42.283.939	€ 307.955		€ 9.854.570	€ 52.138.509
2052	€ 8.481.776	€ 51.569.109	€ 430.456	€ 353.600	€ 10.638.625	€ 62.207.735
2053		€ 52.548.922	€ 430.456		€ 11.069.081	€ 63.618.003
2054		€ 53.547.352	€ 430.456		€ 11.499.536	€ 65.046.888
2055		€ 54.564.752	€ 430.456		€ 11.929.992	€ 66.494.744
2056		€ 55.601.482	€ 430.456		€ 12.360.447	€ 67.961.929
2057		€ 56.657.910	€ 430.456		€ 12.790.903	€ 69.448.813
2058		€ 57.734.410	€ 430.456		€ 13.221.359	€ 70.955.769
2059		€ 58.831.364	€ 430.456		€ 13.651.814	€ 72.483.178
2060		€ 59.949.160	€ 430.456		€ 14.082.270	€ 74.031.430
2061		€ 61.088.194	€ 430.456		€ 14.512.725	€ 75.600.919
2062		€ 62.248.870	€ 430.456		€ 14.943.181	€ 77.192.051

2063		€ 63.431.598	€ 430.456			€ 15.373.636	€ 78.805.235
2064		€ 64.636.799	€ 430.456			€ 15.804.092	€ 80.440.891
2065		€ 65.864.898	€ 430.456			€ 16.234.548	€ 82.099.445
2066		€ 67.116.331	€ 430.456			€ 16.665.003	€ 83.781.334
2067		€ 68.391.541	€ 430.456			€ 17.095.459	€ 85.487.000
2068		€ 69.690.981	€ 430.456			€ 17.525.914	€ 87.216.895
2069		€ 71.015.109	€ 430.456			€ 17.956.370	€ 88.971.479
2070		€ 72.364.396	€ 430.456			€ 18.386.825	€ 90.751.222
2071		€ 73.739.320	€ 430.456			€ 18.817.281	€ 92.556.601
2072		€ 75.140.367	€ 430.456			€ 19.247.737	€ 94.388.103
2073		€ 76.568.034	€ 430.456			€ 19.678.192	€ 96.246.226
2074		€ 78.022.826	€ 430.456			€ 20.108.648	€ 98.131.474
2075		€ 79.505.260	€ 430.456			€ 20.539.103	€ 100.044.363
2076		€ 81.015.860	€ 430.456			€ 20.969.559	€ 101.985.419
2077		€ 82.555.161	€ 430.456			€ 21.400.014	€ 103.955.176
2078		€ 84.123.710	€ 430.456			€ 21.830.470	€ 105.954.179
2079		€ 85.722.060	€ 430.456			€ 22.260.925	€ 107.982.985
2080		€ 87.350.779	€ 430.456			€ 22.691.381	€ 110.042.160
2081		€ 89.010.444	€ 430.456			€ 23.121.837	€ 112.132.281
2082		€ 90.701.642	€ 430.456			€ 23.552.292	€ 114.253.935
2083		€ 92.424.974	€ 430.456			€ 23.982.748	€ 116.407.721
2084		€ 94.181.048	€ 430.456			€ 24.413.203	€ 118.594.251
2085		€ 95.970.488	€ 430.456			€ 24.843.659	€ 120.814.147
2086		€ 97.793.927	€ 430.456			€ 25.274.114	€ 123.068.042
2087		€ 99.652.012	€ 430.456			€ 25.704.570	€ 125.356.582
2088		€ 101.545.400	€ 430.456			€ 26.135.026	€ 127.680.426
2089		€ 103.474.763	€ 430.456			€ 26.565.481	€ 130.040.244
2090		€ 105.440.783	€ 430.456			€ 26.995.937	€ 132.436.720
2091		€ 107.444.158	€ 430.456			€ 27.426.392	€ 134.870.550
2092		€ 109.485.597	€ 430.456			€ 27.856.848	€ 137.342.445
2093		€ 111.565.823	€ 430.456			€ 28.287.303	€ 139.853.127
2094		€ 113.685.574	€ 430.456			€ 28.717.759	€ 142.403.333
2095		€ 115.845.600	€ 430.456			€ 29.148.215	€ 144.993.815
2096		€ 118.046.666	€ 430.456			€ 29.578.670	€ 147.625.336
2097		€ 120.289.553	€ 430.456			€ 30.009.126	€ 150.298.679
2098		€ 122.575.055	€ 430.456			€ 30.439.581	€ 153.014.636
2099		€ 124.903.981	€ 430.456			€ 30.870.037	€ 155.774.017
2100		€ 127.277.156	€ 430.456			€ 31.300.492	€ 158.577.649
2101		€ 129.695.422	€ 430.456			€ 31.730.948	€ 161.426.370
2102		€ 132.159.635	€ 430.456			€ 32.161.403	€ 164.321.039
2103		€ 134.670.668	€ 430.456			€ 32.591.859	€ 167.262.527
2104		€ 137.229.411	€ 430.456			€ 33.022.315	€ 170.251.726
2105		€ 139.836.770	€ 430.456			€ 33.452.770	€ 173.289.540

Appendix XI. Structural checks

Determination of the representative loads

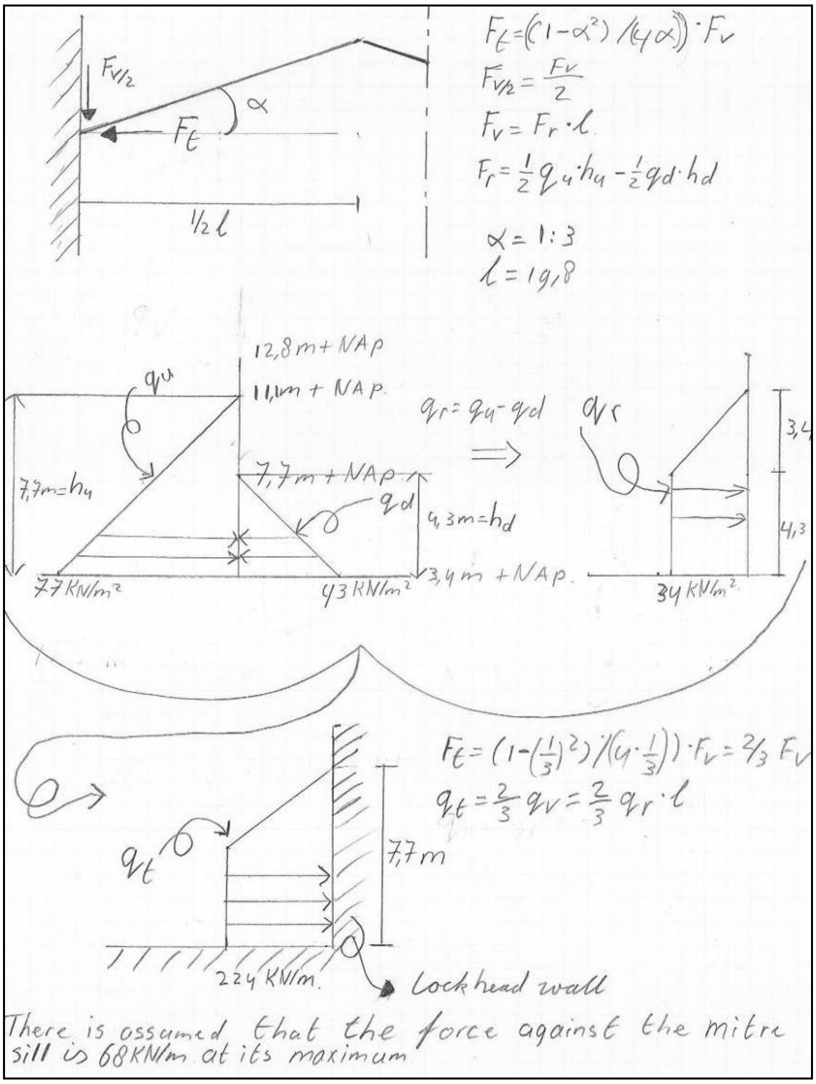


Figure XI 1: Distributed load on the wall through the gate [Glerum 2000]

The uplifted weight of the gate is determined in chapter 8 and applied in Figure XI 2 to determine the forces that act upon the lock wall.

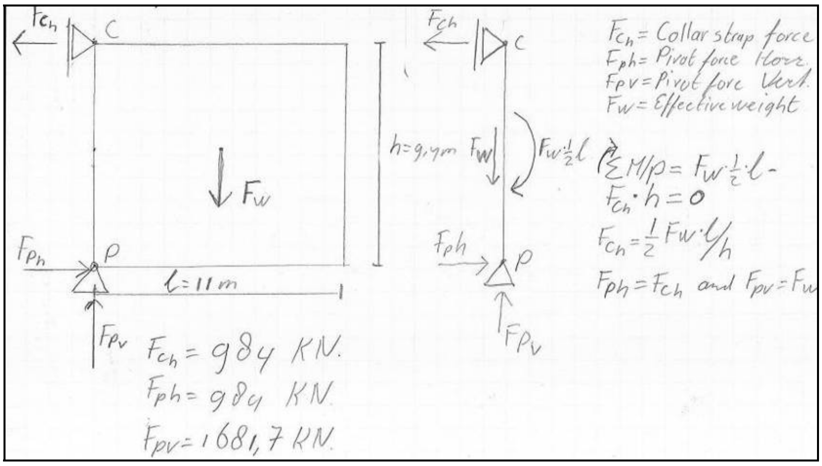


Figure XI 2: Maximum reaction force on the pivot and the collar strap

The resulting moments and forces for the next three cross-sections in this appendix are calculated with lower load factors for the variable loads than represented in the report (Table 12.1 and Table 12.2). The ground load, the soil pressure and the weight of the gates were multiplied by 1,5 in this appendix, but in a re-calculation these loads should be multiplied by 1,2 according to the report. Besides the load factors, also the concrete cover and the average bar diameter differ from the report. The cover and the average bar diameter are 30 mm and 16 mm in this appendix, but in a re-calculation these measures have to be changed in respectively 40 mm and 20 mm.

Cross section AA'

Load combinations

In Figure XI 3 the schematisation of cross-section AA' is shown. The loads in the different load combinations can be seen in Figure XI 4 till Figure XI 9. The loads are in the Serviceability Limit State (SLS).

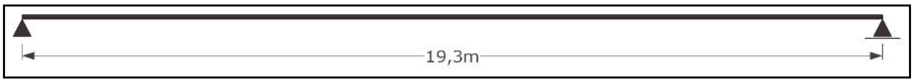


Figure XI 3: Schematisation of cross-section AA'

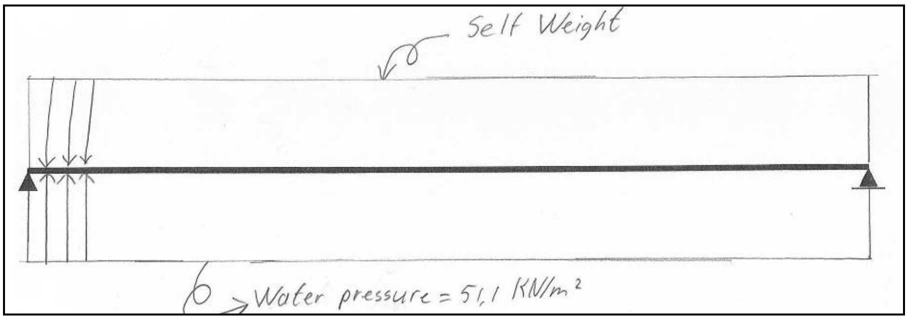


Figure XI 4: Cross-section AA', Load combination C.1a

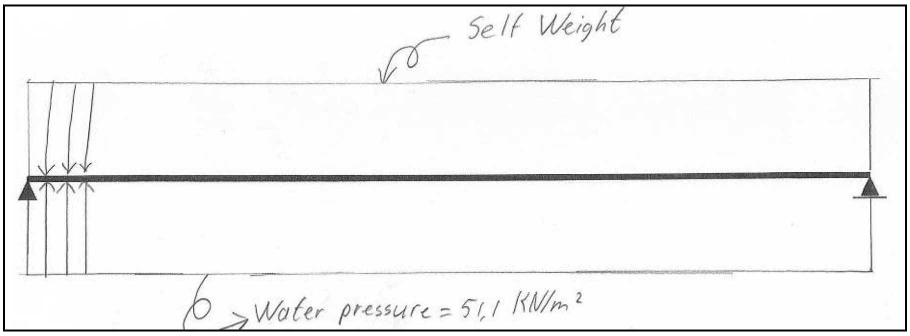


Figure XI 5: Cross-section AA', Load combination C.1c

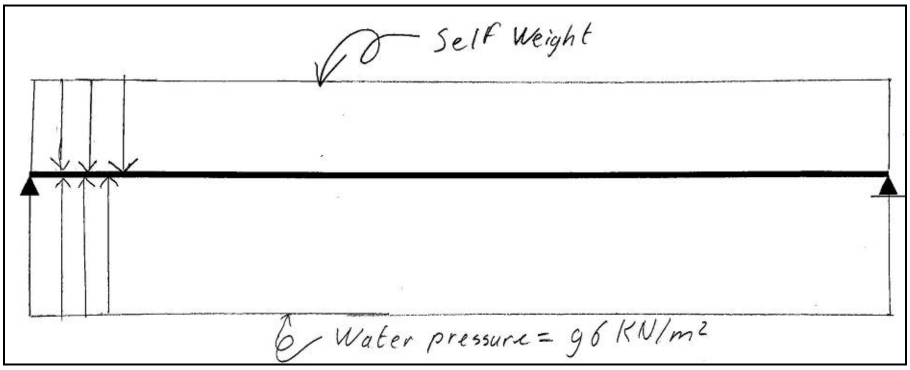


Figure XI 6: Cross-section AA', Load combination C.2

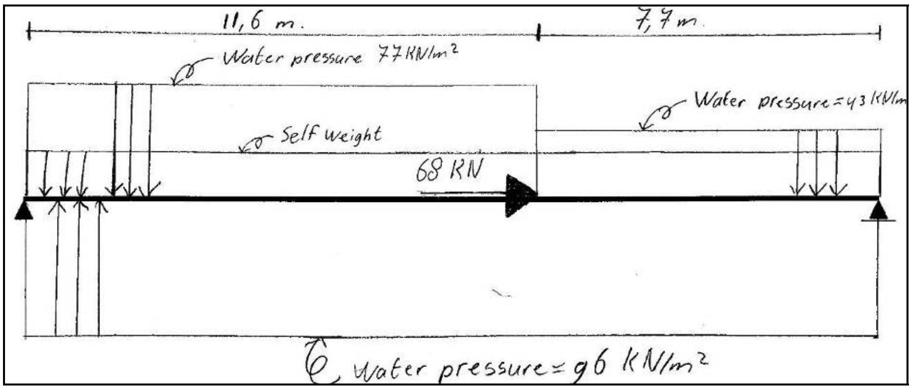


Figure XI 7: Cross-section AA', Load combination U.3

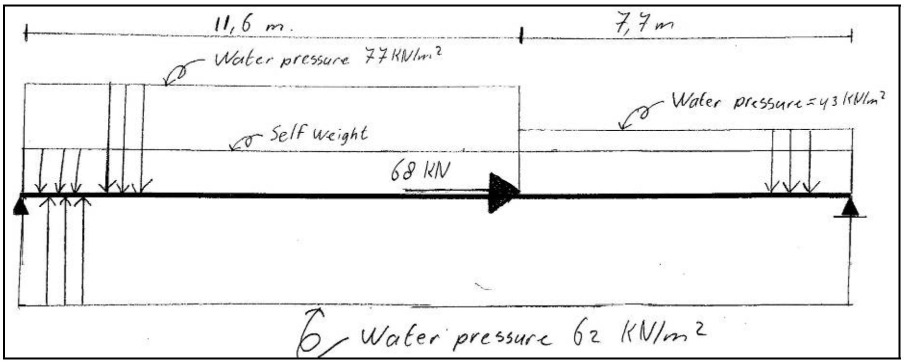


Figure XI 8: Cross-section AA', Load combination U.4

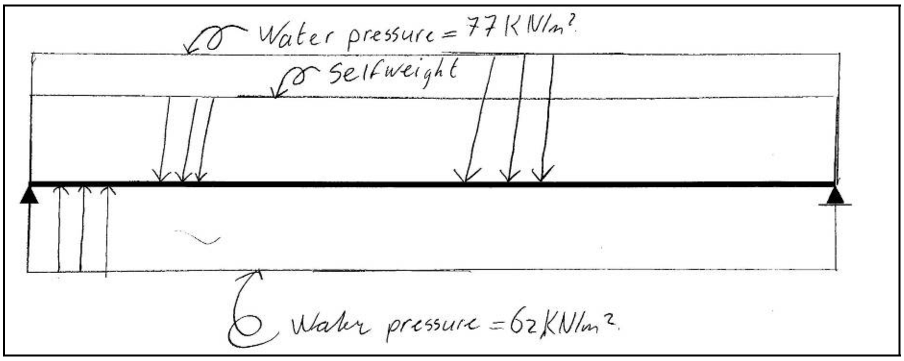


Figure XI 9: Cross-section AA', Load combination U.5

Resulting forces, moments and deformations

In Figure XI 10 till Figure XI 16 all the forces, moments and deformations in the Ultimate Limit State (ULS) and the SLS can be found.

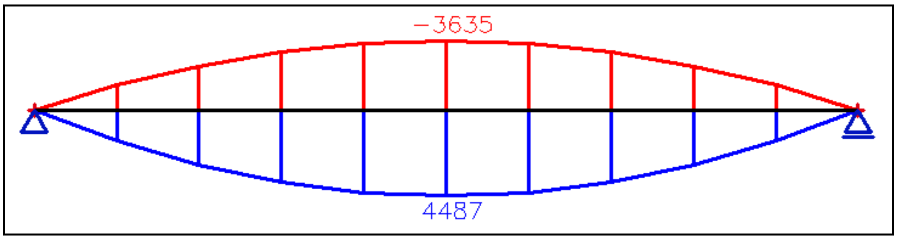


Figure XI 10: Cross-section AA', ULS moment envelope (kNm)

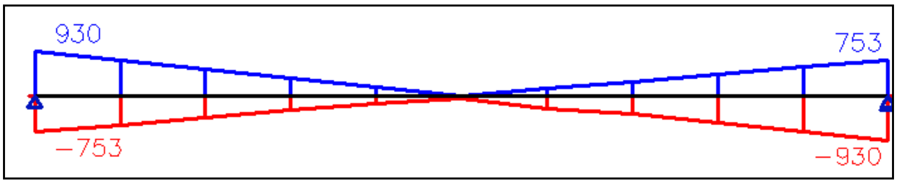


Figure XI 11: Cross-section AA', ULS shear force envelope (kN)

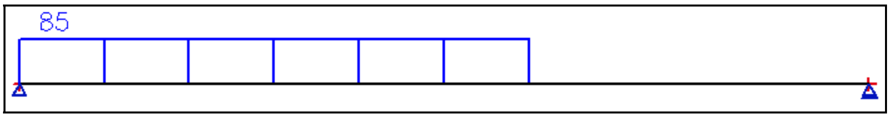


Figure XI 12: Cross-section AA', ULS normal force envelope (kN)

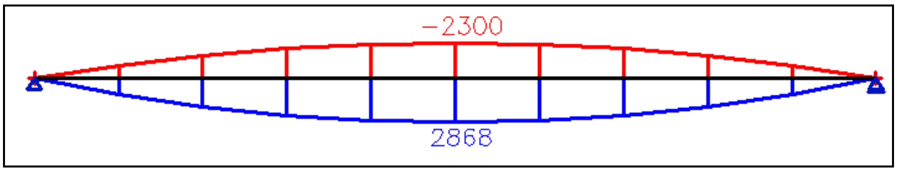


Figure XI 13: Cross-section AA', SLS moment envelope (kNm)

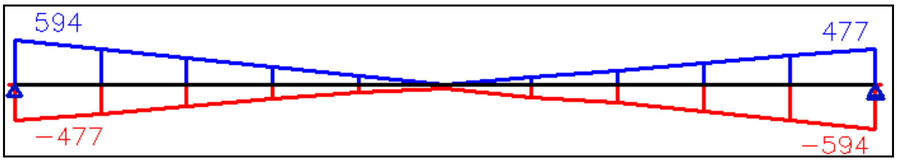


Figure XI 14: Cross-section AA', SLS shear force envelope (kN)

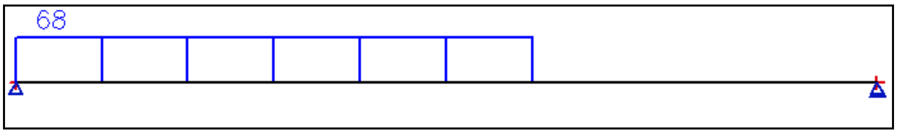


Figure XI 15: Cross-section AA', SLS normal force envelope (kN)

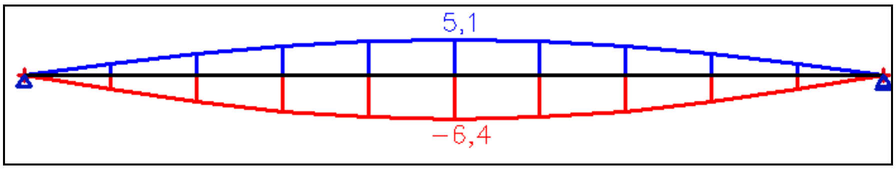


Figure XI 16: Cross-section AA', deformations envelope (mm)

Checks

In Table XI 1and Table XI 2 the different checks can be seen for respectively the top side and the bottom side of this cross section. The deformations that can be seen in Figure XI 16 are within the prescript maximum deformation of 30 mm.

Table XI 1: Cross-section AA' topside check.

Parameters						
Variable parameters				Locked parameters		
h	1,9	m		c	30	mm
Ø	32	mm		f_s	435	N/mm^2
s	140	mm		τ_1	0,56	N/mm^2
(ULS)				τ_2	4,2	N/mm^2
M_d	3635	kNm		b	1000	mm
N_d	85	kN		ξ	1	-
V_d	655	kN		k_1	3750	-
(SLS)				k_2	750	-
M_d	2300	kNm				
N_d	68	kN		Economic reinforcement values		
V_d	542	kN		0,50%	< ω >	0,75%
Effective height				*Bending and Normal force (ULS)*		
d	1854	mm		N_s (ULS)	2263,5	kN
				z	1,7	m
Shear force (ULS)				A_s Required	5203	mm^2
τ_d	0,35	N/mm^2		A_c	1900000	mm^2
No Stirrups Needed				A_s Selected	5745	mm^2
				ω	0,30%	

Cracking [NEN 6720] (SLS)			
N_s (SLS)	1446	kN	**The cross section is Sufficient**
σ_s (SLS)	252	N/mm^2	**but not Economic**
Condition 1	NOT OK		
Condition 2	OK		
Acceptable cracking			

Table XI 2: Cross-section AA' bottom side check

Parameters

Variable parameters			Locked parameters		
h	1,9	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	125	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	4487	kNm	b	1000	mm
N_d	85	kN	ξ	1	-
V_d	930	kN	k_1	3750	-
(SLS)			k_2	750	-
M_d	2300	kNm			
N_d	68	kN	Economic reinforcement values		
V_d	594	kN	0,50%	< ω >	0,75%

Effective height			*Bending and Normal force (ULS)*		
d	1854	mm	N_s (ULS)	2774,1	kN
			z	1,7	m
Shear force (ULS)			A_s Required	6377	mm^2
τ_d	0,50	N/mm^2	A_c	1900000	mm^2
No Stirrups Needed			A_s Selected	6434	mm^2
			ω	0,34%	

Cracking [NEN 6720] (SLS)			
N_s (SLS)	1446	kN	**The cross section is Sufficient**
σ_s (SLS)	225	N/mm^2	**but not Economic**
Condition 1	NOT OK		
Condition 2	OK		
Acceptable cracking			

In the tables can be seen that the required reinforcement percentages are under the economic reinforcement value. For this cross section a thinner floor should be a good optimisation.

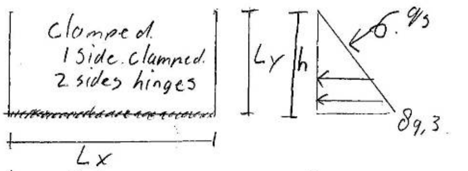
Cross- section BB'

Load combinations

Only the floor is modelled for this cross-section. The walls are checked by a hand calculation for the load combinations where this is needed. In Figure XI 17 and Figure XI 18 these calculation for respectively the gate chamber wall and the outer side caisson wall are shown.

In Figure XI 17 is assumed that the floor acts as a clamp and the supporting caisson walls as a hinge, because the floor is much thicker than the supporting walls. This is a conservative assumption. The calculated moments are applied in the load combinations.

Load Combination C.3 (Platte III/3/6) (d=1,2 m)



(S/S) $q_s = K_0 \cdot \gamma_{wet} \cdot h = 0,5 \cdot 19 \cdot 9,4 = 89,3$ KN/m

$L_y = h = 9,4$ m
$L_x = 6,8$ m

$N = \frac{d^3}{12} E_c = \frac{1,2^3}{12} \cdot 31 \cdot 10^6 = 4,46 \cdot 10^6$ KN/m²

$\frac{L_y}{L_x} = \frac{9,4}{6,8} = 1,4 \Rightarrow m_y = 9,8 \; m_x = 22,3 \; K_w = 235$

(S/S) $= K = \frac{q_s \cdot L_x \cdot L_y}{2} = \frac{89,3 \cdot 6,8 \cdot 9,4}{2} = 2854$

$M_y = \frac{2854}{9,8} = \boxed{291}$ KNm. $M_x = \frac{2854}{22,3} = 128$ KNm (SLS).

$w = \frac{K \cdot L_x^2}{k_w \cdot N} = \frac{2854 \cdot 6,8^2}{235 \cdot 4,46 \cdot 10^6} = 1,26 \cdot 10^{-4} m \Rightarrow w = 0,13$ mm

Load Combinatation U.1 (Platte III/3/6) (d=1,2m)

The condition are the same are the same as in C3 only there two q loads considered. $q_s = 89,3$ KN/m² (load C.3).

and $q_{gl} = K_0 \cdot q_g \cdot h = 0,5 \cdot 20 \cdot 9,4 = 94$ KN/m² (Groundload).

$q_s \Rightarrow M_y = \boxed{291}$ KNm $M_x = 128$ KNm (S/S)

$q_{gl} \Rightarrow K = \frac{94 \cdot 6,8 \cdot 9,4}{2} = 3004$

$M_y = \frac{3004}{9,8} = \boxed{307}$ KNm. $M_x = \frac{3004}{22,3} = 135$ KNm.

$w_{gl} = \frac{3004 \cdot 6,8^2}{235 \cdot 4,46 \cdot 10^6} = 1,33 \cdot 10^{-4} m \Rightarrow w_{gl} = 0,13$ mm

$w_{u1} = 0,13 + 0,13 = 0,26$ mm

$M_{y.u1} = 598$ KNm (S/S)

$M_{y.u1} = 809,7$ KNm (U/S)

Load Combination U.5 (Platte /3/6) (d=1,2m).

Same Soil en Ground load moment, but a resisting water pressure moment.

$q_w = 7,7 m \cdot 10 KN/m^2 = 77$ KN/m²

$L_y = 7,7 \; L_x = 6,8 \quad \frac{L_y}{L_x} = \frac{7,7}{6,8} = 1,1$

$m_y = 9,0 \quad m_x = 21,6. \quad K_w = 200.$

$(S/S) = K = \frac{77 \cdot 6,8 \cdot 7,7}{2} = 2016$

$M_y = -224$ KNm $M_x = -93.1$ KNm (S/S).

$w = \frac{2016 \cdot 6,8^2}{200 \cdot 4,46 \cdot 10^6} = -1,04 \cdot 10^{-4} \Rightarrow -0,10$ mm.

$W_{u5} = 0,16$ mm.

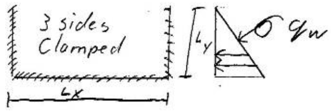
$M_{y u5} = 374$ KNm (S/S).

$M_{y u5} = 529,7$ KNm (U/S).

Figure XI 17: Cross-section BB': Gate chamber wall calculations [Wippel, 1983]

In Figure XI 18 the reaction moment of the outer caisson wall is calculated for the different load combinations. In this case is assumed that all three the connections are clamped, because the walls and the floor have both the same thickness.

Platte III/1/b $(d = 0{,}4)$



<u>Load Combination C.1a</u> (two sides). $q_w = 51{,}1\ kN/m^2$

$L_y = 5{,}11m\ L_x = 6{,}8m\ \ \frac{L_y}{L_x} = 0{,}8 \Rightarrow m_y = 13{,}0.\ m_x = 16{,}7$

$K = \frac{q_w \cdot L_y \cdot L_x}{2} = \frac{51{,}1 \cdot 5{,}11 \cdot 6{,}8}{2} = 887{,}8$

$M_y = \frac{887{,}8}{13{,}0} = 67\ kNm.\ \ M_x = 53\ kNm.$

<u>Load Combination C.1b.</u>

one side $M_y = 67\ kNm$ (C.1a)

one side $M_y = 170\ kNm$ (C.2)

<u>Load Combination C.2.</u> (two sides) $q_w = 96\ kN/m^2$

$L_y = 9{,}6m\ L_x = 6{,}8m\ \ \frac{L_y}{L_x} = 1{,}4\ \ m_y = 18{,}4\ \ m_x = 17{,}0.$

$K = \frac{96 \cdot 9{,}6 \cdot 6{,}8}{2} = 3133$

$M_y = \frac{3133}{18{,}4} = 170\ kNm.\ \ M_x = 184\ kNm.$

<u>Load Combination (C.3, U4 and U5)</u> (two sides) $q_w = 62\ kN/m^2$

$L_y = 6{,}2m\ \ L_x = 6{,}8m\ \ \frac{L_y}{L_x} = 0{,}9 \Rightarrow m_y = 13{,}5\ \ m_x = 16{,}5$

$K = \frac{62 \cdot 6{,}2 \cdot 6{,}8}{2} = 1307.$

$M_y = \frac{1307}{13{,}5} = 97\ kNm.\ \ M_x = \frac{1307}{16{,}5} = 80\ kN/m.$

Figure XI 18: Cross-section BB' Side caisson wall calculations [Wippel, 1983]

In Figure XI 19 a schematisation of cross section BB' is given. After that the loads in the different governing load combinations are shown in Figure XI 20 till Figure XI 25. All the represented loads are in the Serviceability Limit state (SLS). To calculate the Ultimate Limit State (ULS) these loads are multiplied by the in paragraph 12.2.1 given load factors.

The weight of the soil on the bottom of the side caissons is only considered in load combination U.5, because the side caissons are assumed to be supported in all the other load combinations, by the foundation.

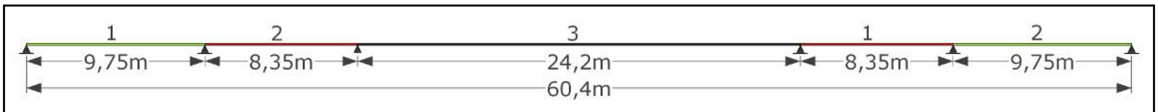


Figure XI 19: Schematisation of cross-section BB'

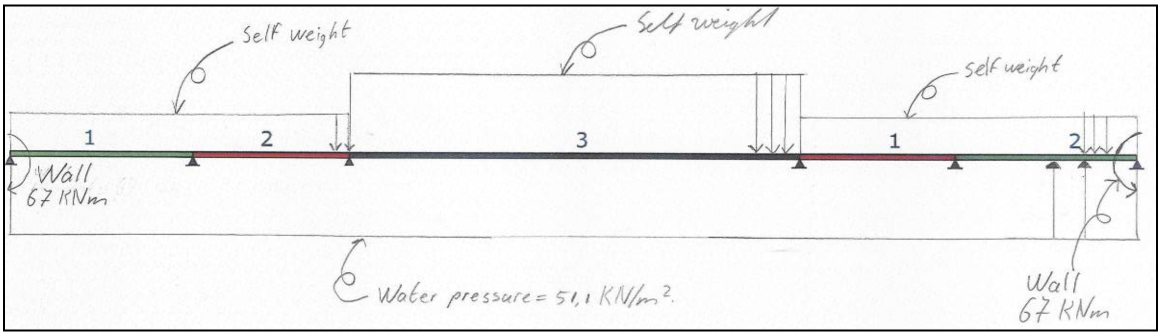


Figure XI 20: Cross-section BB', load combination C.1a

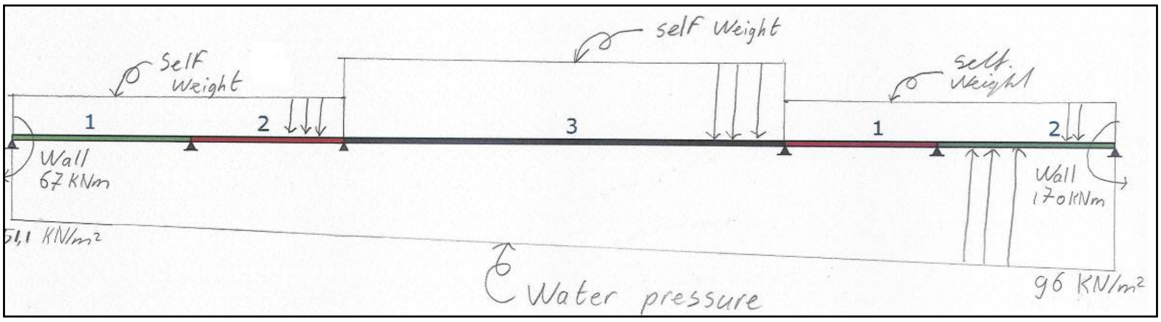


Figure XI 21: Cross-section BB', load combination C.1b

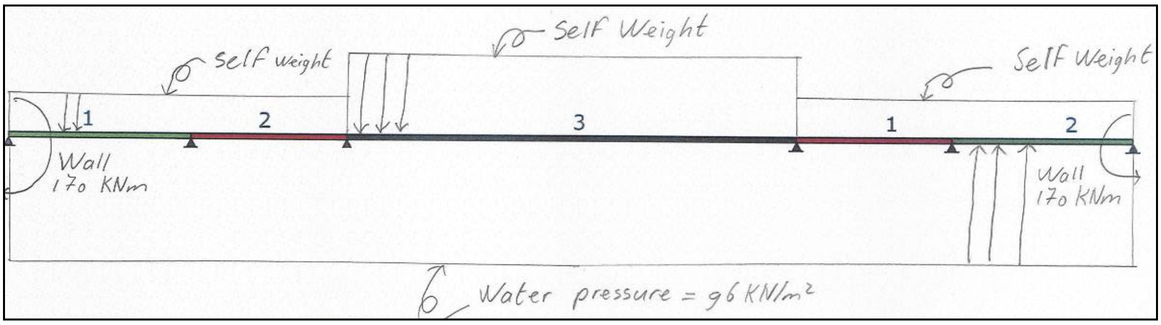


Figure XI 22: Cross-section BB', load combination C.2

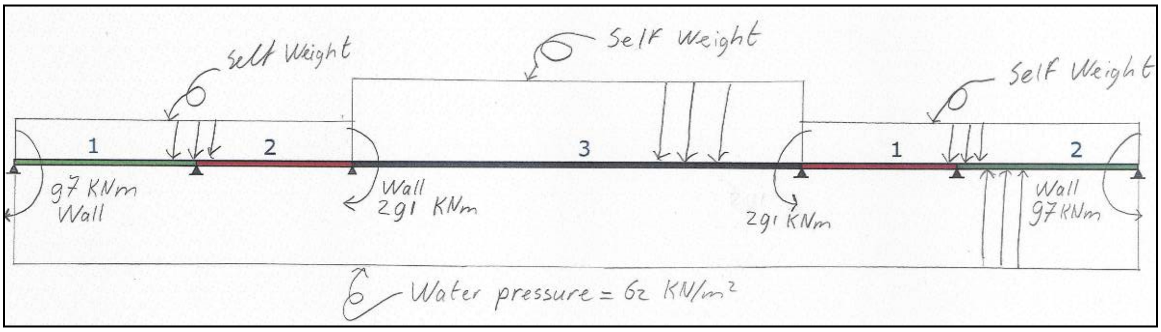


Figure XI 23: Cross-section BB', load combination C.3

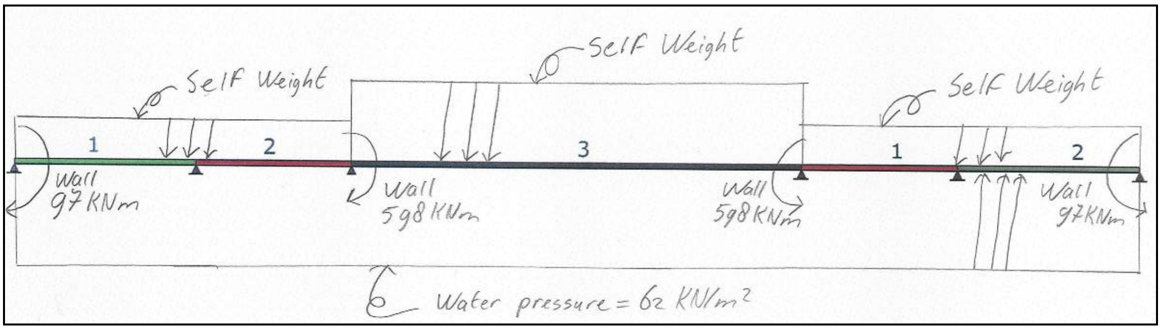


Figure XI 24: Cross-section BB', load combination U.1

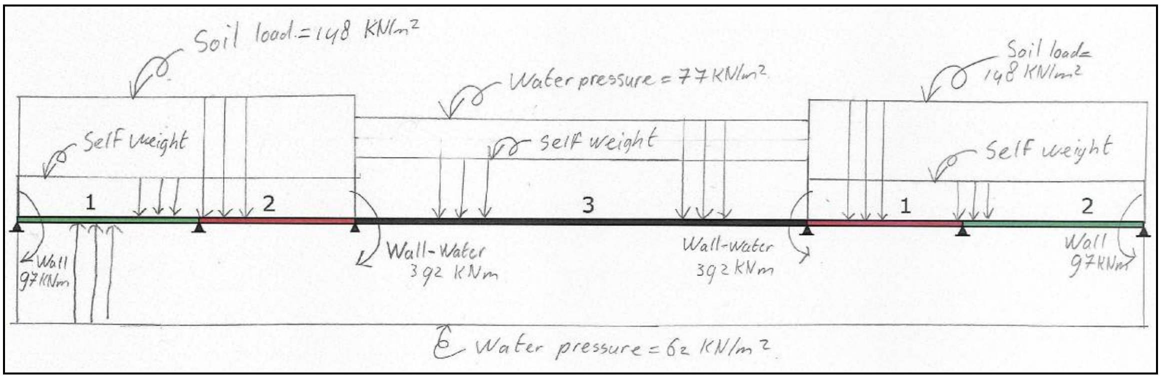


Figure XI 25: Cross-section BB', Load combination U.5

Resulting Moments and deformations

The resulting moments in the ULS and SLS and the deformations are represented in Figure XI 26 till Figure XI 30. The moments that occur in section 1 and 2 are not representative, because the side caissons will react as a plate that is supported on 4 sides and not as the two sides supported beam that is assumed in these calculations. In Appendix VII in Figure VII 8 the floors and the walls of the caissons are checked on the maximum moments and deformations, also the shear between the gate chamber and the side caissons is checked in this appendix. So only the gate chamber is of great importance (section 3).

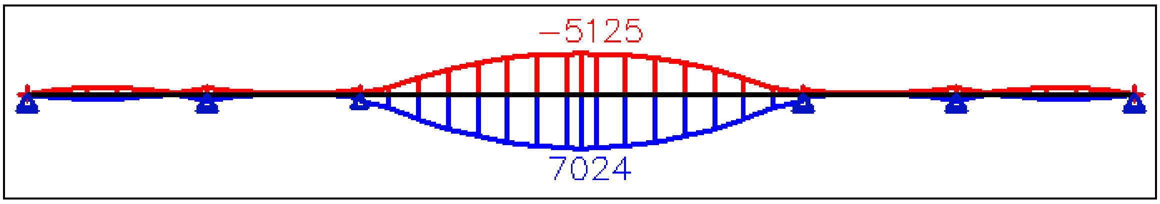


Figure XI 26: Cross-section BB', ULS moment envelope (kNm)

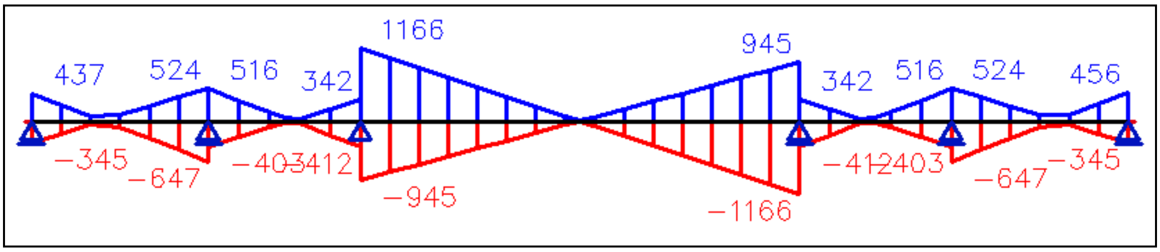


Figure XI 27: Cross-section BB', ULS shear force envelope (kN)

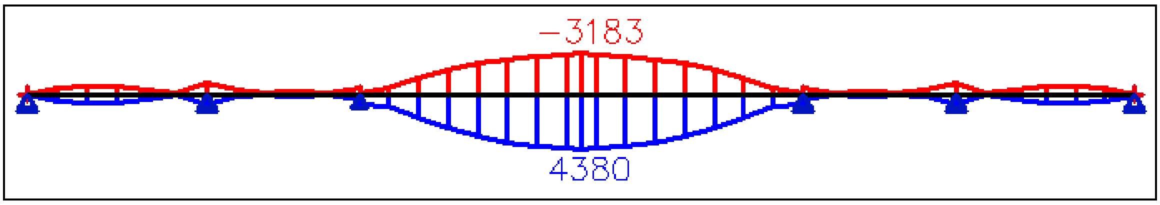


Figure XI 28: Cross-section BB', SLS moment envelope (kNm)

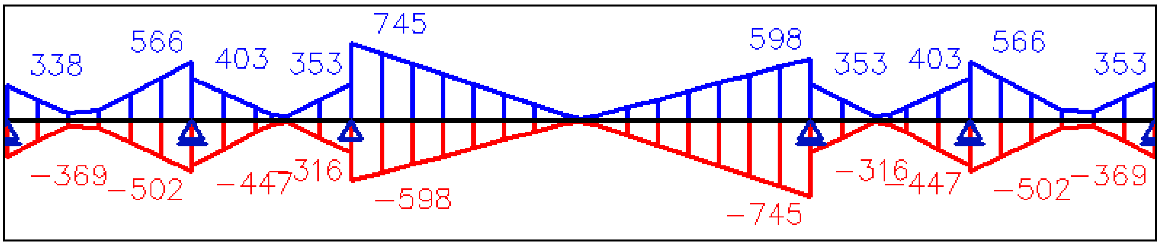


Figure XI 29: Cross-section BB', SLS shear force envelope (kN)

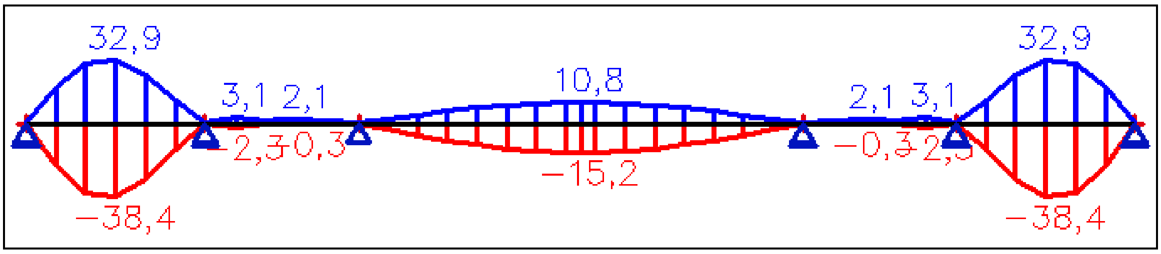


Figure XI 30: Cross-section BB', deformations (mm)

Check

The floor and the gate chamber are checked in Table XI 3 till Table XI 5. The deformation of both the wall and floor are within the norm, as can be seen in Figure XI 17 and Figure XI 30. The thin outer walls of the side caissons and the thin floors of these caissons are already been checked in Figure VII 8 and in the last part of this appendix (Shear force check).

Table XI 3: Cross-section BB' topside check.

Parameters					
Variable parameters			Locked parameters		
h	1,9	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	110	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	5125	kNm	b	1000	mm
V_d	1166	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	3183	kNm	k_2	750	-
V_d	745	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1854	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	3071,4	kN
τ_d	0,63	N/mm^2	z	1,7	m
Stirrups Needed			A_s Required	7061	mm^2
			A_c	1900000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	7311	mm^2
N_s (SLS)	1908	kN	ω	0,38%	
σ_s (SLS)	261	N/mm^2			
Condition 1	NOT OK		**The cross section is Sufficient**		
Condition 2	OK				
Acceptable cracking			**but not Economic**		

Table XI 4: Cross-section BB' bottom side check.

Parameters					
Variable parameters			Locked parameters		
h	1,9	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	80	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	7024	kNm	b	1000	mm
V_d	1166	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	4380	kNm	k_2	750	-
V_d	745	kN			

			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1854	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	4209,5	kN
τ_d	0,63	N/mm^2	z	1,7	m
Stirrups Needed			A_s Required	9677	mm^2
			A_c	1900000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	10053	mm^2
N_s (SLS)	2625	kN	ω	0,53%	
σ_s (SLS)	261	N/mm^2			
Condition 1	NOT OK		**The cross section is**		
Condition 2	OK		**Sufficient**		
Acceptable cracking			**and Economic**		

Table XI 5: Cross-section BB' Gate chamber wall check.

Parameters					
Variable parameters			Locked parameters		
h	1,2	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	80	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	810	kNm	b	1000	mm
V_d	875	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	598	kNm	k_2	750	-
V_d	646	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1154	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	779,9	kN
τ_d	0,76	N/mm^2	z	1,0	m
Stirrups Needed			A_s Required	1793	mm^2
			A_c	1200000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	10053	mm^2
N_s (SLS)	576	kN	ω	0,84%	
σ_s (SLS)	57	N/mm^2			
Condition 1	OK		**The cross section is**		
Condition 2	OK		**Sufficient**		
Acceptable cracking			**but not Economic**		

As can be seen in tables the reinforcement in the walls and the floor of the gate chamber is less than the economic value. Both can be dimensioned thinner, this is in line with the conclusion from cross section AA'. Also the wall can be constructed thinner in a possible optimisation.

Cross section CC'

Load combinations
In Figure XI 31 the forces that acting on the wall are shown. The triangle buttress is so stiff that no moments will be transferred from the wall to the floor.

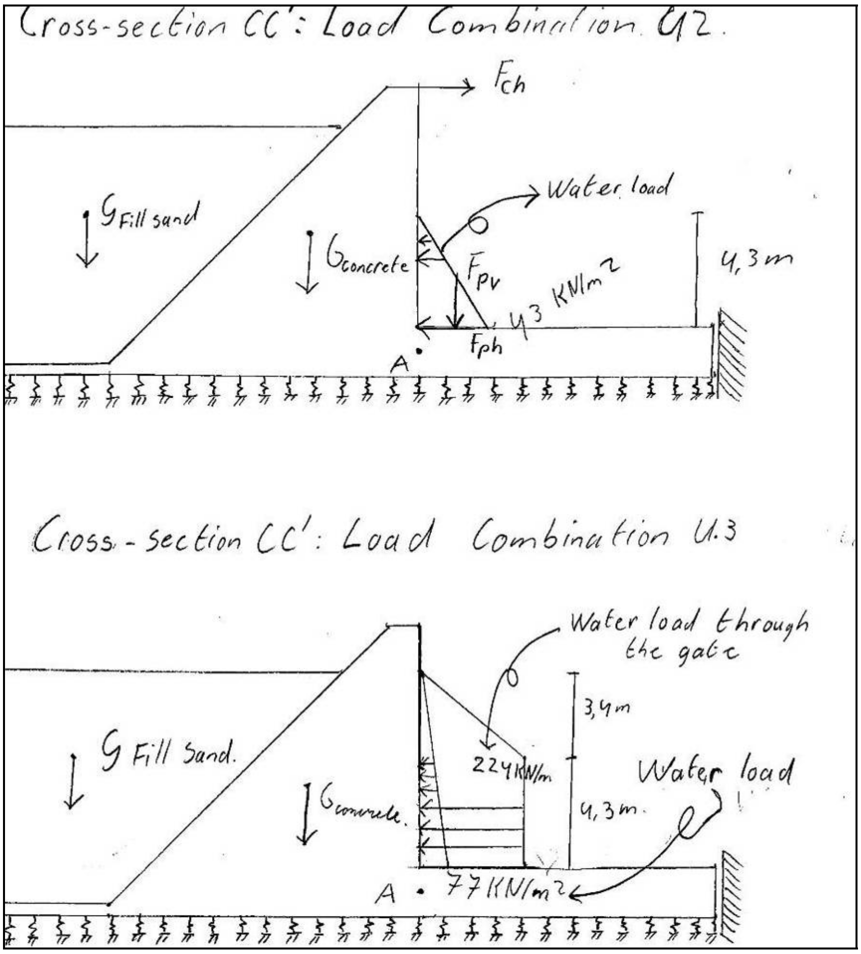


Figure XI 31: The cross section CC' force distribution on the wall

In Figure XI 32 the schematisation of Cross-section CC' is shown and in Figure XI 33 till Figure XI 37 the loads that are acting on this schematisation can be seen.

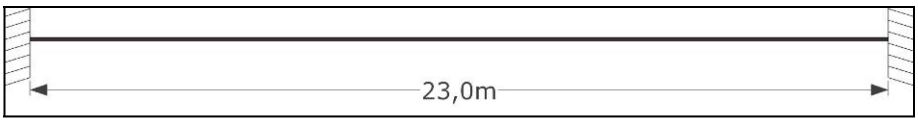


Figure XI 32: Schematisation of cross-section CC'

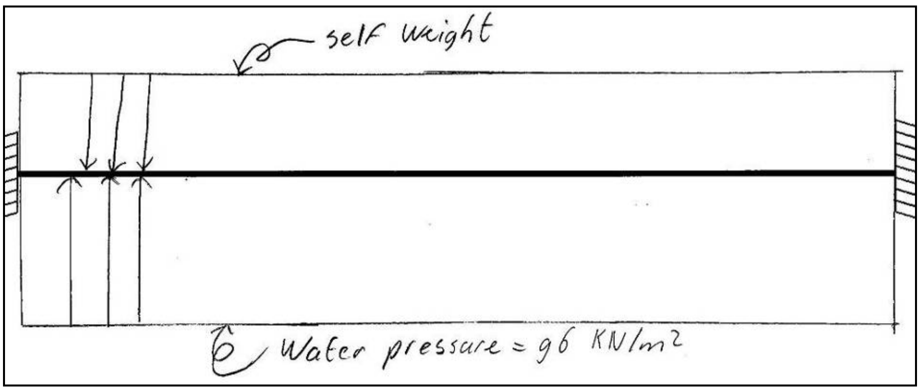


Figure XI 33: Cross-section CC', Load combination C.2

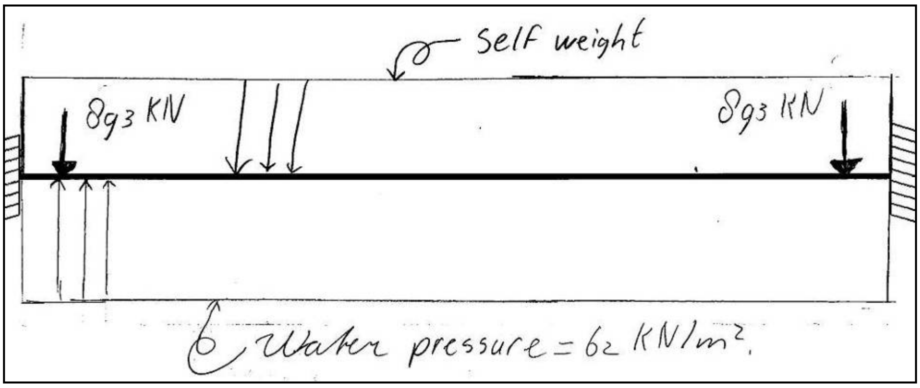


Figure XI 34: Cross-section CC', Load combination U.1

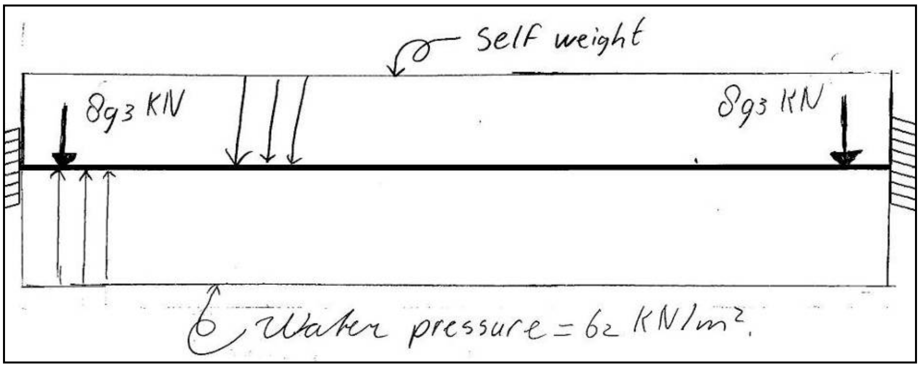


Figure XI 35: Cross-section CC', Load combination U.2 and U.4

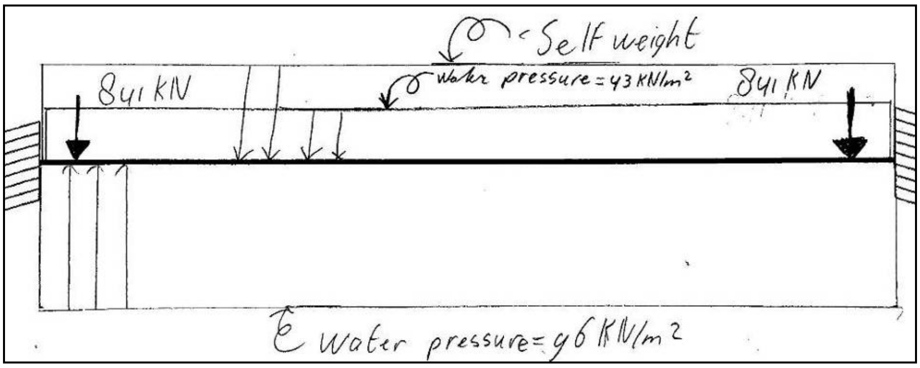


Figure XI 36: Cross-section CC', Load combination U.3

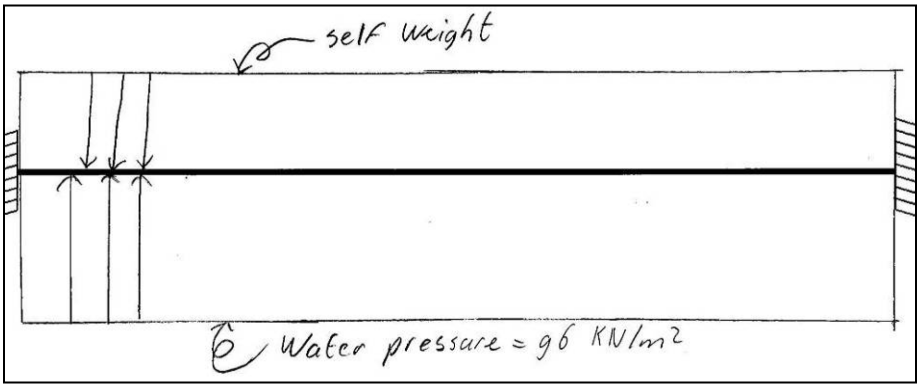


Figure XI 37: Cross-section CC', Load combination U.5

Resulting forces and moments
In Figure XI 38 till Figure XI 42 the ULS and SLS moments, shear forces and deformations are shown.

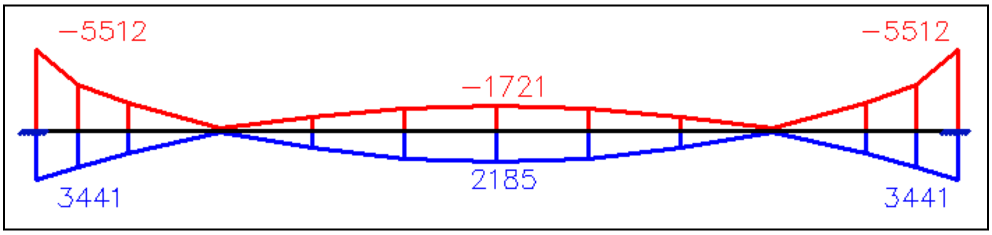


Figure XI 38: Cross-section CC', ULS moment envelope (kNm)

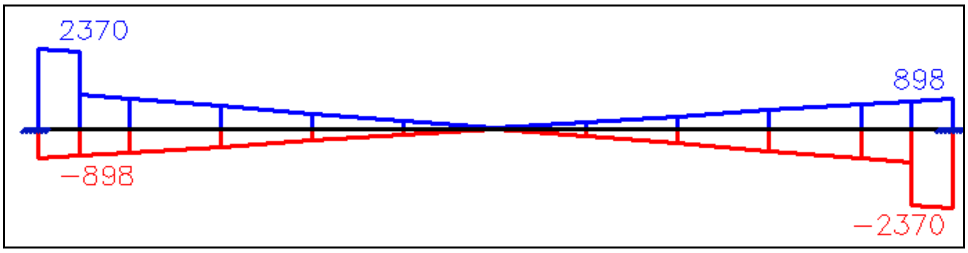


Figure XI 39: Cross-section CC', ULS shear force envelope (kN)

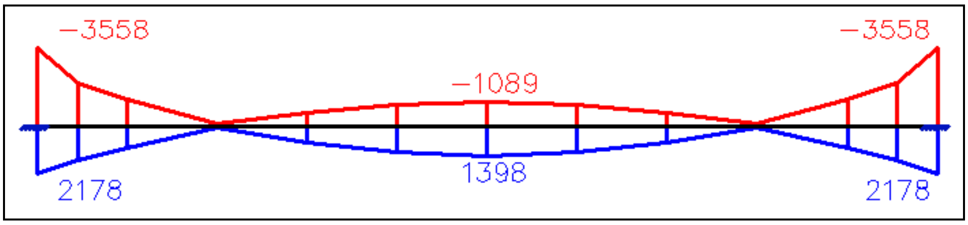


Figure XI 40: Cross-section CC', SLS moment envelope (kNm)

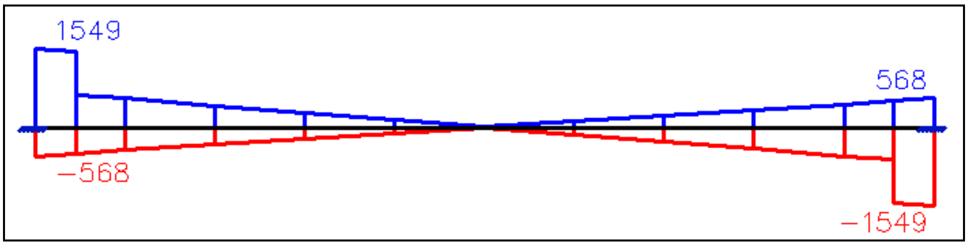


Figure XI 41: Cross-section CC', SLS shear force envelope (kN)

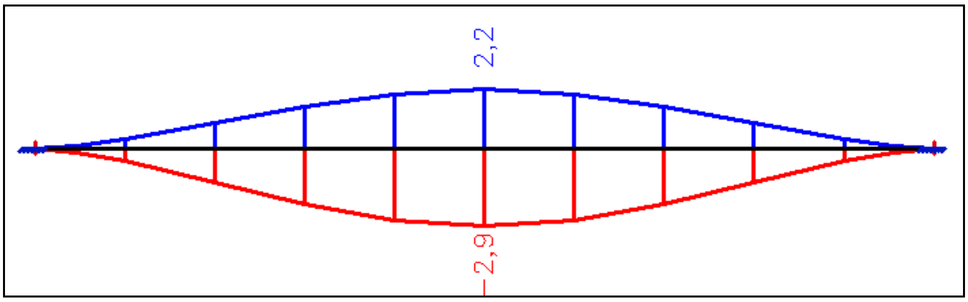


Figure XI 42: Cross-section CC', deformations (mm)

Checks
In Table XI 6 and Table XI 7 the checks of the top and bottom side of the lock head floor are shown. From this two tables follows that the floor good be optimised by constructing it thinner, because than the economic value is reached.

Table XI 6: Cross-section CC' top side check.

Parameters					
Variable parameters			Locked parameters		
h	1,9	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	100	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	5512	kNm	b	1000	mm
V_d	2370	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	3558	kNm	k_2	750	-
V_d	1549	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1854	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	3303,4	kN
τ_d	1,28	N/mm^2	z	1,7	m
Stirrups Needed			A_s Required	7594	mm^2
			A_c	1900000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	8042	mm^2
N_s (SLS)	2132	kN	ω	0,42%	
σ_s (SLS)	265	N/mm^2			
Condition 1	NOT OK		**The cross section is**		
Condition 2	OK		**Sufficient**		
Acceptable cracking			**but not Economic**		

Table XI 7: Cross-section CC' bottom side check.

Parameters					
Variable parameters			Locked parameters		
h	1,9	m	c	30	mm
Ø	25	mm	f_s	435	N/mm^2
s	100	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	3441	kNm	b	1000	mm
V_d	2370	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	2178	kNm	k_2	750	-
V_d	1549	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1858	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	2058,3	kN
τ_d	1,28	N/mm^2	z	1,7	m
Stirrups Needed			A_s Required	4732	mm^2
			A_c	1900000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	4909	mm^2
N_s (SLS)	1303	kN	ω	0,26%	
σ_s (SLS)	265	N/mm^2			
Condition 1	NOT OK		**The cross section is**		
Condition 2	OK		**Sufficient**		
Acceptable cracking			**but not Economic**		

Shear force check

Besides the three cross sections the shear force between the side caissons and the gate chamber is checked. The connection between the side caissons and the gate chamber must be able to cope with the total shear force, in case of a maximal uplift and an empty lock head or in a floating position. This total shear force is checked below.

(SLS) Weight of the concrete	= 1310 m^3 * 24 kN/m^3	= 31440 kN
Connected walls/struts(one) (d=10,5m w = 2,1m)	= 10,5*2,1	= 22 m^2
Connected floor (one) (d=0,362 w= 19,3)	= 0,362*19,3	= 7 m^2
b*d	= (22+7)*2*10^6	= 58 *10^6 mm^2

Construction phase

Uplift	= 449 m^2 * 51,1 kN/m^2	= 22944 kN
Resulting force upwards	= 0,9*22944 – 1,2*32381	= - 18208 kN

Resulting shear stress $= \tau_d = \frac{V_d * 10^3}{b * d} = \frac{18208 * 10^3}{58 * 10^6} = 0{,}31\ N/mm^2$

Users phase

Uplift	= 449 m^2 * 96 kN/m^2	= 43104 kN
Resulting force upwards	= 1,25*43104 – 0,9*32381	= 24737 kN

Resulting shear stress $= \tau_d = \frac{V_d * 10^3}{b * d} = \frac{24737 * 10^3}{58 * 10^6} = 0{,}43\ N/mm^2$

No stirrups are needed in the connection between the side caissons and the gate chamber, because both the shear stresses are below 0,56 N/mm^2

Appendix XII. Optimisation of the large lock heads

In this appendix the same checks as in Appendix XI are performed with the same forces and moments, but with the optimised dimension. This is done to check if the optimised construction from Figure 12.18 is sufficient and economic. The checks can be seen in Table XII 1 till Table XII 7. Finally in Table XII 8 and Table XII 9 the draft and the resistance against uplift can be seen of the optimised lock heads.

Table XII 1: Cross-section AA' topside optimisation check.

Parameters					
Variable parameters			Locked parameters		
h	1,5	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	100	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	3635	kNm	b	1000	mm
N_d	85	kN	ξ	1	-
V_d	655	kN	k_1	3750	-
(SLS)			k_2	750	-
M_d	2300	kNm			
N_d	68	kN	Economic reinforcement values		
V_d	542	kN	0,50%	< ω >	0,75%
Effective height			*Bending and Normal force (ULS)*		
d	1454	mm	N_s (ULS)	2862,8	kN
			z	1,3	m
Shear force (ULS)			A_s Required	6581	mm^2
τ_d	0,45	N/mm^2	A_c	1500000	mm^2
No Stirrups Needed			A_s Selected	8042	mm^2
			ω	0,54%	
Cracking [NEN 6720] (SLS)					
N_s (SLS)	1826	kN	**The cross section is**		
σ_s (SLS)	227	N/mm^2	**Sufficient**		
Condition 1	NOT OK		**and Economic**		
Condition 2	OK				
Acceptable cracking					

Table XII 2: Cross-section AA' bottom side optimisation check.

Parameters					
Variable parameters			Locked parameters		
h	1,5	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	90	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	4487	kNm	b	1000	mm
N_d	85	kN	ξ	1	-
V_d	930	kN	k_1	3750	-
(SLS)			k_2	750	-
M_d	2300	kNm			
N_d	68	kN	Economic reinforcement values		
V_d	594	kN	0,50%	< ω >	0,75%

Effective height				*Bending and Normal force (ULS)*		
d	1454	mm		N_s (ULS)	3513,9	kN
				z	1,3	m
Shear force (ULS)				A_s Required	8078	mm^2
τ_d	0,64	N/mm^2		A_c	1500000	mm^2
Stirrups Needed				A_s Selected	8936	mm^2
				ω	0,60%	
Cracking [NEN 6720] (SLS)						
N_s (SLS)	1826	kN		**The cross section is**		
σ_s (SLS)	204	N/mm^2		**Sufficient**		
Condition 1	NOT OK			**and Economic**		
Condition 2	OK					
Acceptable cracking						

Table XII 3: Cross-section BB' topside optimisation check.

Parameters						
Variable parameters				Locked parameters		
h	1,5	m		c	30	mm
Ø	32	mm		f_s	435	N/mm^2
s	80	mm		τ_1	0,56	N/mm^2
(ULS)				τ_2	4,2	N/mm^2
M_d	5125	kNm		b	1000	mm
V_d	1166	kN		ξ	1	-
(SLS)				k_1	3750	-
M_d	3183	kNm		k_2	750	-
V_d	745	kN				
				Economic reinforcement values		
Effective height				0,50%	< ω >	0,75%
d	1454	mm				
				Bending and Normal force (ULS)		
Shear force (ULS)				N_s (ULS)	3916,4	kN
τ_d	0,80	N/mm^2		z	1,3	m
Stirrups Needed				A_s Required	9003	mm^2
				A_c	1500000	mm^2
Cracking [NEN 6720] (SLS)				A_s Selected	10053	mm^2
N_s (SLS)	2432	kN		ω	0,67%	
σ_s (SLS)	242	N/mm^2				
Condition 1	NOT OK			**The cross section is**		
Condition 2	OK			**Sufficient**		
Acceptable cracking				**and Economic**		

Table XII 4: Cross-section BB' bottom side optimisation check.

Parameters					
Variable parameters			Locked parameters		
h	1,5	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	80	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	7024	kNm	b	1000	mm
V_d	1166	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	4380	kNm	k_2	750	-
V_d	745	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1454	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	5367,6	kN
τ_d	0,80	N/mm^2	z	1,3	m
Stirrups Needed			A_s Required	12339	mm^2
			A_c	1500000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	10053	mm^2
N_s (SLS)	3347	kN	ω	0,67%	
σ_s (SLS)	333	N/mm^2			
Condition 1	NOT OK		**The cross section is**		
Condition 2	OK		**Not Sufficient**		
Acceptable cracking			**and Economic**		

Table XII 5: Cross-section BB' Gate chamber wall optimisation check.

Parameters					
Variable parameters			Locked parameters		
h	0,7	m	c	30	mm
Ø	25	mm	f_s	435	N/mm^2
s	130	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	810	kNm	b	1000	mm
V_d	875	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	598	kNm	k_2	750	-
V_d	646	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	658	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	1368,8	kN
τ_d	1,33	N/mm^2	z	0,6	m
Stirrups Needed			A_s Required	3147	mm^2
			A_c	700000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	3776	mm^2
N_s (SLS)	1011	kN	ω	0,54%	
σ_s (SLS)	268	N/mm^2			
Condition 1	NOT OK		**The cross section is**		
Condition 2	OK		**Sufficient**		
Acceptable cracking			**and Economic**		

Table XII 6: Cross-section CC' topside optimisation check.

Parameters					
Variable parameters			Locked parameters		
h	1,5	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	80	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	5512	kNm	b	1000	mm
V_d	2370	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	3558	kNm	k_2	750	-
V_d	1549	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1454	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	4212,1	kN
τ_d	1,63	N/mm^2	z	1,3	m
Stirrups Needed			A_s Required	9683	mm^2
			A_c	1500000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	10053	mm^2
N_s (SLS)	2719	kN	ω	0,67%	
σ_s (SLS)	270	N/mm^2			
Condition 1	NOT OK		**The cross section is**		
Condition 2	OK		**Sufficient**		
Acceptable cracking			**and Economic**		

Table XII 7: Cross-section CC' topside optimisation check.

Parameters					
Variable parameters			Locked parameters		
h	1,5	m	c	30	mm
Ø	32	mm	f_s	435	N/mm^2
s	100	mm	τ_1	0,56	N/mm^2
(ULS)			τ_2	4,2	N/mm^2
M_d	3441	kNm	b	1000	mm
V_d	2370	kN	ξ	1	-
(SLS)			k_1	3750	-
M_d	2178	kNm	k_2	750	-
V_d	1549	kN			
			Economic reinforcement values		
Effective height			0,50%	< ω >	0,75%
d	1454	mm			
			Bending and Normal force (ULS)		
Shear force (ULS)			N_s (ULS)	2629,5	kN
τ_d	1,63	N/mm^2	z	1,3	m
Stirrups Needed			A_s Required	6045	mm^2
			A_c	1500000	mm^2
Cracking [NEN 6720] (SLS)			A_s Selected	8042	mm^2
N_s (SLS)	1664	kN	ω	0,54%	
σ_s (SLS)	207	N/mm^2			
Condition 1	NOT OK		**The cross section is**		
Condition 2	OK		**Sufficient**		
Acceptable cracking			**and Economic**		

Table XII 8: The upstream lock head dimensions

Gate chamber		
Width	19,8	m
Length	19,3	m
Height (inside)	9,4	m
Floor thickness	1,5	m
Wall thickness	0,7	m
Wall width	2,3	m
Floor volume	573	m^3
Wall volume (1)	238	m^3
Total volume	1049	m^3

Gate recess (2)		
Length	11,9	m
Width	1,6	m
Height	9,4	m
Volume (1)	-179	m^3

Caisson recess (4)		
Length	2	m
Width	1,6	m
Height	10,5	m
Volume (1)	-33,6	m^3

Side caissons (2)		
Width	14,0	m
Length	19,3	m
Height (inside)	9,3	m
Floor thickness	0,4	m
Inner wall thickness	0,3	m
Outer wall thickness	0,4	m
Outer wall length	46,5	m
Inner wall length	45,1	m
Volume floor(1)	108	m^3
Volume walls (1)	238	m^3
Total volume (1)	346	m^3

Gate struts (2)		
Angle	45	o
Thickness	1,0	m
Height	10,5	m
Length	10,5	m
Volume (1)	55,1	m^3

Wall struts (2)		
Total thickness	1,1	m
Reduction volume caissons	61	m^3
volume (1)	61	m^3

Upstream head		
Volume	1973	m^3
Width	52,4	m
Length	19,3	m
Top level	12,8	m NAP
Top level floor	3,4	m NAP
Top level bottom	1,9	m NAP
Concrete Weight	47347	kN
Total floating weight	50594	kN

Bulk head (2)		
Surface (1)	162	m^2
Weight (1)	1624	kN

Water pressure		
Surface	1011	m^2

Sand		
Volume	4527	m^3
Weight	76964	kN

Resistance against uplift		
Water pressure	93041	kN
Total weight	124311	kN
OK		

Floatation level		
Floatation level	5,00	<5,2 m

Table XII 9: The downstream lock head dimensions

Gate chamber		
Width	19,8	m
Length	19,3	m
Height (inside)	9,4	m
Floor thickness	1,5	m
Wall thickness	0,7	m
Wall width	2,3	m
Floor volume	573	m^3
Wall volume (1)	234	m^3
Total volume	1042	m^3

Gate recess (2)		
Length	11,9	m
Width	1,6	m
Height	9,4	m
Volume (1)	-179	m^3

Caisson recess (4)		
Length	2,1	m
Width	1,6	m
Height	10,5	m
Volume (1)	-35,3	m^3

Side caissons (2)		
Width	14,0	m
Length	19,3	m
Height (inside)	6,3	m
Floor thickness	0,4	m
Inner wall thickness	0,3	m
Outer wall thickness	0,4	m
Outer wall length	46,5	m
Inner wall length	45,1	m
Volume floor(1)	108	m^3
Volume walls (1)	175	m^3
Total volume (1)	283	m^3

Gate struts (2)		
Angle	45	o
Thickness	1,0	m
Height	10,5	m
Length	10,5	m
Volume (1)	55,1	m^3

Wall struts (2)		
Total thickness	1,1	m
Reduction volume caissons	28	m^3
volume (1)	61	m^3

Upstream head		
Volume	1839	m^3
Width	52,4	m
Length	19,3	m
Top level	12,8	m NAP
Top level floor	3,4	m NAP
Top level bottom	1,9	m NAP
Concrete Weight	44131	kN
Total floating weight	46190	kN

Bulk head (2)		
Surface (1)	103	m^2
Weight (1)	1030	kN

Water pressure		
Surface	1011	m^2

Sand		
Volume	3425	m^3
Weight	58224	kN

Resistance against uplift		
Water pressure	77872	kN
Total weight	102355	kN
OK		

Floatation level		
Floatation level	4,57	<4,6 m