Adaptation of marine locks against sea level rise

MSc Thesis of Douwe Huijsman

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Preface

This thesis is the final step of my master program Hydraulic Engineering at the faculty of Civil Engineering and Geosciences at the Technical University Delft. The thesis is undertaken during an internship at the company Sweco Nederland.

My initial thanks thus goes out to Arjan Frens, team leader at Sweco, who has offered me the opportunity to execute this internship, and helped me deriving a subject. Even though the circumstances of my internship have been far from ideal due to the pandemic, I very much appreciate the effort to include me within the team as much as has been done. I would also like to note my gratitude for Richard Roggeveld, who has been my supervisor at Sweco, and has been able to offer me great support throughout the whole process. At last I would like to thank my University instructors: Mark Voorendt, Erik-Jan Houwing, and Bas Jonkman. Their insights and advice have helped me a great deal in setting out each step in my work plan, and structuring my work schedule and report. They have also made it very easy to get in contact with other people who could help me further where they felt less qualified.

Douwe Huijsman
Leiden, January 5, 2021

Summary

There is a lot of uncertainty in climate change developments, and therefore in its consequences. For the Netherlands, an important consequence of climate change is the sea level rise. The many coastal areas of the country are protected against the sea by an impressive flood protection system, consisting of i.a. sea dykes, dunes, flood barriers, and marine locks. The reliability of these elements is however threatened by the rising sea level. Due to the uncertainty of sea level rise, it is unsure whether their integrity will actually be jeopardised, and when and how they need to be adapted.

Marine locks are important elements of the flood protection, as they both function to preserve the safety of the hinterland, and to allow transportation of vessels between the inland and the sea. Marine locks are large, rigid, and intricate structures, with a required lifespan of at least 100 years. Therefore often design decisions have to be made for many decades, which is not made easy by the uncertainty of climate change. When a lock no longer suffices due to sea level rise, should its lifetime be prolonged by means of adaptations, or is it more beneficial to simply replace the lock? And if a lock is being replaced, should it be designed based on a conservative sea level rise scenario, risking having spent too much money, or should it be done optimistically, risking having to adapt or replace again long before the required 100 years of lifetime? The objective of this thesis is to develop a method to support decision making on adapting and replacing locks while taking the uncertainty of sea level rise into account.

Analysing influence of sea level rise on failure probability

To support decisions on adapting lock structures, different adaptation alternatives are to be studied, and therefore have to be developed. Before these concepts can be developed, it has to be known what lock characteristics first require attention. To explore this, the initial step is to investigate how the failure probability of a marine lock will alter due to this sea level rise, what failure mechanisms contribute most to the total failure probability, and at what point a lock no longer suffices. Thereto, a failure probability model is made, in which all major failure mechanisms of a marine lock which are influenced by sea level rise are included. The calculations performed by the model are based on Rijkswaterstaat's WBI2017 manuals. These are guidelines composed to assess the structural sufficiency of a hydraulic structure. After implementing the lock properties and hydraulic boundary conditions of a specific case into the model, the development of the failure probability due to sea level rise is derived through a probabilistic Monte Carlo simulation. The model is validated by means of a case study, the Western lock in Terneuzen. For this lock, the maximum failure probability which is acceptable by the safety standard is reached at a sea level rise of 1.1 m. The significant failure mechanisms are the failure due to vibration of the gate during overflow, failure of the bed protection during overflow or inflow, surpassage of the storage capacity after overflow or inflow, and structural failure of the gates due to a large head difference. From these principle failure mechanisms, the critical lock characteristics can be derived which are to be considered in the development of adaptation concepts. These characteristics include the retention height, the strength of the gate, the bed protection composition, and the piping path length. The latter is added because although piping is no relevant mechanism for the case study, it cannot be ignored that sea level rise can have a grave influence on the piping probability of other locks.

Developing support for deciding to adapt or replace

When a lock no longer suffices, it either has to be adapted, or replaced. For each critical lock characteristic derived in the failure probability analyses, multiple adaptation concepts are developed. A method is derived which can be used to compare the concepts to one another, and make a decision on what adaptation to apply. Initially, the concepts are evaluated based on several important criteria. These criteria include the replacement time, replacement nuisance, familiarity of the construction procedure, the sustainability, and the circularity. For each project, a different weighting can be assigned to the criteria, depending on the requirements of the project, and a different grading can be derived for each adaptation concept, depending on their applicability per specific lock case. In addition, the required costs of the adaptations are determined, based on the required materials and executed construction procedure. By comparing both the value and costs of the alternatives, a selection can be made which adaptation concepts are to be considered when a lock no longer suffices due to sea level rise.

From this process, two adaptation procedures are derived for the case study, the Western lock in Terneuzen. The first is a minor adaptation including a wall of 0.5 m on top of the lock head, and a gate extension of an equal height. The second is a drastic adaptation including a modular extension of the lock design (for which a strengthening of the lock head and chamber walls is required, in addition to an elevation of the bridge crossing the outer head), a replacement of the gate with a taller one, and the penetration of the bed protection with colloidal concrete. By means of the minor adaptation and drastic adaptation, respectively another 0.5 m and 1.5 m of sea level rise can be withstood by the lock. For both adaptation options, it is analysed whether it is beneficial to prolong the lifetime of a lock rather than to replace it (with a conservative lock). The advantages and disadvantages of the adaptations are listed below. Unless there is another reason than sea level rise to replace a lock, it is recommended to always first apply a minor adaptation, as it has very low costs and little impact on the residents and lock users during construction.

- \+ The minor adaptation is profitable, regardless of the additional yearly maintenance costs of an old lock relative to a new replaced lock.
- +/- The drastic adaptation can be profitable, depending on the functional and structural state of the lock.
- \+ Postponing the replacement by means of an adaptation results in a smaller uncertainty in sea level rise which has to be considered by the time the lock is eventually being replaced.
- \+ Regarding the collective of Dutch locks, postponing some of the lock replacements which have to be executed in the upcoming decades, adapting some helps with spreading the expenses over a longer stretch of time.
- \- During an adaptation, the lock has to be closed off temporarily (much longer for a drastic adaptation), reducing the capacity of the waterway, while the neighboring residents and lock users experience nuisance of the construction.
- \- The regarded adaptations only solve a single issue, whereas multiple issues can be solved by means of a lock replacement.

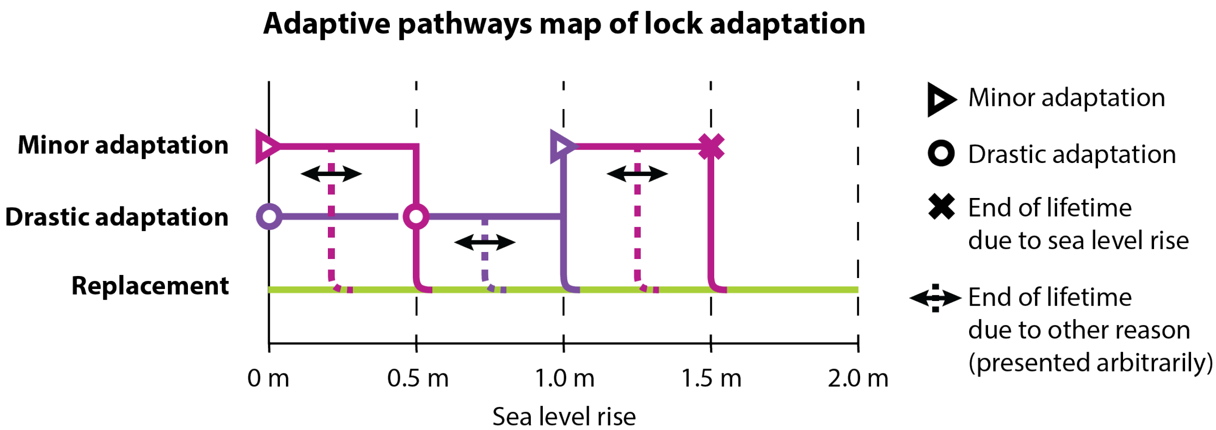


Figure 1: Adaptive pathways map for a lock which either has the structural capacity for a drastic adaptation, or can be strengthened through an additional adaptation

Developing support for deciding on conservative or adaptive design
When a lock is finally replaced, regardless of the underlying reasons, it has to be decided how the uncertainty of sea level rise is implemented in the lock design. Three alternatives are developed. The first is a conservative lock, with a high implemented sea level rise. The second is a "strong" adaptive lock, which has no conservative retention height, but components designed strong enough for a possible increment of the retention height in case of a severe sea level rise scenario. These components include the lock heads, chamber walls, foundation, and operating mechanism of the gates. The third alternative is a "weak" adaptive lock, which has no conservative retention height, and no sufficient strength for a height increment. The lock is however designed to allow for its components to be strengthened in case of a severe scenario, after which the retention height *can* be increased.

The costs and value of the lock alternatives have been compared using an adaptive pathways map. The advantages and disadvantages of the adaptive designs are listed below. Based on these remarks, it seems attractive to apply an adaptive design to a lock, but this still has to be determined per individual case.

+ Money is saved by applying an adaptive design in case of a mild sea level rise scenario
- Additional money is spent in case of a severe sea level rise scenario due to required adaptations
+ The payment is spread in case of a severe sea level rise scenario, as the adaptation is required later in the lifetime of the lock
- A section or the entirety of the waterway has to be closed off temporarily during adaptation in case of a severe scenario, and additional nuisance is experienced by neighboring residents and users of the road crossing the lock
- The possibility of an adaptation in the future requires stricter monitoring of the lock and hydraulic boundary conditions

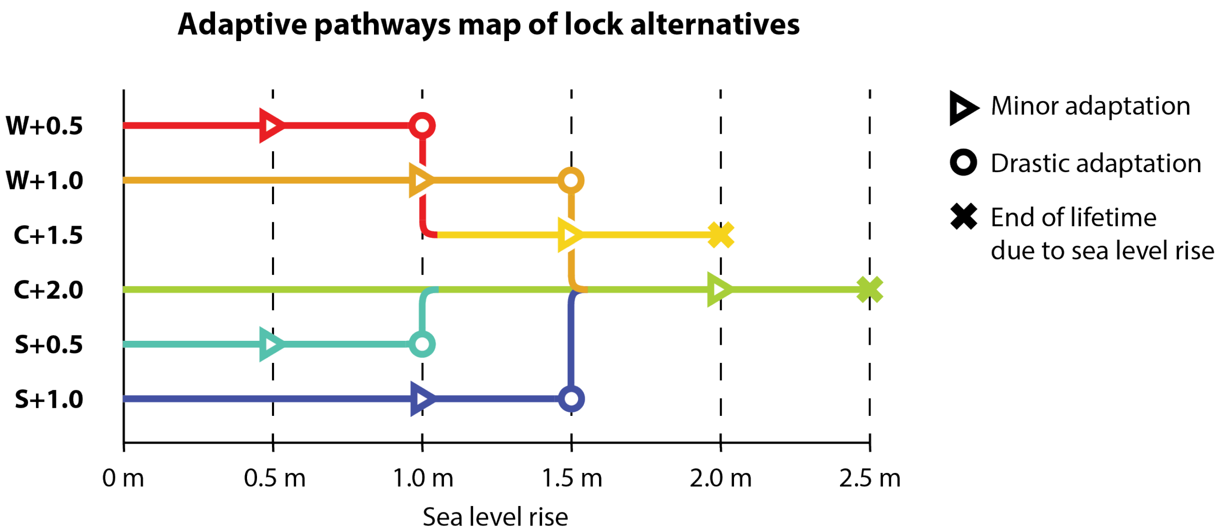


Figure 2: Adaptive pathways map for lock replacement alternatives weak adaptive (W+0.5 and W+1.0), conservative (C+2.0), and strong adaptive (S+0.5 and S+1.0)

Contents

1
Introduction

To be able to effectively protect ourselves against the dangers of climate change, the extent of its consequences must be predicted. Thereto, many researchers and institutes have for many years been gathering information and creating models which provide possible projections of these global and local phenomena. However, even with the greatest knowledge and effort, these projections will always be accompanied by a substantial uncertainty (Stocker et al., 2013).

The profuse number of aspects that play a role in the global and local impact can be overwhelming. In addition, there can always be significant facets yet unconsidered or undiscovered. But above all, the largest uncertainty of all remains humankind. How the trends in our social and political structures will unfold over the upcoming decades will dominate the playing field (Van Vuuren et al., 2011).

In conclusion, the only real certain aspect of climate change seems to be uncertainty. Although decreasing this uncertainty will remain of importance to tackling the provoked issues, it may be time to shift our focus on the matter. We could give in to a certain level of uncertainty, and put effort into finding solutions which are flexible and applicable regardless of the eventual outcome.

For the Netherlands, amongst the facets for which adaptable solutions may be in order, are the elements which make up the flood protection (Rijkswaterstaat & Dutch Ministry of Economic Affairs, Agriculture and Innovation, 2012). A rising sea level, high river discharges in winter and low discharges in dryer summers will change the conditions to which the structures are subjected. Important elements of the flood protection are the locks. Specifically marine locks allow the passage of vessels through the Dutch coastline flood defence, and are therefore essential structures for almost all occupation of the water system in the country. These structures form the only connection between the sea and the inland water system, meaning their functionality must be guaranteed. Global warming will however cause changes in both the sea level and the river levels, possibly resulting in unforeseen loading conditions for which the locks have not been designed (Haasnoot et al., 2018). In case the consequences become too severe, entire lock systems may have to be rebuilt to preserve the flood safety of the hinterland. Replacing all Dutch marine locks is obviously a robust and costly solution to the issue. In addition, the uncertain future makes it impossible to predict to what extent locks will have to be improved, likely resulting in either under- or overdimensioning. Thus, it could be beneficial to investigate if more flexible solutions can be applied to the problem, and if adaptive strategies can be developed to avoid having to make bold assumptions regarding the consequences of such uncertain events. The motivation of this thesis is the current lack of investigation regarding solution strategies for marine locks with respect to consequences of global warming.

2
Problem Analysis

Before assessing possible design solutions, it should be understood how uncertain the issue actually is, and to what extent the changes can influence the functionality of the locks. Thereto, it must first be analysed how the loading conditions of marine locks will be altered due to a changing climate, and how this may affect their failure mechanisms. This chapter provides insights on the current knowledge regarding these topics.

2.1. Climate change uncertainties

As mentioned, climate change is governed by many uncertainties. The following sections depict where these uncertainties originate from, and how it develops through the consequences which endanger the flood protection of the Netherlands.

2.1.1. Climate change causes

Climate change is a process triggered by an increment of radiative forcing (Rosenzweig, Neofotis, & Vicarelii, 2008). This is a unit indicating the balance between the energy entering the Earth's atmosphere and the energy which is reflected back outward. The radiative forcing is expressed in W/m^2, i.e. the energy absorbed per square meter of the Earth's surface. This energy is mostly emitted by the sun, and dictates the surface temperature of our planet. The more energy is contained, the larger the radiative forcing, and the higher the global mean temperature. The reflective behaviour of the atmosphere is governed by its composition, which has been rapidly changing ever since the industrial revolution.

There have been many attempts at predicting the forthcoming global warming, all including different projections of emissions, caused by varying expectations of i.a. population growth, wealth, and energy consumption. To summarise the extent of these predictions, four Representative Concentration Pathways (RCPs) have been composed by a collective of climate modellers, indicating the radiative forcing to be expected by the year 2100: RCP2.6, RCP4.5, RCP6, and RCP8.5 (Van Vuuren et al., 2011). The numbers in the titles of the RCPs indicate the predicted radiative forcing by 2100 in W/m^2 with respect to the radiative forcing present in 1750, i.e. pre-industrial times. Thus, in scenario RCP8.5, radiative forcing in 2100 is predicted to be 8.5 W/m^2 higher than in 1750. To comprehend what this conveys: the additional radiative forcing in 2018 was estimated to be equal to 3.1 W/m^2 (Butler & Montzka, 2019). The projections are presented at the left in Figure 2.1. Obviously, RCP2.6 is the scenario with the least severe consequences, whereas RCP8.5 will pose the largest threat.

Using these four scenarios, many other projections have been made regarding climate change driven developments, i.e. the global average surface temperature, as presented at the right in Figure 2.1. Not only does it present how rapidly and to what extent the temperature can grow, but it also clearly depicts the large uncertainty concerning climate change. Even each different scenario can result in a wide range of outcomes by the year 2100. As many other processes are affected by the change in temperature, our future is very uncertain. A number of these developments relevant for marine navigation lock design are discussed in the following sections.

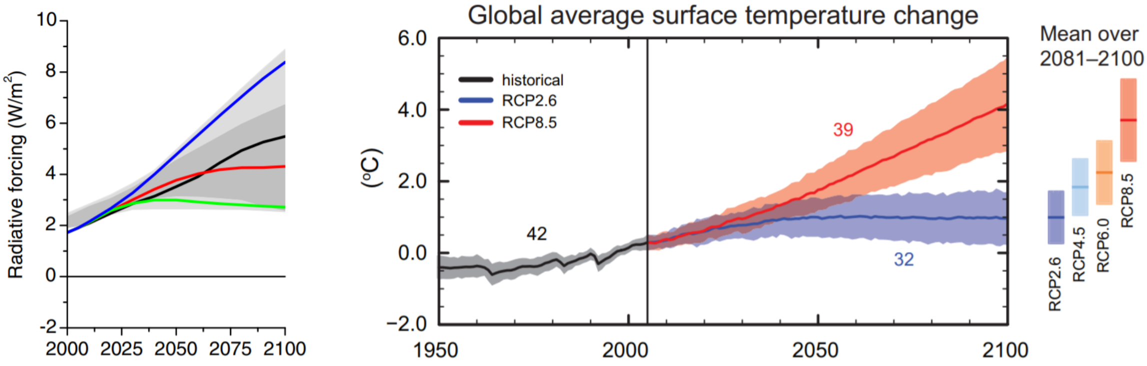


Figure 2.1: LEFT: Projection radiative forcing of RCP2.6 (green), RCP4.5 (red), RCP6 (black), and RCP8.5 (blue). Retrieved from Van Vuuren et al. (2011). RIGHT: Projection global average surface temperature. Retrieved from Stocker et al. (2013) (edited).

2.1.2. Uncertainties hydraulic boundary conditions at sea

One of the most familiar consequences of global warming is sea level rise. This is however not the only hydraulic boundary condition of marine locks that will change. In order to recognise how the sea level can change due to climate change, it is initially important to understand what elements comprise the sea level. Not only is it constantly changing due to the astronomical tide, but many additional processes can influence the height of the waterline, as listed below (Beem et al., 2000).

- The **Normative High Water** (NHW) is the high water level which is to be taken into account given the return period specified for each segment of the Dutch dyke system (Dutch Ministry of Transport, Public Works and Water Management, 2017d). These have been determined through extrapolation of measured data, and for coastlines concerns the highest astronomical tide and storm setup.
- **Sea level rise** is a result of global warming. This process has been well known and accepted for years, and taken into consideration of flood protection design. The extent of sea level rise may however have been well underestimated, as further discussed in the consecutive paragraph.
- **Local wind set up** is an inclination of the water line caused by local wind gusts and a closed boundary condition, preventing the wind from carrying the water any further.
- **Seiches** are standing wave patterns occurring in closed basins with a size proportional to the wave length.
- **Tidal resonance** can occur when a tidal wave is reflected in an estuary, bay, or river mouth, and interacts with a new incoming wave, and thus creating resonance.
- **Short term atmospheric depression** are differences in water level caused by low pressure zones travelling at an equal velocity with incoming waves, elevating the sea level for short periods of time (Molenaar & Voorendt, 2018).

The combined effects of local wind set up and short term atmospheric depressions are called **storm surges** (Molenaar & Voorendt, 2018). Whereas these processes concern changes of the sea level over periods of time of at least minutes, marine structures are also constantly disturbed by shorter oscillations: **waves**.

As most of these factors are partially determined by climate, changes are expected to arise to the sea level due to global warming. In the following sections, the currently performed research on the topic is analysed. Seiches and tidal resonance are not included, as these are very specifically determined by the geometry of basins.

Sea level rise due to global warming

Most Dutch locks were constructed several decades ago. At that time, the potential extent of climate change consequences was not yet recognised, and only a small degree of sea level rise has been taken into account of the designs. Thus, an accelerated sea level rise is likely to endanger the functionality and structural integrity of the coastline protection.

Sea level rise is mainly caused by two phenomena. The first is oceanic thermal expansion (Stocker et al., 2013). As the Earth's surface temperature rises, the oceans absorb a larger quantity of energy, causing the water bodies to heat up, and expand. This evidently results in a climbing sea level. The second phenomenon is glacier melting, concerning the imbalance of the seasonal fluctuation of glacier masses, such as the ice sheets of Greenland and Antarctica. Due to global warming of the Earth's surface, the volume of ice which melts during the summer period increases, while the volume regained during the cold winter decreases. The loss of ice volume contributes to the volume of the oceans, and thus increases the sea level.

Sea level rise will not be uniform over the entire surface of the Earth, due to i.a. gravitational pull and oceanic topography. For the Netherlands, much research has been performed based on sea level rise scenarios made by the Royal Netherlands Meteorological Institute (KNMI) in 2014. The most severe of these scenarios included a sea level rise of 1.0 m in 2100 relative to the sea level in 1995 (Haasnoot et al., 2018). It has however come to light that the Antarctic ice shelf is melting at a higher rate than formerly assumed. This is caused by hydrofracturing (melted surface ice seeping through cracks in the ice and damaging the internal coherency) and ice-cliffing (ice caps collapsing under their own weight) (Le Bars, Drijfhout, & De Vries, 2017). Taking these processes into consideration, the KNMI has calculated new projections regarding the sea level rise in the Netherlands (Haasnoot et al., 2018). These are presented in Figure 2.2. It should be noted that this is the most extreme, but one of several projections made regarding the new insight on the melting of the Antarctic.

As the magnitude of the consequences of hydrofracturing and ice-cliffing are yet to be thoroughly understood, these new insights currently provide much uncertainty. This can easily be recognised from the difference in confidence of the projections in Figure 2.2. What can also be recognised is that the sea level rise could develop much quicker than thought before, with the most extreme value being 3.2 m by the year 2100: three times more than originally predicted.

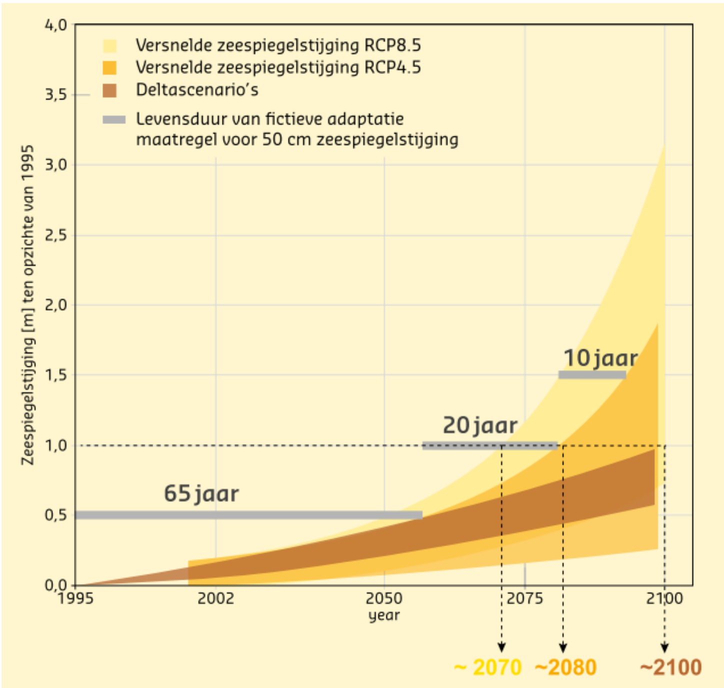


Figure 2.2: Projections sea level rise in the Netherlands relative to 1995 for RCP8.5 (light yellow), RCP4.5 (saturated yellow), and the outdated Delta scenarios (brown). Retrieved from Haasnoot et al. (2018).

The lifetime of a lock is meant to be at least 100 years in the Netherlands (Rijkswaterstaat, 2010). It is therefore important to also look beyond the year 2100, especially because when regarding the worst scenario, RCP8.5, the velocity of the sea level rise is clearly not decelerating near the end of the projection presented in 2.2.

Figure 2.3 presents projections drafted more recently, showing a less drastic sea level rise by the year 2100, but giving a broader view over a longer period of time. Here by the year 2100, it ranges between 0.3 and 1.1 meters, but by the year 2300 a sea level rise of even 5.4 meters is plausible in case emissions are not reduced. This graph clearly depicts how one should look further than 2100, because for scenario RCP8.5, the climate change issue has barely even commenced by that time. The difference between the predictions presented in Figure 2.2 and 2.3 (both credited to Le Bars, published in 2018 and 2019 respectively) shows both the uncertainty of these models and how fast research can change the outcome of the projection models.

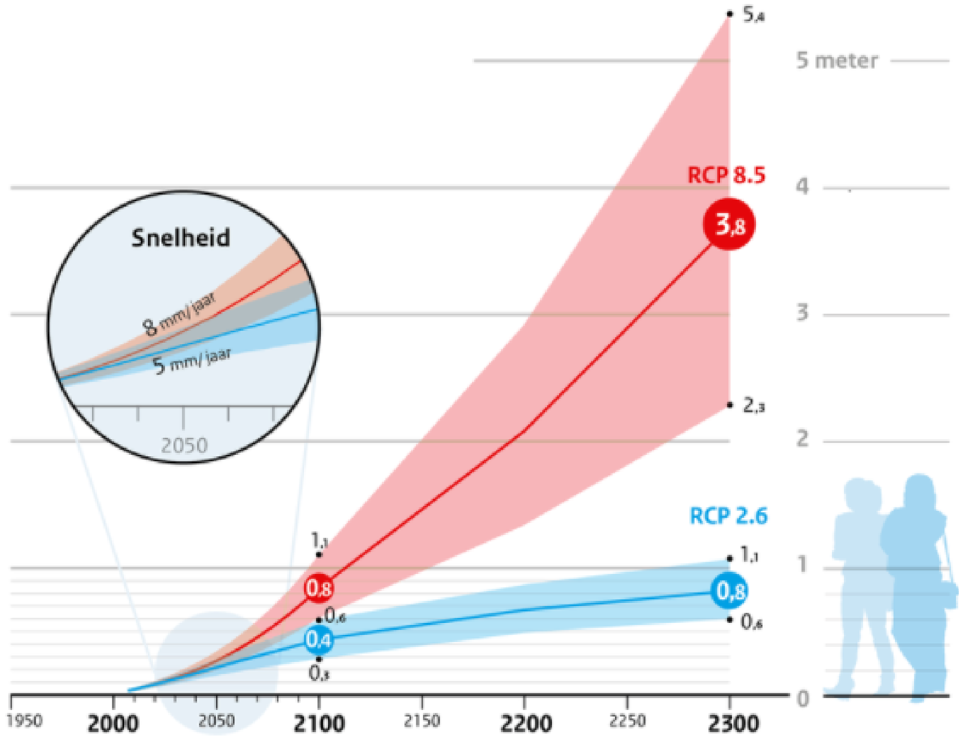


Figure 2.3: Sea level rise projections until 2300. Retrieved from Le Bars (2019) (edited).

Storm surge growth

Storm surges are oceanic water level differences caused by low pressure zones. For the Dutch coast, these commonly originate from storms at the North Sea. The global and local storm patterns might however be affected by climate change. Due to the difference in heating between the poles and the mid-latitudes, storm tracks are expected to migrate northwards on the Northern Hemisphere (Groll, Grabemann, & Gaslikova, 2013). This would essentially increase the frequency and intensity of westerlies that reach the North Sea trough the Atlantic Ocean. However, the decrease in temperature difference between the mid-latitudes and Arctic regions will additionally have a reducing effect on the same parameters, as pressure zones are driven by the magnitude of the temperature drop. It has therefore been suggested that these two effects might balance each other. In order to investigate the precise effects of climate change on the frequency and intensity of storms in the North Sea, researchers have studied past trends in storm occurrences, and have tried to computationally model the meteorological developments of the upcoming decades. By means of the former method, the conclusion is often made that, although there are clear decadal fluctuations in storm severity in the North Sea, no increasing trend can be observed from past data (Weisse, von Storch, Dieter Niemeyer, & Knaack, 2011). Figure 2.4 even suggests a declining trend line of average winter wind speeds when regarding more then a century of data. From the graph can however additionally be observed that the yearly variation is too inconsistent to draw well substantiated conclusions from the slightly declined trend line. With the same reasoning, one can also not exclude the possibility that the accelerated global warming actually has a stimulating effect on the North Sea winds. The fluctuations are clearly defined by other less consistent processes. The effect of climate change might also surface further in the future, as emission levels have been increasing exponentially in the past years (Van Vuuren et al., 2011).

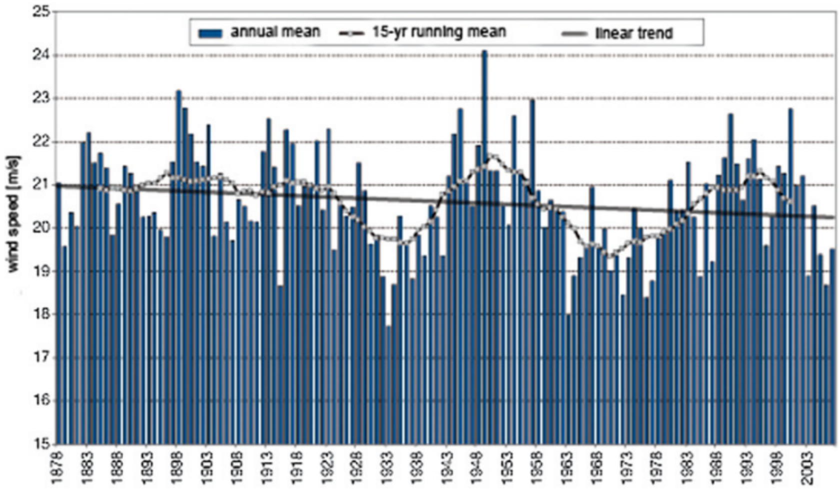


Figure 2.4: Annual average winter wind speed (in m/s) in the German Bight (North Sea) from 1878 through 2008, combined with a 15 year running mean and a linear trend. Retrieved from Weisse et al. (2011).

The other method applied to study future storm patterns is by means of computational models. Several model studies have concluded that the before mentioned meteorological phenomena seem to balance one another to a large extent (Haarsma et al., 2013). A study specifically based on the 2006 KNMI scenarios suggests there will be no change in north-west winds and only a minor increment of south-west winds (Sterl, van den Brink, de Vries, Haarsma, & van Meijgaard, 2009). The latter are known not to affect the storm surge levels in the southern North Sea due to the narrow trough of the English Channel.

A different study has however indicated that, while agreeing with a minor change in average wind speed, a larger number of hurricanes is to be expected to influence the water level in the North Sea. (Haarsma et al., 2013).

A later study confirms that the 99$^{\text{th}}$ percentile wind speed will increase for their meteorological models for two different SRES climate scenarios in combination with different control climates (Gaslikova, Grabemann, & Groll, 2013). The SRES scenarios are former emission projections created by the IPCC in 2000, but the range of expected emissions throughout the SRES scenarios is comparable to that of the before mentioned RCP scenarios (Nakićenović et al., 2000). Results of the study can therefore be considered relevant regarding the RCP scenarios. In the mentioned research, a link is made between the increasing extreme wind speeds and storm surges in the North Sea. These are presented in Figure 2.5. The referenced locations of the study do not include the Dutch coastline, but give an indication of the extent of storm surge change and variability over different climate scenarios. A maximum increment of 11 cm is predicted for SRES scenario A1B_1, which is most in line with RCP6. In the research a 95% confidence interval is claimed of approximately 6 cm for most projections, settling a range of 5 cm to 17 cm of the 99$^{\text{th}}$ percentile storm surge increment. It should be noted that no extreme prediction alike RCP8.5 has been studied, meaning storm surge developments could be worse then presented.

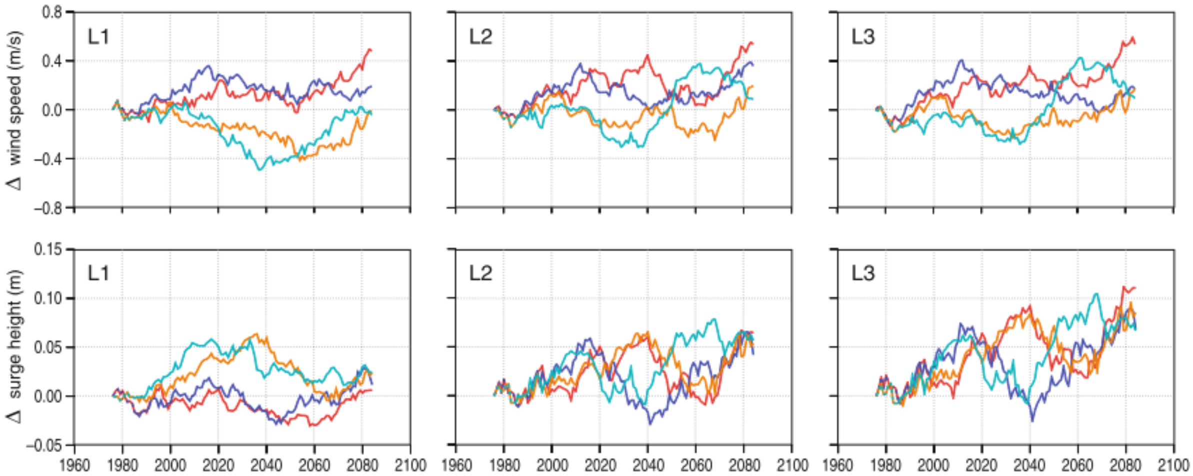


Figure 2.5: Projections of the change in 99$^{\mathrm{th}}$ percentile wind speed (top) and storm surge (bottom) in a location at the far northwest North Sea (L1) and two locations at the German Bight (L2 and L3) for SRES scenarios A1B_1 (red), B1_1 (dark blue), A1B_2 (orange), and B1_2 (turquoise). Retrieved from Gaslikova et al. (2013).

Wave height growth
Varying storm tracks will not only change the intensity of storm surges, but also alter wave characteristics. As waves are essentially initiated by wind, a similar change in wave intensity can be expected as for the storm surges. Several efforts have been made to model the North Sea wave climate regarding climate change projections. Figure 2.6 presents the results from a simulation for a variety of SRES climate scenarios (Grabemann, Groll, Möller, & Weisse, 2013). The results of the study show a large variation in wave development throughout the different SRES scenarios and the geography of the North Sea. Also different initial conditions and local climates have been taken into consideration, resulting in large differences per scenario. For the Dutch coast however, the model mostly suggests an increment of the yearly maximum wave height.

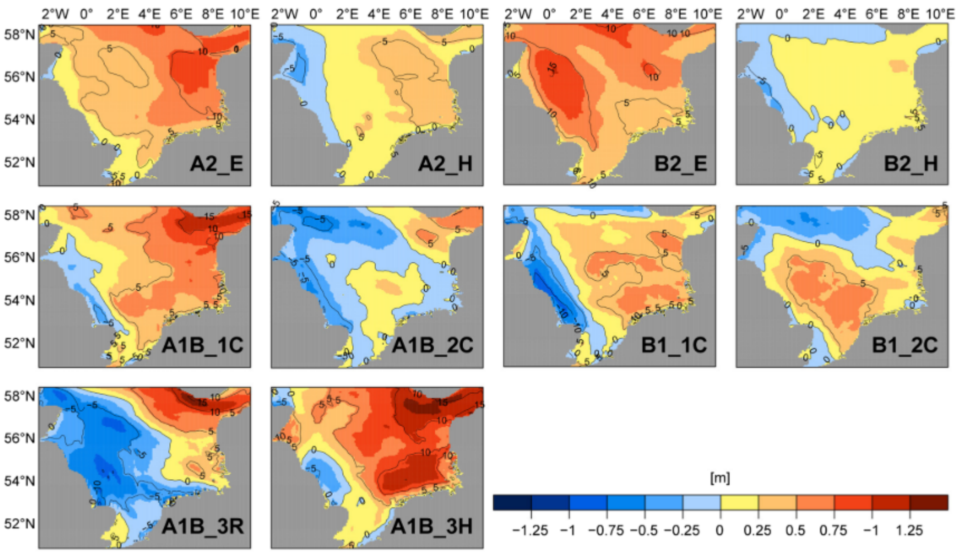


Figure 2.6: Modelled prediction of distribution of 30-year averaged highest yearly wave height in the North Sea in 2071-2100 relative to 1961-1990 for a variety of SRES scenarios. Retrieved from Grabemann et al. (2013).

The uncertainties portrayed by the results in Figure 2.6 are both reinforced and opposed by other studies.

While there are models and scenarios resulting in increased maximum wave heights (Weisse et al., 2011), other models give more positive predictions. A model for SRES scenario A1b (a moderately positive scenario) concluded in virtually no change of the maximum yearly wave height, but a decrease in maximum wave period for the Dutch coastline (De Winter, Sterl, de Vries, Weber, & Ruessink, 2012). A recent study based on RCP8.5 has resulted in a prediction of a decrease of both the 95$^{\text{th}}$ percentile significant wave height and 95$^{\text{th}}$ percentile significant wave period in the southern North Sea in the year 2100.

In conclusion, there are many possible different models which can be applied to predict the future wave climate of the Dutch coast, providing a range of outcomes per climate scenario, either positive or negative. This further underlines the uncertainties of the ocean developments due to climate change.

River discharge fluctuation

In addition to the sea level, the river water level is a defining parameter in the flood protection of a lock. Due to the accelerated global warming, the intensity and timing of precipitation, evaporation, and snow melting may change significantly. These factors largely influence the discharge which reaches the Dutch river system, and therefore the river levels to be expected at the marine locks.

Inflow of water into the Dutch river system is dictated by the discharge of two rivers: the Meuse and the Rhine. The Rhine and Meuse have very different characteristics, such as their origin, length, damping features such as lakes and flood planes, and adjacent dyke systems. To understand how the water level at the Dutch navigation locks will change, the developments of these rivers must be studied. Deltares has performed an analysis in order to create projections of the discharge of both rivers based on five KNMI'14 climate scenarios: GL, GH, WL, WH and WH$_{\text{dry}}$ (Klijn, Hegnauer, Beersme, & Sperna Weiland, 2015). The difference between these scenarios is either a moderate (G) or warm (W) climate, and a low change in air circulation (L) or a high change in air circulation (H) (Klein Tank, Siegmund, & Van den Hurk, 2014). The scenario WH$_{\text{dry}}$ was later added as the precipitation decrease that follows from climate change was underrated in the initial scenarios (Lenderink & Beersma, 2015). How these scenarios fit within the RCP scenarios is presented at the left in Figure 2.7. The rivers will therefore be differently affected by a changing climate. Below, the changes in average discharge through these rivers which were found in the Deltares study are discussed. The development of extreme river water level conditions is analysed in Appendix A.

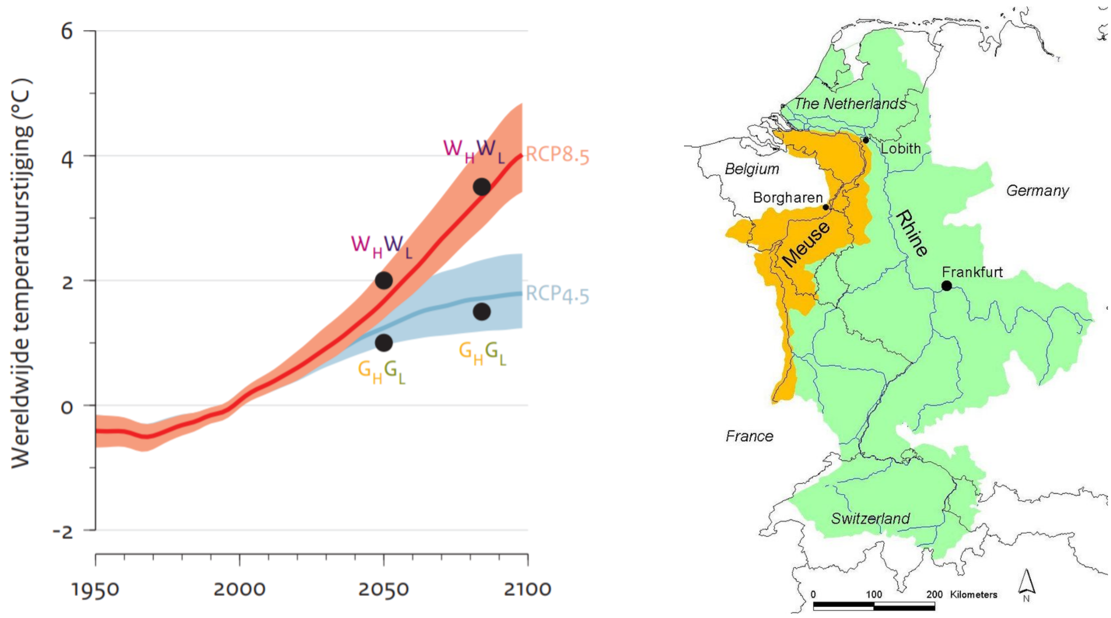


Figure 2.7: LEFT: Fitting of KNMI'14 scenarios within RCP scenarios by means of the global temperature change over time. Retrieved from Klein Tank et al. (2014). RIGHT: River basins of the Rhine (green) and Meuse (orange). Retrieved from Kwadijk (2007).

Figure 2.8 presents projections of the monthly averaged discharge of the Rhine at Lobith (the location where the river enters the Netherlands) in 2050 and 2085. The current fluctuation pattern (in black) shows a relatively consistent yearly river flow in comparison to the Meuse (see Figure 2.9). This is caused by the large river basin of the Rhine, the lakes which act as a damping factor for the river flow, and delayed melting of the snow in the Swiss Alps. These aspects contribute to a dispersion of discharge throughout the year, and only a minor seasonal deviation (Klijn et al., 2015).

This behaviour could however alter drastically under the influence of climate change, as can be observed from Figure 2.8. There is a very distinctive difference between the discharge variations per scenario. For the winter period, a range of discharge increment can be seen of approximately 20% (GH and GL) to 40% (WH) between the scenarios by the year 2085. In the summer period, a range of 0% (GL) to 30% ($\mathrm{WH_{dry}}$) reduction is presented.

The Meuse is contrarily a less complex river. It has a much smaller river basin (presented on the right in Figure 2.7), lacks the extent of damping features that the Rhine possesses, and originates from far less elevated ground than the Alps, and therefore receives less smelt water. This results in a more defined seasonal discharge pattern than the Rhine, as can be observed from the black line in Figure 2.9. This figure additionally shows that in the winter, an increment between 10% (GL and $\mathrm{WH_{dry}}$) and 25% (WH) is to be expected, and in the summer a reduction range of 0% (GL) and 60% ($\mathrm{WH_{dry}}$).

Not only do these projections show the severity of the changes that could occur, the results are also very diverse, demonstrating the high uncertainty of the issue.

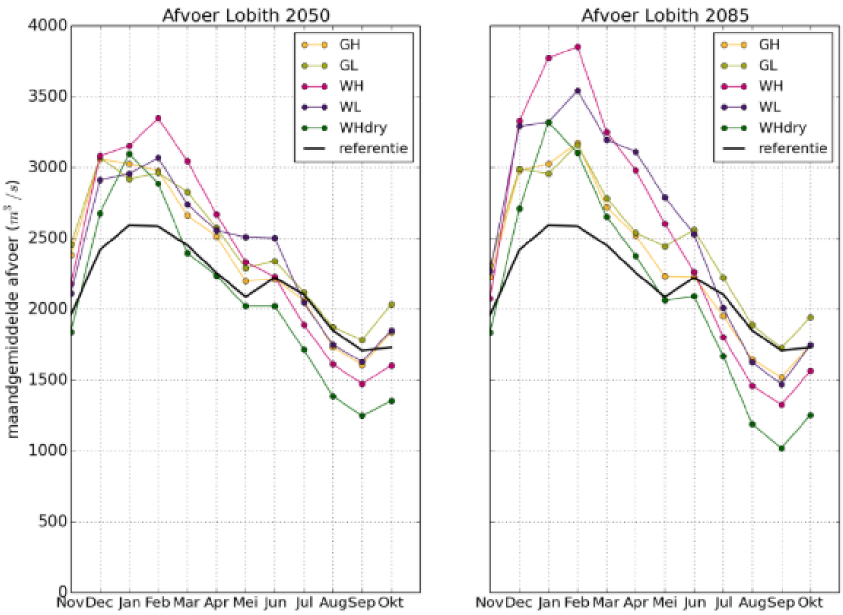


Figure 2.8: Projections monthly averaged discharge (in $\mathrm{m^3/s}$) of Rhine at Lobith in 2050 and 2085 per climate change scenario. Retrieved from Klijn et al. (2015).

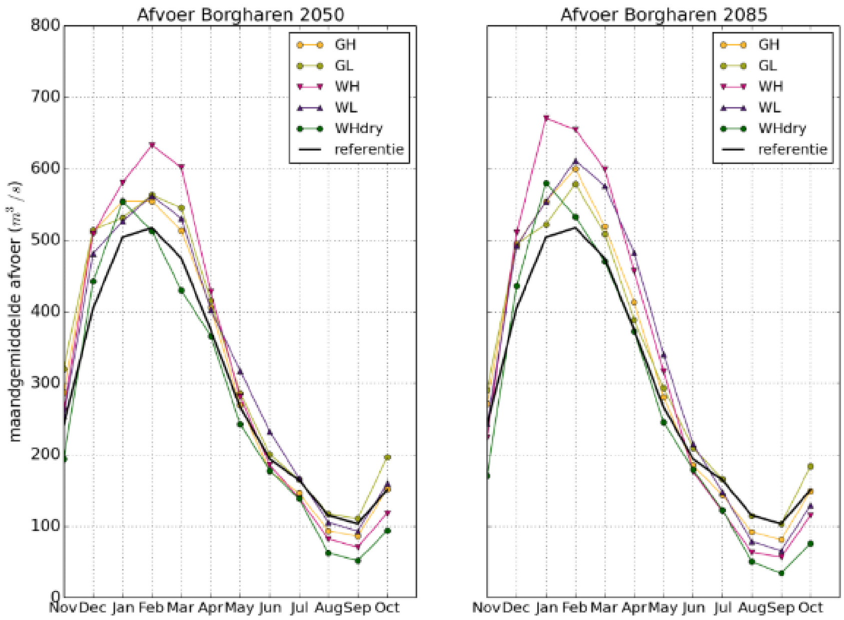


Figure 2.9: Projections monthly averaged discharge (in m^3/s) of Meuse at Borgharen in 2050 and 2085 per climate change scenario. Retrieved from Klijn et al. (2015).

2.2. Sea level rise integration in lock design

Selecting a correct sea level rise for a lock design is a difficult but very important task. This sole parameter will largely determine the effectiveness and efficiency of the design over its lifetime. On one hand, it may seem economically profitable to restrict the height as much as possible, as less material would be used for the construction. On the other hand, in case the height reveals not to be sufficient due to an unforeseen sea level rise, the lock might have to be adapted, or maybe even be replaced in its entirety, more than doubling the costs of the original replacement.

Adding to the difficulty of the issue is the large uncertainty surrounding all climate change driven developments, as described in Section 2.1.2. This uncertainty originates from three sources: the lack of knowledge on nature's response to the heating of the global and local average temperature, the societal and political development in human's effort to reverse their emission patterns, and the uncertainty of the computational models.

2.2.1. Options in choosing a sea level rise prediction

Two decisions are to be made to determine the design water level rise for a lock. The first is the scenario, implying the prediction of human emission over the years to come. Currently the RCP-scenarios are most commonly used (Nakićenović et al., 2000). Of those projections, RCP2.6 is the most optimistic and RCP8.5 the most pessimistic (Van Vuuren et al., 2011). Choosing an optimistic scenario can likely lead to an insufficient structure which has to be adapted or replaced long before its desired lifetime, while on the other hand adopting a pessimistic scenario can lead to overdimensioning and wasting large sums of money.

The second decision to be made is the likelihood of occurrence within a scenario. Per scenario the sea level rise is still very uncertain, due to a lack of knowledge on nature's physical response to an increased global and local temperature, and the inaccuracy of the computational models trying to mimic those responses. Over a span of 100 years, this uncertainty can make a large difference between the median of a scenario and its 95% likelihood (Oppenheimer et al., 2019).

How these factors can be used to determine the design sea level rise of a lock replacement is presented here by means of Figure 2.10 and the steps listed below. In this example the decision is made to design a lock for RCP8.5 and a 95% likelihood of non-exceedance. Important to note that this is one of many projections made in the previous years, and should not be conceived as a definitive range of possible futures. This particular prediction is used here because it reaches far into the future.

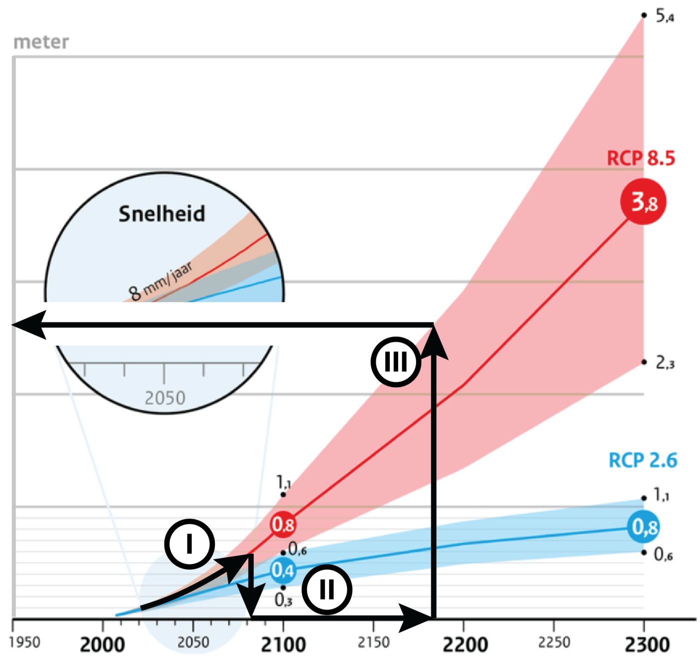


Figure 2.10: Steps to determine design sea level rise for a lock. RED: range between p5 and p95 for RCP8.5. BLUE: identical range for RCP2.6. Retrieved from Le Bars (2019) (edited).

(I) Time will pass until the sea level rise has aggravated to the point at which replacement of the lock is required. For this example, this is at 0.6 m sea level rise since the year 2000. The year at which this is reached is not linked to any projection in this figure, but to the actual development of sea level rise which would occur, and is chosen arbitrarily for the sake of this example.

(II) A number of years before the sea level rise has reached 0.6 m, design of a new lock will be initiated. Upon the year at which the replacement will be executed (2082 in this example), a lifetime of 100 years is added (2182).

(III) From the projection (which is likely to have changed by the year 2082), the sea level rise is read for the requested scenario and likelihood (RCP8.5 and 95% in this example) by the end of the lifetime of the new structure. In this case the lock would have to be designed to withstand 2.6 m of sea level rise.

2.2.2. Current policy Rijkswaterstaat

At the moment, when designing a lock, the policy of Rijkswaterstaat is to implement the most extreme degree of sea level rise covered by the projections at the time of design. The demanded lifetime of a lock is 100 years, and fulfilling this goal is regarded more as a necessity than an ambition (Rijkswaterstaat, 2010). Whether this strategy is the best option is debatable.

Figure 2.11 presents how a conservative design may be economically less sufficient than an optimistic design, regarding the exponential rise of the sea level. The figure originates from a letter from the “Expertise Netwerk Waterveiligheid” (Expertise Network Water Safety) to the Dutch Ministry of Infrastructure and Water Management, in which advise is provided on how to deal with sea level rise in drafting design criteria. What can be seen is that the years of lifetime lost when climate change becomes more severe than has been assumed in the design, are far less than the years gained in lifetime in case the opposite happens. Initially it may not seem as an issue that years of lifetime are gained, but it is important to understand that this is merely the lifetime with respect to the sea level, and not the general lifetime of a structure. In the design of a hydraulic structure, many different aspects determine the general lifetime, for instance the traffic capacity of a lock. When those other aspects require a replacement of the structure, the additional years in lifetime with respect to the sea level rise are wasted, and costs have been lost to overdimensioning.

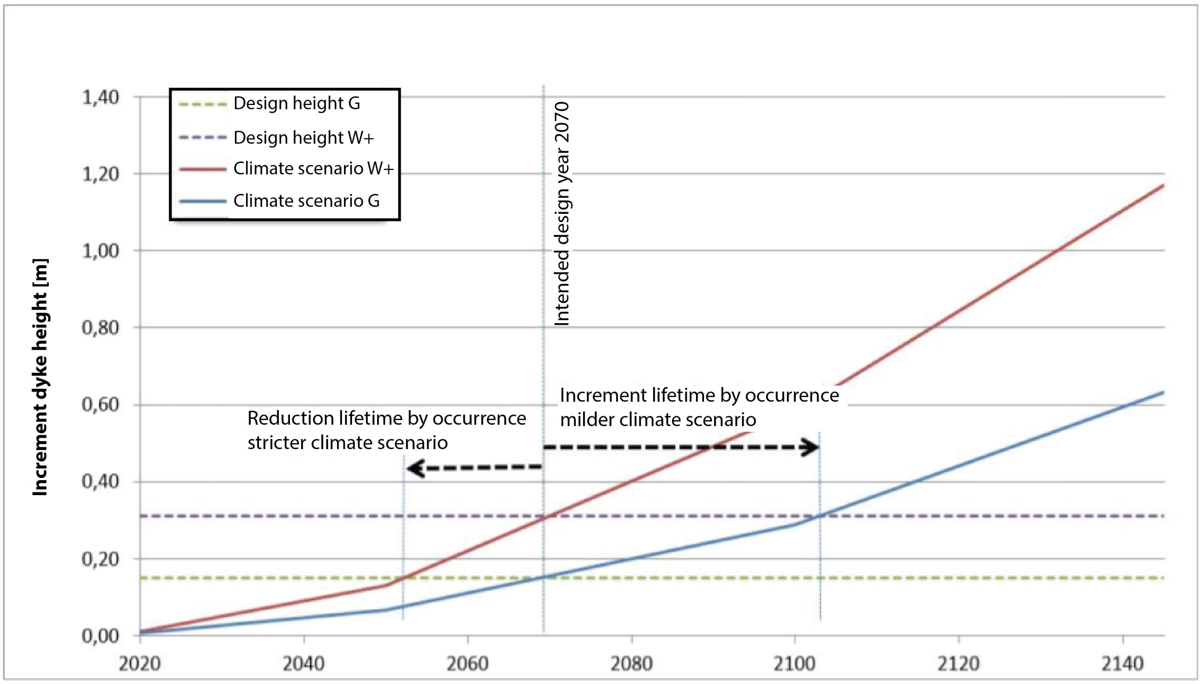


Figure 2.11: Depiction of lifetime reduction and increment in case of the occurrence of a different scenario than the one integrated in a design. Retrieved from Expertise Netwerk Waterveiligheid (2019) (edited).

However, in this letter it is also mentioned that for a structure as intricate as a lock, it is probably still beneficial to implement a conservative sea level rise. If the design is no longer sufficient, adaptation or replacement is probably more expensive than the costs saved by implementing an optimistic sea level.

Nonetheless, this shows that the most conservative alternative is not in all cases the optimal one, which encourages to think about alternatives. The intricacy of the lock structure is apparently a key aspect in economical decision making, but perhaps the intricate components can be avoided by means of adaptive or flexible constructions.

2.3. Inventory Dutch marine locks

In the previous sections it has been presented that the future of the Dutch marine locks may not be certain due to climate change driven water level extremes increasing the probability of the failure mechanisms. To understand the magnitude of the issue that is being faced in the upcoming decades, it is also important to have primary knowledge on the amount and variety of marine locks there are in the Netherlands.

In this section, a brief inventory is provided of the major characteristics of the Dutch marine locks. All locks are regarded which are directly in contact with the water level of the sea. Locks behind the Eastern Scheldt barrier and Haringvliet dam are also taken into consideration, as these have to retain the sea level in case the barriers are opened. These locks are however unlikely to be subjected to extreme sea levels, as the barriers will then be closed off. The number of chambers per lock, chamber dimensions, and gate type are presented in Table 2.1, and their locations are depicted in Figure 2.12. All information was retrieved from the website “Vaarwegeninformatie”, property of Rijkswaterstaat (n.d.-d).

Table 2.1 shows there are 25 marine lock complexes in the Netherlands, composed of 38 lock chambers. Adapting or replacing all of these locks to withstand more extreme water levels could be a big challenge. In addition, there are many other locks located in river sections which will be influenced by the sea level, but are not in direct contact with the sea (and thus not regarded as marine locks). This further underlines the need to find more flexible and convenient solutions. There are several types of gates in use, and the dimensions of the chambers vary significantly. A large variation in lock properties could mean that not all locks will be appropriate for identical solutions to the climate change issue, further complicating the problem.

Table 2.1: Inventory Dutch marine locks. Retrieved from Rijkswaterstaat (n.d.-d).

	Complex	Chamber	Width [m]	Length [m]	Gate type	Location
1	Terneuzen	Western	38	290	Roll	Coast
	Terneuzen	Middle	24	140	Mitre & roll	Coast
	Terneuzen	Eastern	24	280	Mitre	Coast
2	Vlissingen	Nr. 1	23.4	140.7	Mitre	Coast
	Vlissingen	Nr. 2	8	64.8	Mitre	Coast
3	Hansweert	Nr. 1	24	280	Roll	Coast
	Hansweert	Nr. 2	24	280	Roll	Coast
4	Bergsediepsluis		6.5	37	Pivot	Eastern Scheldt
5	Goesche Sas		10	86	Mitre	Eastern Scheldt
6	Zandkreeksluis		20	152	Mitre	Eastern Scheldt
7	Roompotsluis		16	100	Roll	Coast
8	Krammersluizen	Nr. 1	24	285	Roll	Eastern Scheldt
	Krammersluizen	Nr. 2	24	285	Roll	Eastern Scheldt
	Krammersluizen	Nr. 3	9	80	Pivot	Eastern Scheldt
9	Grevelingensluis		16	139	Mitre	Eastern Scheldt
10	Volkeraksluis	Nr. 1	24.1	331.5	Mitre	Haringvliet
	Volkeraksluis	Nr. 2	24.1	329	Mitre	Haringvliet
	Volkeraksluis	Nr. 3	24.1	329	Mitre	Haringvliet
	Volkeraksluis	Nr. 4	16.1	142	Mitre	Haringvliet
11	Middelharnis		15	29	Mitre	Haringvliet
12	Hellevoetsluis		25	40	Mitre	Haringvliet
13	Goereesluis		16	144.5	Mitre	Coast
14	Hartelsluis		24	280	Mitre	Coast
15	Rozenburgsluis		24	306.4	Mitre	Coast
16	IJmuiden	Southern	18	100	Mitre	Coast
	IJmuiden	Middle	25	200	Mitre	Coast
	IJmuiden	Northern	47.3	400	Roll	Coast
17	Zeedoksluis		20.1	42	Mitre	Coast
18	Koopvaardersschutsluis		16	86	Mitre	Coast
19	Stevinsluis		13	120	Mitre	Coast
20	Lorentzsluizen	Nr. 1	13	120	Mitre	Coast
	Lorentzsluizen	Nr. 2	8.2	67	Mitre	Coast
21	Tsjerk Hiddessluis	Nr. 1	12	135	Mitre	Coast
	Tsjerk Hiddessluis	Nr. 2	7	49.5	Mitre	Coast
22	Robbengatsluis		9.1	47.6	Mitre	Coast
23	Farmsum	Nr. 1	7	123	Mitre	Coast
	Farmsum	Nr. 2	16	174	Mitre	Coast
24	Termunterzijldiep		8	50	Mitre	Coast
25	Nieuw Statenzijl		8.6	67	Pivot	Coast

Figure 2.12: Locations Dutch marine locks. Retrieved from *https://heloohaloo.blogspot.com/2018/10/74-mooi-blanco-kaart-van-nederland.html* (edited).

2.4. Problem Statement

It is uncertain with what loading conditions the Dutch marine locks have to cope in the upcoming decades. Climate change is causing the sea level to rise and river discharge to become more extreme. The most recent projections show a large uncertainty for both aspects. The locks have not been designed on these altering conditions, raising the question whether adaptation or replacement of the locks will be necessary in order to ensure the flood protection of the hinterland.

Thus far, there has not been much investigation on this topic. It has not been well studied how the complex system of failure mechanisms is influenced by these factors, and there has therefore not been much thought as to how solutions can be applied. The relevancy of these facets is however constantly gaining, and action is better to be taken prematurely than late. The water network of the Netherlands is protected by a significant number of marine locks, and strengthening the entire system is a big task which requires innovative thinking to assure the flood safety and avoid unnecessary expenses.

3

Objective and Methodology

Sequentially to the problem statement, an objective and scope are defined for this thesis, followed by the method which will be applied to reach the objective.

3.1. Objective

The objective of this report is to develop a method to support decision making on adapting and replacing locks while taking the uncertainty of sea level rise into account. Two decisions are specified which can be considered when a lock no longer suffices to its functional or structural requirements. Initially the choice has to be made whether to prolong the lifetime of a lock by means of adapting the lock characteristics which are most critical regarding sea level rise, or to replace the entire lock with a variant in which those characteristics are improved (also allowing any other property to be improved). Secondly, when the choice is made to replace a lock, it has to be decided how the uncertainty of sea level rise is implemented in the design of the lock. This can be done conservatively, considering a severe sea level rise scenario, or adaptive, applying a moderate or mild scenario in the design requirements, but assuring the possibility of easy adaptation in the future in case severe sea level rise develops. To be able to develop support for these two decisions, first the influence of sea level rise on the failure probability of a marine lock has to be analysed. From this can be derived what characteristics require improvement (either by adaptation or replacement). This thesis will thus essentially consist of three elements: an initial research towards the influence of sea level rise on the failure probability of a lock, the development and evaluation of adaptation concepts, and the development and evaluation of replacement concepts.

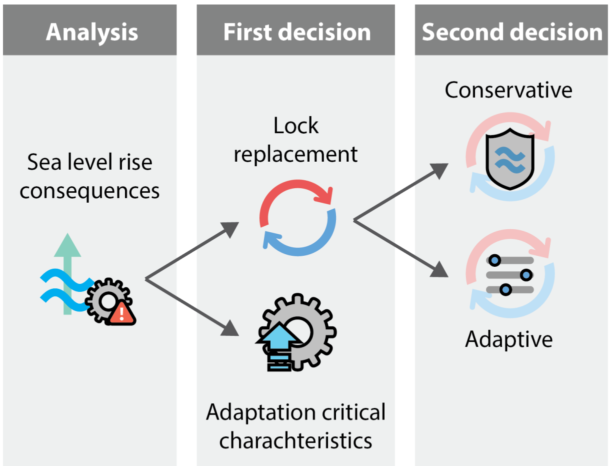


Figure 3.1: Flow chart of thesis elements

The primary step is to analyse how the failure probability of a lock will be altered when the sea level rises due to global warming. Thus the extent of the issue is determined, resulting in knowledge on the critical characteristics of a lock at which the reliability of a lock will most likely be jeopardised first, and at what water levels the structures can be expected to fail. Although the water levels on the river side of locks are also expected to become more extreme in the upcoming years, this is left out of the scope of this thesis. Focus lays on the sea level rise, regarding locks located at the entrance of regulated channels, for which the water level is assumed not to change. Thus, the following research question can be answered, which can also be divided into sub-questions:

1. *What are critical lock characteristics of a marine lock which have to be improved first in order to maintain a sufficient flood protection function with respect to a rising sea level?*
   - 1.1 *What failure mechanisms of a marine lock are relevant in the assessment of the effect of sea level rise on the sufficiency of its flood protection function?*
   - 1.2 *To what extent does sea level rise alter the flood protection failure probability of a marine lock relative to its legal norm, and how does the development of each individual failure mechanism contribute to this advancement?*
   - 1.3 *What lock characteristics are critically related to the failure mechanisms which contribute significantly to the development of the failure probability?*

Knowing what the exact consequences will be for marine locks, and what characteristics are to be improved to counteract this, solutions can be developed. In the second stage of this thesis, adaptation concepts are developed and compared to one another. The application of the optimal adaptation concepts is then compared to an immediate lock replacement. This part of the thesis is captured in the following design question, and sub-questions:

2. *Is it effective to prolong the lifetime of a lock by means of an adaptation, instead of directly replacing the lock entirely?*
   - 2.1 *In what manners can the critical characteristics be improved by means of an adaptation?*
   - 2.2 *What is per critical characteristic the optimal adaptation alternative, based on their value and costs?*
   - 2.3 *How does the implementation of the optimal adaptations relate to a replacement, based on time dependent progression of their value and costs?*

At last, a similar process is applied to replacement concepts. Here altering replacement designs are developed and compared to one another based on the progression of their costs and value during their lifetime. The section is framed by the following design-question and sub-questions:

3. *Is it effective to replace a lock with an adaptive alternative, rather than a conservative one, when facing an uncertain sea level rise?*
   - 3.1 *What are possible design deviations from a regular conservative lock which can be applied to help against the uncertainty of sea level rise?*
   - 3.2 *How do the costs and value of these alternatives compare in their initial state, and their adapted state which is applied in a severe sea level rise scenario?*

3.2. Scope

A scope is defined to clearly specify the limits of this project.

- Only locks in **the Netherlands** will be regarded. Global warming consequences will be geographically inconsistent, so a decision has to be made on the investigated location.
- Only **marine locks** will be assessed. Loading conditions are expected to change very differently for marine locks and inland locks. The distinction is thus made to provide explicit conclusions on the matter.

- Only **sea level rise** is regarded, and no changes to the water level on the other side of a lock, or changes to the occurring wave heights. Exclusively marine locks which are located at a channel with a regulated water level are taken into consideration, implying no changes in the water level distribution of the channel.
- For this study, only the improvement of **existing locks** will be investigated. No assessment will be made of design choices for new structures.
- Solely the **flood protection function** of the locks will be studied, disregarding any consequences for the navigation capacity, water management, crossing traffic, or ecology.
- It will be assumed that the sea dykes forming the coastal protections will not flood, and maintain their structural integrity. The dykes adjacent to the river are presumed not to be altered in dimensions.

3.3. Methodology

The methodology consists of three stages: failure probability research, adaptation concept development and evaluation, and replacement concept development and evaluation. It is expected that the execution of these steps will provide a different outcome depending on the regarded lock, as the Dutch locks show a large variety in properties. As the thesis is limited in time, not all Dutch marine locks can be studied. Throughout the thesis, the Western lock in Terneuzen is used as case study. Information on this lock is readily available, as it was also used by Kemper (2019).

1. Failure probability development analysis

The first stage in the design process is a research on the increment of probabilities of the failure mechanisms of the locks. This is performed using the equations provided by the WBI 2017 regulation (De Waal, 2019). Initially a selection of failure mechanisms is made, based on the influence that sea level rise has on their probability of occurrence. Consecutively, a failure probability model is made to study the development of the probability of the selected failure mechanisms. The product of this phase is an overview of the development of the failure probability of a marine lock, and the contribution of the individual failure mechanisms. From this, a number of critical characteristics of a lock can be selected which are to be improved in order to lower the failure probability sufficiently.

2. Decision to adapt or replace

Subsequently the scientific design method will be applied to develop adaptation concepts for the critical characteristics derived in the previous stage. This method consists of the composition of the base of design, the development of a variety of concepts, the verification of the solutions by means of the base of design, evaluation of the concepts based on their value and costs, and a selection. Throughout this process, the case study is applied to give insight in the application of the method.

After a selection of optimal adaptations, it can be analysed whether the application of these adaptations is preferable over the replacement of a lock. This is done using an adaptive pathways map. This is a method which projects possible time-dependent adaptive policies, derived from implementing different measures at different time slots (Haasnoot, Kwakkel, Walker, & ter Maat, 2013). This helps visualising and understanding the issue. Using the possible pathways in this map (i.e. the options to adapt or replace), the cost and value progression of the policies can be derived and analysed. At the end of this process, a recommendation is made whether to adapt or replace a lock which no longer suffices due to sea level rise.

A simple example of an adaptive pathways map is presented in Figure 3.2, demonstrating what this map could look like for this issue. Mitigation actions are presented on the coloured lines, and on the x-axis an inclining sea level rise is given. At the left, a choice is to be made between a method of mitigation when the current norm of a structure is surpassed. As time passes, the sea level will rise, and solutions may become obsolete. At this point a jump is made to a different solution. The map can thus be followed from left to right over several pathways. Each pathway has a certain value and comes with different costs for the solutions applied during the lifetime of the lock. These can be compared with one another, from which a decision can be made as to what pathway is optimal. It should be noted that because climate change in uncertain, the map is most likely not followed all the way to the right side. Thus, certain pathways could be beneficial if sea level rise does not become severe, but become insufficient in case it does.

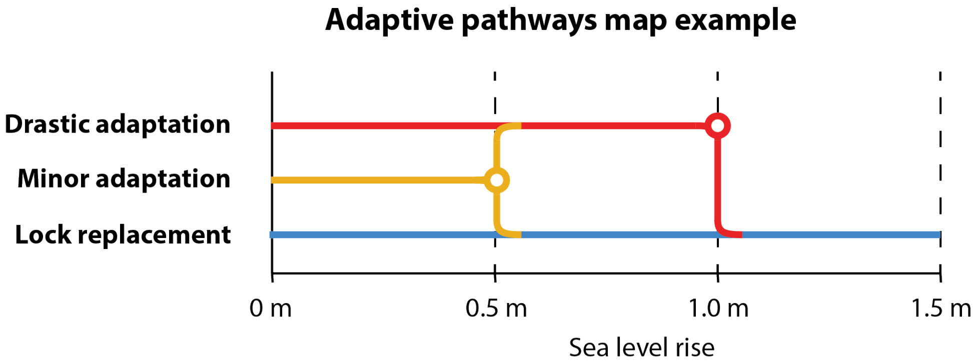


Figure 3.2: Example of an adaptive pathways map applied to the issue of this report

3. Effectiveness of adaptive replacement alternatives
In the last stage of the thesis, the effectiveness of adaptive lock alternatives is analysed. Thereto, a similar process is applied as in stage 2. The method consists of a base of design, the development of lock alternatives, the verification of those alternatives based on the base of design, the creation of an adaptive pathways map, and an evaluation of the pathways based on their value and costs. The effectiveness of an adaptive design in comparison to a conservative lock is time dependent, because it is influenced by the development of sea level rise. Therefore, the lock alternatives are not firstly evaluated based on value and costs without the aspect of time (which *is* done for the adaptation concepts). This means no selection of alternatives is made before the drafting of the adaptive pathways map. Again, the same case study is used to apply the replacements to. The stage is closed off with a recommendation on whether to apply any of the adaptive replacement alternatives over a conservative one.

3.4. Report structure

The report structure follows the structure of the design phases explained in the methodology. The first research question is treated in the first chapters, thereafter the second question, and thereafter the third. The structure is depicted by means of a flow chart in Figure 3.3. The chapters are coloured by the research question which they concern. Additionally, it is noted in which step the answer to which questions is derived.

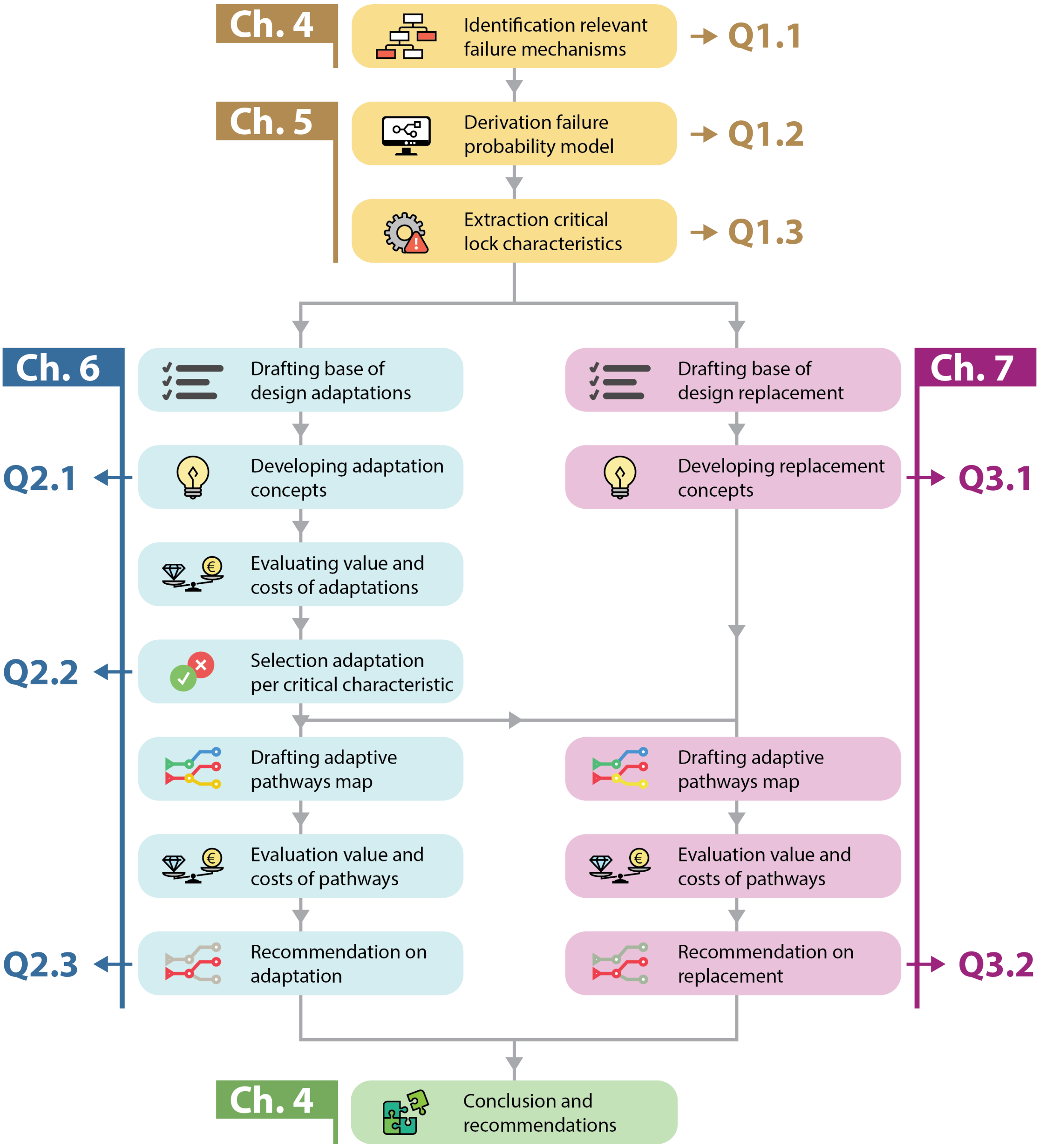


Figure 3.3: Flow chart of report structure

4

Influence sea level rise on failure mechanisms marine lock

This chapter explores how sea level rise can have an impact on the varying failure mechanisms of a marine lock related to its flood defence function. To do so, initially the manner in which a lock contributes to a flood protection system is depicted in Section 4.1. Sequentially, all relevant failure mechanisms and their causality are described in Section 4.2. At last, for each mechanism the significance and relevancy with respect to sea level rise is derived in Section 4.3, concluding in a set of failure mechanisms which are regarded throughout the remainder of this thesis.

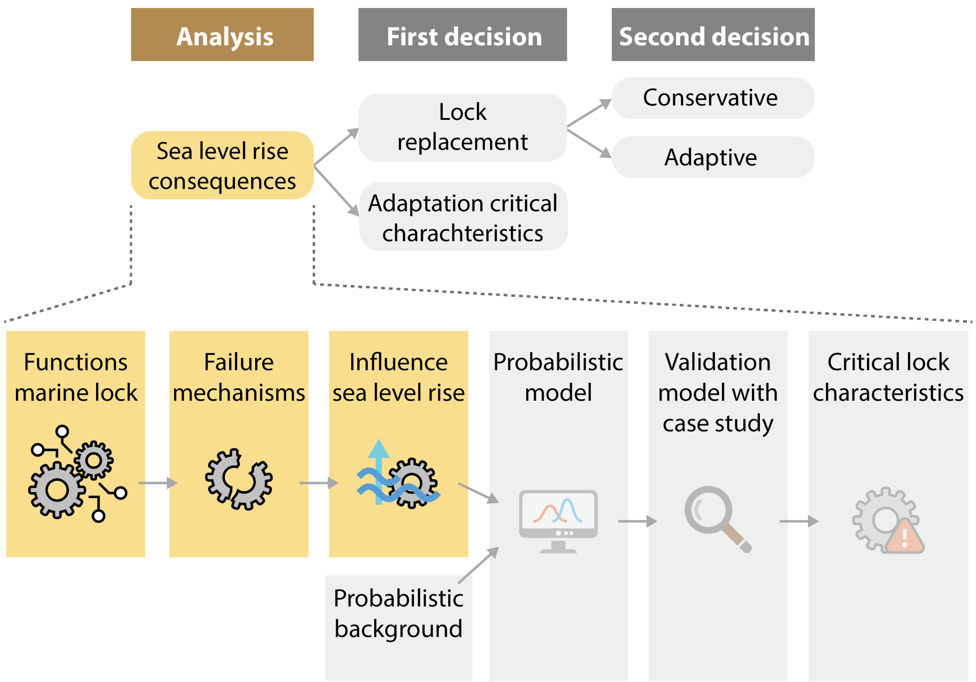


Figure 4.1: Flow chart Chapter 4

4.1. Function description marine lock

To explain the manners in which a marine lock can fail, it first has to be derived what the functions of a marine lock are. In this section it is described what the functions of a lock are, and how it executes those.

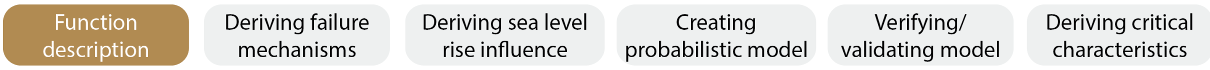


Figure 4.2: Chapter progression Section 4.1

Marine locks form the gateway between the oceans and the inland water system. The structures are both flood protection and waterway, and can even serve more purposes, according to the requirements of the site (Beem et al., 2000).

- Locks are meant to allow **navigation** of vessels between two bodies of waters with varying water levels. A lock therefore consists of at least two gates, between which a vessel waits until the water level between the gates is lowered or elevated through a filling- and emptying system.
- Since a lock is the gateway between two different water levels, it has a **retention** function. While some locks are part of a flood defence system, others simply allow passage between two differently elevated canals. Obviously for the former category the retention function is much more essential.
- Some locks also fulfill a **water management** function, either by limiting water loss, discharging excessive water, or separating water bodies with different concentrations of i.a. salt. This also includes the prevention of salt intrusion to a certain extent, in order to preserve ecological systems.
- Many locks do not only serve as a passage for vessels, but also **crossing** of "dry" traffic. Lock gates can be designed with integrated infrastructure on top, allowing people to traverse over the construction from one side of the passage to the other.

For this thesis only the flood protection related consequences of climate change will be further analysed. Although the other functions of locks might also be affected by global warming consequences, these do not concern the scope of this study. To understand how a marine lock performs its retention function, it should be established what role it plays in the flood defence system as a whole.

4.1.1. Lock as element of flood defence

Most of the Netherlands is protected from the river and ocean by means of dykes and dunes. Without these natural and man-made structures, extreme water levels would flow from the oceans and rivers into the hinterland and cause immense damages and lives. Storms above the ocean can carry extreme water conditions towards the coast. If these conditions could reach far into the rivers and channels, adjacent dykes would have to be built very tall for long stretches of the waterways, far into the main land. To avoid this, the coastal defence line is built to withstand extreme conditions very unlikely to occur. Because this first line of defence is so rigid, the structures behind can be constructed for less extreme conditions to meet equal safety norms.

However, inward and outward transport via the waterways does have to remain possible, requiring an opening in the line of dykes and dunes at the mouths of these rivers and channels. Therefore, marine locks are built. These structures allow passage of vessels, while retaining the water levels from both the ocean and the river side.

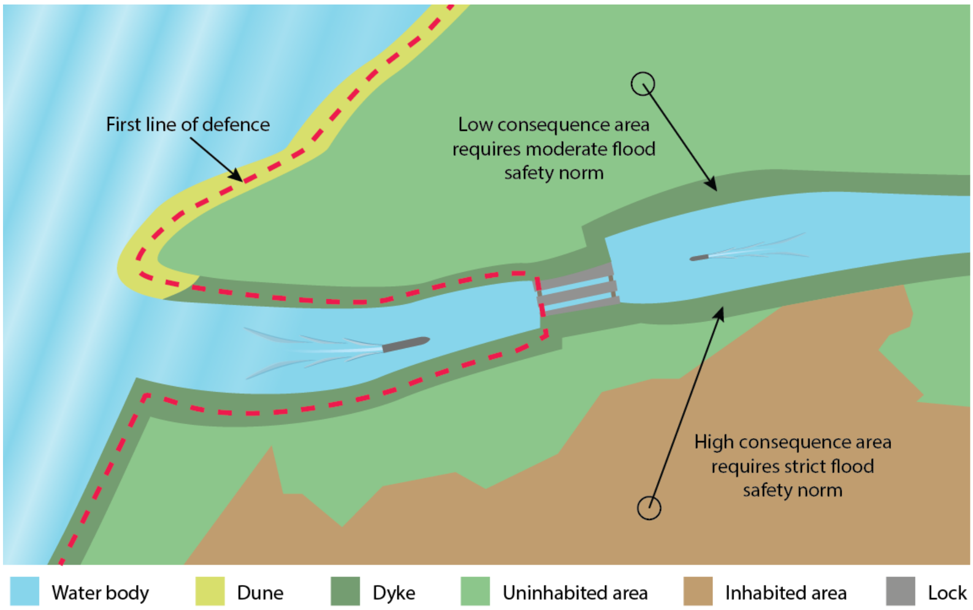


Figure 4.3: Marine lock in flood defence system

A flood protection system has a legally mandatory minimum for its failure probability. These norms have been included in the Dutch Water Act, and vary for the large collection of dyke and dune sections in the country (Dutch Ministry of Transport, Public Works and Water Management, 2017c). The norms are derived from the value and population of the the hinterland. The costs of the damages caused by a flood can become much higher for densely populated areas with more economic worth than a mostly empty region, and a higher number of lives is in danger. These areas therefore require a lower safety norm, following the standard risk equation (Vergouwe, 2015).

$$risk = probability \cdot consequences \tag{4.1}$$

The combination of the structures in a flood defence system must together maintain a failure probability below the legal norm. To ensure this safety, each structure is assessed around every 10 years using the WBI 2017 guidelines, created by Rijkswaterstaat (De Waal, 2019). Using these guidelines, the probability of each failure mechanism of the structures can be determined, eventually allowing the calculation of the total failure probability of the system. In case the requirement is not met, an adaptation or replacement of one or more of the components is in order.

Before exploring the manners in which a lock can fail its flood protection function, a brief description is required of the elements which compose the lock. By understanding the purpose of each component, it becomes more evident how these could fail at maintaining the flood protection of the hinterland.

4.1.2. General lay out navigation lock

Figure 4.4 presents the general lay out of a navigation lock. The structure is divided into five sections: an approach channel and lock head on both the upstream and downstream end, with in the midst the lock chamber. A vessel approaches the lock via the approach channel, where it will slow down and wait for the nearest gate(s) - situated in the lock head - to open. Once the passage is clear, it enters the lock chamber, where it again pauses. The gate(s) now behind the vessel close, and the lock chamber becomes completely separated from the channel. Thus, the water level inside the chamber can be levelled in a controlled manner. The water level is adjusted from by means of a filling- and emptying system, often through openings in the gates or a culvert system located in the lock heads and lock chamber walls. Once the water level is equal to that in the approach channel in front of the vessel, the gate(s) on this side of the lock can open, allowing the vessel to advance.

Throughout the remainder of this section, the components of a lock which are vital for the understanding of its failure mechanisms are briefly explored .

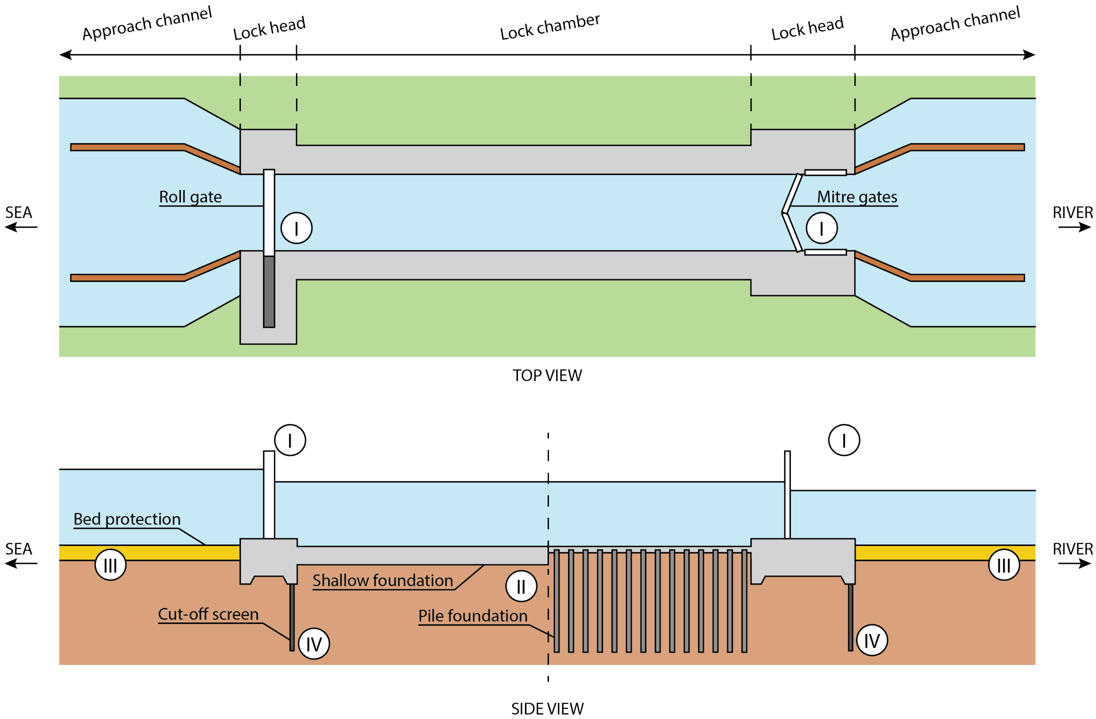


Figure 4.4: General lay out navigation lock

(I) Gates

The gates are located at the lock head. They are to retain water in closed condition, and allow passage in open condition. For the former, it must have sufficient strength to withstand the pressures provided by the water levels and waves. To alternate between an open and closed state, an operating mechanism is installed in the lock head (Beem et al., 2000).

For marine locks, mostly two types of gates are applied: roller gates and mitre gates. Roller gates (depicted on the left side in Figure 4.4) are often used for locks with large dimensions. From its open state, in which it is stored inside the lock head, it can be rolled outward over rails to the opposite side. The gate type can retain water from both sides, contrary to the mitre gate. The diagonal arrangement of mitre gates only allows one-sided retention, for which often two sets of gates are installed at a marine lock (as in Figure 4.4).

(II) Foundation

To assure the vertical, horizontal and rotational stability of the structure, a foundation is installed. This can either be a shallow foundation, consisting of a thick concrete slab, a pile foundation, or a combination of the two. Which type is most efficient to be applied depends on many factors, amongst which the type of subsoil and expected forces during installation and the lifetime of the structure.

(III) Bed protection

Water will flow in and out of the lock chamber by means of its filling- and emptying system. Depending on the flow velocity, grains of the subsoil can be lifted up and transported elsewhere, evidently resulting in the development of a scour hole at the toe of the structure. In case the scour hole becomes deep enough, the soil beneath and in front of the structure will slide, causing instability of the lock. To prevent a scour hole from forming within dangerous distance of the lock, a bed protection is placed, consisting of layers of rocks which are large enough not to be lifted by the expected occurring flow velocities. A scour hole will still form at the toe of the bed protection, but too far away from the lock to endanger its stability.

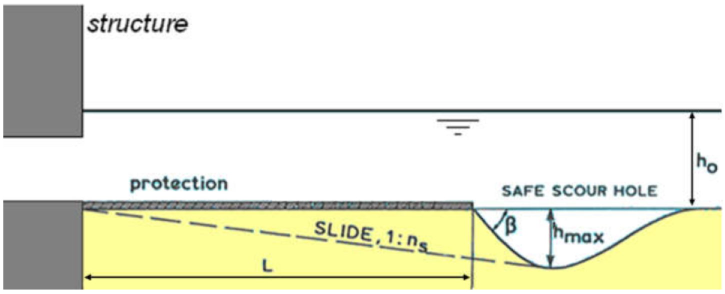


Figure 4.5: Development of scour hole at safe distance from structure. Retrieved from Molenaar and Voorendt (2018).

(IV) Cut-off screen
Cut-off screens are placed beneath a structure to obstruct piping from occurring. Piping is the mechanism of water seeping underneath a structure from one side to the other. This can cause erosion of the bed protection, and can thus jeopardise the stability of a lock. Seepage is driven by a difference in pore pressure, which for a lock is mostly related to the water levels on each side. Another main factor of the process is the length over which water has to transport from a body with a higher pressure to a body with a lower pressure. In case this seepage path is too long, In case a lock is not sufficiently long enough to stop piping from occurring, cut-off screens can be installed to lengthen the seepage path.

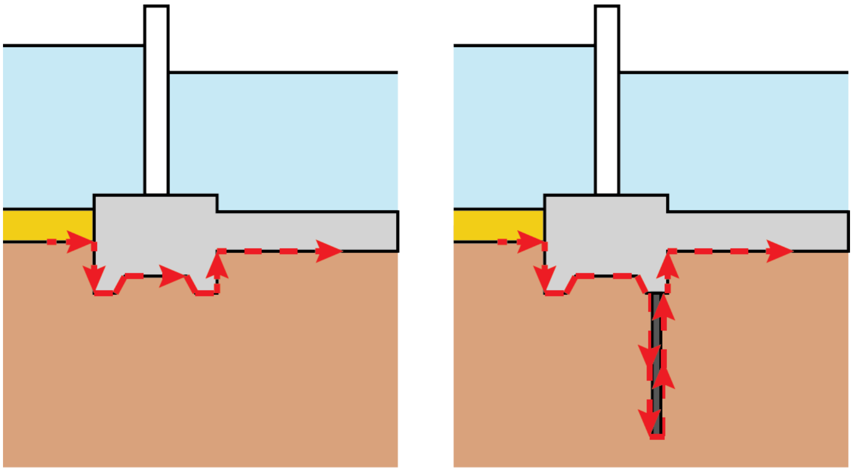


Figure 4.6: Seepage path with and without cut-off screen

4.2. Flood protection failure mechanisms of marine lock

Understanding how the components of a lock contribute to its functions, and how the lock functions within a flood defence system, it can be derived how a lock can fail at its flood protection function. Throughout this section, the basic failure mechanisms of a marine lock that lead to flood protection failure are explicated.

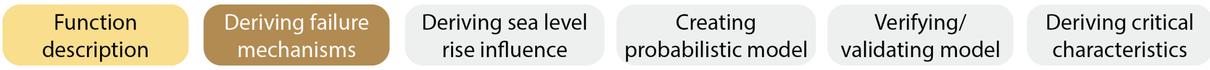


Figure 4.7: Chapter progression Section 4.2

To derive the failure mechanisms of the flood protection function of a marine lock, the WBI manuals are used as reference (De Waal, 2019). These are guidelines from Rijkswaterstaat used to assess whether hydraulic structures require a adaptation or replacement. Therefore, these govern the perspective on the flood protection function of a lock in this thesis. In addition, the assessment of the IJmuiden lock complex is referenced to address how the WBI manuals are used in practise (Van den Berg et al., 2019).

4.2.1. Sub-mechanisms of flood protection failure

There are in principle two mechanisms which can cause water to pass the lock: retention failure of the gates, and structural instability (De Waal, 2019). The latter can cause a translation of the entire structure, leaving room for water to freely enter the system. This can be caused by piping or a head difference which has not been anticipated in the design. Retention failure of the gates, however can be subdivided into many other failure mechanisms, which make a complex and large fault tree. For the sake of simplicity, only the top of the fault tree is presented in Figure 4.8. The layers below "Retention failure gates" are derived throughout the remainder of this section. A complete overview of the fault trees related to flood protection failure is provided in Appendix B. A thorough description of how the probability of each failure mechanism can be derived is provided in Appendix C. For most mechanisms, this includes a limit state function, which can be used in a Monte Carlo analysis (see Section 5.1.2 and 5.1.3). For other mechanisms, a probability of occurrence can directly be derived from generally applicable statistics and assumptions. Table 4.2 provides a brief description of each mechanism and the parameters on which they depend.

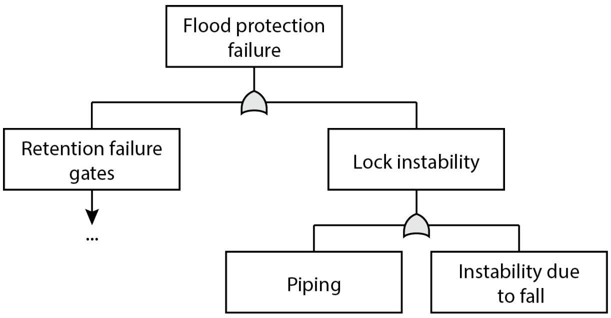


Figure 4.8: Top of fault tree "Flood protection failure"

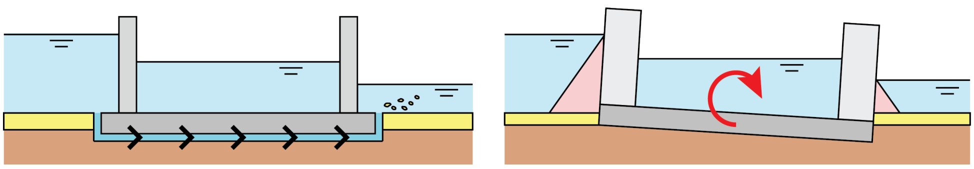


Figure 4.9: Piping and instability of the lock structure

4.2.2. Retention failure gates

The causes and consequences of gate failure form an intricate system of processes. Depending on the amount of gates that have failed and the manner in which they have failed, a different probability of overflow or inflow arises. In addition, the manner in which one gate fails can either exclude or entail a manner in which the other gate fails. Nonetheless, all gate failure mechanisms can result in water flowing into the system behind the lock. This can have a negative effect on the flood protection in two manners: erosion of the bed protection and filling of the storage capacity (Rijkswaterstaat, 2019b). The interplay between these steps is explored in the remainder of this section.

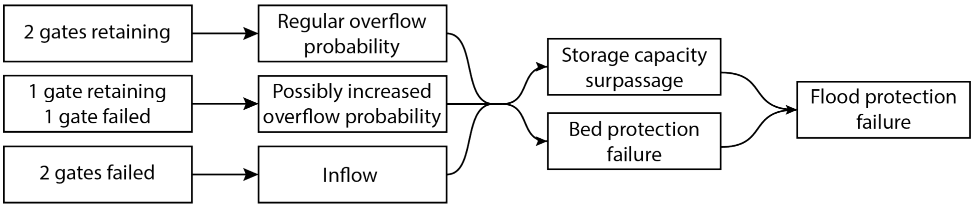


Figure 4.10: Flow chart of gate failure consequences

Gate failure
There are in principle four manners in which a gate can lose its retention function, as listed below (Rijkswaterstaat (2019b) & Rijkswaterstaat (2019c)):

A **Failure due to head difference**: In case the difference between the inner- and outer water level becomes larger than has been accounted for in the design, the gate can fail structurally. For further explanation, see Appendix C.4.

B **Failure due to vibration**: If water flows over a gate, this can cause the gate to vibrate, which can eventually lead to structural failure. See Appendix C.5 for more information.

C **Failure due to collision**: When a vessel enters a lock chamber, there is a possibility of it colliding with the gate in front of it. In case the collision energy is too large, this leads to structural failure of the gate. This is further derived in Appendix C.8.

D **Closure failure**: The closure of a gate depends on both the operator and the machinery. Both could fail their purpose, resulting in an open gate while it is meant to be closed. See Appendix C.7.

The failure of a single gate is not likely to cause the entire system to fail, as the remaining gate will maintain its retention function. For locks with an inner gate shorter than the outer gate, the probability and intensity of overflow does increase in case the outer gate fails to retain, but a far more hazardous situation occurs in case both the inner- and outer gates no longer function. Not all four failure mechanisms listed above are independent of one another. The occurrence of one mechanism can entail the occurrence of another, or completely exclude it.

An overview of the failure mechanisms that can occur simultaneously for the inner- and outer gates is provided in Table 4.1. "Closure failure" can coincide with all three other mechanisms. "Failure due to collision" will only cause inflow in combination with "Closure failure" (De Waal, 2019). The other mechanisms, "Failure due to head difference" and "Failure due to vibration" can be assumed not to occur simultaneously with a collision, as a lock will not be operating during the extreme conditions required for these mechanisms to arise.

If the outer gate fails due to the head difference, the inner gate will necessarily fail in the same manner. When extreme environmental conditions are occurring, the gates of a lock may be temporarily closed. To decrease the head difference to which the gates are subjected, the water level inside the lock chamber is regulated to form a cascading development over the structure, as presented in Figure 4.11 (Van den Berg et al., 2019). Because of this, in case one of the gates fails, the other(s) are thereafter subjected to a much larger head difference than before, and can be assumed to fail sequentially. For marine locks with dissimilar gates, the policy can be applied to initially open the (weaker and shorter) inner gate, relieving the full pressure on the outer gate. Obviously for such cases an extreme head difference which causes the outer gate to fail cannot be retained by the less compatible inner gates and results in inflow through the lock chamber.

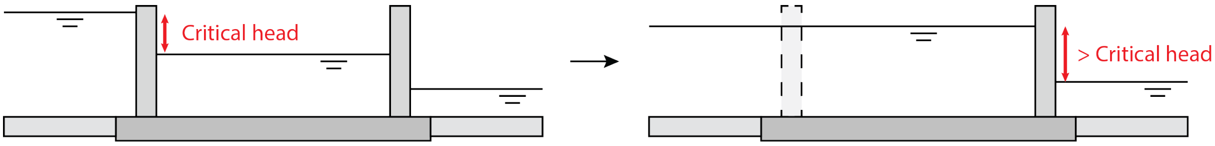


Figure 4.11: "Failure due to head difference" at cascading water levels

A similar principle applies for "Failure due to vibration", i.e. failure of the outer gate entails failure of the inner gate. In case the water flow over the first gate is high enough for the vibrations to cause it to fail structurally, the same will happen to the other gate. This failure mechanism almost entirely depends on the overflow discharge (see Appendix C.5), which will be equal or even larger for the inner gate, depending on its height in relation to the outer gate.

Both "Failure due to head difference" and "Failure due to vibration" are dependent on the water levels, as are their consequences, "Bed protection failure" and "Storage capacity surpassage". The occurrence of these failure mechanisms therefore entail a higher probability of the failure of the bed protection and storage capacity. Thus, for these cases the probability of the consequences has to be derived *given* the occurrence of failure due to head difference or vibration.

Overflow and inflow
The failure of the gates determines the volume of water which will flow into the system. If both gates are operating ordinarily, overflow might still occur when the sea level rises above the height of the outer gate during a storm.

When the outer gate has failed, the determining factor for overflow is the height of the inner gate. For some locks this is identical to that of the outer gate, and for some it is smaller. In the last situation, the probability of overflow increases.

When both gates have failed, water is able to flow freely through the lock chamber. A large volume of water can be expected to enter the system at a high velocity. The flood protection function of the system is thus very likely to fail.

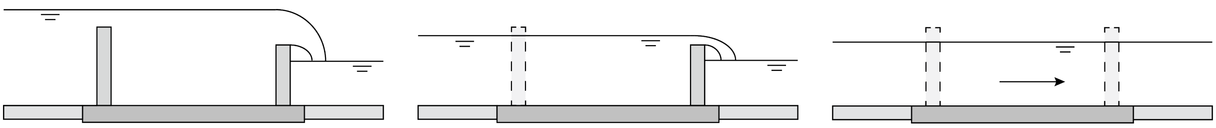


Figure 4.12: From left to right: Overflow outer gate; Overflow inner gate; Inflow

Consequences of overflow and inflow
The possible consequences of overflow and inflow are erosion of the bed protection and surpassage of the storage capacity of the water system (e.g. a river, channel or lake). Only when this storage capacity is filled by the flow through or over the lock to the point where water spills over the dykes, does the hinterland flood. The consequences of erosion of the bed protection (sliding and instability of the subsoil and thus the structure) are however so severe and hard to mitigate that it is considered as flood protection failure, regardless of the storage capacity (Rijkswaterstaat, 2019b).

Storage capacity surpassage and bed protection failure can be directly derived from the overflow or inflow, and require no further explanation as to how these are affected by the varying failure mechanisms of the gates. The processes are further analysed in Appendix C.2 (storage capacity surpassage) and C.3 (bed protection failure).

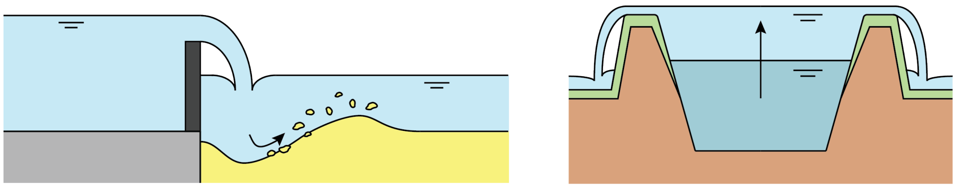
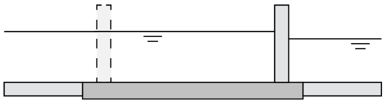
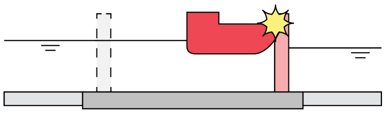
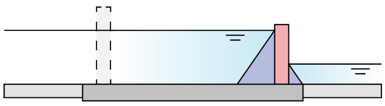
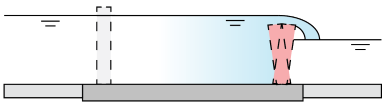
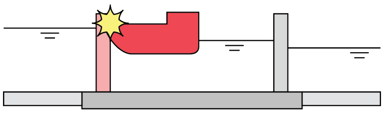
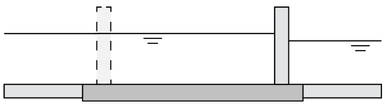
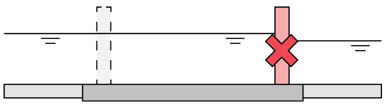
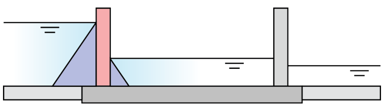
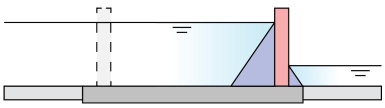
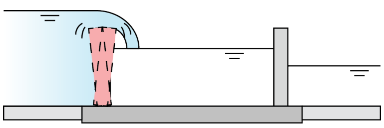
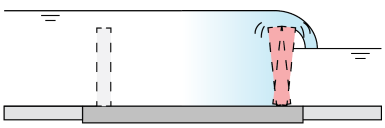


Figure 4.13: LEFT: Erosion of the bed protection due to overflow. RIGHT: Surpassage of the storage capacity

Table 4.1: Possible combinations of failure mechanisms for outer- and inner gates and their consequences

Failure outer gate	Failure inner gate	Consequence
None	None	Possible overflow outer gate
Closure failure 	None 	Possible overflow inner gate
	Collision failure 	Inflow
	Failure due to head difference 	Inflow correlated to heads
	Failure due to vibration 	Inflow correlated to vibration
Collision failure 	None 	Possible overflow inner gate
	Closure failure 	Inflow
Failure due to head difference 	Failure due to head difference 	Inflow correlated to heads
Failure due to vibration 	Failure due to vibration 	Inflow correlated to vibration

4.3. Relation failure mechanisms and sea level rise

In the previous section it has been described by means of what failure mechanisms a lock can fail at its retention function, and how these mechanism are influenced and caused by one another. To analyse which failure mechanisms are relevant to take into account for this thesis, it must be understood what variables (such as lock properties, characteristics of the channel, and hydraulic boundary conditions) determine the probability of these mechanisms. In this section, the mechanisms on which sea level rise has an influence are derived.

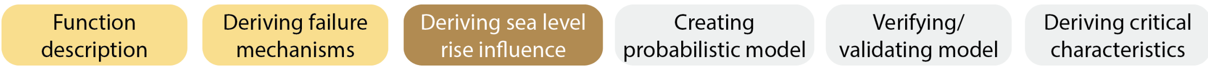


Figure 4.14: Chapter progression Section 4.3

In Appendix C a thorough description is provided on the derivation of each failure probability. Tables 4.2 and 4.3 provide a brief summary of this. The important variables on which the mechanisms depend are accentuated in **bold**.

Table 4.2: Brief description of failure mechanisms (1/2)

Failure mechanism	**Description**
Failure storage capacity	The storage capacity is determined by the **storage area** and the **storage height**. The former is the area of the water system to which the volume flowing over or through the lock can be transported. The storage height is the available height between the water level in the crest of the adjacent dykes. Whether the storage capacity will be surpassed depends on both the available volume in the water system and the volume of **inflowing water**.
Failure bed protection	Two types of water flow can cause the bed protection to erode: **free flow** and **spilling flow**. The former occurs when a lock is emptied, and in case water flows directly through the lock when the gates are failing their retention function. Spilling of water occurs at overflow. When overflow occurs, the spilling water creates a jet when hitting the water body behind a lock, which can cause erosion. Whether the flow velocity actually causes erosion, depends on the **critical flow velocity** of the bed protection composition.
Failure due to head difference	The **strength of a gate** depends on the loading conditions which it has been designed for, and the partial factors that have been applied. The latter depends both on the material, and the year of design. Obviously when **hydraulic boundary conditions** become more extreme, the failure probability increases.
Failure due to vibration	This failure mechanism is approached by means of a very simple critical discharge over the gate at which it is expected that some damage could be done: 1 $m^3/s/m$. The mechanism is thus fully dependent on the **retention height** and **hydraulic boundary conditions**.
Failure closure after structural damage	After structural damage of the gate caused by either the head difference or vibration, a gate could still be closed in case of no severe deflections. This probability is however conservatively set at 1.
Closure failure	The probability of closure failure comes from an estimation of the initial chance that a gate is not already closed, and the probability that the closure fails. This probability is regularly calculated from the empirically derived failure probability of the individual components in the operating mechanism of a gate.
Failure due to collision	For this mechanism, three things have to occur: a vessel has to collide with a gate, the energy of the collision is higher than the critical energy of the gate, and the damages to the gate are so severe that it can no longer be closed. The yearly probability of a vessel collision depends on the **yearly amount of vessels passing the lock**, the **type of vessels** that pass the lock, and the **type of gates**. For the energy of the collision to be critical, an estimation of 1/100 is derived. Whether the gates can still be closed afterwards depends on the gate type and the **flow velocity** after the collision has occurred.

Table 4.3: Brief description of failure mechanisms (2/2)

Instability due to head difference	Instability of a structure can occur in three degrees of freedom: horizontal, vertical and rotational. Because of the large dimensions of a lock, it can be assumed that instability in any of these degrees of freedom is highly unlikely.
Piping failure	The occurrence of piping is dependent on the **piping length**, the **subsoil**, and **the hydraulic gradient** over the length of a lock. Sea level rise could cause the hydraulic gradient to become larger than has been anticipated in the design.

As stated in Table 4.2, "Failure closure after structural damage" is assumed to be equal to 1. Contrarily, mechanism "Instability due to head difference" is deemed very improbable for a navigation lock. Both mechanisms can thus be disregarded.

For the remaining failure mechanisms, it is described below whether these are positively, negatively, or not influenced by a rising sea level:

- **Failure storage capacity**
  Although the available storage capacity of a water system is not directly influenced by sea level rise, the probability of a large volume of water entering the system during a storm is increased.
- **Failure bed protection (by flow or jet)**
  As for the storage capacity, nothing changes for the integrity of the bed protection because of sea level rise. However, higher flow velocities become more probable due to a larger head difference over the navigation lock.
- **Gate retention failure:**
  - **Failure due to head difference**
    With a rising sea level, the extreme head differences over the navigation lock will increase, and thus pose a larger load on the gate(s).
  - **Failure due to vibration**
    The possibility of an overflow larger than 1 $m^3/s/m$ increases when the sea level rises, at which it is expected that vibration can cause damage to the gate(s).
  - **Closure failure**
    The possibility of closure failure does not increase due to sea level rise. However, in case the inner gate is shorter than the outer gate, closure failure of the outer gate might impose magnitudes of hydraulic boundary conditions on the inner gate to which it has not been designed. This probability increases with sea level rise.
  - **Failure due to collision**
    Sea level rise does not increase the possibility of a collision. Even more so, the probability could decrease, depending on the procedure of a lock. Most locks have a maximum sea level at which they operate, as flow velocities accompanied by the filling and emptying of the lock chamber become unsafe for the vessels in more extreme conditions. As this maximum water level becomes more frequent with a rising sea level, the operability of a lock decreases, and thereby also the probability of a collision.

    It should be noted that if the average sea level is higher, the point of collision on the gate also generally lays higher, which can have an effect on the probability of an impact being critical. However, a probability of 1/100 is assumed regardless for this mechanism, so in the extent of this thesis, this has no influence.
- **Piping failure**
  When the sea level rises, the probability of a head difference that can cause piping also increases.

From the dependencies described above, it can be concluded that all listed failure mechanisms are negatively influenced by sea level rise , except for "Failure due to collision". The latter is henceforward disregarded, as it will not play a significant part in the threat of sea level rise. All other mechanisms listed above are taken into consideration.

5

Development failure probability model

The first step of this thesis concerns the development of the failure probability of a lock. To calculate how the probability of each failure mechanism changes with a rising sea level, a computational model is to be made. This chapter embodies the development of that model.

To understand the probabilistic methods used in the model, Section 5.1 provides theoretical background. Section 5.2 depicts the basic structure of the model. A verification of the script is presented in Section 5.3. A validation is required to examine whether the model renders a reasonable outcome when provided with realistic input parameters. Thus, the model is validated by means of a case study of a lock closely resembling the Western lock in Terneuzen 5.4. The results of this validation additionally function as an insight in the quantitative relevancy of each failure mechanism with respect to sea level rise, from which several critical lock characteristics are drawn in Section 5.5. These form the basis for the lock adaptations which are developed in Chapter 6.

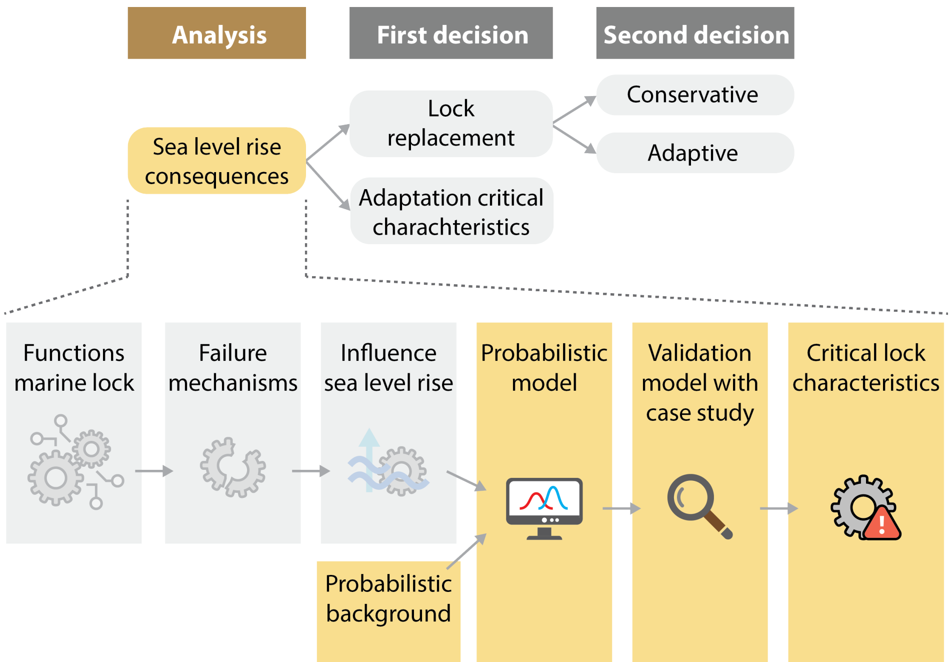


Figure 5.1: Flow chart of Chapter 5

5.1. Probabilistic background

The probabilistic method applied in this thesis to define the failure probability of the flood protection function of a navigation lock is depicted in Figure 5.2. The knowledge applied per individual step of this flow chart is set out throughout this Section. The method is initiated by the determination of the distributions of the input parameters, i.e. the properties of the lock, its environment and loading conditions (Section 5.1.1). These function as input for the Monte Carlo analysis, in which they are implemented in limit state functions (Section 5.1.2). The output of the Monte Carlo analysis (Section 5.1.3), i.e. the separate probabilities of the failure mechanisms of a lock, function as input for the fault trees (Section 5.1.4). These describe the relation between the different failure mechanisms, and provide the total failure probability as final product.

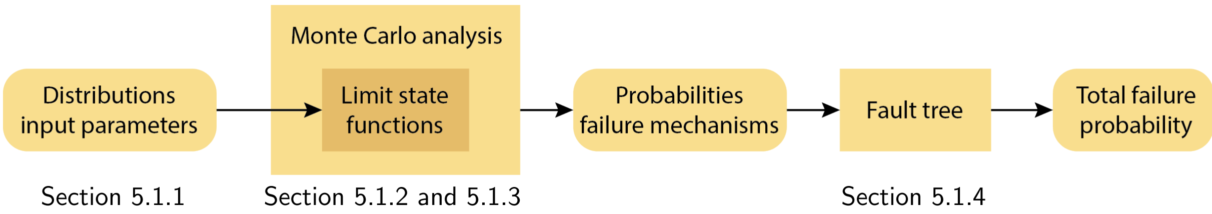


Figure 5.2: Flow chart probabilistic method

5.1.1. Probability density function

For the Monte Carlo method, many continuous random variables will be used. These are variables which are not limited to discrete values, but can take every possible value (sometimes limited by a range). A wave for instance will never be equal to precisely one or two meters, but can be everything in between. As the variable is continuous, the probability is also described by a continuous function: the distribution function $F_X(x)$, presented in Equation 5.1 (Jonkman et al., 2017).

$$F_X(x) = P(X \leq x) \tag{5.1}$$

For many objectives, it is more helpful to use the probability density function $f_X(x)$, which is obtained by differentiating the distribution function, presented in Equation 5.2.

$$f_X(x) = \frac{dF_X(x)}{dx} \tag{5.2}$$

Different variables often follow similar patterns in terms of probability. This has lead to the derivation of a large number of standard probability density functions (PDFs). As an example, the Gaussian- or Normal Distribution is presented here. This is one of the most commonly found distributions in reality. The distribution is described by Equation 5.3, which will, depending on the input parameters, provide the bell shaped curves shown in Figure 5.3.

A distribution most often has one or more parameters which determine the magnitude, shape, or translation of the function. For the Normal Distribution (and many others) these are the mean μ and standard deviation σ, but for some distributions these can be completely different parameters.

$$f_X(x) = \frac{1}{\sqrt{2\pi}} \frac{1}{\sigma} \cdot e^{\left(-\frac{(x-\mu)^2}{2\sigma^2}\right)} \tag{5.3}$$

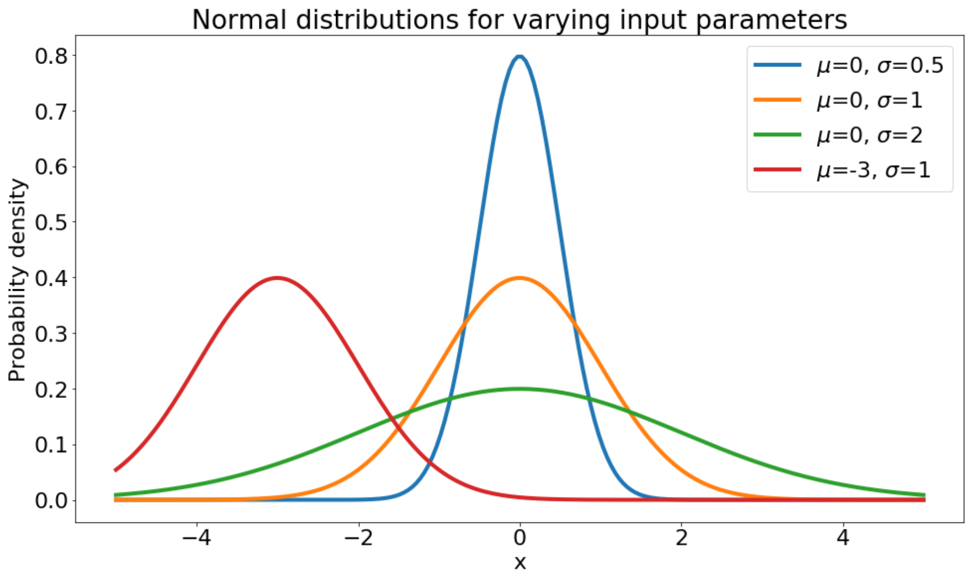


Figure 5.3: Plots of normal distributions

Many variables are naturally distributed like one of the common distributions, for example a Normal Distribution. For most parameters required for determining the failure probability of a navigation lock, the distributions have been empirically derived or require only the determination of one parameter, such as the mean. For others (mainly environmental variables like the wave height and water level), the distribution is not fixed and has to be be obtained per individual case.

Unfortunately, the data from which these distributions are to be derived are not always conclusive. One could try to find a distribution type which fits the data as close as possible, but this can be time consuming, and does not necessarily lead to a distribution with an negligible error. Luckily, the computational nature of the Monte Carlo method allows a trick to be applied which discards the need for a common distribution.

As will be explained, the Monte Carlo method is based on the function many programs have to select a random value from a given distribution. This function does however not work in case no distribution type can be provided. Alternatively, this function can be mimicked without the use of a distribution. What these functions essentially do, is take a random variable from a uniform distribution with a range between 0 and 1, and fill this value into the equation of the variable's distribution (see Figure 5.4).

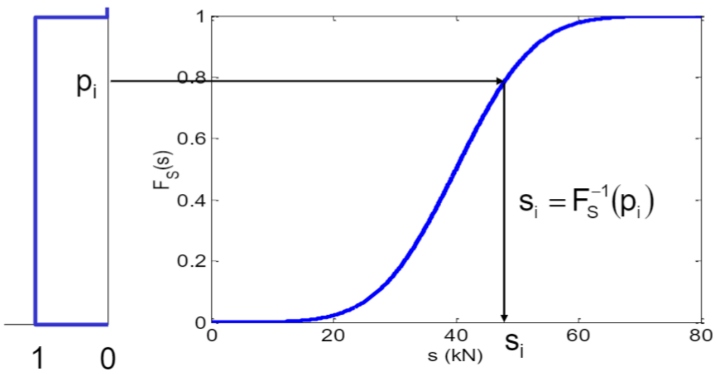


Figure 5.4: Drawing a random variable by means of a uniform distribution. Retrieved from (Jonkman et al., 2017).

In case the distribution is unknown, this method can be applied to a regression line. A regression line is the line which has the smallest error with a given set of data points, given a certain base equation. This base equation can be linear ($y = ax + b$), exponential ($y = ae^b$), or anything else. In some cases, a data-set can not be well defined by a single regression line. This is no issue, as the data can be fitted with multiple regression lines with varying domains. This is presented by means of an example in Figure 5.5.

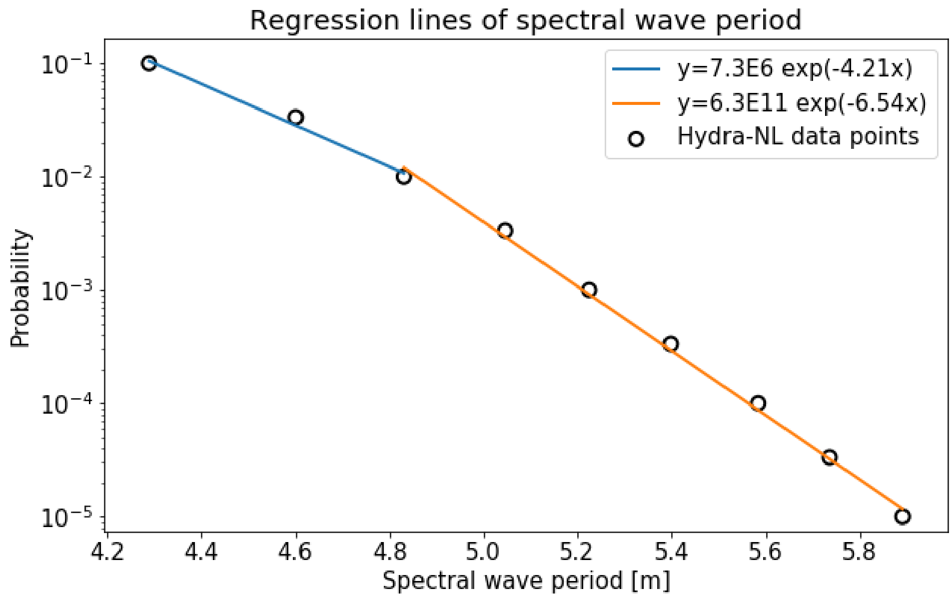


Figure 5.5: Example of the fitting of two regression lines on discrete data points of a wave period. Data retrieved from Hydra-NL.

5.1.2. Limit state functions

Another important probabilistic term to explore is the limit state function. This function is a.o. a device to derive failure probabilities. All limit state functions follow the same basic structure, presented in Equation 5.4 (Jonkman et al., 2017). In this Equation, the R represents the Resistance of a system, whereas the S represents the Solicitation, often referred to as "load". These do not necessarily concern resistance and load in the literal sense. For instance, regarding the storage capacity of a river, the resistance is the storage volume, while the load is the inflow volume.

$$Z = R - S \tag{5.4}$$

The principle of the limit state function is that a system fails in case the load exceeds the resistance. Thus, for a deterministic approach, a negative value for Z represents a failure. For a probabilistic approach, however, the resistance and load are drawn from distributions, and do not provide such a definitive outcome. For example the storage volume of a river is highly dependent on its water level, which is uncertain and ever changing. Therefore, depending on the state of the river level, the system might fail or not in case inflow occurs. The failure probability can thus be described using Equation 5.5.

$$P_f = P(S > R) \tag{5.5}$$

There are several manners in which the limit state function can be used to derive the failure probability. The following Section will explore one of these methods.

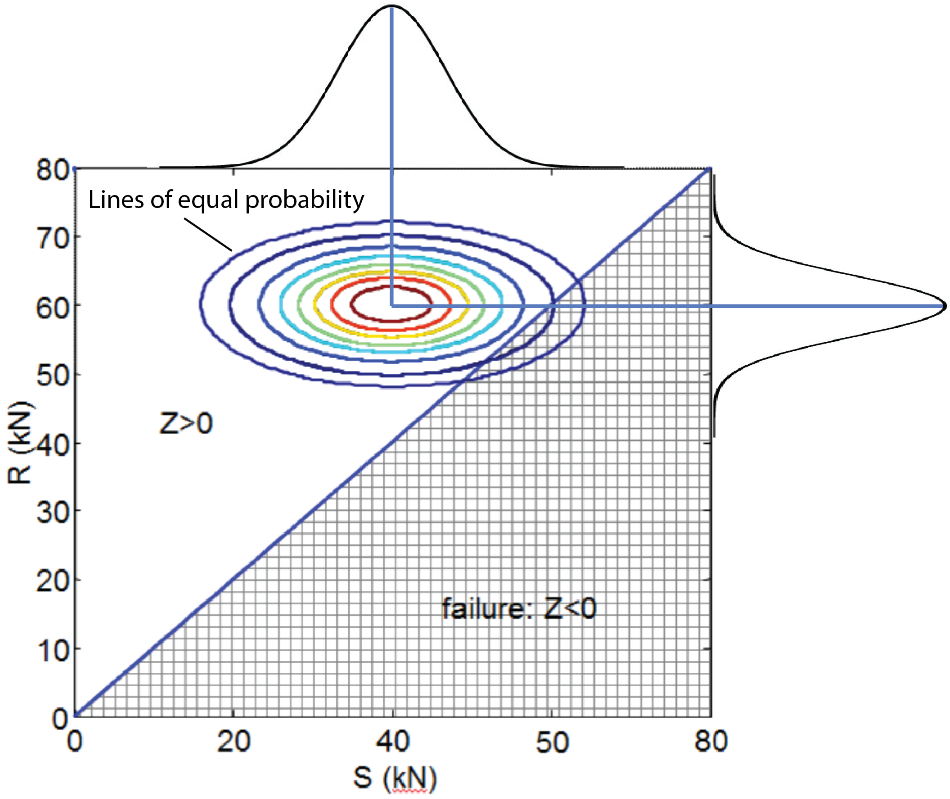


Figure 5.6: Graphic depiction of failure probability by means of the limit state function. Retrieved from Jonkman et al. (2017) (edited).

5.1.3. Monte Carlo simulations

A very applicable method to derive a system's failure probability is by means of Monte Carlo simulations (Jonkman et al., 2017). This method is used as it can be easily applied to any limit state function (regardless of the amount of variables in the equation), it is not labour intensive, yet still very precise. The method uses the ability many computational programs and languages provide to draw a random number from a given distribution. Using such a function, a value is drawn for all parameters in the limit state function which are defined by a distribution. By filling these values into the limit state function, a deterministic outcome for Z is derived. As explained, a negative Z indicates a failing system. Applying this simulation once obviously does not yet provide a failure probability of the system, as only one combination of the variables is analysed. Thus, the process is iterated an N number of times, providing an N number of variable combinations and an N number of outcomes for Z. For each iteration, Z is either lower than zero, or it is not.

Sequentially, the amount of iterations for which Z is below zero is divided by the total amount of iterations. This provides an approximation of the failure probability.

$$\hat{P}_f = \frac{N_{Z<0}}{N} \tag{5.6}$$

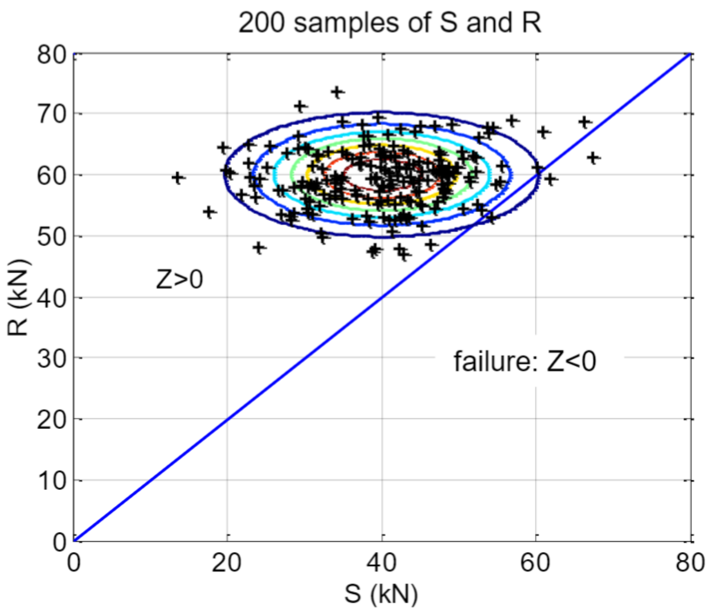


Figure 5.7: Depiction of Monte Carlo method. Retrieved from Jonkman et al. (2017).

The outcome of the method depends on the random selection of variables from a distribution, which does not lead to an exact outcome, but merely an approximation. In general, if a system has a smaller failure probability, it requires more iterations to provide a solid approximation. Secondly the accuracy of the Monte Carlo method always increases with the number of iterations applied. This is captured in the Equation for the coefficient of variation of $\hat{\mathrm{P}}_{\mathrm{f}}$.

$$V_{\hat{P}_f} = \frac{1}{\sqrt{NP_f}} \tag{5.7}$$

After performing the Monte Carlo method, it should always be checked whether the number of iterations performed and the derived failure probability indicate a desired accuracy, by means of Equation 5.7.

5.1.4. Fault trees

Once the individual probabilities of the failure mechanisms of a system have been derived, the total failure probability can be calculated. Thereto, it must be understood what the relations between the failure mechanisms are. What mechanisms provoke different mechanisms to happen, and whether they can occur separately, or have to occur simultaneously. A good manner of graphically portraying the relations between the causes and consequences of a system's failure mechanisms is by means of a failure tree. A failure tree is structured such that from the bottom to the top, causes are followed by their consequence. A consequence can have multiple causes, which are connected through a certain gate-type.

An AND-gate signifies that all causes have to occur for the consequence to follow. Contrarily, an OR-gate concerns a system where only a single cause is enough to set the consequence in motion.

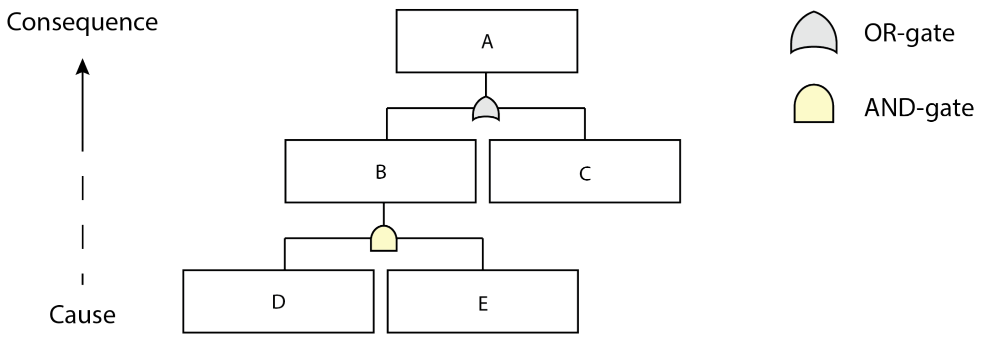


Figure 5.8: Example of fault tree

Table 5.1: Equations used for OR- and AND-gates. Retrieved from (Jonkman et al., 2017).

Gate	Mutually exclusive	Independent	Fully dependent
OR	$\sum_{i=1}^{n} P_i$	$1 - \prod_{i=1}^{n}(1 - P_i)$	$max(P_i)$
AND	0	$\prod_{i=1}^{n} P_i$	$min(P_i)$

5.2. Structure of failure probability model

To derive the influence of sea level rise on the probability of the failure mechanisms of a lock which can lead to flooding, a computational failure probability model is made. In this section, the structure of that model is provided.

Figure 5.9: Chapter progression Section 5.2

The probabilities of the individual failure mechanisms are derived using the Monte Carlo method, as described in Section 5.1.3. A computational model is constructed, both because the Monte Carlo method requires computational power, and because it gives the possibility to automate the other steps of the process depicted in Figure 5.2.

This model lends itself for multiple steps in this thesis. It is initially used to derive the failure probability of a lock, and to depict how this will increase when the sea level rises. It can thus provide insight of how much sea level rise will cause the failure probability of a lock to surpass the legal norm, meaning an adaptation or replacement is required. Thereafter, the same model can be used to calculate to what extent the failure probability will decrease once an adaptation or replacement has been performed. Solely the input parameters will have to be altered.

The model is created using the coding language Python, and is in its entirety presented and explained in Appendix D. To illustrate the steps which are executed by the computational model, a flow chart is presented below, alongside an explanation of the process. The colours in the flow chart signify the step to which the processes belong. The box labeled "Limit state functions" refers to the equations derived in Appendix C. The box labeled "Fault tree" refers to the one depicted in Appendix B.

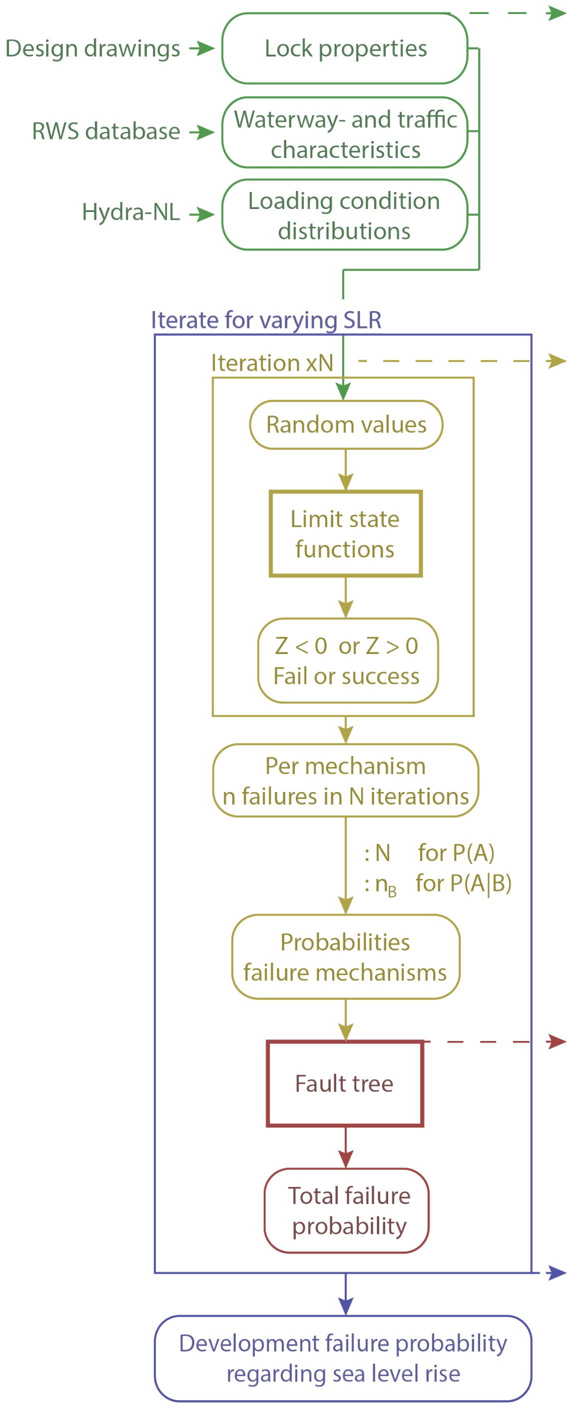


Figure 5.10: Flow chart of computational model

Step 1: Providing input distributions
The dimensions of the lock are obtained from design drawings, or estimated with calculations. The properties of the waterways and traffic are derived from RWS databases. The distributions of the hydraulic boundary conditions are acquired from Hydra-NL, and fitted by means of regression lines.

Step 2: Monte Carlo Analysis
For each variable described by a distribution, a random value is drawn. For each failure mechanism, these values are provided as input to derive required parameters and eventually the limit state function.

The output of the limit state functions is a value for Z, which is either below or above 0, implying a failure or success respectively. The process of random selection and limit state calculations is iterated N times. The amount of failures n_i is counted for each individual failure mechanism. For the probability of a failure mechanism *given* the occurrence of another mechanism, only the times are counted at which *both* mechanisms provide a failure.

To derive a regular probability, P(A), the amount of failures n_A is divided by the amount of iterations N. To derive a failure probability *given* another failure, P(A|B), $n_{A\&B}$ is divided by n_B.

Step 3: Processing fault tree
With the probabilities of the failure mechanisms, the fault tree is worked through from bottom to top using the mathematical expressions of AND- and OR-gates. Thus the total failure probability is provided, and also all relevant steps in between.

Step 4: Depicting influence sea level rise
Finally, the entire process is repeated for a range of discrete values of sea level rise. Thus, a graph can be made to present the development of the probability of any failure mechanism or the total failure against the sea level rise.

5.3. Verification of the failure probability model

It has to be verified whether any mathematical mistakes or typos have been made in the model. To do so, the results of the model have been compared to a number of example calculations acquired from the WBI manuals, and several hand calculations. This verification is presented in Appendix E.

5.4. Validation of the failure probability model with case study

Finally, the model is validated by means of an example. This section presents the required input parameters and obtained results using the limit state functions presented in the previous sections in combination with the Monte Carlo method. The example is based on the Western lock in Terneuzen, to assure a considerably realistic result.

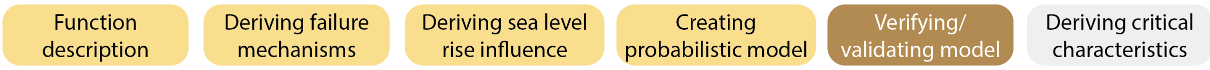


Figure 5.11: Chapter progression Section 5.4

5.4.1. General properties

The main properties of the Western lock chamber in Terneuzen are presented in Table 5.2. Most of these parameters have for been retrieved from the thesis of Arno Kemper, who also studied this lock complex. A depiction of the main dimensions of the lock is presented in Figure 5.12.

Table 5.2: General properties of hypothetical lock, based on the Western lock chamber in Terneuzen

Lock properties	
Outer gate height	NAP+6.0 m
Inner gate height	NAP+6.0 m
Sill height	NAP-12.8 m
Lock chamber width	38 m
Gate type	Roll
Gate material	Steel
Construction year*	1968
Gate policy	Cascaded head
Environmental properties	
Bottom level approach channel	NAP-14.0 m
Bottom level canal	NAP-10.4 m
Average canal width**	140 m
Canal length**	32 km
Traffic properties	
Yearly levellings*	5050
Seagoing vessels***	33%
Commercial transport***	95%
Piping related properties	
Horizontal seepage length	439 m
Vertical seepage length	27 m
Soil type	Very fine sand

* Retrieved from Pfaff-Wagenaar and De Wit (2015)
** Retrieved from *www.rijkswaterstaat.nl/water/vaarwegenoverzicht/kanaal-van-gent-naar-terneuzen* (last accessed on 23-10-2020)
*** Retrieved from Bückmann et al. (2019)
All other parameters are retrieved from Kemper (2019).

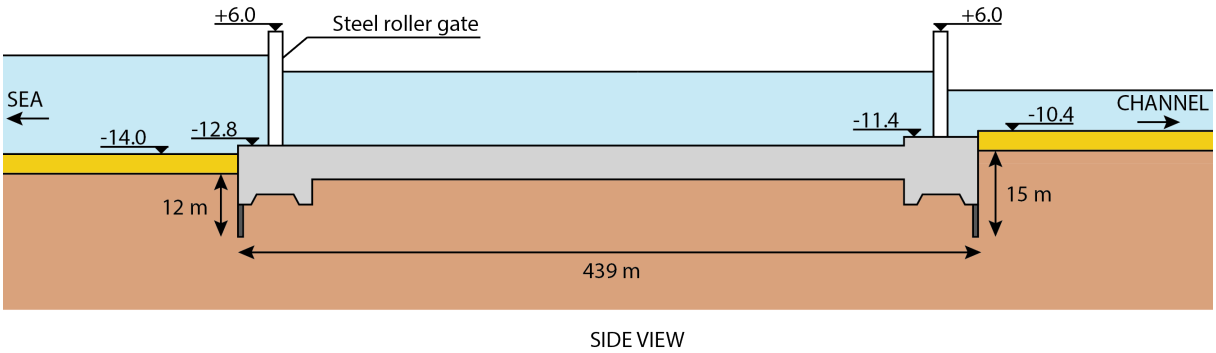


Figure 5.12: Depiction of main dimensions and elevations of the Terneuzen Western lock chamber. Values retrieved from Kemper (2019).

For the purpose of this example, the storage area is simplified to the area of Canal Ghent-Terneuzen. The storage area is composed from its average width and length, as presented in Table 5.2: 4.48 km^2. An average dyke height has been derived from multiple cross sections presented in Figures 5.13 and 5.14 as NAP+4.2 m.

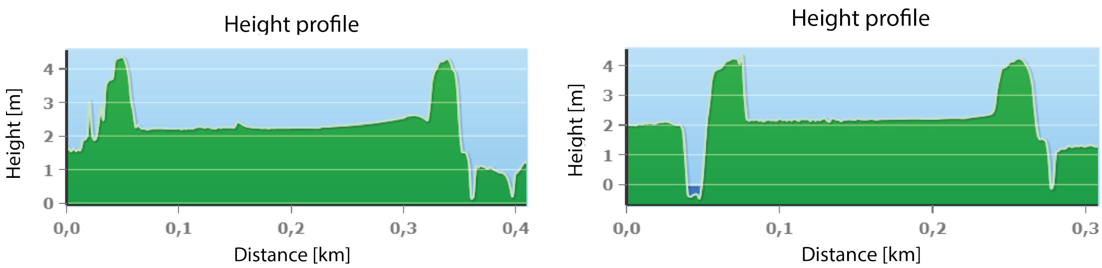


Figure 5.13: Two cross sections of Canal Ghent-Terneuzen. Retrieved from https://ahn.arcgisonline.nl/ahnviewer/ (Accessed on July 27, 2020) (edited).

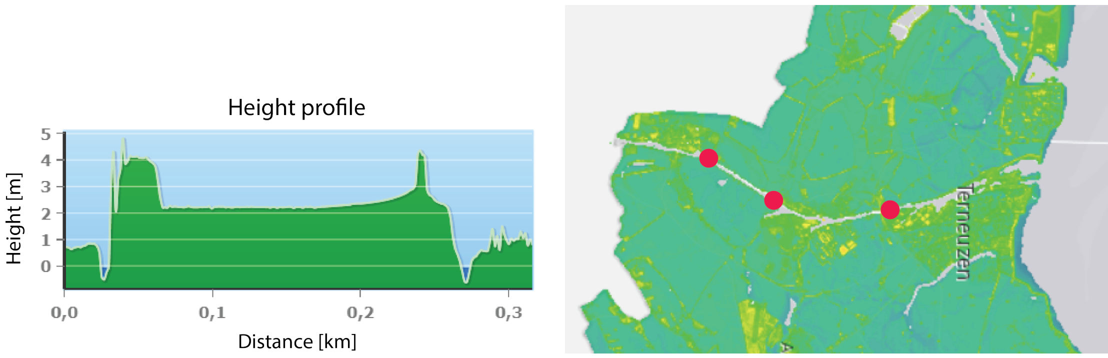


Figure 5.14: LEFT: Third cross section of Canal Ghent-Terneuzen. RIGHT: Locations of cross sections. Retrieved from https://ahn.arcgisonline.nl/ahnviewer/ (Accessed on July 27, 2020) (edited).

The D_{n50} of the bed protection behind the lock has been derived using the limit state function for "Bed protection failure due to flow" presented in Equation C.10 in combination with a Shield's parameter equal to 0.05 (some grain movement allowed) and the original design conditions, which are derived in the last paragraph of Section 5.4.3. It is thereby assumed that the bed protection has been designed for a situation in which water flows through the lock at extreme conditions. By filling these parameters into Equation C.10, a minimum required mean of the Normal Distribution of D_{n50} is equal to 0.39 m. This corresponds with a standard sorting of 15-300 kg (Capel, 2015), for which a D_{n50} range of 0.30 m to 0.43 m is available.

5.4.2. Failure probability norms

The Terneuzen lock complex is part of dyke section 32-3, located at the Western Scheldt in the province of Zeeland in the Netherlands. For this dyke section, two failure probability norms for the flood protection function of the lock have been included in the Dutch Water Act. One indicates the signaling value, at which the minister is to be informed about the receding sufficiency of the lock. This is equal to 1:3000 per year (Dutch Ministry of Transport, Public Works and Water Management, 2017b). The other is the actual norm which should not be surpassed by law to sustain the safety of the population. This is equal to 1:1000 per year (Dutch Ministry of Transport, Public Works and Water Management, 2017c).

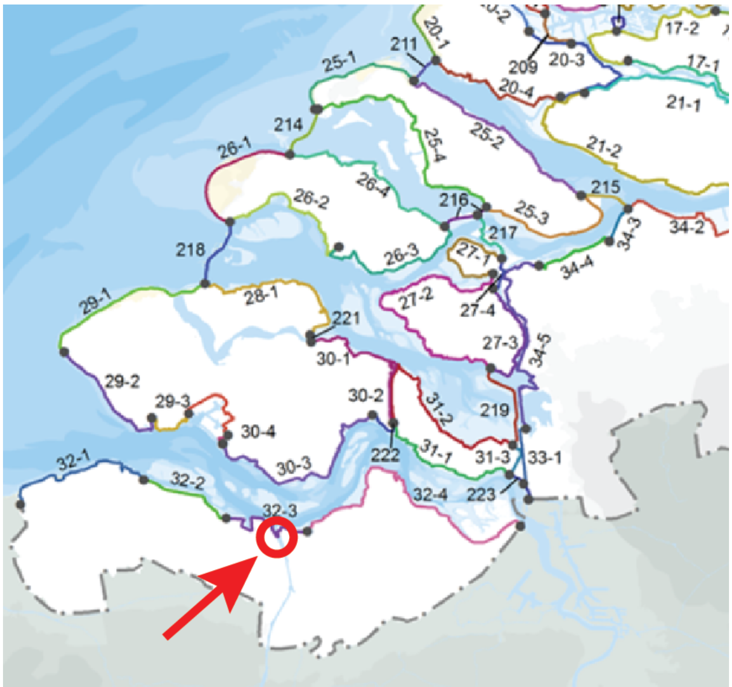


Figure 5.15: Location of dyke section 32-3. Retrieved from Dutch Ministry of Transport, Public Works and Water Management (2017a) (edited).

5.4.3. Hydraulic boundary conditions

The following step is the assessment of the hydraulic boundary conditions in the area of the lock complex. This is done using the computer program Hydra-NL. This can retrieve location-specific parameters such as water levels, wave heights and wave lengths, based on data and probabilistic calculations. Figure 5.16 presents the measurement location that is used for this thesis, point 40 of dyke section 32-3. Unfortunately there is no data on the hydraulic boundary conditions at the lock itself, but only at the entrance of the approach channel.

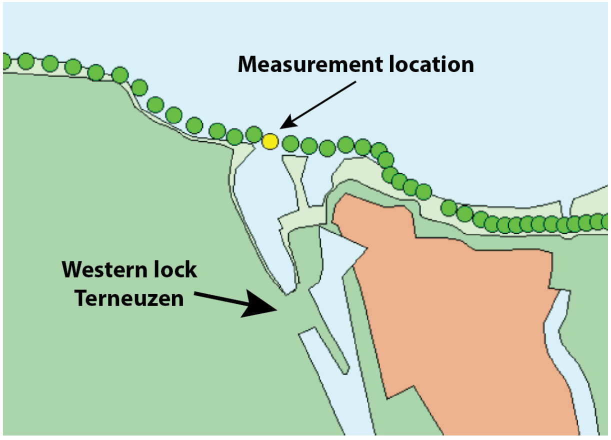


Figure 5.16: Measurement location of hydraulic boundary conditions. Retrieved from Hydra-NL.

Hydra-NL provides data on hydraulic boundary conditions by means of their return periods. To make use of the data, it firstly has to be converted from "wave height per return period" to "yearly probability per wave height". The program provides a number of discreet values for return periods between 10 years and 100,000 years. By dividing 1 with these return periods, the yearly probability is found (ranging from 10^{-1} to 10^{-5}). By these means, the Hydra-NL data is converted to the data points seen in Figure 5.17.

Using these data points, one or multiple regression lines can be retrieved using the built-in function of Excel, "Trendline". In this particular case, one exponential regression line was found to provide a reasonable fit to the data. An obvious note to make is that no wave height with a return period smaller than 10 years has been taken into account, albeit that this is the load most frequently applied. The assumption is however made that the norm of the navigation lock has been long surpassed once wave heights of such frequencies of occurrence start to play an important role in the failure probability. On the other hand conditions with a return period larger than 100,000 years are not of significance to a lock with a yearly norm of 1:1000. The data is thus regarded as sufficient for the derivation of the failure probability of the flood protection function.

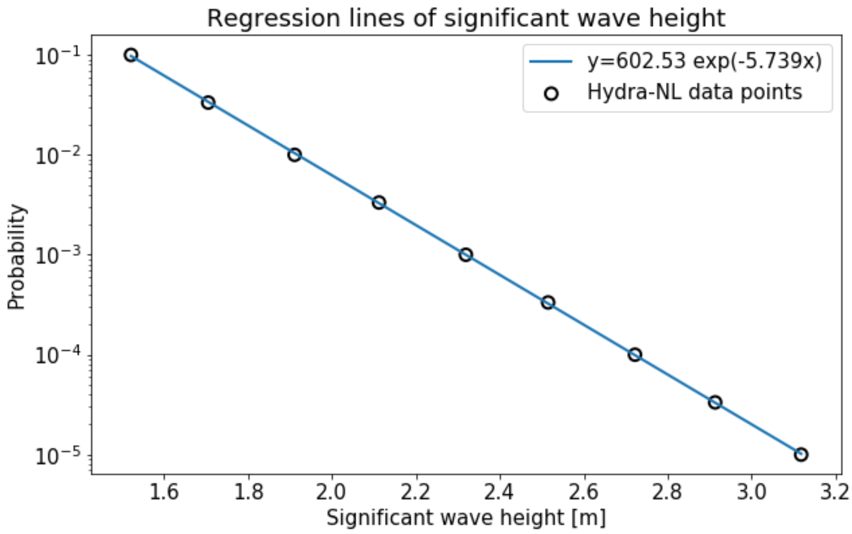


Figure 5.17: Regression line of wave height. Data retrieved from Hydra-NL.

In the same manner, data is retrieved from Hydra-NL on the water level, from which the results are presented in Figure 5.18. The figure additionally presents how sea level rise is applied to the data, i.e. a direct translation of the water level equal to the sea level rise.

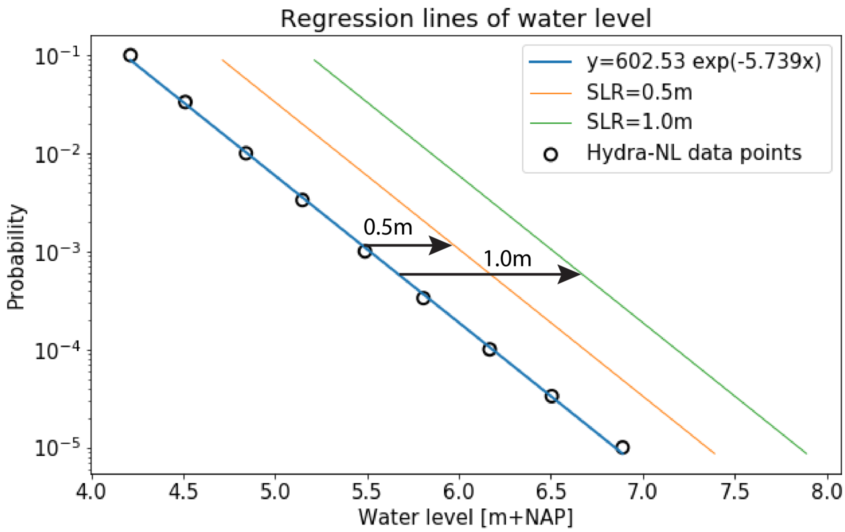


Figure 5.18: Regression lines of water level. Data retrieved from Hydra-NL.

For the water level on the other side of the navigation lock, a different strategy is used. The Terneuzen lock complex is connected to Canal Ghent-Terneuzen, which is a regulated water way for which the water level is maintained around NAP+2.13 m. As for now the focus lays on the sea-side of the lock, a simple normal distribution is assumed for the canal water level with a standard deviation of 0.10 m.

Finally the regression lines of the data on the spectral wave period are presented in Figure 5.19.

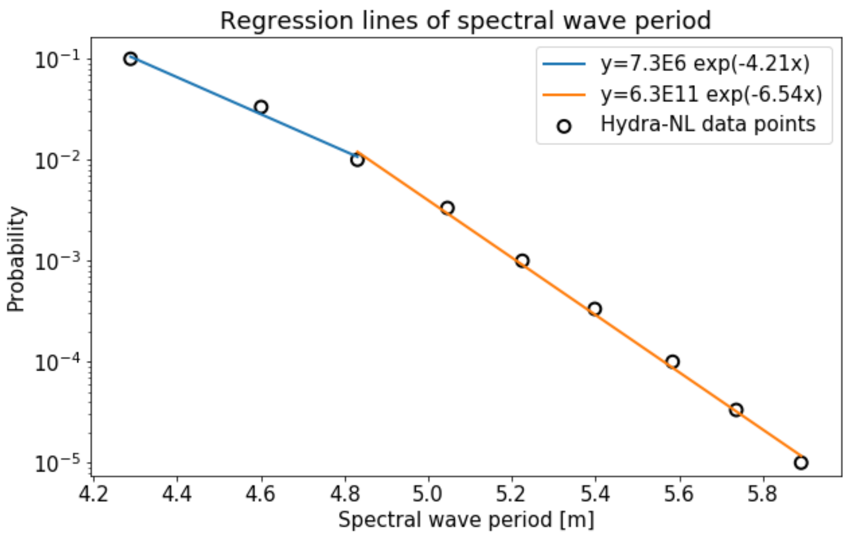


Figure 5.19: Regression lines of wave period. Data retrieved from Hydra-NL.

For the purpose of wave pressure calculation, the wave period must be converted into a wave length. This can be done for any water depth using the dispersion relationship, as presented in Equation 5.8 (Holthuijsen, 2007). As this contains the wave length on both sides of the equal-sign, it is to be solved iteratively, which can fortunately easily be implemented in the method due to the computational nature of the Monte Carlo simulations.

$$L = \frac{gT^2}{2\pi} tanh\left(\frac{2\pi d}{L}\right) \tag{5.8}$$

The last aspect of the hydraulic boundary conditions is the angle of incidence of the waves. Due to the geometry of the approach channel of the Western lock, most waves do not reach the lock. Only waves coming in from the north can progress in the direction of the lock without having to be reflected. Because it can be assumed that waves are far from fully reflected on the dykes, only waves from the north are taken into consideration. In general, over the range of return periods, only 1.5% of the waves at the measurement location have an angle of incidence of 0°(see Figure 5.20). Therefore, a probability of 1.5% is implemented of wave impact on the gates, and no wave impact in all other situations. Although there will still be waves at the lock, these will not be of significance to any of the failure mechanisms in relation to the waves approaching from the north.

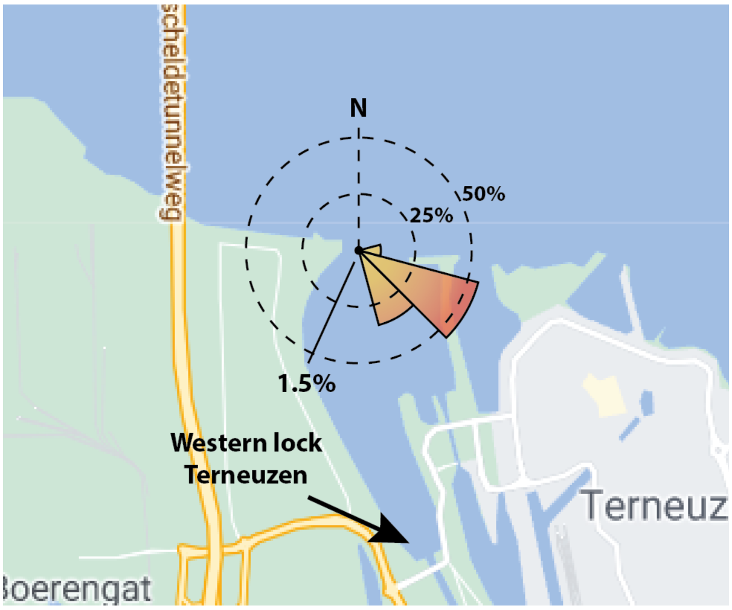


Figure 5.20: Contribution percentages of wave directions at Terneuzen. Data retrieved from Hydra-NL. Image retrieved from https://www.google.com/maps (Accessed on 28-10-2020) (edited)

For the derivation of the resistance of the gates, the hydraulic boundary conditions on which the structure has originally been designed have to be provided. For this hypothetical case, it as been assumed that the gates are designed to withstand the conditions with a probability of occurrence equal to the legal norm, 1:1000. This can be read directly from the data acquired from Hydra-NL, and is summarised in Table 5.3. The design wave length has been derived from the design wave period and dispersion relationship (Equation 5.8).

Table 5.3: Original design conditions

Parameter	Value
Wave height	2.32 m
Water level	NAP+5.49 m
Wave period	5.23 s
Wave length	42.4 m

5.4.4. Examination results case study

Using the limit state functions and parameter distributions presented in Appendix C, the Monte Carlo method is applied with 1,000,000 iterations for a range in sea level rise between 0 and 2 m. The outcome has thereafter been processed through fault trees as presented in Appendix B, from which the results are presented in this section.

Main failure mechanisms

Figure 5.21 presents the individual development of the main failure mechanisms. Clearly the main contribution to the total failure probability comes from "Failure due to vibration". As verification of the script, the coherency between the failure probability development expectations and actual results is described below per mechanism.

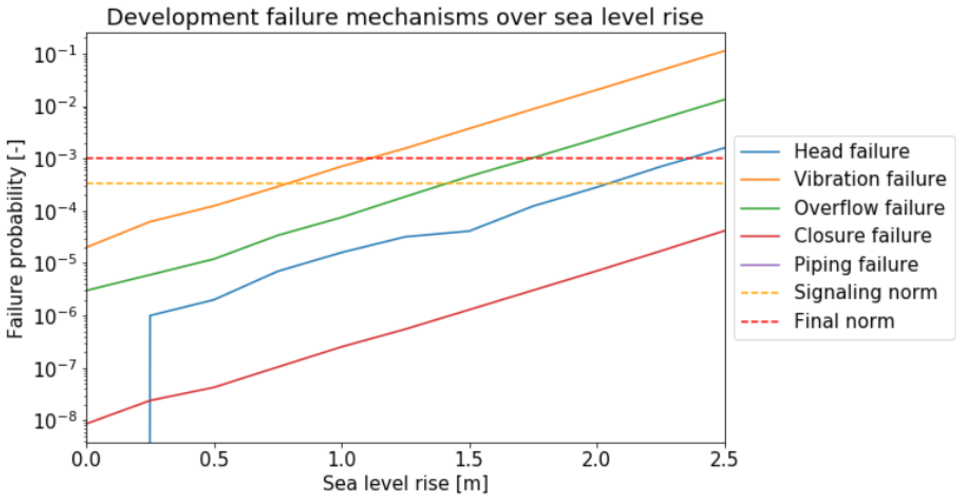


Figure 5.21: Development main failure mechanisms over sea level rise of the hypothetical case

- **Vibration failure**
  "Vibration failure" is caused by an extreme discharge over a gate. It can be expected that when the sea level rises, such an extreme discharge is more frequently reached. Because of the exponential distribution of the water level and wave height (presented in Figure 5.18 and 5.17), the probability of this mechanism also increases exponentially with the sea level rise.
- **Failure due to head difference**
  Similarly as for vibration, "Failure due to head difference" is mostly related to the water level and wave height. Therefore it is as expected that due to the exponential distribution of these parameters, the probability of this failure mechanism also increases exponentially with the sea level rise. The failure probability is however much lower, mostly due to the safety factors regarded in the gate design.
- **Overflow failure**
  Obviously this failure mechanism is driven by the same hydraulic conditions as "Vibration failure", and therefore increases exponentially with the sea level rise. The consequences of this mechanism do however become relevant at a later stage. This is explained by the results presented in Figure 5.22.
- **Closure failure**
  The failure probability of this mechanism does increase, like all others, exponentially. This is however not due to the probability of a single gate failing to close, but due to the combination with any of the other failure mechanisms at the remaining gate which is closed. Therefore, as the failure probability of the mechanisms at the other gate exponentially increases, the total probability of "Closure failure" additionally increases. However, because the probability of a gate failing to close is relatively low, this mechanism does not contribute significantly to the total failure probability of the flood protection function of the lock.
- **Piping**
  The failure probability of this mechanism is equal to 0 for the entire range of analysed sea level rise, and therefore does not present itself in the graph (a 0 is not presented in a logarithmic scale). A quick calculation using the equations in Section C.10 shows that due to the installed cut-off screens at this lock, piping would only start occurring at a head difference of 13.6 m, implying a (nearly) empty channel.

Sub-mechanisms of overflow
"Overflow failure" can be initiated by either failure of the bed protection or surpassage of the storage capacity. Their contributions are presented in Figure 5.22. Both possible bed protection failure mechanism are is not present in the graph (except for a single failure due to a jet at 1.75 m and 2.25 m sea level rise), indicating it is equal to 0 (which does not show on a logarithmic scale).

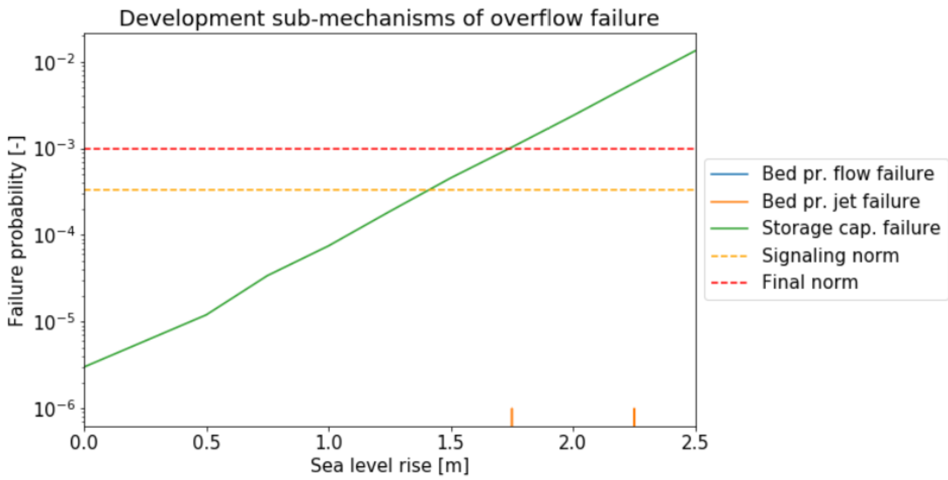


Figure 5.22: Development sub-mechanisms "Overflow failure" over sea level rise of the hypothetical case. The probability of bed protection failure due to flow or a jet are mostly equal to 0, which does not appear on a logarithmic scale.

- **Storage capacity surpassage due to overflow**
  The frequency and quantity of overflow is expected to increase when the sea level rises. As has been established, the overflow volume increases exponentially due to the exponential distribution of the hydraulic boundary conditions. The storage capacity is surpassed when a sufficiently large volume of water enters the system. It is therefore coherent that this failure mechanism increases exponentially with the sea level rise.
- **Bed protection failure due to overflow**
  In case of overflow, the flow velocity remains low. Therefore, "Bed protection failure due to flow" does not occur at any sea level rise. The bed protection can mostly only erode due to the jet created by water spilling over the gate. The water level of the channel behind the locks in Terneuzen is deliberately maintained around NAP+2.13 m, which amounts to a depth of 12.5 m behind the Western lock. One can imagine that with such a large depth, the jet of spilling water has lost most of its energy through turbulence by the time it reaches the bottom. Apparently at the considered range of sea level rise, the erosion of the bed due to a jet is incredibly rare.

Sub-mechanisms of inflow
For mechanisms "Failure due to head" and "Failure due to vibration", both gates lose their retention function. In case water is able to flow freely through the navigation lock, both probabilities of "Surpassage storage capacity" and "Bed protection failure" are larger than the legal norm, given any magnitude of sea level rise. This is presented in Figure 5.23. Thus, failure of both retaining elements leads to flood protection failure regardless.

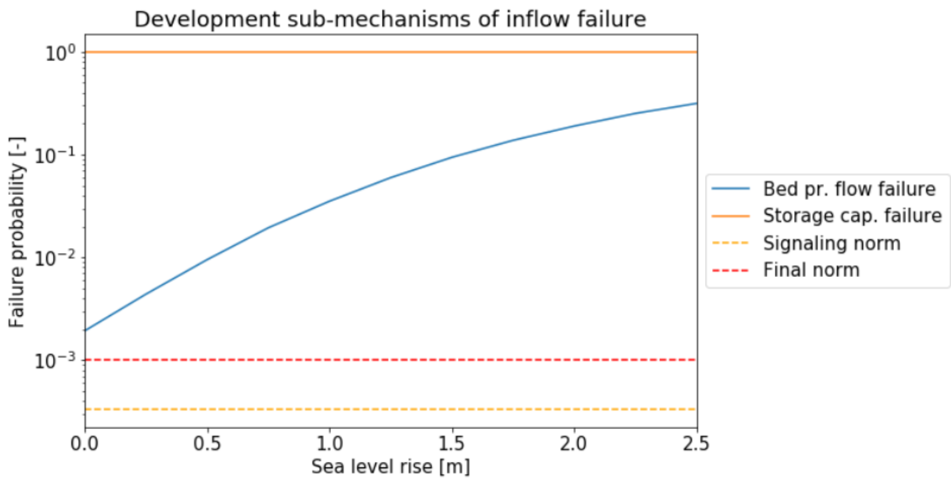


Figure 5.23: Development sub-mechanisms "Inflow failure" over sea level rise of the hypothetical case

- **Storage capacity surpassage due to inflow**
  Contrarily to overflow, for inflow the sea level does not have to be higher than the retention height of the lock. This failure mechanism is related to a situation in which all gates of a lock are failing their retention function. Unobstructed by any gate, a large volume of water can enter the system without requiring a sea level higher than the crest of the gates. It is therefore not unexpected that the probability of storage surpassage is equal to 1, regardless of the sea level rise.
- **Bed protection failure due to inflow**
  Similarly, the large discharge accompanied by inflow is related to very high flow velocities. The bed protection has not been designed to withstand these flow velocities, and when the sea level rises, the probability of erosion even approaches 1.

Sub-mechanisms of "Closure failure"
Failure mechanism "Closure failure" is defined in Section 4.2 as the combination of the closure failure of the outer gates with the consequential failure probabilities of the inner gates (which can be subjected to different conditions once the outer gate has failed). The contribution of each secondary failure mechanism to the total "Closure failure" is presented in Figure 5.24. It should be noted that the probabilities presented here are the *yearly* probabilities, which are reworked to a 12-hourly probability later in the process, as it is expected that a closure failure is mitigated within 12 hours (see Appendix B.1).

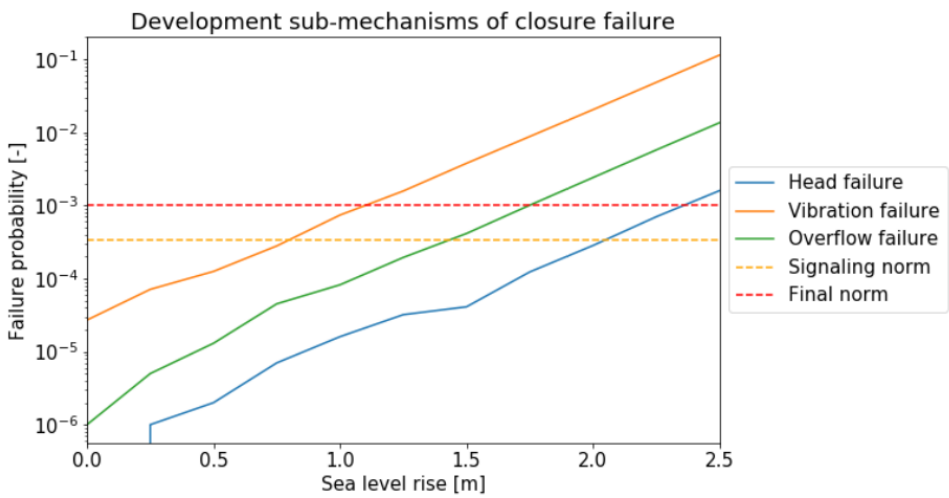


Figure 5.24: Development sub-mechanisms "Inflow failure" over sea level rise of the hypothetical case

The mechanisms show similar patterns as in Figure 5.21. This is to be expected, as the inner gates of the case study are identical to those in the outer head. However, because the mechanisms have to occur within a time span of 12 hours after a closure failure, the total contribution of this mechanism is insignificant.

Total failure probability flood defence function
At last the development of the total failure probability of the flood protection of the lock is presented in Figure 5.25. The legal norm is surpassed at a sea level rise of 1.1 m. As this is a case containing a lot of assumptions, the confidence of this value is still very low. What can be concluded, however, is that the failure probability grows exponentially with an increasing sea level. Around 1.75 m sea level rise, the yearly failure probability is already at 1:100, tenfold the current the norm. Because the case study is only *based* on the Western lock in Terneuzen, and no detailed design specifications have been retrieved, it cannot be validated whether the critical sea level rise found with the model is equal to the design sea level rise of the lock design.

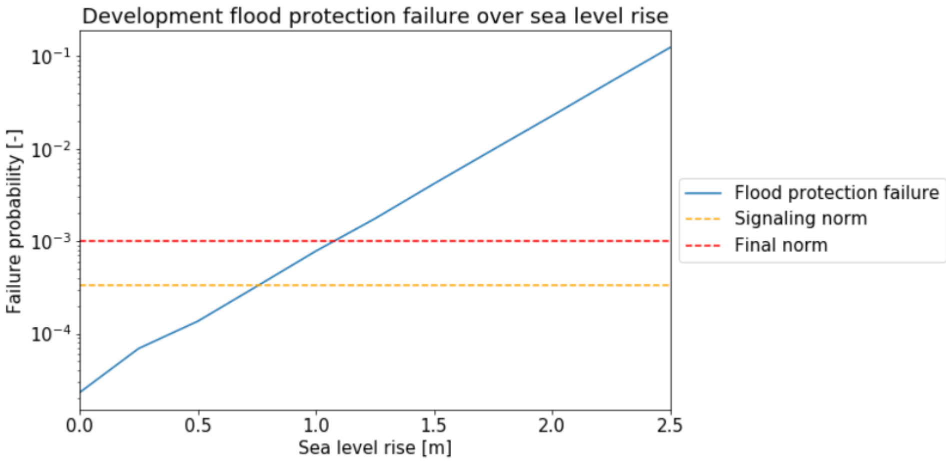


Figure 5.25: Development flood protection failure over sea level rise of the hypothetical case

5.5. Critical characteristics for structural adaptations

From the results presented in the previous section, a conclusion can be drawn as to what characteristics of the lock are trivial to the total failure probability of the flood protection function. These characteristics are the ones which are to be improved in case the sea level rises too much. These improvements are treated in the following chapter.

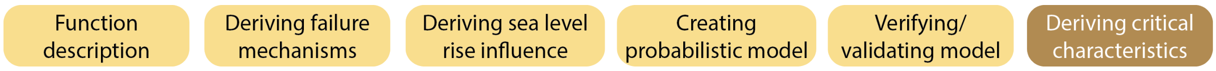


Figure 5.26: Chapter progression Section 5.5

"Vibration failure" contributes the most to the total failure probability. The only parameter of the lock involved in this mechanism is the retention height.

When the probability of "Vibration failure" is lowered, the mechanisms "Failure due to head" and "Overflow failure" become most significant. For the latter, however, the retention height also plays a significant role. Thus, when resolving "Vibration failure", "Overflow failure" is simultaneously also treated. To reduce the probability of "Failure due to head", the strength of the gate(s) will have to be increased, or they have to be replaced with stronger substitutes.

For this particular case, piping did not contribute to the total failure probability of the lock. However, it can be expected that this is not the case for all marine locks. The mechanism is initiated by a large head difference, which is undeniably more likely to occur when the sea level rises. For the study case, the head difference which would be present when the channel is nearly empty, would still not cause piping, due to the long piping length. This is a very rough requirement (maybe due to the construction method), which does not apply for all locks. Thus, for locks which have been designed with a less conservative piping path, this mechanism is likely to become a threat to its integrity. Therefore, it is important to also consider the length of the piping path as an important parameter when developing structural adaptations.

When a lock is adapted to withstand a larger head difference caused by sea level rise, and it is desired that the lock is still operable at this head difference, the bed protection may have to be improved in order to withstand the higher flow velocities accompanied by the emptying of the lock. One could argue that a lock could simply not operate at the new conditions made possible by a heightening of the lock, but when the sea level rises with a meter, this would imply that an entire meter is lost from the range of operable water levels. It could therefore be beneficial to assure that the lock can still operate at its original range of head differences. This would require an improvement of the composition of the bed protection, assuming it has not been conservatively designed initially.

To reflect, the critical characteristics for structural adaptations are presented in Figure 5.27, in relation to the relevant failure mechanisms. For the case study, the length of the piping path is however not included.

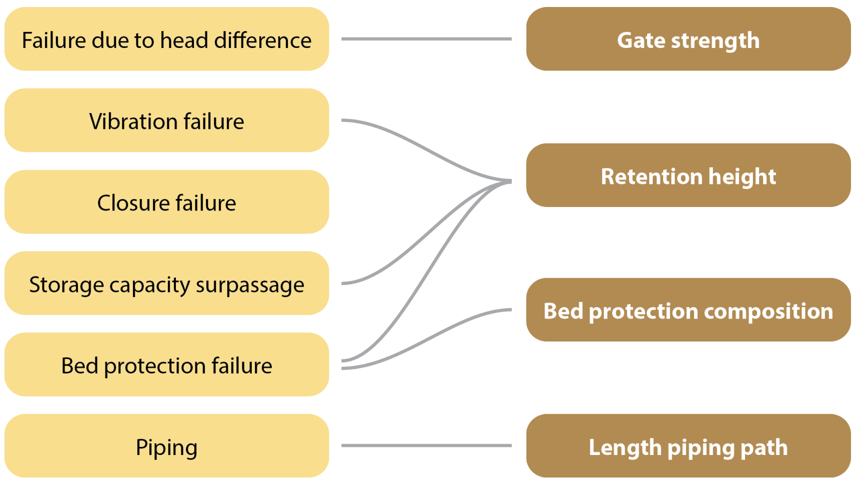


Figure 5.27: Relation between failure mechanisms and critical lock characteristics

6

Determining effectiveness of prolonging lifespan lock with adaptations

This chapter concerns the second step in the methodology, i.e. the development of support for making the decision to adapt or replace a lock to deal with sea level rise. In the previous chapter, the critical lock characteristics were derived which are to be improved in case sea level rise aggravates. In this chapter, it is determined whether it is effective to improve those characteristics by means of a combination of adaptations. From the critical characteristics, a base of design is drafted in Section 6.1. Thereafter, several adaptation concepts are developed *per characteristic* in Section 6.2. A method for evaluating the value and costs of these concepts is explicated in Section 6.3, based on which a selection is made *per characteristic*. A combination of selected adaptations (combining all characteristics) is than implemented in an adaptive pathways map, together with the alternative of a conservative lock replacement in Section 6.5, to compare their time dependent effectiveness in Section 6.6. After this, a recommendation on the decision is made in Section 6.7.

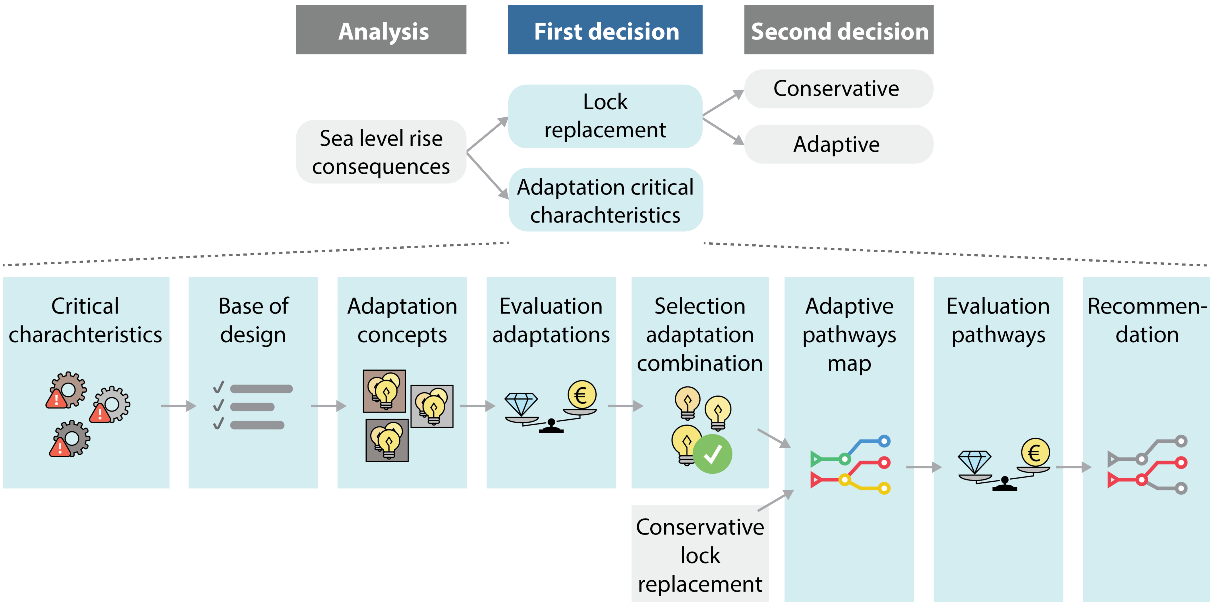


Figure 6.1: Flow chart of Chapter 6

6.1. Drafting base of design for adaptation concepts

To be able to develop adaptations which can improve the critical characteristics of a marine lock, a base of design is drafted. In the base of design, requirements, starting points, and boundary conditions for the structural adaptations are provided. A general base of design is drawn up, which can be applied to any marine lock. To present how the general base of design can be applied to a specific project, the case study of the Western lock in Terneuzen is used again. The application of the requirements is presented in italics in this section.

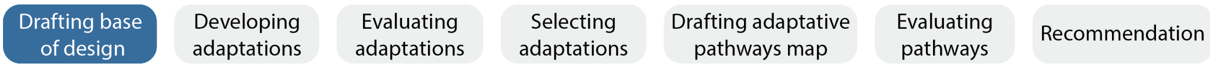


Figure 6.2: Chapter progression Section 6.1

6.1.1. Structural requirements

- Each adaptation must improve one or more of the critical characteristics derived in the failure probability development study presented in Chapter 5.

  For the case study, it was derived in Section 5.5, that these critical characteristics include the retention height, gate strength, and bed protection composition (the piping length would regularly be a critical characteristic, but not for this specific case).

- For an adaptation, the existing structure must be able to bear the additional weight and forces applied through the adaptation, or the components which do not have this structural capacity should be included in the adaptation.

  The assumption is made that the lock heads and chamber walls of the Western lock in Terneuzen cannot bear any substantial force addition. The shallow foundation, however, can carry a larger weight, as the downward direction load is often not the critical load condition for this foundation type, but instead the upward directed pore pressure during the construction period when the lock is emptied (Molenaar & Voorendt, 2018).

- The adaptation must be constructable:
  - ◇ The components which are being adapted must be accessible, or made accessible through excavation or demolition of the obstructing elements.
  - ◇ The construction has to be executable with existing equipment and construction techniques.

  No specific additional notes can be given for the case study.

- The structure must be sufficiently strong, stable, and stiff.

  The adaptation must suffice to the EN EuroCode norms (European Commission, n.d.), and to the ROK 1.4 in case Rijkswaterstaat is the client (Rijkswaterstaat, 2017).

6.1.2. Functional requirements

- The operability of a lock may only decrease minimally after the implementation of the adaptation.

  The maximum sea level at which currently leveling is allowed at the Western lock in Terneuzen is NAP+3.5 m. When the sea level rises substantially, this maximum water level should be increased as well to attain a similar operability. The structure of the lock must allow for this to happen.

- The dimensions of the lock chamber should not be adapted in any manner which would obstruct the sailing of any vessel which can currently pass through the structure.

  The current maximum vessel dimensions are a length of 265 m, a width of 34 m, and a draught of 12.5 m. This concerns a lightened Panamax with adapted length (De Gans, 2007).

- If the lock functions as an important cross-over for "dry" traffic, this function should be maintained.

  The case study has two bridges crossing over it: one between the two gates in the outer head, and one at the inner head, outside the gates on the side of the channel. These both have to remain functioning, to assure "dry" traffic can cross while vessels enter the lock, for which only one bridge has to be opened (De Gans, 2007).

- No electrical or mechanical element should face the probability of malfunctioning caused by water or debris transported by water.

  For the case study, this concerns the operating mechanism of the roller gates, and the bridges.

6.1.3. Starting points

- Preference goes to a construction which has little impact on the ecosystem in its direct area.

  For the case study, this concerns the ecosystem of the Western Schelt, Canal Ghent-Terneuzen, and above the water, the area surrounding the lock. The Western Scheldt is a protected area under the Natura 2000 (Ministry of Agriculture & Quality, n.d.).

- Preference goes to a construction method which does not provide much nuisance to the neighboring residents.

  The case study is located next to the city of Terneuzen. The inhabitants of this city are to be considered in the development of a construction method.

- Preference goes to a construction in which circular elements are included.

  No specific additional notes can be given for the case study.

6.1.4. Boundary conditions

- In the design, the specific boundary conditions of an individual project and location should be incorporated
  - ◇ Hydraulic boundary conditions on both sides of the lock
  - ◇ Ground water level
  - ◇ Subsoil composition

  The hydraulic boundary conditions of the case study were derived in Section 5.4.3, but can change over the years. The conditions should be analysed by the time the adaptation is actually designed. The same is true for the ground water level. The subsoil composition could also change due to settlement of the subsoil, as a new lock is currently constructed next to the Western lock in Terneuzen (Pfaff-Wagenaar & De Wit, 2015).

6.2. Development adaptation concepts for critical lock characteristics

In this section, adaptation concepts are developed based on the base of design drafted in the previous section. From the results of the failure probability analysis performed on the Western lock in Terneuzen in Section 5.4, a number of critical characteristics were listed which should be included in structural adaptations: the retention height, gate strength, length of the piping path, and bed protection composition. In this section, several adaptation concepts are developed per characteristic. Further in this chapter, a combination of those is selected based on an evaluation, which can then be compared to a lock replacement.

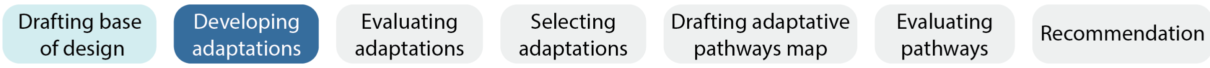


Figure 6.3: Chapter progression Section 6.2

The adaptation concepts have been derived by brainstorming how the critical characteristics can be improved. The concepts have been derived with the base of design in mind, and therefore all suffice to all functional and structural requirements.

This section is divided on basis of the critical characteristics. However, because for both the retention height and gate strength, the gates have to be improved, these requirements are fulfilled by the same concepts, and are therefore covered in a single section. Tables 6.1 and 6.2 provide an overview and brief description of the developed adaptations.

Table 6.1: Adaptation alternatives per lock characteristic (1/2). Adaptations "Gate replacement" and "Modular gate" also improve the gate strength.

Characteristic	Adaptation	Description
Retention height		
Lock head	Wall on outer lock head	A concrete retention wall of 0.5 m high installed on the outer lock head. The height is limited due to the low strength of the wall construction.
	Fixed lock extension	Vertical extension of the existing lock construction, i.e. the lock heads and chamber walls, accomplished by pouring concrete on the structure in combination with an adhesive. If the existing lock structure is not strong enough for the additional forces applied through the adaptation, it requires additional support, which is discussed under "Additional" in Table 6.2.
	Modular lock extension	Vertical extension of the lock structure by means of interlocking concrete modules and supporting landfill. As for the fixed extension, this adaptation is limited not by its own strength, but by that of the existing lock structure, which may require strengthening.
Gates	Gate extension	Small extension of the existing gate with a height of 0.5 m, to be used in combination with the wall on the outer lock head.
	Gate replacement	The gates can be replaced by taller and stronger ones. This will increase the weight on the support system of the gates, which therefore also have to be replaced by stronger ones.
	Modular gate	Instead of a regular gate, a modular gate can be installed. With such a design, material is initially spared, but can be added by means of a module in case of a severe sea level rise scenario.

Table 6.2: Adaptation alternatives per lock characteristic (2/2).

Additional	Structure support	The fixed and modular lock extension may require additional support of several lock elements. The lock heads and chamber walls can be enforced by means of sheet pile walls and diagonal support piles. A pile foundation can be improved by boring piles into solid lock head and chamber wall constructions. A shallow foundation can be assumed not to require any strengthening.
Piping path	Cut-off screen	Often locks already have cut-off screens installed. Additional ones can be implemented to lengthen the piping path. The screens can be installed in front of or behind the lock, and connected to the existing structure with underwater concrete.
	Sandtight filter	The piping path can be elongated by means of a sandtight filter beneath the bed protection. To install this, the bed protection will first have to be removed, and deposited back on top of the filter once it has been placed.
Bed protection	Colloidal concrete	Colloidal concrete has small grains, which can fit in between the rocks of a bed protection. By penetrating the top layer of a bed protection with colloidal concrete, the rocks can no longer erode. Colloidal concrete can be poured underwater.
	Replacement	Instead of improving the existing bed protection, it can be replaced. This alternative can be well combined with the placement of a sandtight filter, which improves the piping path.

6.2.1. Retention height increment and gate strengthening

Both the lock structure and the gate have to be improved. For the lock structure, three concepts are provided: "wall on outer lock head", "fixed lock extension", and "modular lock extension". All concepts include an increment of the retention height of the lock. For the gate, also three alternatives are developed: "gate extension", "gate replacement", and "modular gate".

6.2.1.1 Wall on outer lock head

A fairly simple and inexpensive manner to increase the retention height of a lock is by mounting a small concrete wall on the edge of the structure. This method has been applied at the Prins Bernard locks to increase the retention height with 0.5 m (Rijkswaterstaat, 2019). This solution has a negligible effect on the integrity of the existing structure, as its weight can be assumed to be insignificant. It can thus be applied to any lock, regardless of any surplus of bearing capacity and stability available in its design.

The solution is not applicable for heightening the structure with much more than the 0.5 m of wall, like has been installed at the Prins Bernard locks. Any additional height allows for a larger head difference to be retained, resulting in a larger force on the gate, and thus its connection to the lock head. In case of a small heightening, the additional force transmitted to the lock head is minimal, and can be assumed to be borne by the structure. However, for a larger force increment this assumption falls short, and either detailed calculations have to be performed, or the entire lock head has to be modified. For this thesis it is assumed that the 0.5 m is the maximum allowable additional retention height without requiring additional alteration.

Figure 6.4: Retrieving wall at the Prins Bernard lock. The openings in the wall are closed off during extreme conditions. Retrieved from Rijkswaterstaat (2019).

6.2.1.2 Fixed lock extension

To increase the retention height of a lock, the current cross section of its lock heads and chamber walls can be vertically extended. To do so, a new layer of concrete can be pored onto the original structure. This will not naturally bond to the originally present concrete. Therefore, an adhesive layer has to be applied on top of the original structure before poring the new concrete on top. The adhesive material will initiate a chemical reaction which causes the layers of concrete to bond to one another (NEN-EN 1504-4, 2005).

Before applying the new concrete, however, the new reinforcement cage has to be installed and connected to the original structure. This can be done by drilling holes into the concrete, in which the rebars can be inserted (EOTA, 2015). Obviously this does not provide a sufficient bond between the reinforcement and the concrete. Therefore, the hole is filled beforehand with mortar, which acts as a binding medium. The holes are required to be at least as long as the mandatory anchorage length of the bars.

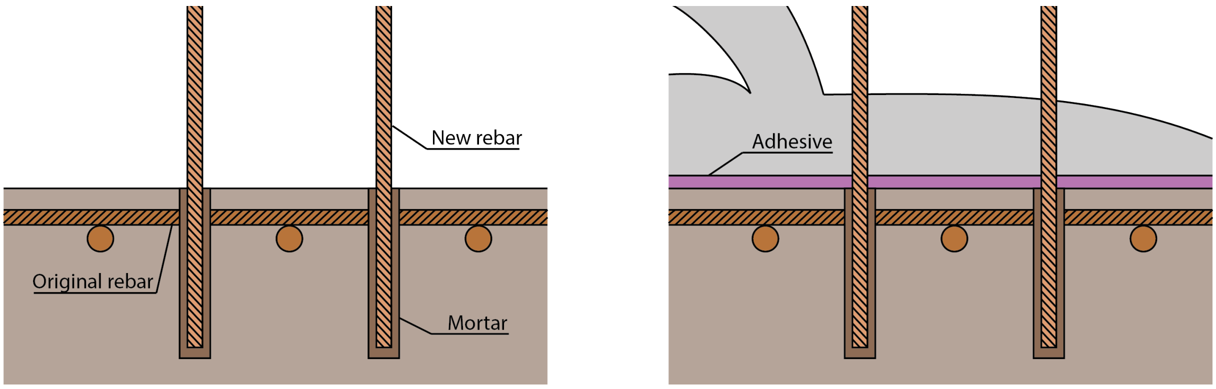


Figure 6.5: Stages of applying new layer of reinforced concrete

In addition to the heightening of the lock structure, the soil around it also has to be supplemented to the height of the lock. No special measures have to be taken for this. Soil can simply be delivered and treated as would be done regularly.

Many marine locks also function as crossing for "dry" traffic, and therefore have one or two bridges crossing the structure. If the lock is heightened, the bridge has to be elevated along with it. The bridge structure currently in place can be dismounted, and placed back after the subsoil, abutment, operating mechanism have been improved, replaced or elevated.

The weight of the extended lock structure can be rather substantial. The original lock components will not necessarily be capable of carrying this additional load, if any. It has to be analysed per project whether any surplus is available in the existing structure to bear the additional load. Thus, when applying this concept, additional adaptations might have to be performed to support the structure, if the arrangement of the structure allows this. These are discussed in Section 6.2.1.7. The height to which this concept can be applied is therefore also case specific.

6.2.1.3 Modular lock extension

Instead of extending the existing design of a lock structure, a modular alternative can be applied. An advantage of this is that heightening the structure is a simple matter of placing new modules on the existing construction. It does not matter if the height reveals not to be sufficient, as a second or third layer can easily be added on top of the structure.

In addition to this, the use of identical modules increases the circularity of the structure. In case the modules are no longer required, they can be re-used for other locks or even completely different structures, depending on the type of module used.

Modules could be applied in different matters and sizes. A practical example is the use of interlocking concrete or stone blocks. Such modules have previously been applied for the construction of retention walls, as presented in Figure 6.6 (CPM, n.d.-c), and might also lend themselves for increasing the retention height of a lock. Modules can be arranged at the edge of the lock structure, with landfill behind them. It should be noted that it seems that such modules have to this date not yet been used in the construction of locks. This solution serves merely as a concept.

Figure 6.6: LEFT: Example of build-up interlocking modules. RIGHT: Example of application modules to retention wall. Retrieved from CPM (n.d.-c).

Figure 6.8 presents how interlocking modules could be incorporated to increase the height of a lock. A concrete segment can be bonded to the original lock structure by means of an adhesive layer, initiating a chemical reaction between old and new concrete (NEN-EN 1504-4, 2005). This segment will have an identical locking mechanism as the modules that will be used (this mechanism can vary). Sequentially the modules can be attached to the lock head and chamber walls, and the space in between can be filled with soil to provide support.

The seal between the gate and lock head requires extra attention, as this regularly extends from the rectangular cross section of a lock head, and can be made from a different material, like timber. Timber modules could be incorporated in the structure, perhaps also with an interlocking system. This would however require an adjustment to some of the concrete blocks, specific for the lock, decreasing the re-usability of a number of modules.

There are interlocking modules in existence to which reinforcement bars can be applied (CPM, n.d.-a). These are partly hollow, but have to be filled with concrete to create a bond between the module and the rebar. Thus, all circularity of the concept is completely lost, and if a new layer is to be added atop the structure, a similar process as depicted in Figure 6.5 has to be executed. Without any reinforcement, the function of the modules is merely to increase the retention height of the structure, and no additional moment can be transmitted through the structure. Modules can also be fitted with a geogrid, affirming a connection between the landfill and the block (CPM, n.d.-b). For such modules, adding an additional layer is again simply a matter of placing the components, and no circularity is lost. What modules are most fitting to increase a lock's height may vary according to the requirements of a project.

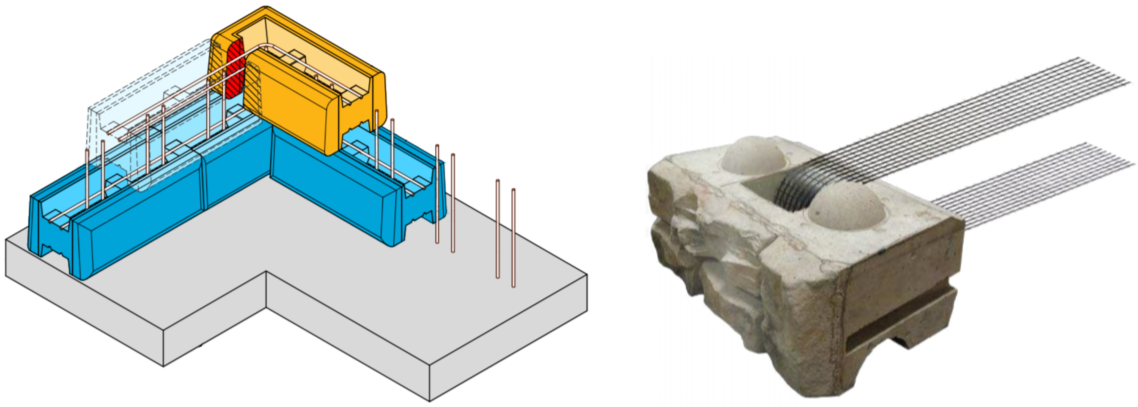


Figure 6.7: LEFT: Modules equipped with reinforcement. Retrieved from CPM (n.d.-a). RIGHT: Module equipped with geogrid. Retrieved from CPM (n.d.-b).

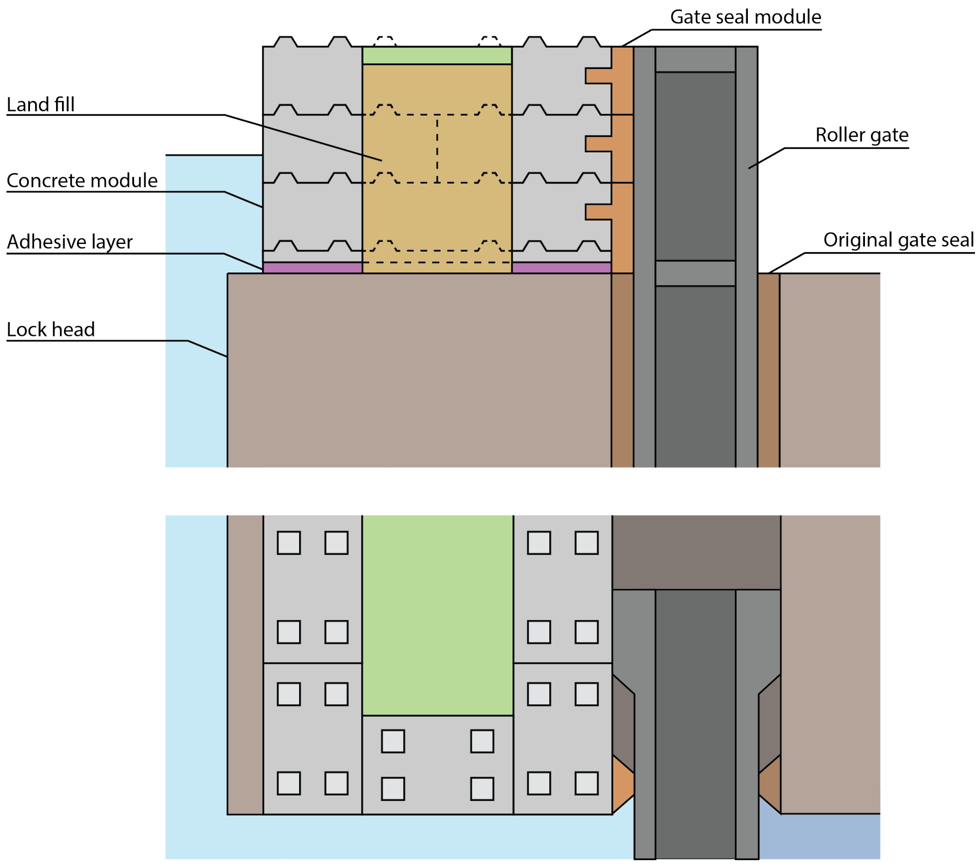


Figure 6.8: Example use of interlocking modules for lock heightening. TOP: Side view. BOTTOM: Top view.

As for the fixed lock extension, it has to be analysed whether the existing lock components are able to bear the additional forces accompanied by a modular lock extension. If the arrangement of the lock allows it, strengthening of the components might be applied, as explained in Section 6.2.1.7. Also if any present bridges are located at the lock, these will have to be elevated along with the heightening of the lock structure.

6.2.1.4 Gate extension

To attain an increased retention height over the entire width of the structure, the gates have to be heightened simultaneously with the lock head. Concept "Wall" only includes a small addition of 0.5 m, and therefore also requires no extensive adaptation of the gate. A structurally unimpressive extension with an identical height can be mounted on the currently employed gate. Similarly to the wall extension atop the head, the additional weight and forces applied by a head difference increment of 0.5 m are assumed not to exceed the structural capacity of the gate, the gate connections, and the lock head.

6.2.1.5 Gate replacement

In case the retention height has to be increased with more than 0.5 m, and a fixed or modular lock extension is installed, a more severe gate adaptation has to be undertaken. One option is to completely replace the gate by a taller one. Because the strength of the gate is also a critical characteristic with respect to sea level rise, the replacing gate should be made stronger, able to withstand forces accompanied by the newly acquirable head difference.

A larger gate means a larger weight on the gate support, i.e. the rails in case of a roller gate and the bearings in case of mitre gates. An increment of the load on these elements can lead to faster wearing of the steel. It might be necessary to also reinstall a stronger support system for the gate, and maybe even a better operating mechanism to pull the larger weight of the gate. This is further discussed in Section 6.2.1.7.

6.2.1.6 Modular gate

Instead of a regular gate, a modular gate can be installed. TU Delft student M. Levinson (2018) has performed his thesis on the design of a modular lock gate. This was executed within the scope of the standardisation of lock gates throughout the Netherlands. By mounting a different amount of modules on top of one another, a different gate height can be achieved. Thus, the gate can be installed for locks around the country with varying retention demands.

This gate design could also be used against sea level rise. It would be beneficial if every time the sea level tends to become too high, heightening the gate is a simple matter of mounting another module onto the gate. Not only does this make the operation a lot less complicated, it also cuts production costs, as no entire new gate has to be manufactured.

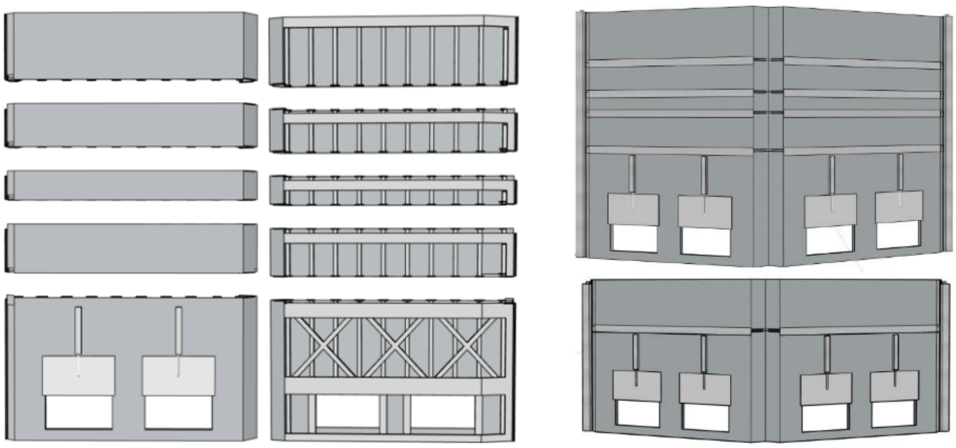


Figure 6.9: Modular rendition of mitre gates. LEFT: Front and back of modules. RIGHT: Two different combinations of modules to attain a different retention height. Retrieved from Levinson (2018).

It should be noted that the design made by M. Levinson is an adaptation of a regular set of *mitre gates*. Many marine locks are equipped with mitre gates, but roller gates are also often applied (see Table 2.1). For this thesis, it is assumed that a roller gate can be built effectively using modules in a similar manner as the design by M. Levinson. If this would not be possible, it is at least fathomable that a non-modular roller gate could be made so that a module can be attached at its crest. The adaptation of the gate therefore remains equal: an extension of a gate module. Only the initially installed gate would vary. This is further explored in the replacement concepts, where one of the gate types initially has to be installed (see Section 7.2.2)

As for a regular lock replacement, the addition of a module to the gate increases its weight, and allows for larger forces to be applied by an increased retainable water level difference. To maintain the simplicity of the heightening procedure of a modular gate, the support and operating mechanism should be adapted to be able to bear these larger forces at the initial installation of the gate.

6.2.1.7 Additional heightening adaptation requirements

For the heightening concepts which include a significant addition to the retention height, and therefore a significant addition to the forces on the structure, some adaptations might be required to strengthen the construction. Whether the lock already contains the capacity to transfer these loads, it is henceforth referenced as a strong lock (S). If it does require strengthening, it is called weak (W). The strengthening adaptations are discussed in this section.

Lock head and chamber walls

When installing an extension of the lock head and gate, the existing structure has to be able to bear both the additional weight, and the increment of forces applied by the head difference which can be retained due to the increased height. For this thesis, it is assumed that the lock head, chamber walls, foundation, floor, gate bearing, and operating mechanism have to be adapted to be able to bear these additional forces. Depending on the arrangement of a lock, such modifications of the structure might be possible.

For the strengthening of the lock head and chamber walls, the renovation project of the Southern lock in IJmuiden is regarded (Beem et al., 2000). This lock had to be able to withstand a larger head difference, an issue similar to sea level rise. For this project, the lock head was enhanced by means of a combi wall and diagonal piles (seen at the left in Figure 6.10), whereas the chamber walls were improved with sheet piles (on the right in Figure 6.10). For such constructions to be possible to be applied, a sufficient area next to the lock is required, and the lock chamber has to be emptiable to reduce the horizontal pressure on the walls when the soil has been excavated. Thus, the gates have to be designed to withstand the large head difference present when the lock chamber contains no water.

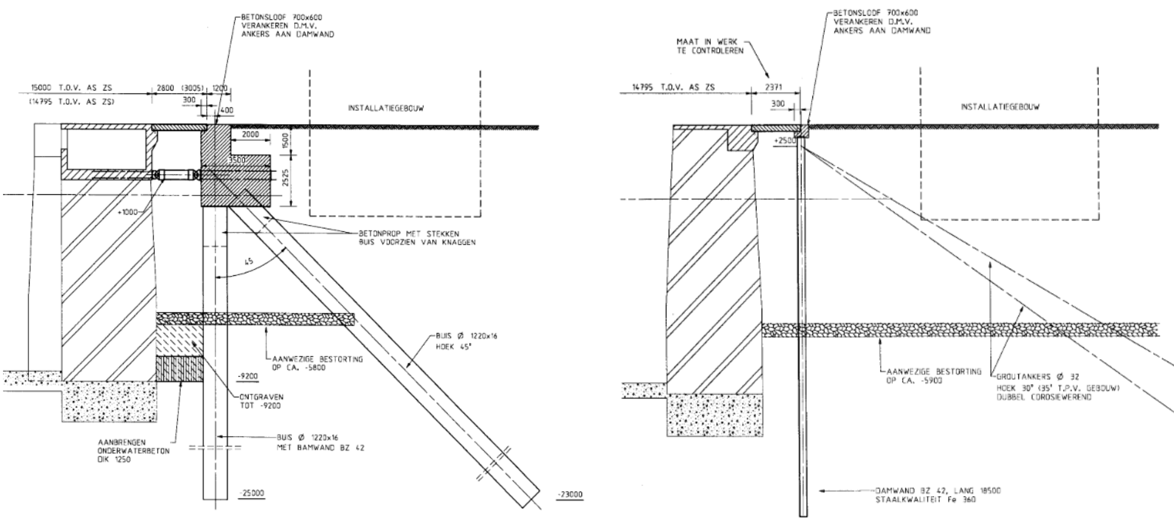


Figure 6.10: LEFT: Enhancement of lock head construction at the Southern lock in IJmuiden. RIGHT: Enhancement of chamber wall. Retrieved from Beem et al. (2000).

Foundation

The foundation is hard to access, because it is positioned beneath the floor. For a lock, two types of foundations can be applied: shallow or deep. A shallow foundation consists of a large concrete plate, transmitting the weight to a large area of the subsoil. A deep foundation is comprised of piles, which transfers the load to the soil both through the tip and the shaft of the piles (Molenaar & Voorendt, 2018).

For shallow foundations, the vertical load is rarely the critical loading, but rather the upward pore pressure occurring when the lock is emptied during construction or maintenance. Therefore, a lock with a shallow foundation is assumed to be able to bear the additional weight from the fixed or modular extension.

For pile foundations, on the other hand, the bearing capacity is a critical parameter in its sufficiency. For this component, the renovation of the Oranje locks in Amsterdam is regarded (Beem et al., 2000). Here bore piles were installed through the lock head into the subsoil to support the heightening of the lock head and chamber walls. The bore piles could be installed into this particular structure because of its massive masonry, in which the bore piles could be well secured.

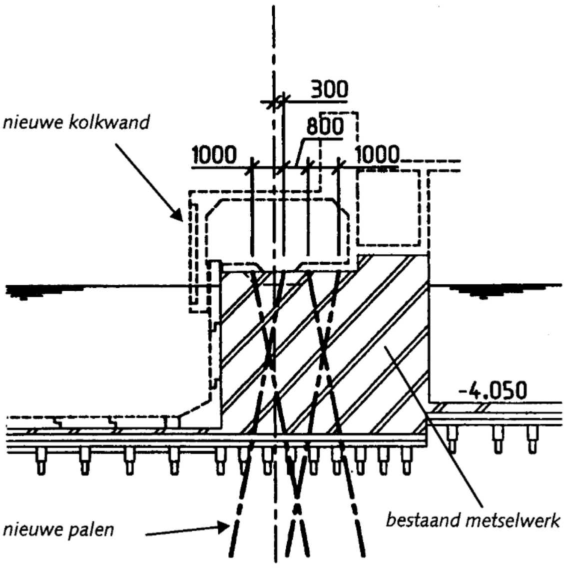
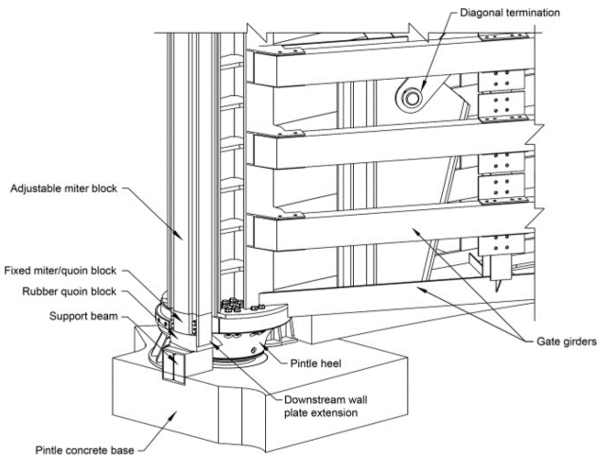


Figure 6.11: Addition of bore piles at the Oranje locks. Retrieved from Beem et al. (2000).

Gate bearing

By heightening the gate, larger forces are allowed to be transmitted to the lock head. Therefore, the system connecting the gate to the lock head also has to be able to bear such forces. The type of bearing system in place depends on the type of gate. A mitre gate is connected to the lock head at two pivot points, but often the bearing is located at the bottom (see Figure 6.12 on the left). Roller gates are supported by roller carriages, which transmit the weight of the gate to a rails (see Figure 6.12 on the right). For both systems, it is for this thesis assumed that the additional weight of the larger gate causes too much wear of the the bearing systems, meaning these have to be replaced.



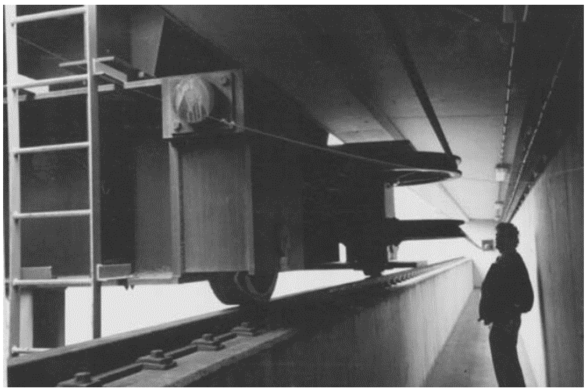
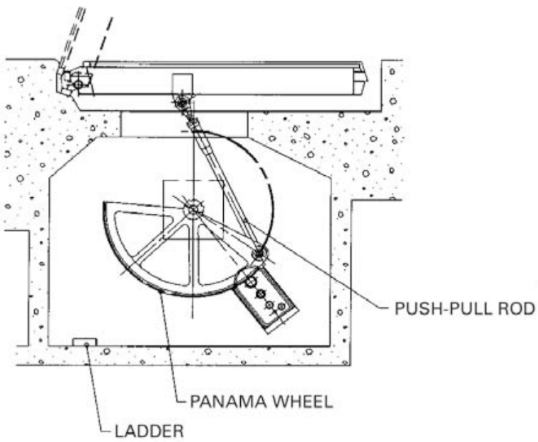


Figure 6.12: LEFT: Bearing of mitre gate. RIGHT: Roller carriage and rails of roller gate. Retrieved from Beem et al. (2000).

Operating mechanism
Depending on the gate type, a different operating mechanism will be in use. Mitre gates are closed and opened using one of three systems: a Panama wheel, rack-bar, or hydraulic cylinder (Beem et al., 2000) (see Figures 6.13 and 6.14). All are situated in a chamber in the lock head, and move the gate through a hole in the structure. If the water level becomes too high, it can enter this hole and cause damages. Electrical equipment can be stored in a dry room, but for the Panama wheel and rack-bar, debris and dirt can clog the gears, obstructing the mechanism from performing. For the hydraulic press, this is not an issue.

If mitre gates are replaced with taller ones to retain higher water levels, the chambers where the operating mechanisms are located are more likely to be flooded. For a Panama wheel or rack bar, this would mean that the entire chamber and mechanism would have to be relocated at a higher place. In case of a modular lock extension, creating a chamber in the modules and landfill is not optional. A better alternative is to equip mitre gates with hydraulic presses. The room required for this system is smaller than for a Panama wheel or rack-bar, meaning enough space should be available in the lock head.

If new gates would be much heavier than the original ones, it has to be checked whether the operating mechanism is still able to move the gates at an acceptable rate, if at all. When this is not the case, the critical elements of the mechanism have to be replaced with more effective ones.



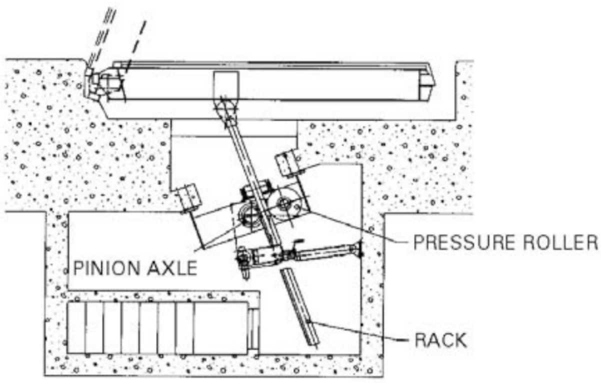


Figure 6.13: LEFT: Panama wheel. RIGHT: Rack-bar mechanism. Retrieved from Beem et al. (2000).

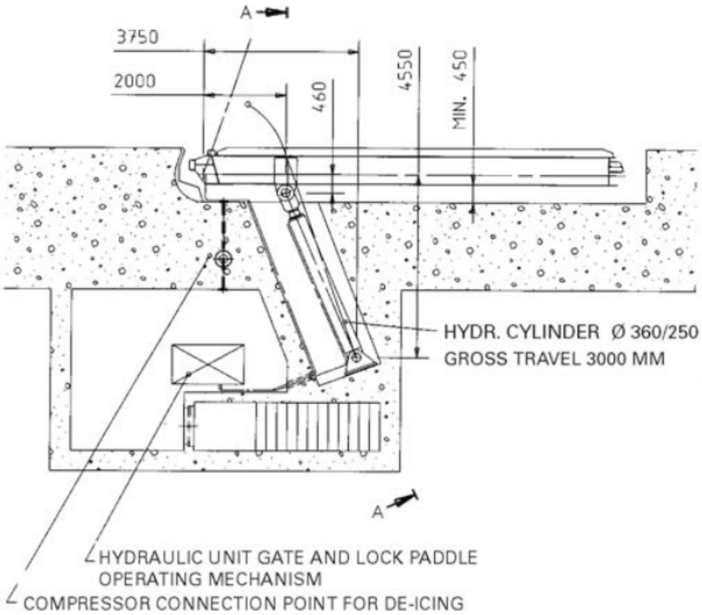


Figure 6.14: Hydraulic press mechanism. Retrieved from Beem et al. (2000).

For a roller gate, the operating mechanism is rather different, as presented in Figure 6.15. Roller gates are positioned on roller carriages, which can move horizontally over a rails. The gate is connected to cables which can pull the gate open or closed by means of one or two cable drums (Beem et al., 2000). The cables are positioned at the top of the gate, as these are not to be in contact with water. If the cables become wet, they will rust. If the sea level rises, the probability of the cables getting wet increases. When a taller gate has to be is installed, the cables have to be elevated to the new top of the structure. If it is not possible to redirect the cables from the cable drum, the entire mechanism has to be relocated to a higher level. The cable drums are located in watertight chambers, and in principle require no relocation unless this is provided to assure that the cables outside the chamber are always dry.

As for the mitre gates, a check has to be performed when taller or stronger gates are to be installed, to assess if the operating mechanism is still effective enough to move the gate. If not, critical components should be replaced. The same applies to the roller carriages and rails beneath the gate.

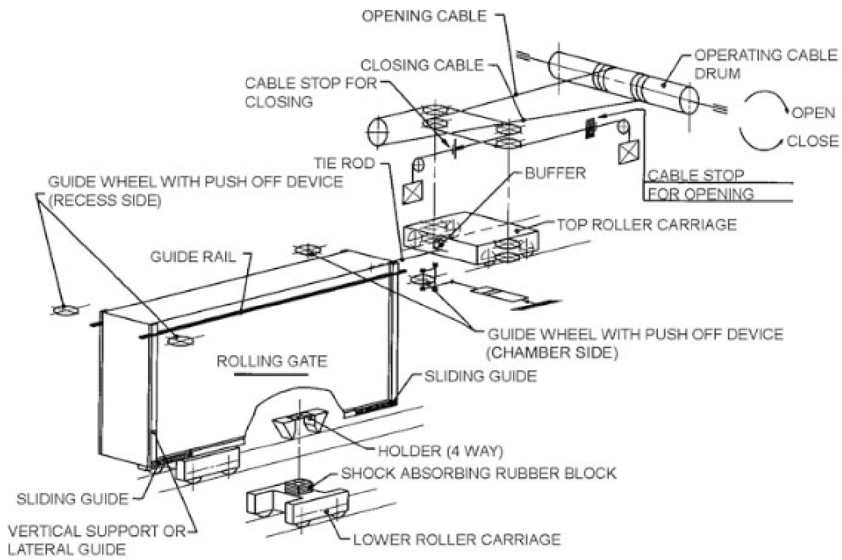


Figure 6.15: Roller gate operating mechanism. Retrieved from Beem et al. (2000).

6.2.2. Elongation piping path

The length of the piping path is the shortest route from one side of the lock to the other over which grains can be transported. This route always horizontally follows the bottom of the lock, and in case cut-off screens are implemented also vertically along the screens. Here two manners are discussed in which the piping length of an existing lock can be elongated.

6.2.2.1 Additional cut-off screen

Some locks already have cut-off screens installed to assure piping does not occur. In case these are no longer sufficient due to sea level rise, an additional screen can be installed. Regularly these would be installed beneath the lock head, but as this is no longer accessible, it will have to be located right before the construction. The cut-off screen has to be drilled or driven into the subsoil, depending on the soil-type and noise restrictions. After installation, a connection has to be made between the cut-off screen and the lock head, to obstruct water from flowing over the screen. This can be done by poring concrete over the installation site, thus extending the lock head horizontally.

If this method is applied, a conservative additional length will be implemented with respect to the sea level rise. In theory it is possible to install even more cut-off screens, or to drive the screen further into the soil and attach a second one at the top to even further elongate the piping length. It would however be more sensible to initially assure that the additional cut-off screen will suffice for the rest of the life time of the lock.

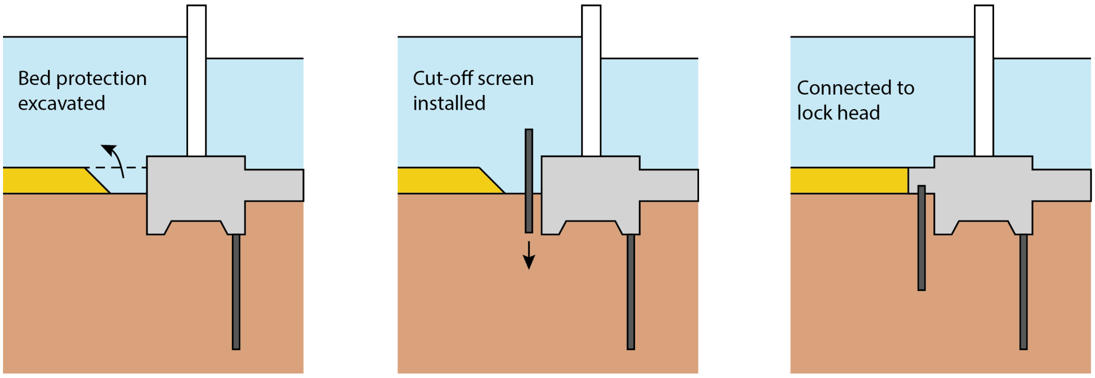


Figure 6.16: Installation of additional cut-off screen

6.2.2.2 Sandtight filter

Instead of adding vertical distance to the piping length, it could be elongated horizontally, by means of an impermeable filter beneath the bed protection. Such a filter can be applied using geotextiles: a variability of woven or non-woven porous synthetic materials which can be fabricated with a range of pore sizes (Schierink & Verhagen, 2016). By selecting a material which has such a permeability that it allows water to flow through it, but not the smallest of granular particles present, the piping path is lengthened.

A geotextile has to be installed beneath the bed protection, which is obviously inaccessible. Therefore, for this solution, an area of the bed protection, as large as the piping path requires to be elongated, has to be excavated. Once the filter is placed, the bed protection can be deposited back in place.

Similar to the cut-off screen, it would be very insensible to not conservatively elongate the piping length. This operation is rather drastic, and should not be executed multiple times.

6.2.3. Improvement bed protection

To improve a bed protection, the flow velocity at which the top layer of the composition erodes has to be increased. Additional larger rocks cannot simply be dropped upon the original top layer, as this would decrease the available draught for passing vessels. For this thesis it is assumed that no height can be added to the bed profile. The other available options to enhance the bed protection are described in this section.

6.2.3.1 Colloidal concrete

Instead of adding heavier rocks to the composition, the existing top layer can be penetrated with colloidal concrete (Betonhuis, n.d.). In this concrete natural polymers are present which ameliorate the attraction of small particles. By filling the pores between the rocks of a bed protection with colloidal concrete, the composition is bonded to a coherent mass, not allowing the rocks to be eroded, regardless of high flow velocities. The concrete is available as both a watertight substance, and an open texture, which is appropriate depending on the bed protection requirements. The concrete can only be penetrated in case the pores between rocks in the top layer are large enough for the concrete to be pored in between. For a marine lock it can be assumed that the top layer of the composition consists of large enough rocks for this procedure.

It is assumed for this thesis that the penetration with colloidal concrete is always optional, as the only requirement is a top layer with a large enough grain diameter. There is however an important downside to the alternative. When poring colloidal concrete, a portion of it will be carried off by the present water flow, and end up downstream of the site. Thus the area downstream of the bed protection is contaminated with concrete, which might affect the ecosystem. This alternative is therefore not perfectly sustainable.

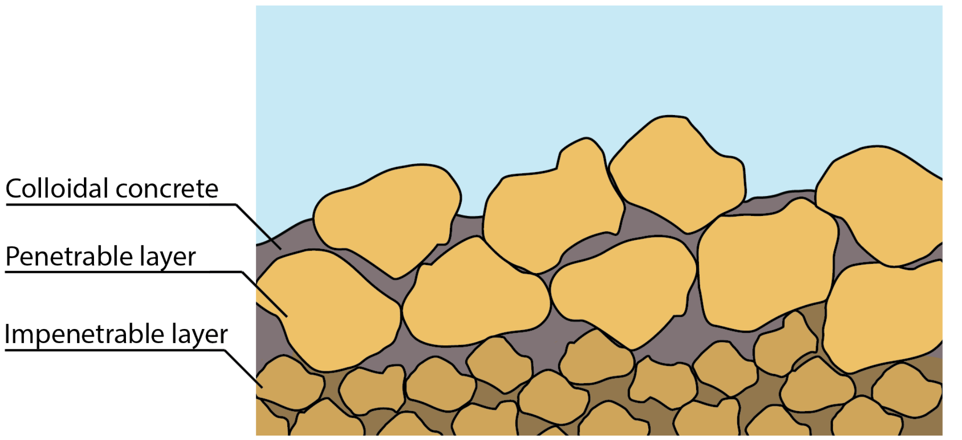


Figure 6.17: Bed protection penetrated with colloidal concrete

6.2.3.2 Replacement bed protection

Another available option is to entirely replace the bed protection. By doing so, the composition of the layers can be redesigned to withstand larger flow velocities. If the top layer of the bed protection has a larger nominal grain diameter, a larger flow velocity is required to lift it off the bed. More of the subsoil has to be excavated to allow for more layers of bed protection to be placed on top of one another without reducing the available draught. This also allows for a sandtight geotextile to be placed beneath the bed protection in order to elongate the piping path.

6.2.4. Application adaptations to case study

To review if and how these concepts can be implemented at an actual lock, the adaptations are applied to the case study of the Western lock in Terneuzen. This is thoroughly described in Appendix F. The most notable distinctions and nuances of the applications are described in this section.

Retention height increment and gate strengthening

The retention height concerns both the lock structure and the gate. The lock is heightened by means of three possible adaptations: a wall on the outer lock head, a fixed extension, and a modular extension. These can all be applied to the case study. However, for the lock extensions, it is assumed that the existing lock cannot bear the additional weight and additional horizontal forces applied by the higher retainable head difference. Therefore, the lock heads and chamber walls have to be supported by means of sheet pile walls and additional diagonal bearing piles for the heads. The foundation requires no further strengthening, as the lock has a shallow foundation, for which often the downward load is not critical, but the upward pore pressure during the construction period, when the lock is emptied.

The gates (of which two are present in the outer and inner lock head) can be heightened with a gate extension, gate replacement, or gate module. For the latter, an already competent gate has to be installed, specifically manufactured to bear the additional forces applied to the module. All adaptations can be applied to the case study. For the gate replacement and module, an operating mechanism which can bear a larger weight than the original gate has to be already installed or replaced together with the adaptation of the gate.

Elongation piping length
The case study requires no application of any adaptation to elongate its piping length. The current piping length is already long enough to assure no piping will occur at a head difference of over 13 m. This is far larger than any head difference which would occur logically at the site.

Improvement bed protection
To improve the bed protection, it can either be completely replaced, or penetrated with colloidal concrete. The replacing bed protection requires a larger nominal grain diameter in its top layer, which can withstand a larger flow velocity. The colloidal concrete does not have to be applied to the entire length of the bed protection. The flow velocity does decrease with distance from the lock, and therefore, the concrete only has to be pored in the area close to the outlet.

6.3. Development and application cost-value comparison method

Because multiple lock characteristics have to be improved, and the adaptation concepts only concern one or two of those characteristics, several adaptation concepts have to be combined with one another to assure the flood safety function of a marine lock. To be able to effectively select an adaptation combination, the concepts have to be compared to one another *per characteristic*. Thereto, the concepts have to be evaluated. The different concepts developed to improve the retention height, piping length, and bed protection have a similar effectiveness in their purpose to eliminate the effects of sea level rise. They can however perform very differently on other facets, and have different costs. In this section, a method is presented to evaluate the value of the concepts based on several criteria, and compare these to their costs.

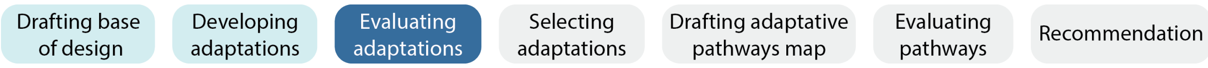


Figure 6.18: Chapter progression Section 6.3

Figure 6.19 presents a flow chart of the process of deriving the value and costs of the concepts. Firstly an ambition web is made, which entails the range of relevant criteria, and their weighting, i.e. their importance relative to one another. Using these criteria and weights, the concepts can consecutively be graded to acquire a single score (value) per alternative. The costs are derived separately from the grading process. At last it can be reviewed based on both the value and costs whether a selection can be made which of the concepts per characteristic are preferable.

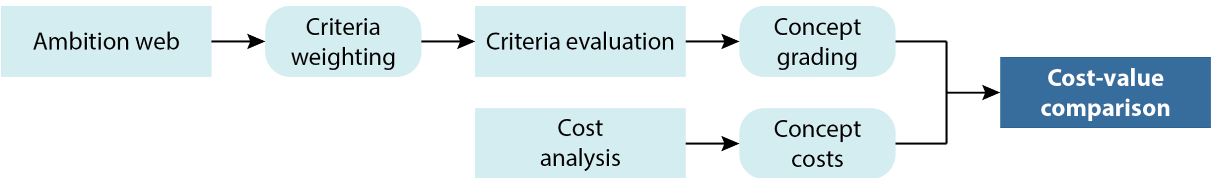


Figure 6.19: Flow chart of value and costs derivation and comparison process

The adaptation concepts which are evaluated in this section are listed below. Because for the case study of the Western lock in Terneuzen the piping length is no critical characteristic, the adaptations improving this length are not included in the example of the application of the evaluation method.

1. **Retention height lock head**
   1.1. Wall on lock head
   1.2. Fixed lock extension S (for sufficiently Strong locks)
   1.3. Fixed lock extension W (for Weaker locks, requiring additional support of components which lack structural capacity)
   1.4. Modular lock extension S
   1.5. Modular lock extension W
2. **Retention height lock head**
   2.1. Gate extension
   2.2. Gate replacement
   2.3. Gate module
3. **Bed protection composition**
   3.1. Colloidal concrete
   3.2. Bed protection replacement

6.3.1. Ambition web

The concepts are evaluated by means of five different criteria, as listed below. These criteria are chosen because of their impact on the functional value and societal value of the adaptations. Not all criteria are equally important for a project. For instance, for a lock which is located closely to a city, the construction nuisance is an important factor, but for a lock in a rural area, it has little significance. Therefore, a weighting, or "ambition", has to be specifically assigned to each criterion for each individual project. By means of this ambition, the significance of the facets is accredited. The case study of the Western lock in Terneuzen is used here as example, with Rijkswaterstaat as its client. The weightings chosen are an indication of what these values could have been in case this method would have been applied to the case study, and are not extracted from the actual design process of the lock. The weighting is visually presented by means of an ambition web, as shown in Figure 6.20.

- **Construction time:** The longer an adaptation takes to be executed, the longer the lock will be out of service. This may completely obstruct vessels from passing through the waterway. However, a lock is often part of a lock complex, meaning that the inoperability of a single lock merely limits the capacity of the waterway, and does not fully diminish it.

  The Western lock in Terneuzen is one of the three locks that make up the entire lock complex. Currently a very large lock is under construction which is to replace an old and outdated lock. The third lock, the Eastern lock, is rather similar in dimensions to the Western lock. Thus, when the Western lock is inoperable due to an adaptation, two substantial locks still allow traffic in and out of Canal Ghent-Terneuzen, albeit not at the desired capacity. Therefore, an ambition of 2/4 is considered for this criterion.
- **Construction nuisance**: During a construction period, the neighboring residents and users of the lock can undergo nuisance in several forms, like traffic obstruction, noise and vibration.

  The studied lock complex of is located directly next to the city of Terneuzen. To guarantee contentment of the residents, their objections in terms of nuisance should be taken into consideration. This allows for a construction procedure without interference. Therefore, an ambition of 3/4 is considered for this criterion.
- **Familiarity procedure**: The less familiar a procedure, the more prone it is to mistakes and hazards during construction. Additionally, more time has to be spent on sorting out how each step in the procedure has to be executed, and how this can be done safely. Thus, using unfamiliar techniques and procedures can be a risk for the costumer, as the occurrence of delays is more probable when there is little experience with a construction method.

  Given the importance of the flood safety of the Netherlands, and given the magnitude of the issue that is sea level rise, the unfamiliarity of a procedure should not stand in the way of a solution. However, it is unlikely that Rijkswaterstaat is very willing to execute a completely unknown procedure. It is expected that the criterion would be considered moderately important, i.e. an ambition of 2/4.
- **Sustainability**: Construction can damage the environment in many ways, such as by emission of greenhouse gasses, pollution of the air, and intoxicating the water. This can affect both the environment and human health.

  Given the current climate change developments, and especially as the regarded issue of this thesis is sea level rise - a direct consequence of climate change - it is important to assure that the concepts are sustainable. Rijkswaterstaat highly values sustainability, and for instance aims to be energy neutral by the year 2030, and reduce CO_2 emissions as much as possible (Rijkswaterstaat, n.d.-c). Therefore, an ambition of 4/4 is considered for this criterion.
- **Circularity**: Circularity concerns the reusability of the components of a construction. Technically, circularity specifically only concerns reusing components in the same state and with identical structural capacity as in the initial construction, but for the sake of this thesis, recyclability is also regarded under this criterion.

  The environmental benefits of circular components are already covered under "Sustainability", but the criterion also has a big financial value. By converting towards a circular economy, and thereby reducing costs on the extraction of materials, billions of euros can be saved annually in the Netherlands (Bastein, Roelofs, & Rietveld, 2013). Rijkswaterstaat has plans to increase the circularity of all constructions in the upcoming decades (Rijkswaterstaat, n.d.-a). In the future this criterion will be of utmost importance. For now an ambition of 3/4 is regarded.

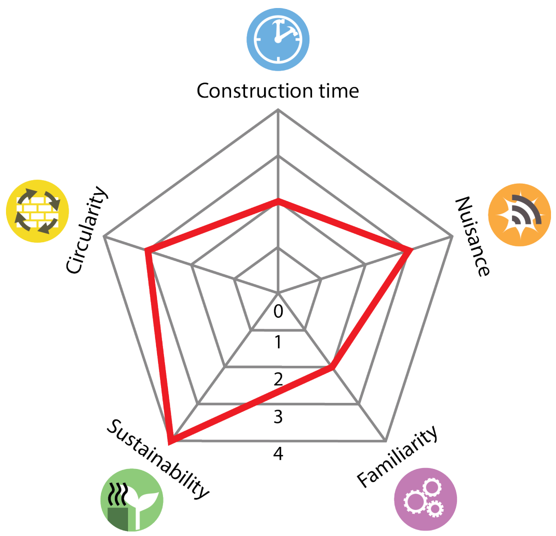


Figure 6.20: Ambition web of case study Western lock in Terneuzen

6.3.2. Criteria evaluation

Consecutively, each concept has to be evaluated per criterion. Therefore, a scale has to be drafted for the individual criteria, based on which the structures can be rated. For instance, for the construction time, a scale between 1 month and 6 moths could be applied, where 1 month is obviously rated the highest, and 6 months the lowest. For different concepts than the ones mentioned in this thesis, or different applications of those concepts, the used scales would have to be reconsidered, and adapted to be relevant for that specific project.

After assigning the evaluation scales, the concepts can be rated. The rating process should be executed objectively, and therefore, depending on the criterion and the applied scale, some calculations might be in order. For the case study of Terneuzen, ratings between 0 and 4 are provided.

Appendix G presents the derivation of the grading of the concepts. Table 6.3 presents the grading, and the final scores of all concepts. To prevent that choices are made solely based on the final score of the concepts, a visual presentation of the evaluation is provided in the remainder of this section, assuring that the meaning behind these numbers is not lost during the evaluation process.

Table 6.3: Grading on criteria of adaptation concepts applied to the Western lock in Terneuzen

Characteristic		Lock height					Gate height			Bed pr.	
	Weight	Wall	Fixed extension S	Fixed extension W	Modular extension S	Modular extension W	Gate extension	Gate replacement	Gate module	Colloidal concrete	Replacement bed pr.
Construction time	2	4	2	1	2	1	4	4	4	4	4
Construction nuisance	3	4	3	1	3	1	3	3	3	4	4
Familiarity procedure	2	4	4	3	2	2	3	4	1	4	4
Sustainability	4	4	2	1	2	1	4	2	4	4	2
Circularity	3	2	2	2	4	3	2	2	2	2	4
Total out of 56		50	35	21	37	22	45	39	41	50	46
		8.9	**6.3**	**3.8**	**6.6**	**3.9**	**8.0**	**7.0**	**7.3**	**8.9**	**8.6**

Lock retention height

The grading of the concepts improving the retention height of the lock structure is presented in Figures 6.21 and 6.22. The additional adaptations required in case of a weak lock have an influence on the grading, and are therefore separately included.

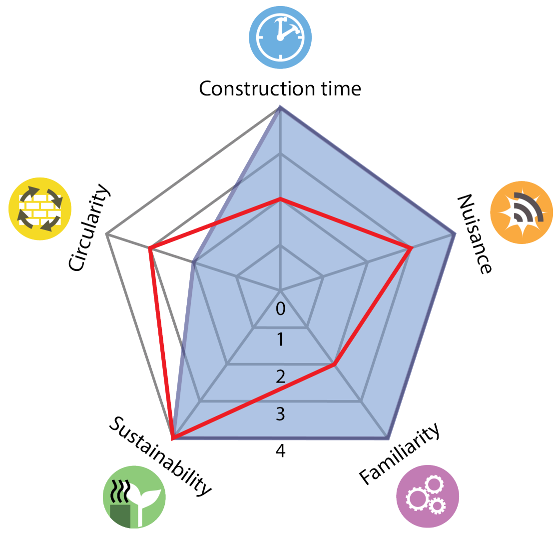


Figure 6.21: Grading of concept "Wall" in indigo. The ambition is presented in red.

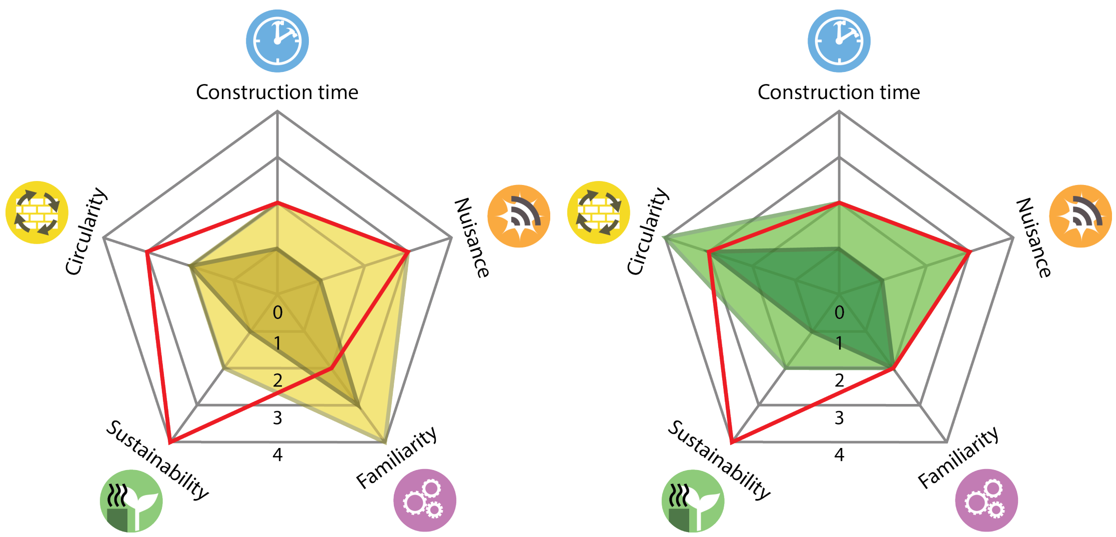


Figure 6.22: LEFT: Grading of concept "Fixed extension" applied to a strong lock in light yellow, and applied to a weak lock in dark yellow. RIGHT: Grading of concept "Modular extension" applied to a strong lock in light green, and applied to a weak lock in dark green. The ambition is presented in red.

Gate retention height

The grading of the concepts improving the retention height of the gate is presented in Figure 6.23. Concept "Gate extension" is placed in a separate chart, both to preserve clarity of the figures, and because the application of the other concepts, "Gate replacement" and "Gate module", have a more comparable impact on the integrity of the retention height.

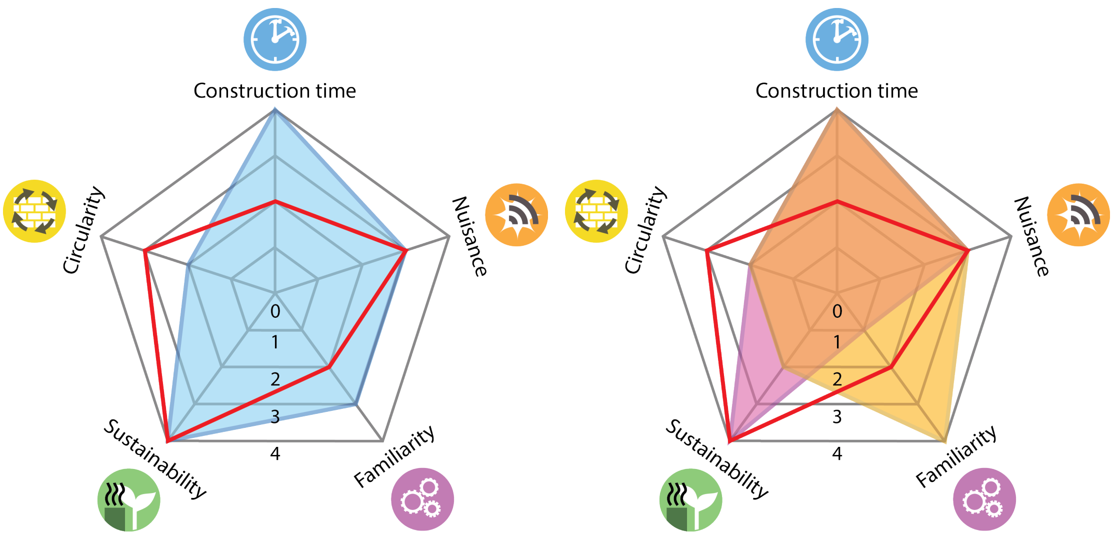


Figure 6.23: LEFT: Grading of concept "Gate extension". RIGHT: Grading of concept "Gate replacement" in orange, and concept "Gate module" in magenta. In both charts, the ambition is presented in red.

Bed protection composition
At last the grading of the two concepts to improve the bed protection composition is presented in Figure 6.24.

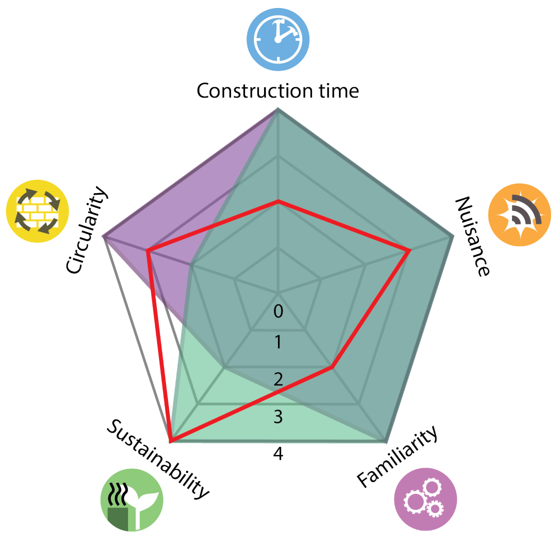


Figure 6.24: Grading of concept "Colloidal concrete" in teal, and concept "Replacement bed protection" in purple. The ambition is presented in red.

6.3.3. Cost analysis

From the criteria evaluation, the value of a concept is drafted. This value is however meaningless without any cost indication. A well performing concept could be incredibly expensive and economically unattractive. Therefore, the costs of each adaptation has to be analysed. These can be derived by means of reference projects or extensive calculations using characteristic monetary values for i.a. material, transport, and execution costs. For each lock, the specific dimensions and properties of the structure have a large influence on the costs. Therefore, the applications of the adaptations have a different price for each different project. It has to be analysed per lock how the costs of the concepts relate to one another. Per example, the costs of the application of the adaptations at the Western lock in Terneuzen is presented here.

Appendix I presents the derivation of the cost estimations of the concepts at the Western lock in Terneuzen. For this thesis, only the direct costs have been taken into account, as estimations of all indirect costs are calculated as percentages of the indirect costs, and would therefore not add any value to the analysis, but only convolute the matter. The total direct costs of the adaptations are presented in Table 6.4, and graphically in Figure 6.25. The costs of the strong (S) and weak (W) fixed and modular extensions all include the elevation of the crossing bridge on the outer lock head. The extensions of a weak lock also include the strengthening of the lock heads and chamber walls. For both the gate replacement and addition of a gate module, the replacement of the operating system has been taken into account.

Table 6.4: Cost estimations of the adaptation concepts at the Western lock in Terneuzen (rounded to thousands)

Characteristic	Concept	Direct costs
Lock height	Wall	€26,000*
	Fixed extension S	€6,505,000
	Fixed extension W	€11.356,000
	Modular extension S	€6.601,000
	Modular extension W	€11,451,000
Gate	Gate extension	€200,000
	Gate replacement	€13,330,000
	Gate module	€5,732,000
Bed pr.	Colloidal concrete	€107,000
	Replacement bed protection	€876,000

*Although the direct costs of this alternative are expected to be low, realistically these costs should be higher than currently derived. However, what matters in this thesis is the relation between the costs of alternatives, which should be that this adaptation is much cheaper than the others. Therefore, the cost item is not derived more thoroughly.

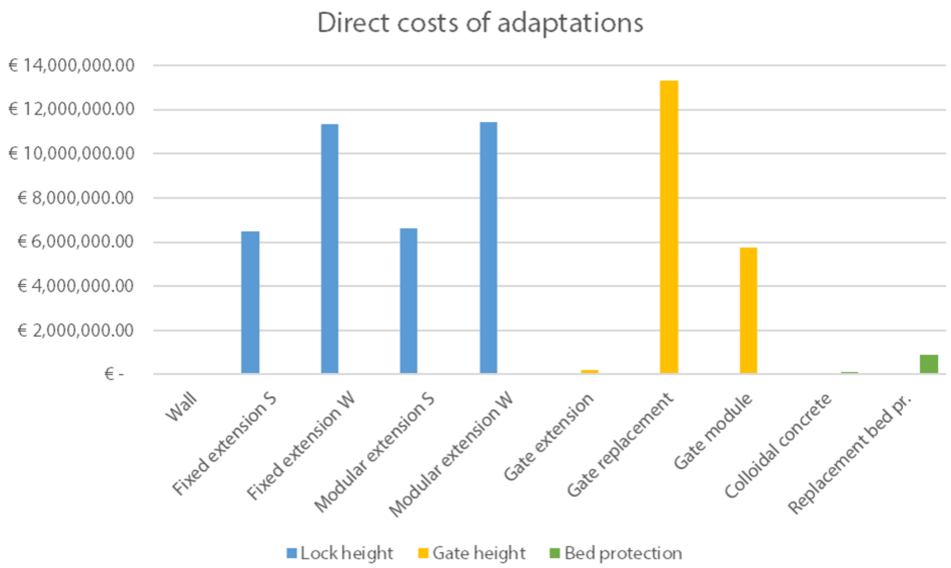


Figure 6.25: Direct cost estimations of adaptations

6.3.4. Cost-value comparison

As stated, the evaluation and cost indications of the concepts are rather incomparable without one another. The evaluation scores have to be set out against the costs in order to fully understand how concepts relate to one another. This can be done with a cost-value graph, with on one axis the costs, and on the other the value derived with from the criteria evaluation. This graphic depiction of both these aspects can provide a clear insight on the relation between the concepts.

For the Western lock in Terneuzen, three cost-value graphs are drafted for the different critical characteristics. These are presented in Figures 6.26, 6.27, and 6.28.

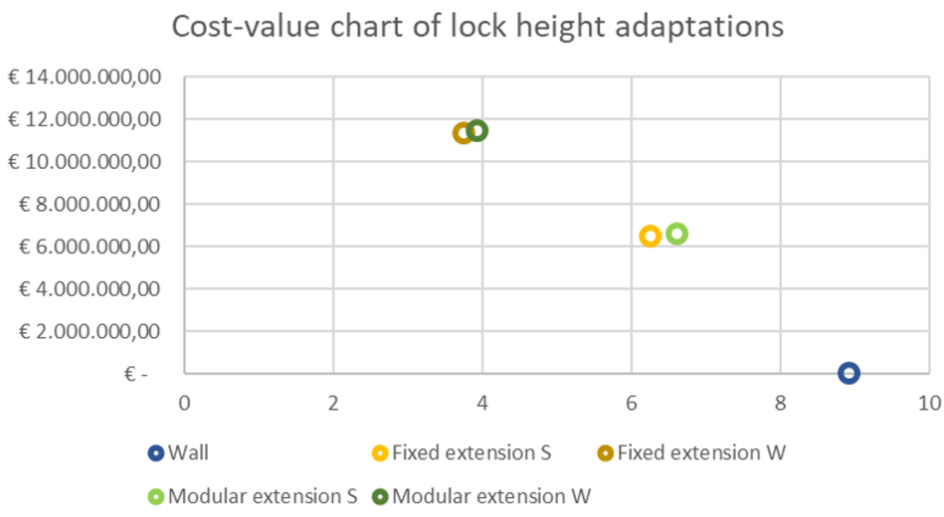


Figure 6.26: Cost-value chart of lock height adaptations

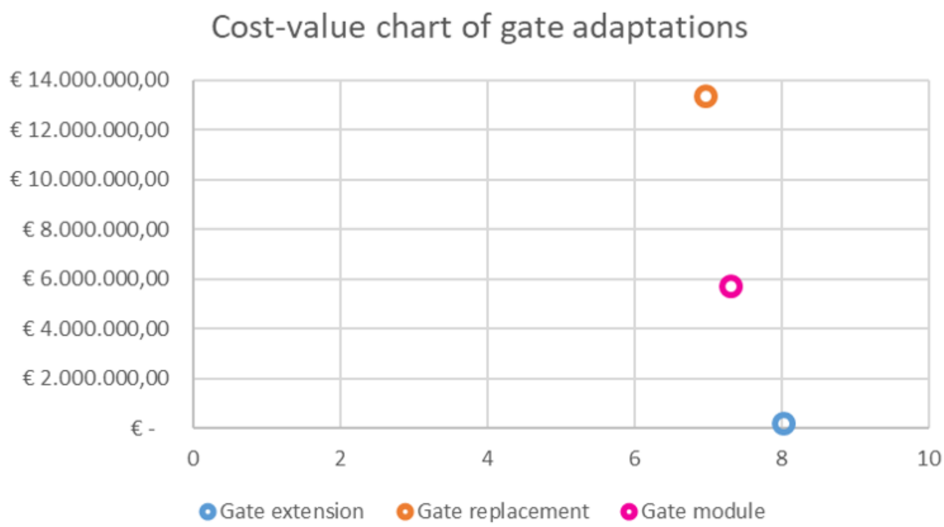


Figure 6.27: Cost-value chart of gate adaptations

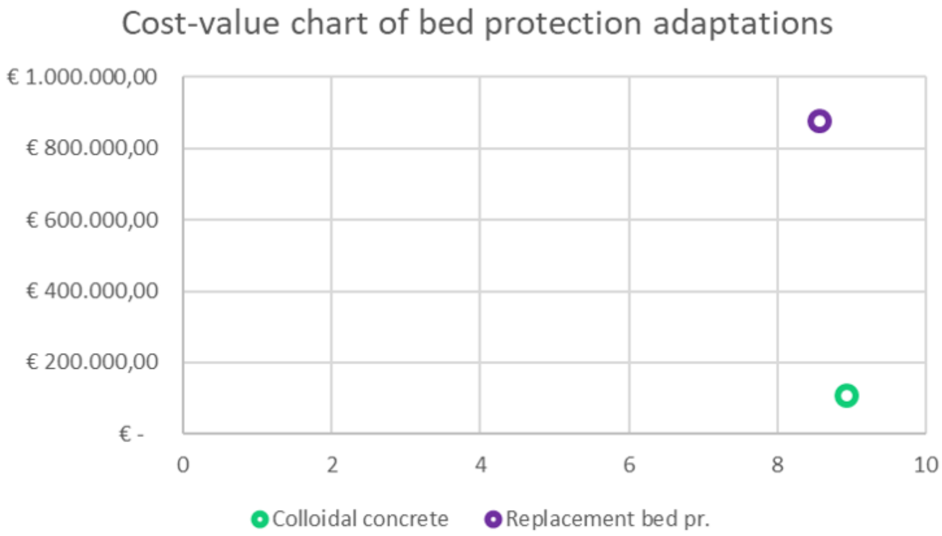


Figure 6.28: Cost-value chart of bed protection adaptations

6.4. Selection of adaptation concepts

Based on the results of the cost-value comparison, a selection can be made for most critical lock characteristics. Because the costs and values presented in this thesis are only relevant for the case study, the selection made here does not suffice for all projects. However, the process described here does reflect how the before executed evaluation can support decision making on what adaptation concepts to use in the drafting of the pathways maps in Section 6.5.

Figure 6.29: Chapter progression Section 6.4

6.4.1. Lock retention height

Observing Figure 6.26, it may seem obvious that concept “Wall” would be the best choice, as it has both the highest value, and is the cheapest. However, this concept can only be applied to increase the retention height with 0.5 m, whereas the other concepts are limited by the strength of the lock. It can however be concluded that concept “Wall” has to be considered in the creation of the adaptive pathways map, especially as it can even be mounted atop the other adaptations.

It also seems that the adaptations for a strong lock are a better choice than those for a weak lock, which have much higher costs, and a worse value. However, the applicability of these adaptations depends on the existing lock. Therefore, no adaptation concept for a weak lock can be can be disregarded based on their value relative to the same adaptation for a strong lock.

What choice does not depend on the existing lock is whether to apply a fixed or modular extension. For both the strong lock and the weak lock, concept “Modular extension” has a higher cost than “Fixed extension”, but with a negligible difference. It does however perform better in the criteria evaluation. From observing the ambition webs in Figure 6.22 can be seen that the modular extension scores better on criterion “circularity”, whereas the fixed extension scores higher on “familiarity”. The latter has a lower importance for this project, confirming that the modular option is preferable over the fixed extension. This concerns both the strong and weak lock.

It should be noted that the cost and value differences between these alternatives are not substantial. This means that a more intensive study of the required materials for the concepts, and a more thorough study of the costs, might result in a different optimal adaptation. However, with the current knowledge, following the developed method, for this particular project, the modular extension is preferable.

Table 6.5: Considered concepts for the time dependent evaluation

Characteristic	Concept
	Wall
Lock height	Modular extension S
	Modular extension W

6.4.2. Gate retention height and strength

For the concepts concerning the gate retention height and strength, identical comments can be made as for the lock extension concepts. The gate extension seems to be the best choice according to the cost-value chart presented in Figure 6.27, but this concept can only be applied up to 0.5 m. For the other concepts, larger retention heights can be achieved. The gate extension can also be mounted atop the other concepts to increase the retention height even further. Therefore it has to be considered for the adaptive pathways map.

Between the concepts “Gate replacement” and “Gate module”, it is very obvious that the costs of the gate replacement are far higher, and that those costs are not made up for by additional value. From the ambition web in Figure 6.23 can even be seen that on the most important criterion, “Sustainability”, the gate replacement scores much worse than the gate module. The gate replacement would be applied to a lock which has initially been fitted with a regular gate with an identical retention height as the lock head. The gate module can be applied to a lock with either a modular gate, or a non-modular gate specifically designed to have a module mounted upon it. Thus, it depends on the initial conditions of a lock whether the gate module can be applied or not. Neither one of the concepts can therefore be disregarded at this moment.

Table 6.6: Considered concepts for the time dependent evaluation

Characteristic	Concept
Gate height and strength	Gate extension Gate replacement Gate module

6.4.3. Bed protection

For the comparison of the bed protection alternatives per characteristic, a choice can directly be made from the cost-value chart presented in Figure 6.28. Observing this chart, it is very clear that "Colloidal concrete" is the better alternative. Not only does it perform slightly better in the criteria evaluation, but it is also much less expensive. It should be noted that from the ambition web in Figure 6.24, one can see that the colloidal concrete scores much worse on criterion "Circularity", which is deemed fairly important. However, the concept is granted a perfect score on all other criteria, partly making up for the lack of sufficiency on that one criterion. Thus, concept "Replacement bed protection" no longer has to be considered.

Table 6.7: Considered concepts for the time dependent evaluation

Characteristic	Concept
Bed protection	Colloidal concrete

6.5. Development pathways of initial adaptation and replacement decision

After the selection of the most suitable adaptation alternatives, it can be analysed whether it is effective to prolong the lifetime of a lock with a combination of adaptations, or whether it is beneficial to simply replace the lock once it no longer suffices due to sea level rise. This is a time dependent issue, which can be clarified by means of an adaptive pathways map (Haasnoot et al., 2013).

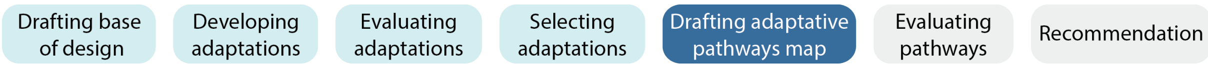


Figure 6.30: Chapter progression Section 6.5

6.5.1. Adaptive pathways maps of adaptations

In this section, the adaptive pathways map for the adaptation alternatives is drafted. Regularly, adaptive pathways maps have years on the horizontal axis. However, as sea level rise is an uncertain factor, it cannot be directly related to a timeline. Therefore, the sea level rise is placed on the horizontal axis in these maps. On the vertical axis, the initial choices are presented. On the maps, each line follows a possible strategy from left to rise, with a rising sea level.

Each concept is indicated with a differently coloured line. If a concept is no longer sufficient for the occurring sea level rise, a jump has to be made to a different concept. In Appendix H, the applicability and compatibility of the adaptations derived in Section 6.2 are determined. This gives insight in the possible manners in which the concepts can be combined with one another. Thus, two possible adaptation combinations are drafted: the minor adaptation and the drastic adaptation. In the adaptive pathways map these are presented respectively by a triangle and a circle. It should be noted that the specifics of these procedures are based on the evaluation applied to the case study, and therefore might deviate for altering locks. For other constructions, for instance, the improvement of the piping length could be included in the drastic adaptation.

- ▶ **Minor adaptation**: This concerns the addition of a wall on top of the lock head, and an extension mounted to the gate. This measure ensures that the flood safety is conserved until an additional 0.5 m of sea level rise. It is assumed that for this minimal change in sea level, no adaptation of the bed protection is yet required. In case the bed protection would not suffice to endure a larger flow velocity, the choice can still be made not to increase the maximum levelling sea level. This would only slightly lower the operability of the lock.

  For the case study, this adaptation includes:

  - ▷ Wall
  - ▷ Gate extension

- ● **Drastic adaptation**: This application concerns the heightening of the lock structure, a replacement of the gate with a taller one, the replacement of the operating mechanism with one that is stronger and more elevated, and the improvement of the bed protection and piping length if necessary for the specific lock. If the structural capacity of the lock is not large enough to support the additional forces applied through the heightening adaptation, a supplementary support of the lock head, chamber walls, and foundation is required, which is therefore included in the drastic adaptation for such locks. In the map presented in Figure 6.31, a drastic adaptation for 1.0 m sea level rise is implemented. Depending on the lock construction, a larger heightening might be possible. The lifetime is however already prolonged by many years by means of this adaptation magnitude, and it is likely that other facets of the lock will necessitate a replacement before a more severe adaptation than the one presented would be required.

  For the case study, this adaptation includes:

  - ○ Modular lock extension
  - ○ Strengthening head and chamber walls
  - ○ Elevation outer bridge
  - ○ Gate and operating mechanism replacement
  - ○ Penetration bed protection with colloidal concrete

- **End of lifetime**: This concerns a situation in which either no more adaptation can be executed, or it is no longer deemed necessary. The former occurs in case a lock which has already been improved with additional support of the lock structure no longer suffices, and requires a drastic adaptation. The lock cannot be strengthened even further, and is therefore out of options. An adaptation is deemed unnecessary in case the lock can already endure the entire range of sea level rise regarded in this thesis.

It is assumed that the case study, the Western lock in Terneuzen, does not have any surplus in its structure, but can be improved by means of additional support of the lock head and chamber walls. For a lock with such characteristics, the adaptive pathways map presented in Figure 6.31 is relevant. Three initial choices can be made: a minor adaptation, a drastic adaptation, or a replacement. If the minor adaptation is chosen (magenta line in the map), the solution works until 0.5 m of sea level rise. At this moment, the choice can be made to either replace the lock, or demolish the minor adaptation and apply a drastic adaptation instead. From this point the pathways follow the lines of the other initial choices. If it is chosen to initiate with applying the drastic adaptation (purple line in the map), this solution works until 1.0 m of sea level rise. At this moment, the choice is made to either replace the lock, or apply a minor adaptation on top of the drastic adaptation. The latter prolongs the lifetime until 1.5 m sea level rise, at which a lock replacement is required. There could however be any moment during the lifetime of the lock at which a different characteristic of the lock - for example the capacity - demands a replacement with a more competent lock variant.

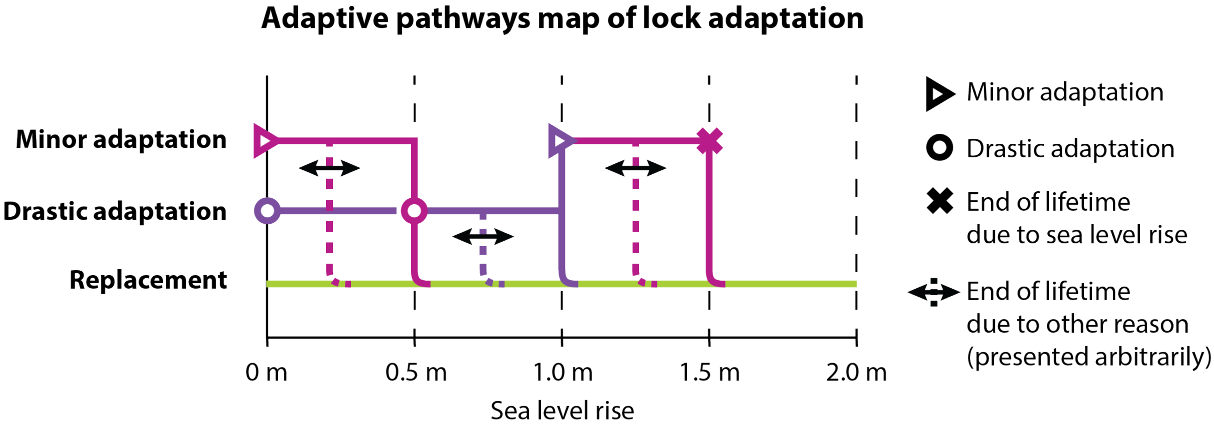


Figure 6.31: Adaptive pathways map for a lock which either has the structural capacity for a drastic adaptation, or can be strengthened through an additional adaptation

The costs of the minor adaptation are insignificant in relation to the costs of a drastic adaptation and especially a lock replacement. Therefore, the assumption is made that - apart from the initial choice - it is always chosen to prolong the lifetime of a lock with a minor adaptation in case possible. This provides four manners to follow the adaptive pathways map from left to right, as presented in Figure 6.32.

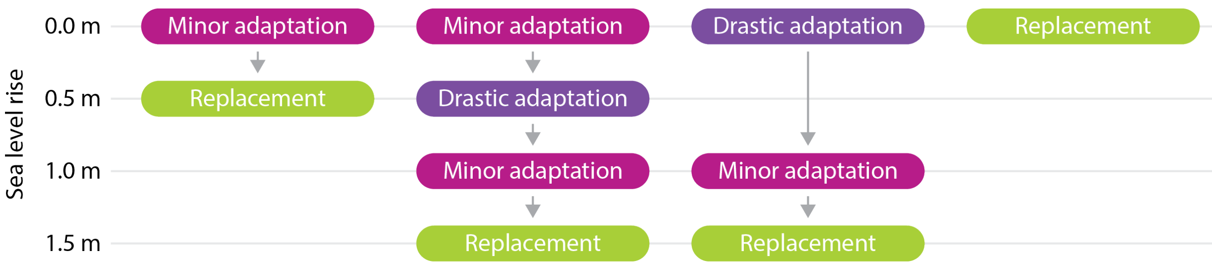


Figure 6.32: All reasonable manners to follow the adaptive pathways map from left to right

6.6. Evaluation of adaptation pathways

The different pathways presented in Figure 6.31 have varying advantages and disadvantages. The pathways are to be compared with one another based on their value and costs, in order to make a recommendation on the best pathway.

Figure 6.33: Chapter progression Section 6.6

6.6.1. Evaluation of adaptation pathways with deduction point system

The value of the pathways has to be evaluated. The evaluation scores derived in Section 6.3.2 cannot be used identically for the evaluation of the pathways. In Appendix G.6, the criteria evaluation of the adaptations is reworked to a system which can be applied to the pathways. Below, the main elements of the system are described, accompanied by a short explanation behind the reason why they are implemented.

- Because the adaptations are now regarded in context of the replacement of an entire lock, not all criteria are as relevant as in their direct comparison. Criteria sustainability and circularity become obsolete. The dimensions of the adapted components are small in relation to the entire lock structure, meaning that their footprint and regained material from circularity are irrelevant in the entire life cycle of the lock.
- Criteria that remain of importance are construction time, construction nuisance, and familiarity of the procedures.
- A system of deduction points is implemented rather than a system of grades, because this allows those deduction points to be added up if multiple adaptations have to be executed sequentially.
- The lock replacement has no deduction points, because it can not be avoided. No matter what pathway is followed, eventually the lock always has to be replaced. Also the issue of inoperability can be avoided during replacement by constructing the lock at a slightly different location than the original one, allowing it to remain opened during the construction period.

Three scenarios are regarded in this analysis, RCP8.5 (severe), RCP2.6 (moderate), and RCP2.6 (mild). For the extreme scenarios, RCP8.5 and RCP2.6, long term projections are available for the Dutch coast. Contrarily, for moderate scenario RCP6, only a long term *global* sea level rise prediction is available. However, this projection can be scaled to an estimation of the Dutch coast sea level rise using the ratio between the Dutch and global projections of RCP8.5 and RCP2.6. This is executed in Appendix J.

As derived in Section 5.4.4, for the case study in Terneuzen, the safety norm of 1:1000 is reached only at a sea level rise of 1.1 m. Considering the RCP projections made by the KNMI, this is reached in 2100 within the 95% likelihood of RCP8.5, in 2145 for RCP6, and 2300 for RCP2.6. This is therefore not the most relevant case regarding sea level rise, and it is likely that other facets demand a replacement before this critical point is reached. Therefore, instead a hypothetical situation is applied to the Western lock, i.e. one in which the sea level rise which is integrated in the design no longer suffices in 2050. Thus, the case is more appropriate to support decision making in current times. The critical characteristics of the lock remain equal, only their point of insufficiency is reached earlier.

According to these projections, the sea level rise which develops between 2050 and 2150 with a 95% likelihood in RCP8.5, RCP6, and RCP2.6 is respectively 0.47 m, 0.84 m, and 1.76 m (see Appendix J).

In Appendix G.6, an overview is provided of the deduction points of all adaptations. By adding those up for each pathway, the values presented in Table 6.8 are found.

Table 6.8: Deduction points for pathways

	Deduction points		
Scenario	RCP2.6	RCP6	RCP8.5
Sea level rise	0.47 m	0.84 m	1.76 m
MR	-5	-5	-5
MDMR	-5	-33	-40
DMR	-28	-33	-33
R	0	0	0

6.6.2. Benefit progression over time

In addition to the evaluation of the pathways by means of criteria, the economical benefits of the decisions has to be derived. It is important to not just know whether a pathway has been beneficial or not at the end of the pathway, but the benefit progression of the entire pathway is required. This is because at any point in the pathway, a different aspect of the lock might require a replacement.

Benefit progression model

Several aspects play a role in the benefits of the adaptations, as listed below.

- The costs of an investment can change over time due to inflation and economical growth. Therefore, the timing of an investment could be an important factor. However, it is estimated that the relative price increase is equal to 0 for public physical investments such as infrastructure (Ministry of Finance, 2020). Therefore, no time related price adjustments are made in this thesis.
- Of course money is saved by postponing the replacement, as the loan required to afford the replacement can also be postponed. It is assumed that the payment of the loan is spread out evenly over the expected minimal lifetime of the lock, i.e. 100 years (Rijkswaterstaat, 2010). A yearly interest of 1% is assumed for the loan.
- The costs of the adaptation would most likely also be payed by means of a loan. However, as these costs are much smaller than those of an entire lock, and have to be spent for these pathways regardless, the progression of the payment of these loans is of no matter to the benefit of the adaptations. Therefore, the costs are implemented in total at the instance of adaptation.
- Because the decision to adapt or replace a lock is mostly an issue for older lock, rather than new lockss, it is assumed that the maintenance costs are higher when deciding to adapt instead of replacing. The cost increment of maintenance is very case specific. The current yearly maintenance costs of the case study are €911,000 (De Pelsmaeker, 2010).

As example, Figure 6.34 presents the progression of the cost items explored in the list above for an increment of yearly maintenance costs of 100%. The yearly gain of postponing paying off a loan for the lock replacement decreases because the interest of the loan decreases with paying off the sum.

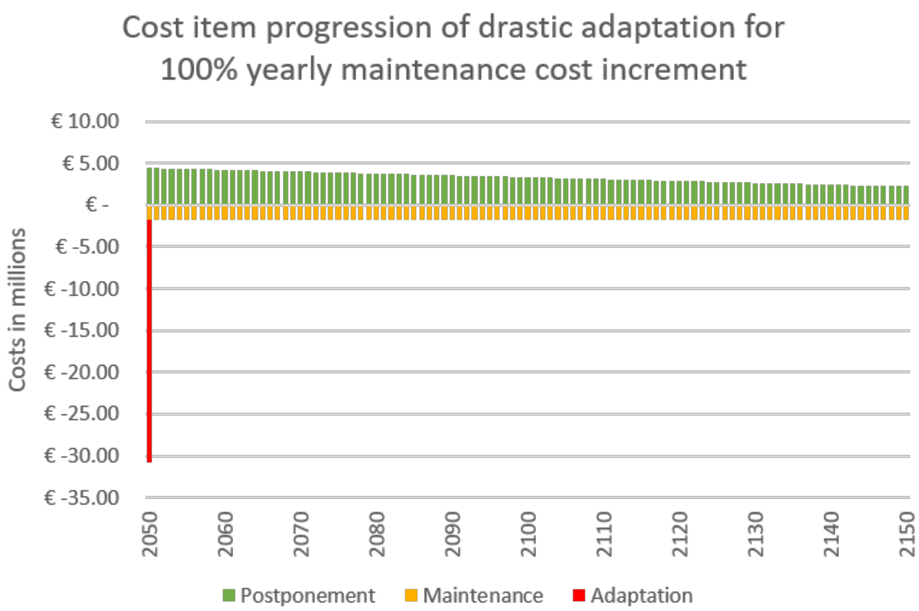


Figure 6.34: Progression of cost items over 100 years

These elements are captured in Equation 6.1, which can be used to determine the benefit of an adaptation over a replacement in a number of years after the decision.

$$benefit = -C_{adaptation} - \sum_{i=0}^{i=years} C_{maintenance,i} + \sum_{i=0}^{i=years} C_{postponement,i} \tag{6.1}$$

The adaptation and replacement costs applied to the model are presented in Table 6.9, as have been derived in Appendix I. The costs of the modular extension include the elevation of the bridge on the outer lock head. The item "other direct costs" in the drastic adaptation are derived from the difference in this cost item between a lock with no sea level rise integrated and a lock with 2.0 m sea level rise integrated in the design.

Table 6.9: Direct cost indications used to derive the cost progression of the pathways

Minor adaptation	
Wall	€26,000*
Gate extension	€200,000
Total direct costs	**€226,000**
Drastic adaptation	
Modular extension	€3,473,000
Lock strengthening	€3,128,000
Gate replacement	€8,000,000
Operating mechanism replacement	€5,332,000
Bridge elevation	€3,128,000
Other direct costs	€4,338,000
Total direct costs	**€27,399,000**
Replacement direct costs	**€222,027,000**

*Although the direct costs of this alternative are expected to be low, realistically these costs should be higher than currently derived. However, what matters in this thesis is the relation between the costs of alternatives, which should be that this adaptation is much cheaper than the drastic adaptation. Therefore, the cost item is not derived more thoroughly.

Benefit progression results

By applying the costs presented in Table 6.9 to the model derived in this section, the progression of the cost difference of adapting the Western lock in Terneuzen instead of replacing it is found, as presented in Figure 6.35. In this graph the cost difference of both the minor and drastic adaptation are shown. What can initially be seen is that both lines are at all moments climbing, because for the implemented maintenance cost increase of 100%, the loan payment which is postponed is always higher than the maintenance costs. For this value of maintenance cost increase, the costs of the drastic adaptation are won back in 11 years, whereas the minor adaptation is immediately profitable due to its low costs.

In the graph is also presented how the addition of a sequential adaptation would alter the use of the graph. On the line of the minor adaptation, two points are indicated at which the adaptation would no longer suffice in scenarios RCP8.5 and RCP6 (in scenario RCP2.6, the adaptation would last over 100 years). If at those moments it is decided to apply a drastic adaptation, a jump can simply be made to the line of the drastic adaptation. Due to the low costs of the minor adaptation, there is virtually no difference between the costs of a pathway in which only a drastic adaptation is applied, and one in which both the drastic and minor adaptation are applied. In case the drastic adaptation no longer suffices, as presented for RCP8.5 (for both RCP6 and RCP2.6, the adaptation lasts over 100 years), and a minor adaptation is applied on top of the drastic adaptation, the line of the drastic adaptation can still be followed, due to the low relative costs of the minor adaptation.

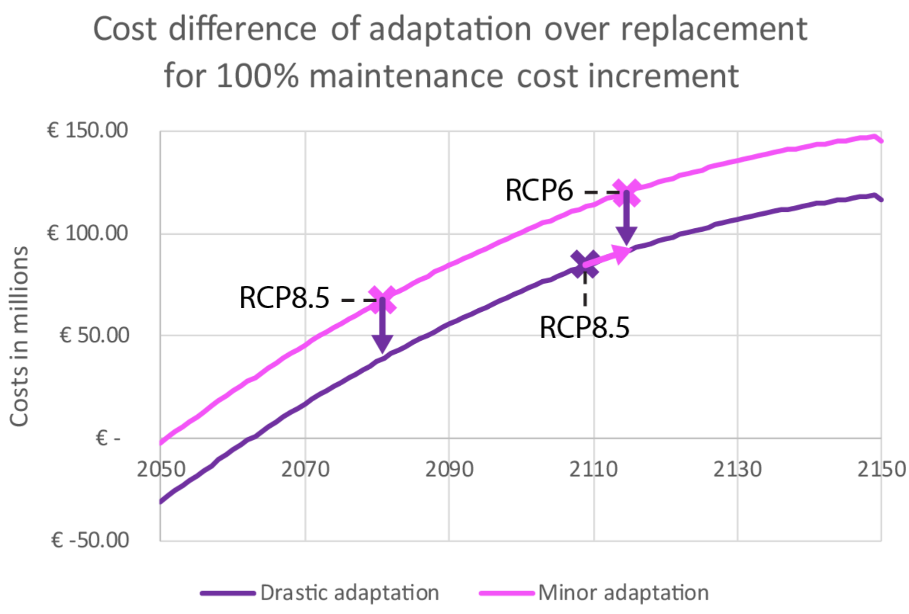


Figure 6.35: Cost difference progression of adapting the Western lock in Terneuzen isntead of replacing it

As stated, it takes around 11 years for the drastic adaptation to become profitable over a lock replacement for a maintenance cost increase of 100%. This means that if within 11 years no replacement of the lock is necessary due to any other reason than sea level rise, the adaptation is profitable. Of course this amount of years changes in case the maintenance costs vary. Figure 6.36 presents how the amount of required years changes for a different maintenance cost increase. It takes at least 8 years for the adaptation to become profitable, even for no yearly maintenance cost increase. For a very extreme situation of a 200% additional maintenance costs relative to a new lock, it takes 19 years for the adaptation to be profitable.

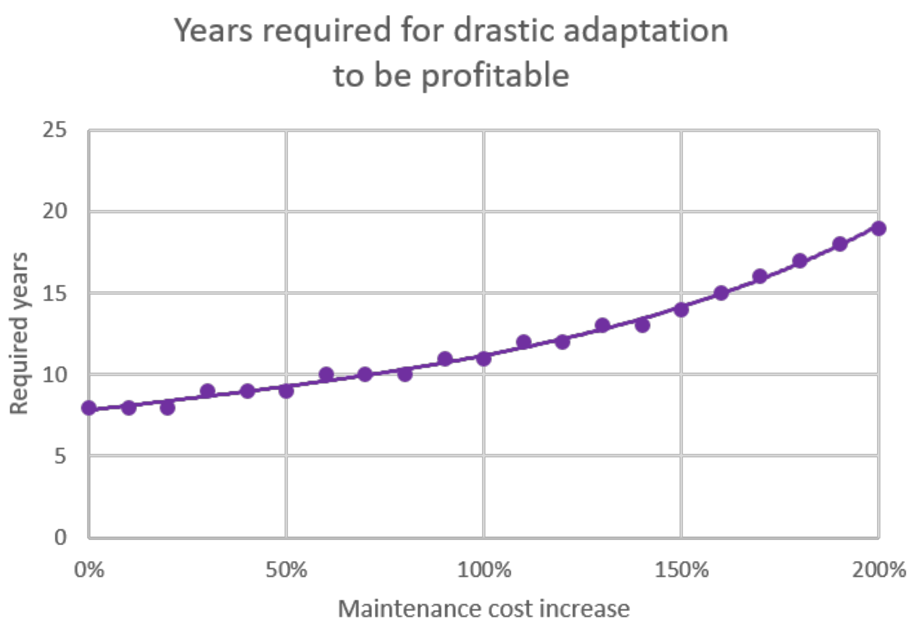


Figure 6.36: Relation between maintenance cost increase and the profitability of a drastic adaptation

6.7. Recommendation for decision to adapt or replace

This step in the method of this thesis concerns the decision to either adapt a lock or replace it when it no longer suffices due to sea level rise. In the previous sections of this chapter, adaptation concepts have been developed, applied to the case study of the Western lock in Terneuzen, have been compared to one another based on costs and evaluation criteria, and have finally been compared to a conservative lock replacement based on time dependent value and cost developments. Based on this, a recommendation on the decision whether to adapt or replace a lock is made. The negatives and positives of the adaptations are listed below, and further explicated in the remainder of this section.

- \+ The minor adaptation is profitable, regardless of the additional yearly maintenance costs of an old lock relative to a new replaced lock. Even if later in the lifetime of the lock the minor adaptation has to be replaced with a drastic adaptation, barely any expenses have been wasted.
- +/- The drastic adaptation can be profitable, depending on the functional and structural state of the lock. If yearly maintenance costs are high, and a replacement of the lock is in order rather quickly after the adaptation, the investment of the adaptation may not yet have been won back.
- \+ Postponing the replacement by means of an adaptation results in a smaller uncertainty in sea level rise which has to be considered by the time the lock is eventually being replaced, as the initial cultural and political response to current climate change insights plays out.
- \+ Regarding the collective of Dutch locks which have to be replaced in the upcoming decades, being able to postpone some of those investments with several decades helps with spreading the expenses over a longer stretch of time.
- \- During an adaptation, the lock has to be closed off temporarily, reducing the capacity of the waterway. In addition, the neighboring residents will experience nuisance from the construction. For a drastic adaptation, this period of obstructions is much longer than for a minor adaptation.
- \- The regarded adaptations only solve a single issue, whereas multiple issues can be solved by means of a lock replacement.

As is clear from Figure 6.35, a minor adaptation is in any situation financially profitable. The application of the adaptation also does not provide much negative side effects during construction, like nuisance. The application remains sufficient for at least 26 years (in a very severe climate change scenario), and might remain sufficient for over 100 years in a very mild scenario. It is likely that no further adaptation of a lock is required for several decades after installing a minor adaptation. Even when the adaptation is no longer sufficient due to a sea level rise of more than 0.5 m, and either a drastic adaptation is applied, or the lock is replaced, barely any money has been wasted on the initial application of the minor adaptation. Therefore, it is recommended to always first apply a minor adaptation.

If after several years, the minor adaptation no longer suffices, a drastic adaptation might be in order. However, several other aspects play a role in deciding whether this is desirable. As presented in Figure 6.36, it will take several years before the investment in a drastic adaptation becomes profitable. The amount of years required for this depends on the yearly maintenance costs of the old lock. For a lock with no additional maintenance costs in relation to a completely new replaced lock, this amounts to 7 years. For a lock with a 50%, 100%, and 200% yearly maintenance cost increase, it takes respectively 9, 11, and 19 years before the postponement of a lock replacement makes the investment profitable. Therefore, it is important to regard whether the functional and structural requirements of the current lock are expected to still be met during those years. If for instance the capacity of the lock is expected to no longer suffice in the upcoming years, it might be better to immediately replace the lock. It then also has to be considered that the construction of the adaptation will require a temporary loss of operability. It also has to be taken into consideration how much money is currently available, and whether there are other projects which require initial attention. If so, it can be of great value to postpone a lock replacement by means of a drastic adaptation.

An additional advantage of postponing a lock replacement, is that by the time the lock is eventually being replaced, the sea level rise projections will most likely have narrowed. Currently, the sea level rise development is very uncertain. However, in the upcoming decades, more research on climate change will result in a reduction of the uncertainty of natures response to emission, and possibly more importantly, it will become clear how extensive the initial human response to the current climate developments will be, narrowing the possible emission scenarios. Thus, if a lock replacement is postponed, the sea level rise uncertainty which has to be considered in its design is reduced.

7

Determining effectiveness of adaptive lock alternatives

In the first step of the method, the critical lock characteristics were derived which have to be improved to remain a sufficient flood protection while the sea level rises. In the second step, it was analysed whether it is effective to prolong the lifetime of a lock with adaptations rather than replacing the lock in its entirety. This chapter concerns the third step in the methodology, i.e. determining whether it is effective to apply an adaptive design when replacing a marine lock, rather than a conservative one (either because of sea level rise developments, or any other reason).

In Section 7.1, a new base of design is drafted for the replacement alternatives, based on the critical characteristics derived in Chapter 5. Thereafter, a conservative and several adaptive lock alternatives are developed in Section 7.2. Because of the similarities of the concepts, they are not evaluated based on criteria. Instead, the adaptive pathways map for this subject is directly drafted in Section 7.3. The pathways on this map are then evaluated and compared to one another in 7.4, based on which a recommendation is made in Section 7.5.

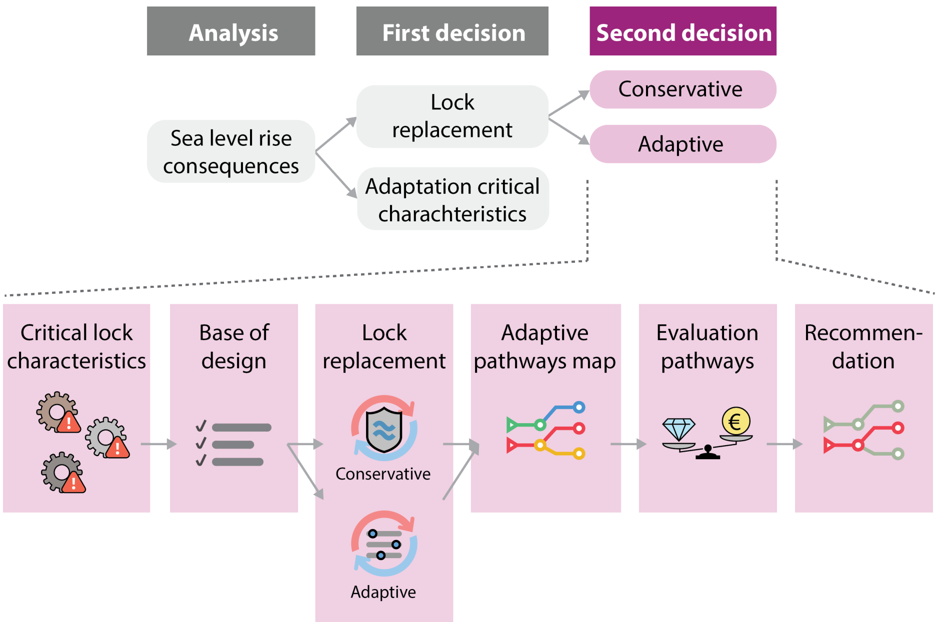


Figure 7.1: Flow chart of Chapter 7

7.1. Derivation requirements replacement concepts

Further in this chapter, lock replacement alternatives are developed. To be able to develop sufficient designs, it must be known what the requirements of the designs are. Thereto, a base of design is drafted. A base of design was already drafted for the adaptation concepts. For the replacement, many of the requirements are similar or identical. Again, the application of the base of design to the case study in Terneuzen is presented in *italics*.

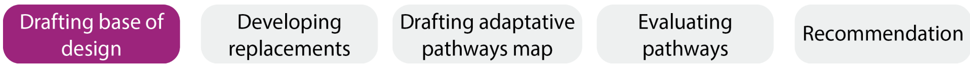


Figure 7.2: Chapter progression Section 7.1

7.1.1. Structural requirements

- Each replacement must improve *all* critical characteristics resulting from the probability failure analysis performed in Chapter 5.

  For the case study, these characteristics are identical for the adaptations and replacements. As explained in Section 6.1, these include the retention height, gate strength, and bed protection composition.

- The replacement must be constructable:
  - ◇ The site of the replacement must be made accessible.
  - ◇ The construction has to be executable with existing equipment and construction techniques.

  No specific additional notes can be given for the case study.

- The structure must be sufficiently strong, stable, and stiff.

  The replacement must suffice to the EN EuroCode norms (European Commission, n.d.), and to the ROK 1.4 in case Rijkswaterstaat is the client (Rijkswaterstaat, 2017).

7.1.2. Functional requirements

- The operability of the lock should not be lower than before the replacement. Often the operability is even required to be increased.

  Similar to the requirement of the adaptation concept, the maximum sea level at which currently leveling is allowed should be increased along with the sea level rise implemented in the lock construction. No operability increment is regarded in this thesis.

- The dimensions of the lock chamber of the new lock should not be smaller than those of the original lock. Often larger dimensions are even required to fulfill new capacity demands.

  The current maximum vessel dimensions are a length of 265 m, a width of 34 m, and a draught of 12.5 m. This concerns a lightened Panamax with adapted length (De Gans, 2007). Although it is likely that when the lock will be replaced, its dimensions will be made larger to increase its capacity, this is not regarded in this thesis for the sake of simplicity.

- If the lock functions as an important cross-over for "dry" traffic, this function should be maintained.

  The case study has a bridge crossing over it at each lock head. These both have to be implemented in the new design, to assure "dry" traffic can cross while vessels enter the lock for which one bridge has to be opened (De Gans, 2007).

7.1.3. Starting points

- In case the lock is an element of a lock complex, it is preferred that the capacity of the remaining locks is not reduced for long stretches of time during the construction.

  The lock in Terneuzen is part of a larger complex, including two other locks. These, and also preferably the original Western lock itself, should mostly remain at their regular operability.

- Preference goes to a construction which has little impact on the ecosystem in its direct area.

  This preference is identical to that in Section 6.1. For the case study, this concerns the ecosystem of the Western Schelt, Canal Ghent-Terneuzen, and above the water, the area surrounding the lock. The Western Scheldt is a protected area under the Natura 2000 (Ministry of Agriculture & Quality, n.d.).

7.1.4. Boundary conditions

- In the design, the specific boundary conditions of an individual project and location should be incorporated. These are identical for the adaptations and replacements, although in the design of a replacement, the development of the boundary condition should be regarded over at least 100 years, as this is the demanded minimal lifespan of a lock (Rijkswaterstaat, 2010).
  - ◇ Hydraulic boundary conditions on both sides of the lock
  - ◇ Ground water level
  - ◇ Subsoil composition

  The hydraulic boundary conditions of the case study were derived in Section 5.4.3, but can change over the years. The conditions should be analysed by the time the replacement concept is actually designed. By then, the development scenarios of both the hydraulic boundary conditions and all other boundary conditions should be considered, to assure that no adaptations are required during the lifetime of the lock.

7.2. Development adaptive replacement alternatives

In this step of the method, the effectiveness of adaptive lock designs is analysed. Thus, obviously, adaptive lock alternatives have to be developed. In addition, a conservative lock design has to be specified, so that it can be compared to the adaptive locks. In this section, one conservative and several adaptive locks are developed, based on the requirements in the base of design drafted in the previous section.

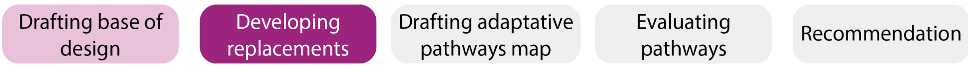


Figure 7.3: Chapter progression Section 7.2

An adaptive design is one which can be altered over time when required. This can be effective for marine locks, because of the uncertainty of the sea level rise development. By means of such a design, material can initially be spared by applying a retention height for an optimistic sea level rise scenario. If later the sea level rise develops more severely, the design can be adapted to withstand the new boundary conditions. In Chapter 6, it was seen that adaptation is not a possibility for all currently existing marine locks, and also that the extent of such an adaptation is limited. Thus, specific adaptive lock designs have to be developed, in order to allow easy adaptation, and reduce their limitations.

As for the adaptation concepts, the replacement concepts have been developed by brainstorming, with the base of design as foundation. All concepts therefore suffice to the functional and structural requirements. The alternatives are listed below.

1. **Conservative lock replacement**
2. **Adaptive lock replacement**
   - 2.1. Strong adaptive: a lock with a non-conservative retention height, but with a base which is strong enough to support a conservative retention height, meaning a less severe adaptation is required when sea level rise aggravates and the retention height has to be increased.
   - 2.2. Weak adaptive: with a non-conservative retention height, and a non-conservative base strength, meaning that when the retention height is increased, the base also has to be adapted to support the additional forces. In the previous chapter it was assumed that a lock can only be strengthened to withstand the forces accompanied by an additional 1.0 m of retention height.

The general characteristics of the lock alternatives are listed in Table 7.1. In the remainder of this section, the alternative designs are further developed and explicated.

Table 7.1: General characteristics of lock alternatives

Alternative	**Conservative**	**Strong adaptive**	**Weak adaptive**
Strength lock base	Conservative	Conservative	Non-conservative
Retention height	Conservative	Non-conservative	Non-conservative
Adaptation limitation	Minor: +0.5 m Drastic: unnecessary	Minor: +0.5 m Drastic: base strength	Minor: +0.5 m Drastic: +1.0 m (assumed strengthening limit)

7.2.1. Conservative lock replacement

The conservative lock replacement concerns a lock which is designed to withstand all realistic sea level rise developments that can occur during the lifetime of the lock. The original structure will be completely demolished, and a new improved lock can be constructed at roughly the same location. Obviously the retention height of the lock has to be tall enough to retain the possible sea level rise, which means that the lock also has to be strong enough to carry the additional weight and the forces accompanied by the tall retention height, as presented in Figure 7.4. This requires sufficient strength in the foundation, lock heads, chamber walls, and gate operating mechanism. According to the latest sea level rise projections of the KNMI (Le Bars, 2019), no climate change scenario results in a sea level rise higher than 2.0 m within the chosen time span between 2050 and 2150. The design is henceforth referred to as C+2.0.

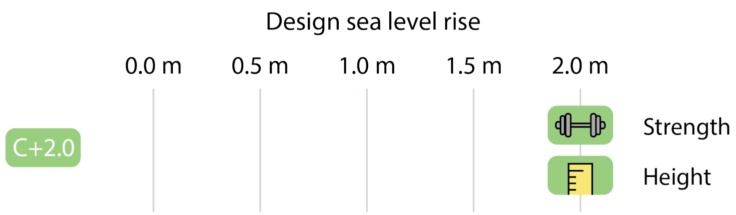


Figure 7.4: Sea level rise implemented in the design of the retention height and strength of a conservative lock alternative

7.2.2. Adaptive lock replacement

Instead of constructing a conservative lock, an adaptive design can be applied. An adaptive design is one which is specifically made to be adaptable over time. Therefore, the lock does not have to be sufficient for all sea level rise scenarios in its original state. In case sea level rise aggravates, the lock can be adapted in order to suffice to the new boundary conditions. Thus, money is saved initially by sparing elements of the construction, but might have to be spent later in the lifetime of the lock on adaptation. Two variations of an adaptive design are analysed in this thesis: a strong adaptive design, and a weak adaptive design.
The specifics of these lock replacement alternatives are described in the following sections.

Strong adaptive lock replacement

For the strong adaptive lock alternatives, the initially applied retention height is lower than would be required in case of a severe sea level rise scenario. However, the strength of the lock is already designed sufficient for a conservative retention height. This means that when a severe scenario does develop, the retention height of the lock can be increased without having to also strengthen the lock. The heightening can be done using the adaptations developed in Section 6.2.

The components which have to be constructed stronger are listed here:

- Lock heads
- Chamber walls
- Foundation and floor
- Gate bearing or rails
- Gate operating mechanism

With the same mindset, the cut-off screen should be initially designed conservatively, so that no adaptation is required later when the sea level has risen.

As explored in Chapter 6, the bed protection can be improved rather simply and inexpensively by penetrating the top layer with colloidal concrete. Therefore, no special measures are to be taken regarding the bed protection composition.

Still a choice is to be made with what retention height the lock is initially constructed, and what eventual possible retention height is considered in the strength and resilience of the structure. The latter is based on the retention height regarded of the conservative lock replacement. The strength of the adaptive lock design will therefore be based on a sea level rise of 2.0 m.

For the initial retention height, two alternatives will be analysed: 0.5 m increment, and 1.0 m increment relative to the retention height of the original lock (which is being replaced). Both are viable options within this concept. Regarding the sea level rise projections, an increment of less than 0.5 m would be rather senseless, and with an initial retention height increment of more than 1.0 m, one might as well just construct a conservative lock.

The considered adaptive lock designs, henceforth referenced as S+0.5 and S+1.0, are summarised as:

- **S+0.5**: Initial retention height for sea level rise of +0.5 m, with a strong enough base for an adaptation to increase the retention height for a sea level rise of +2.0 m.
- **S+1.0**: Initial retention height for sea level rise of +1.0 m, with a strong enough base for an adaptation to increase the retention height for a sea level rise of +2.0 m.

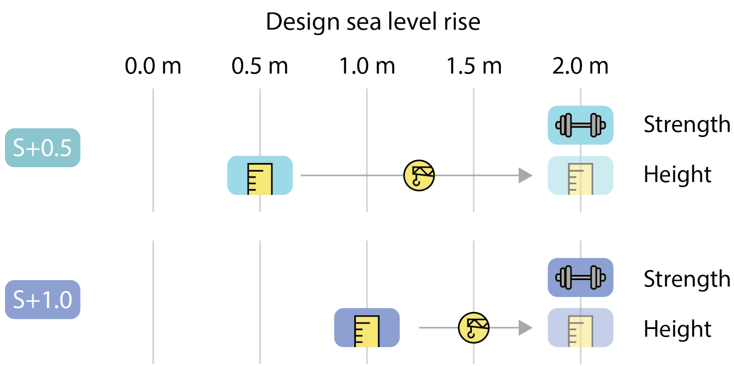


Figure 7.5: Sea level rise implemented in the design of the retention height and strength of a strong adaptive lock alternative

Present-day lock gates can be constructed with a lifespan of 100 years, equal to that of the main lock construction (Fiktorie, 2015). This means that replacing a gate halfway through its lifetime is a substantial waste of money, especially since the gates contribute a significant proportion of the total direct costs of a lock (as is derived further in this report in Section 6.3.3). Instead of installing a regular gate, a gate can be designed which is either entirely modular, or has been designed specifically to to mount a module on top of it. This also implies that the gate is strong enough to endure the additional forces and moments applied to it by the larger retainable head difference. As mentioned in Section 6.2.1.6, a modular mitre gate has been designed by Kemper (2019). Whether this principle is applicable to a roller gate has not yet been analysed.

Weak adaptive lock replacement

The other adaptive lock variant is the weak adaptive lock replacement. Contrarily to the strong adaptive lock, in this design no additional strength is prematurely implemented in the base of the structure. In case the retention height of a weak adaptive lock has to be increased, the base also has to be strengthened. For the design of these locks, a lesson can be learnt from the adaptation possibilities discussed in Section 6.2. Here it was noted that, depending on the original arrangement of a lock, it is feasible to strengthen certain components of the construction, such as the lock head and chamber walls. From these projects can be drawn what structural distinctions allowed for those adaptations to be executed. By implementing those components into the design of a new lock, the possibility is provided to strengthen the structure in a later stage, when sea level rise has aggravated. The required characteristics are listed here:

- Sufficient room alongside lock heads and chamber to install piles and sheet piles
- Structural capacity of chamber walls to empty the lock without failure due to soil pressure
- Create geometry of lock heads and chamber walls such that bore piles can be properly secured in them
- Easily accessible and replaceable gate bearing and/or rails
- Hydraulic cylinder as operating mechanism in case of mitre gates

The height with which a lock can be increased after adaptation of the lock heads, chamber walls, and foundation, is obviously dependent on the magnitude with which these components can be improved. Per indication, the Southern lock in IJmuiden (for which the heads and walls were modified) could after the adaptation bear a head difference increment of 0.75 m relative to its original design condition (Beem et al., 2000). This specific improvement was implemented because it was a new requirement for the structure, but it is unclear whether this was also the absolute structural limit which could be met by means of an adaptation. For this thesis it is assumed that a heightening of 1.0 m is feasible for a lock designed with the properties listed above.

As for the strong adaptive lock replacement, it is optional to initially apply a retention height for a sea level rise of 0.5 m, or 1.0 m. These two alternatives are referenced as W+0.5 and W+1.0. Their characteristics are graphically presented in Figure 7.6

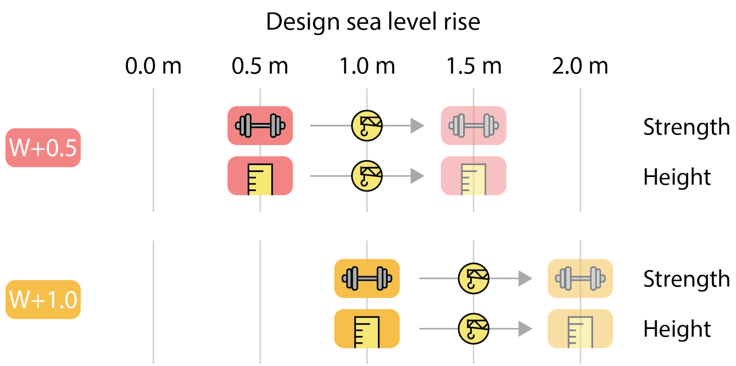


Figure 7.6: Sea level rise implemented in the design of the retention height and strength of a weak adaptive lock alternative

7.2.3. Application replacement alternatives to case study

The sufficiency of these replacement alternatives is analysed by means of the case study used throughout this thesis: the Western lock in Terneuzen. To assure that only the sufficiency regarding the sea level rise is regarded, apart from the retention height, the main dimensions of the current lock are not altered in these designs. The height of the lock is currently 18.8 m from the floor to the height of the lock heads and chamber (Kemper, 2019). Thus, to implement a sea level rise of 0.5 m, 1.0 m, or 2.0 m, the height of the lock should be increased to respectively 19.3 m, 19.8 m, and 20.8 m.

Apart from this, the lock can also be installed with a modular gate, as explained in Section 7.2.2. The width and thickness of that gate is identical to that of the current gate. The height may differ according to the values presented above, depending on the sea level rise implemented in the design.

7.3. Development pathways of replacement alternatives

To derive the effectiveness of the adaptive lock designs, they have to be compared to one another, and to the conservative alternative. For the evaluation of the adaptation concepts, five criteria were analysed: construction time, construction nuisance, familiarity of the procedure, sustainability, and circularity. For the replacement concepts, the differences between the alternatives are too small to realise well founded varying evaluation scores. The only differences between the replacement alternatives are mere small dimensional variations, which would not prompt a different evaluation score at the grand scale of the entire lock. The construction time is roughly identical, and much more influenced by other factors than minor differences in dimensions of the locks. The grading of the construction nuisance, familiarity, and circularity of the alternatives would be equal, as no different techniques are required for them to be constructed. At last, the sustainability would also vary negligibly due to the small differences in dimensions.

Instead of a criteria evaluation, the replacement alternatives can however be evaluated based on an adaptive pathways map. The sufficiency of the alternatives is time dependent, as for some alternatives adaptations are required when the sea level rise aggravates. If a lock has to be adapted during its lifetime, the lock is temporarily inoperable, which is obviously unfavorable. Thus, the lock alternatives can be evaluated by means of their pathways. These are drafted in this section.

Figure 7.7: Chapter progression Section 7.3

An adaptive pathways map is drafted similar to the one presented in Section 6.5. The replacement alternatives are decisions placed on the vertical axis, while the sea level rise development is the horizontal axis. The pathways are initiated by a single replacement alternative. The manners in which adaptations can be combined with one another and with replacements is analysed in Appendix H.

The lifetime of a lock can be prolonged by means of two adaptations, similar to those used in the adaptive pathways maps in Section 6.5.

- **Minor adaptation**: This concerns the addition of a wall on top of the lock head, and an extension mounted to the gate. This measure ensures that the flood safety is conserved until an additional 0.5 m of sea level rise. It is assumed that for this minimal change in sea level, no adaptation of the bed protection is yet required. In case the bed protection would not suffice to endure a larger flow velocity, the choice can still be made not to increase the maximum levelling sea level. This would only slightly lower the operability of the lock.

  For the case study, this adaptation includes:

  - Wall
  - Gate extension

- **Drastic adaptation**: This is applied after the minor adaptation no longer suffices. By means of this adaptation, adaptive locks are “upgraded” to a conservative one. It concerns the heightening of the lock structure, a heightening of the gate with a module in case a gate is installed which allows this, or the replacement of the entire gate and operating mechanism, and the improvement of the bed protection and piping length in case necessary for the specific lock construction. The strong adaptive lock alternatives, S+0.5 and S+1.0 have been constructed strong enough to bare the additional loads applied through these new elements. The weak adaptive locks, however, are not. These require supplementary support of the lock heads and chamber walls, which is therefore included in the drastic adaptation only for the weak adaptive alternatives.

  For the case study, this adaptation includes:

  - Modular lock extension
  - Strengthening head and walls (only for weak adaptive lock alternatives)
  - Elevation outer bridge
  - Gate module (S) or gate and operating mechanism replacement (W)
  - Penetration bed protection with colloidal concrete

- **x End of lifetime**: This concerns a situation in which either no more adaptation can be executed, or it is no longer deemed necessary. The former occurs in case a weak adaptive lock which has already been improved with additional support of the outer lock heads and chamber walls no longer suffices, and requires a drastic adaptation. The lock cannot be strengthened even further, and is therefore out of options. An adaptation is deemed unnecessary in case the lock can already endure the entire range of sea level rise regarded in this thesis.

It is never chosen not to first implement a minor adaptation to the site. The costs of this adaptation are so low, that it would be insensible not to apply it initially. If the adaptation no longer suffices due to sea level rise developments, the wall on the outer lock head has to be demolished, and the gate extension has to be removed. This may seem like a waste, but due to the small magnitude of the measure, this waste is deemed irrelevant.

With the replacement alternatives developed in Section 7.2, and the adaptations described above, the pathways map presented in Figure 7.8.

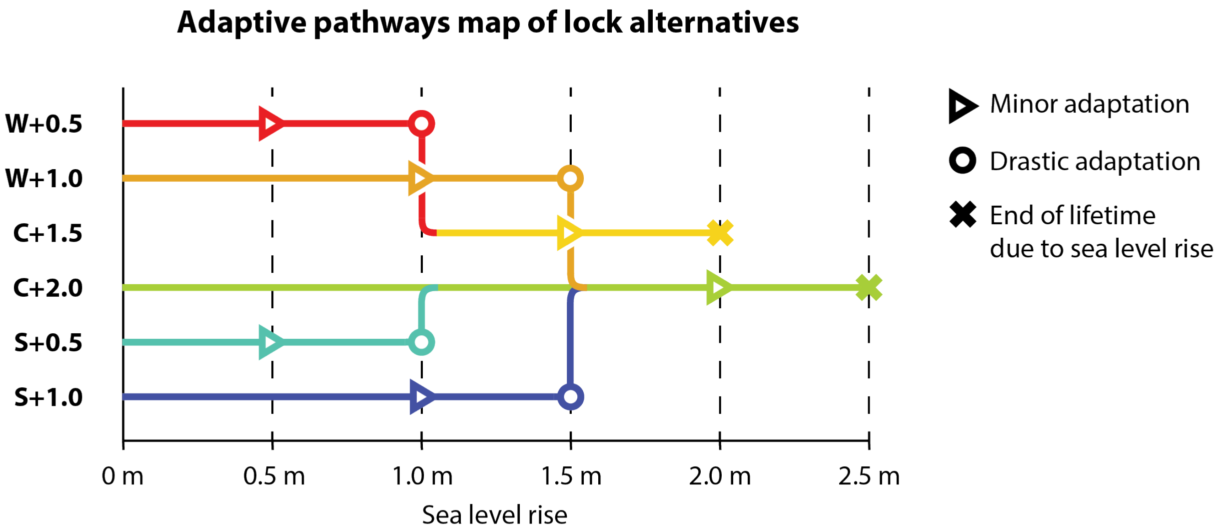


Figure 7.8: Adaptive pathways map for lock replacement alternatives

For all alternatives, the minor adaptation is initially applied when the lock itself is no longer reliable. When sea level rise has aggravated to a point at which the minor adaptation also no longer suffices, the drastic adaptation is implemented. By means of a drastic adaptation, the adaptive lock alternatives are essentially "upgraded" to a conservative lock. The drastic adaptation of the weak adaptive alternatives is limited by an addition of 1.0 m to the retention height, as was assumed to be the maximum head difference for which the base can be strengthened. Thus, W+0.5 is "upgraded" to a conservative lock which is sufficient for 1.5 m sea level rise, i.e. a C+1.5 (0.5 m + 1.0 m), whereas W+1.0 is "upgraded" to a C+2.0 (1.0 m + 1.0 m). For the strong adaptive alternatives, the additional retention height of the drastic adaptation is limited by the initial strength of the base of the structure. This is designed for a sea level rise of 2.0 m. Thus, both strong adaptive designs can be "upgraded" to a C+2.0.

7.4. Evaluation of replacement pathways

In the previous section, an adaptive pathways map was drafted including a conservative lock design and several adaptive lock designs, so that the alternatives can be compared to one another. The pathways of the replacement alternatives are evaluated and compared in a similar manner as the pathways of the adaptation concepts in Section 6.6. First, the pathways are evaluated by means of deduction points, provided to them based on the adaptations that have to be executed during the lifetime of the lock. Second, the required costs of the pathways, and their progression over the years are derived and compared. At last, the alternatives are compared to one another on both the deduction point system and the costs.

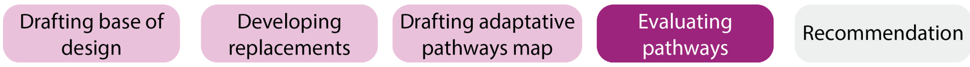


Figure 7.9: Chapter progression Section 7.4

7.4.1. Evaluation with deduction point system

Although the lock alternatives are deemed too similar to compare to one another by means of evaluation criteria, the adaptations executed during the lifetime of a lock do cause temporary inoperability, and therefore harm the operational reliability of a lock. The pathways of the replacement alternatives are therefore also evaluated based on the adaptations that have to be implemented. As mentioned in Section 6.6, for the evaluation of the pathways, a slightly different system is applied than the grading system used to compare the adaptations to one another outside of the pathways. This system is explained and derived in Appendix G.6. Each adaptation is assigned a certain amount of deduction points. By applying this system, in case a pathway includes multiple adaptations, the deduction points of those adaptations can simply be added upon another to derive a final score of the pathway. The deduction points per adaptation are presented in Table 7.2. Note that these are slightly different than for the adaptation pathways, because of the use of a gate module instead of a gate replacement.

Table 7.2: Deduction points for pathways

Measure	Deduction points
Minor adaptation	-5
Drastic adaptation S	-20
Drastic adaptation W	-28

Using zero initial deduction points for each lock alternative, the adaptation deduction points presented above can be implemented in the adaptive pathways map. The progression of the deduction points of the alternatives is presented in Figure 7.10. The sea level rise which is predicted to occur with a 95% likelihood for climate change scenarios RCP2.6, RCP6, and RCP8.5 in 2150 is presented by means of a dash-dotted vertical line. This concerns the desired lifetime of 100 years for a lock constructed in the year 2050. The scenario of RCP6 is used as derived in Appendix J.

Identical for the application of the adaptation pathways in Section 6.6.2, using the actual critical sea level rise of the structure would result in a rather irrelevant case study regarding the RCP scenarios. Therefore, the hypothetical situation is applied in which the lock no longer suffices in the year 2050. The critical characteristics and relevant adaptations remain equal.

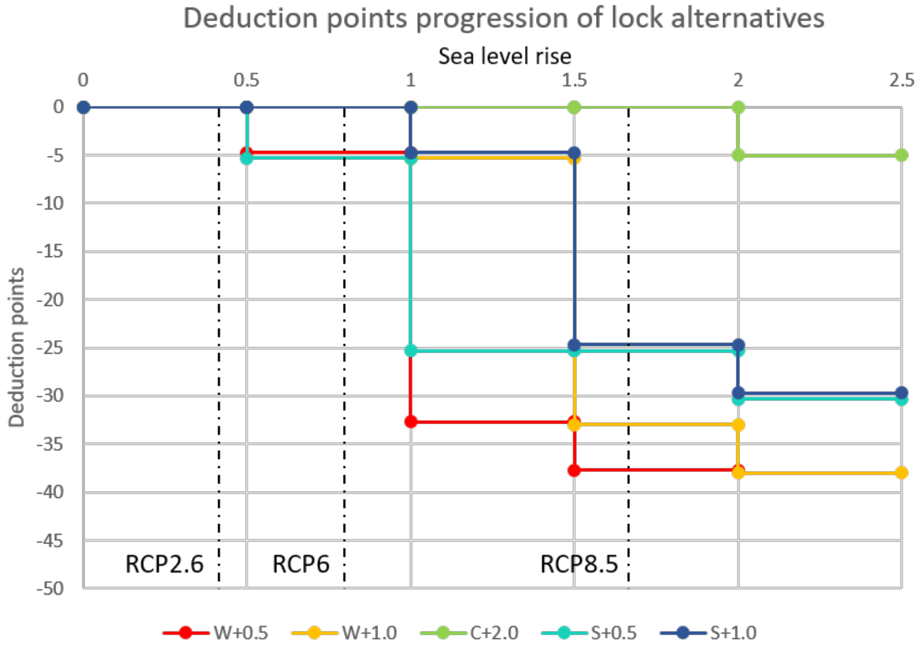


Figure 7.10: Deduction points progression of lock alternatives, with indicated sea level rise in 2150 for three climate change scenarios, RCP8.5, RCP6, and RCP2.6

Several things can be concluded from the progression of the deduction points presented in Figure 7.10. Firstly, for the optimistic scenario RCP2.6, no deduction points are assigned to any of the alternatives, because at that stage, no adaptations are required for any of the locks. For the moderate scenario RCP6, deduction points are assigned to W+0.5 and S+0.5, as these both have had to undergo a minor adaptation. Their initial retention height with an included sea level rise of 0.5 m has become obsolete in this scenario. At last, in scenario RCP8.5, all alternatives except for C+2.0 have required a drastic adaptation. C+2.0 requires no adaptation in any of the climate change scenarios, and therefore never has no deduction points assigned to it.

7.4.2. Cost progression of the pathways

The costs of the replacement alternatives are derived in Appendix I.2, whereas the costs of the adaptations (already discussed in Section 6.3) are derived in Appendix I.1. The costs of the replacements and adaptation are summarised in Table 7.3, and Table 7.4 respectively. The costs of the drastic adaptation are presented for the integration of 1.0 m sea level rise. The costs can be linearly extrapolated to find the costs for for other sea level rise implementations.

Table 7.3: Direct costs of lock alternatives

Concept	Direct costs in millions
W+0.5	€206.0
W+1.0	€211.4
C+2.0	€222.0
S+0.5	€209.8
S+1.0	€213.9

Table 7.4: Direct cost indications used to derive the cost progression of the pathways

Minor adaptation		
Wall	€26,000*	
Gate extension	€200,000	
Total direct costs	**€226,000**	

Drastic adaptation of 1.0 m	**Weak adaptive**	**Strong adaptive**
Modular extension	€3,473,000	€3,473,000
Lock strengthening	€3,128,000	€0
Gate replacement (W) or module (S)	€8,000,000	€400,000
Operating mechanism replacement	€5,332,000	€0
Bridge elevation	€3,128,000	€3,128,000
Other direct costs	€4,438,000	€4,438,000
Total direct costs	**€27,399,000**	**€11,339,000**

*Although the direct costs of this alternative are expected to be low, realistically these costs should be higher than currently derived. However, what matters in this thesis is the relation between the costs of alternatives, which should be that this adaptation is much cheaper than the others. Therefore, the cost item is not derived more thoroughly.

As explained in Section 6.6.2, it is recommended not to implement a relevant price difference over time. Facets like inflation and economical growth are expected to balance one another over long stretches of time (Ministry of Finance, 2020).

Using the costs of the lock alternatives and the costs of the adaptations, the cost progression of the different pathways can be derived for RCP8.5, RCP6, and RCP2.6. The results are presented in Figures 7.11, 7.12 and 7.13. The results are explained and compared below each figure.

Cost progression of pathways for RCP8.5

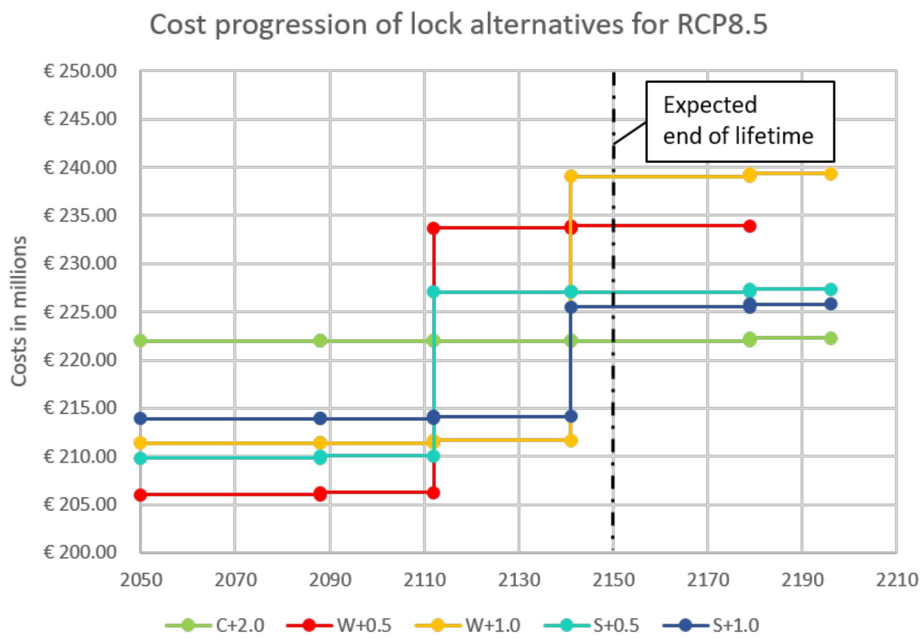


Figure 7.11: Cost progression of the lock alternatives for climate change scenario RCP8.5

- C+2.0: Although this alternative is initially the most expensive, by the end of the lifetime of the lock, it is actually the cheapest, because no adaptation is required at all.

- **W+0.5**: In the initial years, W+0.5 is the cheapest option, obviously because it is the design with the least potential. At a sea level rise of 1.0 m, a drastic adaptation is required, for which the alternative is no longer the cheapest option. By the end of its lifetime, it is 5.4% more expensive than C+2.0, which is than the cheapest.
- **W+1.0**: This alternative is not much more expensive than W+0.5, and lasts until a sea level rise of 1.5 m until a drastic adaptation is required. By the end of its lifetime, it is 7.7% more expensive than C+2.0, which is than the cheapest.
- **S+0.5**: Initially, this alternative is among the cheapest. However, already halfway its lifetime a drastic adaptation is required. Therefore, by the end of its lifetime, it is 2.3% more expensive than C+2.0, which is then the cheapest.
- **S+1.0**: This alternative is never the cheapest option, but remains relatively inexpensive until a sea level rise of 2.0 m, near the end of the lifetime of the lock. By the end of its lifetime, it is 1.6% more expensive than C+2.0, which is than the cheapest.

Cost progression of pathways for RCP6

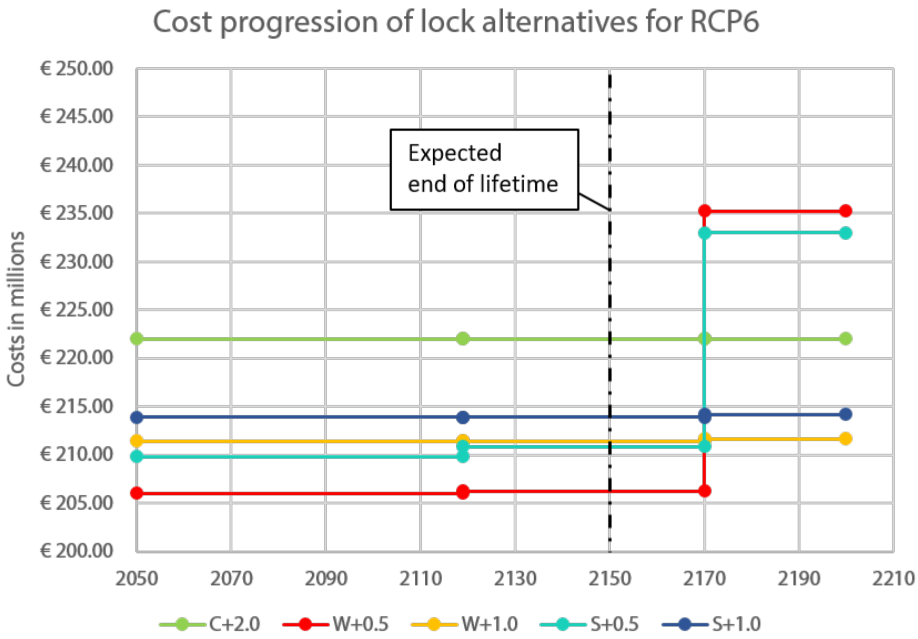


Figure 7.12: Cost progression of the lock alternatives for climate change scenario RCP6

- **C+2.0**: This alternative is the most expensive for this scenario during the expected lifetime, but not extensively. By the end of its lifetime, it is 7.6% more expensive than W+0.5, which is then the cheapest. The design does however show rigidity, and when the sea level rise becomes a little more severe, it is only 4.9% more expensive than W+1.0, which would than be the cheapest.
- **W+0.5**: This alternative remains the cheapest for the entire lifespan of the lock. However, it should be stated that if the lifespan is desired to be prolonged, or if sea level rise is only a little worse than the projected scenario, the required adaptation makes this the most expensive design. It then becomes 11.1% more expensive than W+1.0, which is at that point the cheapest.
- **W+1.0**: This alternative is a little more expensive than W+0.5, the cheapest alternative throughout the lifetime of the lock. By the end of its lifetime, it is only 2.5% more expensive. However, in contrast to W+0.5, the concept could last much longer in case sea level rise becomes more severe. It than actually becomes the cheapest option, as merely a small adaptation is required.
- **S+0.5**: Although this alternative is relatively cheap during the lifetime, if the sea level rises a little more than this scenario, the large required adaptation makes it much more expensive. It is by the end of its lifetime 2.2% more expensive than W+0.5, which is than the cheapest. After the adaptation is is 10.1%

more expensive than W+1.0, which would than be the cheapest.
- **S+1.0**: This alternative is comparable to C+2.0. During its lifetime, it is 3.7% more expensive than W+0.5, which is than the cheapest. In case sea level rise becomes a little worse than the scenario predicts, it is 1.2% more expensive than W+1.0, which is then the cheapest.

Cost progression of pathways for RCP2.6

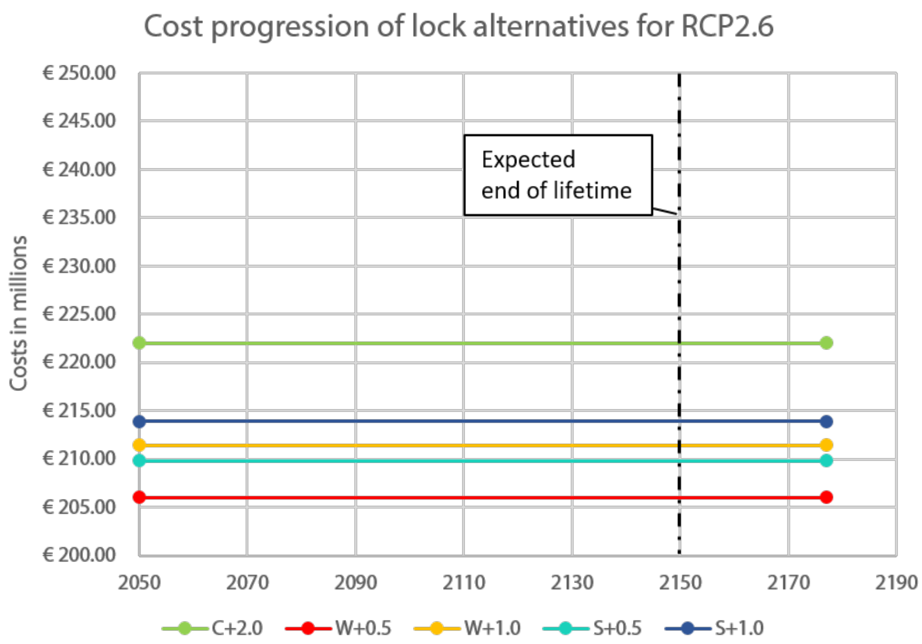


Figure 7.13: Cost progression of the lock alternatives for climate change scenario RCP2.6

- **C+2.0**: This alternative is the most expensive during the entire lifetime of the lock for this climate change scenario. It is 7.8% more expensive than W+0.5, which is the cheapest.
- **W+0.5**: This alternative is the cheapest during the entire lifetime of the lock for this climate change scenario.
- **W+1.0**: This alternative is the *third* cheapest during the entire lifetime of the lock for this climate change scenario. It is 2.6% more expensive than W+0.5, which is the cheapest.
- **S+0.5**: This alternative is the *second* cheapest during the entire lifetime of the lock for this climate change scenario. It is 1.8% more expensive than W+0.5, which is the cheapest.
- **S+1.0**: This alternative is the *second* most expensive during the entire lifetime of the lock for this climate change scenario. It is 3.8% more expensive than W+0.5, which is the cheapest.

Summary of cost progressions

Obviously, the economically optimal alternative is different dependent on the climate change scenario. W+0.5 and W+1.0 perform well in the mild scenarios, but become expensive in case the sea level rise reaches 1.0 m, and 1.5 m respectively. Due to the additional support required for several lock elements, the adaptation costs are higher than for S+0.5 and S+1.0. These alternatives are a little more expensive in the mild scenarios than W+0.5 and W+1.0 respectively, but are cheaper on the long term in case of a severe scenario.

The conservative alternative, C+2.0 is in the mild scenarios obviously the worst performing economically, but not with very large margins in relation to the best performing options (7.6% in RCP6, and 7.8% in RCP2.6). In case of RCP8.5, however, it is the cheapest, but with a smaller margin than any of the best alternatives in the other scenarios.

7.4.3. Cost-value analysis of pathways

The criteria evaluation and cost analysis both individually only tell half of the story. Only by comparing both aspects of the concepts to one another, can a sound recommendation be made. In Table 7.5, a summary is provided of the costs and deduction points derived in Sections 7.4 and 7.4.2. The costs are provided as percentage of the conservative alternative C+2.0, as this provides an easier understanding of the relative differences between the concepts. Because the costs of C+2.0 are equal for all scenarios, the values of different scenarios can still be compared to one another.

Table 7.5: Summary of evaluation and costs of lock alternatives

	RCP8.5		RCP6		RCP2.6	
	Costs relative to C+2.0	Deduction points	Costs relative to C+2.0	Deduction points	Costs relative to C+2.0	Deduction points
W+0.5	+5.4%	-38	-7.1%	-5	-7.2%	0
W+1.0	+7.7%	-33	-4.8%	0	-4.8%	0
C+2.0	-	0	-	0	-	0
S+0.5	+2.3%	-20	-5.0%	-5	-5.5%	0
S+1.0	+1.6%	-20	-3.6%	0	-3.6%	0

The costs and deduction points are also graphically presented in Figure 7.14. Only RCP8.5 and RCP6 are presented here, as the results of RCP2.6 are too similar to those of RCP6, and reduce the clarity of the chart. The outcome of RCP8.5 is displayed by means of circles, whereas RCP6 is illustrated with crosses. The outcome of W+0.5 is identical for both scenarios.

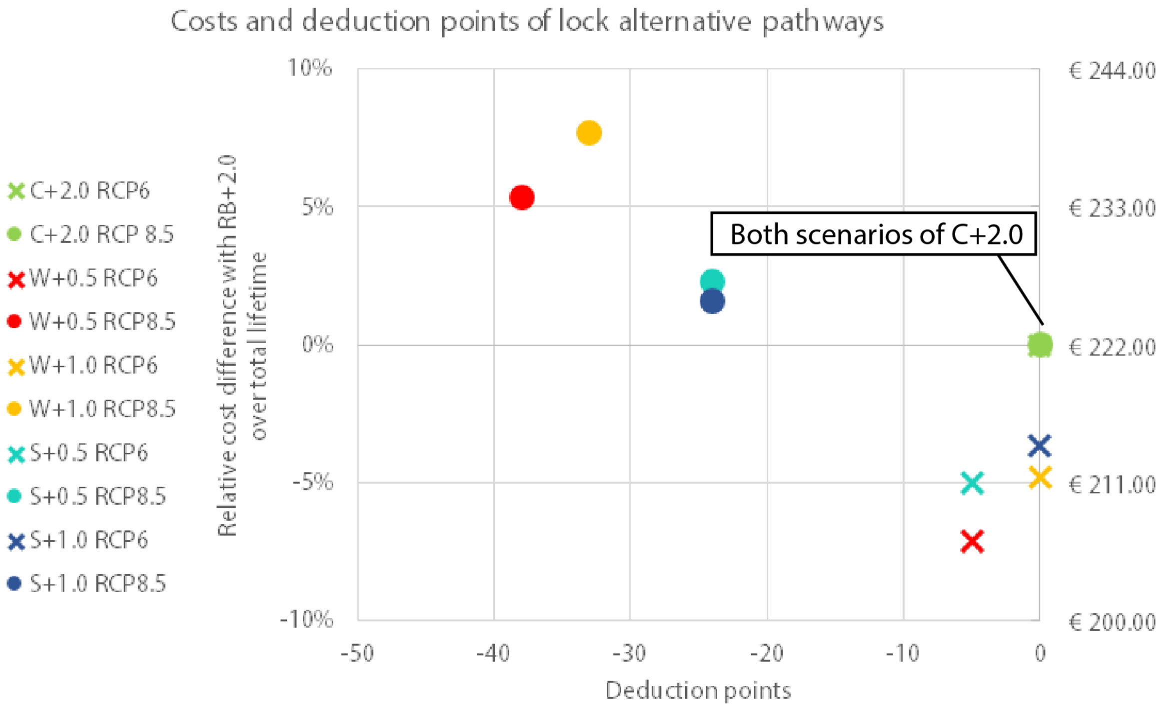


Figure 7.14: Costs and deduction points of lock alternatives

What becomes clear from Figure 7.14, is that, as would be expected, all adaptive alternatives are initially cheaper than the conservative lock. On the other hand, all alternatives are more expensive and get assigned more deduction points in case of a severe scenario. For both the strong adaptive lock alternatives and W+0.5, the costs saved in case of a mild scenario are higher than the costs which can eventually be spent additionally in a severe scenario in relation to the conservative lock design.

7.5. Recommendation for adaptive or conservative replacement

This step of the method concerns the decision whether to replace insufficient locks with an adaptive design, or a conservative design. Throughout the previous sections of this chapter, design alternatives have been developed (one conservative and several adaptive), and have been implemented in an adaptive pathways map, whose pathways have sequentially been compared based both on their value and costs. In this final section, a recommendation is made on deciding whether to construct adaptive or conservative. The advantages and disadvantages of an adaptive design are listed below, and further explicated throughout the remainder of the section.

+ Money is saved by applying an adaptive design in case of a mild sea level rise scenario.
- Additional money is spent in case of a severe sea level rise scenario due to required adaptations.
+ In case of a severe sea level rise scenario, the payment is spread, as the adaptation is required later in the lifetime of the lock.
- A section or the entirety of the waterway has to be closed off temporarily during adaptation in case of a severe scenario, and additional nuisance is experienced by neighboring residents and users of the road crossing the lock.
- The life cycle of a lock becomes more complex, requiring stricter monitoring of the lock and hydraulic boundary conditions, and additional large scale planning (also considering the simultaneous demand of adaptation of other adaptive locks) in case of a severe sea level rise.

As is to be expected, all adaptive designs are initially cheaper than a conservative lock, but become more expensive over their lifetime in case they have to be adapted due to an aggravated sea level rise. For the case study, the costs which can initially be saved vary between 7.1% and 3.6% of the total direct costs of the conservative lock, and the costs which can eventually be spent additionally vary between 1.6% and 7.7%. The cost study performed in this thesis is not very extensive, and should therefore not be weighted too much. It does however function to indicate that the issue does not concern significantly large portions of the total project costs, especially given the large cost uncertainty of projects of this magnitude.

Instead of regarding a single lock, it is also valuable to understand how the application of an adaptive lock can help in the replacement of all Dutch locks. Over time, all locks will eventually require replacement, unless they are no longer required in the waterway. Because there are a lot of marine locks in the Netherlands (see Table 2.1), it can occur that several locks require replacement around the same time. This obviously demands large sums of money, which are not necessarily available. By applying an adaptive lock design to all replacements, the sum of savings becomes rather substantial. Also, even if some or most locks have to be adapted further in their lifetime, and the total costs become larger than would have been for conservative locks, at least some of the payment has been spread out over many years, which is favorable in terms of the availability of money.

Of course a disadvantage of the adaptive design is that, in case a severe sea level rise develops, the construction of the adaptation causes additional obstructions. The lock will have to be closed during a relatively long period (around 1 year), in which the capacity of the waterway is either reduced severely, or even reduced to none. Also the neighboring residents will experience nuisance of the construction, and traffic will have to be diverted temporarily. Depending on the location and arrangement of the lock, this can cause considerable transportation issues.

At last, an important upside of a conservative lock is that when it is constructed, it can be expected that it requires no major adaptation for a long stretch of time. Obviously improvement is always relevant for small components, but the large elements of the lock can mostly remain untouched (except for small renovations). For an adaptive lock, the possibility of an adaptation always lingers. This may require more intensive monitoring of the state of the lock, and the development of the hydraulic boundary conditions. Additionally, if many locks have to be adapted within a relatively short period of time, the planning of these projects may become complex. This issue would however be known long before it arrives, providing a sufficiently long period to work out the logistics of the adaptation operation.

8

Conclusions and recommendations

8.1. Conclusions

Below, a short summary of all conclusions in this thesis is provided. Consecutively, the main research and design questions are treated more extensively in the following sections.

Four critical lock characteristics are to be improved while sea level rise aggravates: the retention height, gate strength, piping path length, and the composition of the bed protection. These characteristics can be improved either by adaptation of the lock, or complete replacement. Unless there is another reason than sea level rise to replace the lock, it is recommended to always initially apply a minor adaptation, i.e. a small wall on the lock head, and a gate extension. If this no longer suffices, it depends on the current functional and structural state of a lock, and the available money, whether it is preferable to apply a drastic adaptation, or replace the lock entirely. When replacing a lock, instead of constructing it conservatively, an adaptive design can be applied. This assures not much money is wasted in case of a mild sea level rise scenario, but allows for easier adaptation in case of a severe scenario. It depends on the lock and waterway arrangement whether the saving of costs in a mild sea level rise scenario, or the spreading of costs in a severe scenario, weighs up against the possibility of temporary lock inoperability and nuisance during the construction of an adaptation in case of a severe scenario.

8.1.1. Development failure probability due to sea level rise

To answer the main research question of this stage of the thesis, initially its sub-questions are to be clarified.

1.1 *What failure mechanisms of a marine lock are relevant in the assessment of the effect of sea level rise on the sufficiency of its flood protection function?*

In Section 4.3, a selection of relevant failure mechanisms was made based on the influence of the sea level on the probability of those mechanisms. The selection includes "Failure storage capacity", "Failure bed protection", "Gate retention failure", and "Piping failure". The retention failure of the gate can be divided into several sub-mechanisms. Of those, the relevant ones are "Failure due to head difference", "Failure due to vibration", and "Closure failure".

1.2 *To what extent does sea level rise alter the flood protection failure probability of a marine lock relative to its legal norm, and how does the development of each individual failure mechanism contribute to this advancement?*

The answer to this question differs per analysed lock. Of course the requirements and boundary conditions to which a lock has been designed have a large influence on the sea level rise which can occur before its failure probability norm is surpassed. For the case study, the Western lock in Terneuzen, this norm is passed around 1.1 m sea level rise. For other locks, this can be derived using the failure probability model developed in Chapter 5.

1.3 *What lock characteristics are critically related to the failure mechanisms which contribute significantly to the development of the failure probability?*

The critical characteristics of a lock are the parameters which can be adapted, and therefore do not include hydraulic boundary conditions. For both "Failure due to vibration" and "Failure due to overflow", a critical feature is the retention height (including both the lock construction and the gates). For "Failure due to overflow", also the bed protection composition and storage capacity could be adjusted, but because the retention height has to be adjusted regardless to reduce vibration failure, these features can be disregarded. However, to maintain the operability of a lock, the head difference at which levelings are allowed has to increase along with the sea level rise. When a lock is emptied at a larger head difference, the flow velocities arise for which the bed protection has not originally been designed. Therefore the bed protection composition *is* actually a critical characteristic. For "Failure due to head difference", the strength of the gate is a critical feature.

Apart from these features, the piping length is considered, because a rising sea level indefinitely has an impact on the piping probability. The case study has a very long piping path, which is the reason why this failure mechanism does not contribute to the failure probability of this particular lock.

Now that the sub-questions are answered, the main research question of this stage an be treated:

1. *What are critical lock characteristics of a marine lock which have to be improved first in order to maintain a sufficient flood protection function with respect to a rising sea level?*

The critical features which are required to be improved are:

1. Retention height
2. Gate strength
3. Piping length
4. Bed protection composition

8.1.2. Decision to adapt or replace

As for the previous stage, the sub-questions are initially to be answered before the main design question can be regarded.

2.1 *In what manners can the critical characteristics be improved by means of an adaptation?*

The adaptation alternatives which were developed in Section 6.2 are listed in Table 8.1.

2.2 *What is per critical characteristic the optimal adaptation alternative, based on their value and costs?*

The adaptation concepts have to be applied and evaluated for each lock specifically. For different locks with different properties, it can vary what concepts are optimal. It is however likely that a set of minor adaptations and a set of drastic adaptations can be assembled. The adaptation concepts presented in Table 8.1 were evaluated in Section 6.3 for the case study, i.e. the Western lock in Terneuzen. The value of the concepts was evaluated based on several criteria, and was compared to their costs. A selection could thus be made on the adaptation concepts which were relevant to consider in the adaptive pathways map. These are indicated by means of a check mark in Table 8.1. Because for the case study, the piping length is no critical feature, it is not regarded in the evaluation.

Table 8.1: List of adaptation concepts developed in this thesis, sorted by critical feature. The concepts which were considered in the development of the adaptive pathways map are indicated with a check mark. Because the piping length was no issue for the case study, this feature is disregarded.

Retention height lock construction	
Wall of 0.5 m on outer lock head edge	✓
Fixed extension of existing lock design	x
Modular extension of lock	✓
Retention height gates and gate strength	
Gate extension of 0.5 m	✓
Gate replacement with taller duplicate	x
Gate replacement with (modular) gate upon which modules can be mounted	✓
Piping length	
Additional cut-off screen(s)	-
Sandtight filter below bed protection	-
Bed protection composition	
Bed protection replacement	x
Penetration of top layer bed protection with colloidal concrete	✓

2.3 *How does the implementation of the optimal adaptations relate to a replacement, based on time dependent progression of their value and costs?*

In Section 6.5, an adaptive pathways map was drafted. In these maps, pathways are present for initiation with a minor or drastic adaptation, and for direct replacement. The costs required for an adaptation are won back within several years of postponing the replacement. The lock is however not operable during the period of adaptation. For a minor adaptation (addition of wall on outer lock head and gate extension with a height of 0.5 m), this period is short, but the execution of a drastic adaptation (modular lock extension, gate replacement, and penetration of bed protection with colloidal concrete) can take around 1 year. If a lock is not part of a complex, but forms the only way into a waterway, this causes inaccessibility for a significant period. If a lock *is* part of a complex, this still causes a capacity reduction of the waterway.

2. *Is it effective to prolong the lifetime of a lock by means of an adaptation, instead of directly replacing the lock entirely?*

Although it has to be considered per individual case whether the inoperability of a lock during the construction time of the drastic adaptation is worth the benefit of the procedure, it is assumed that a minor adaptation will always be beneficial. A minor adaptation applied in 2050 can still prolong the lifetime of a lock with almost 40 years in the most severe climate change scenario. In the least severe scenario, the lifetime is prolonged with over 100 years. It is therefore recommended to always apply a minor adaptation instead of directly replacing a lock. Even if a drastic adaptation is found to be beneficial, it is still recommended to first apply a minor adaptation. If a severe sea level rise develops, and the measure no longer suffices, it can easily be demolished, which is barely any waste due to the small magnitude of the components.

8.1.3. Effectiveness of adaptive replacement alternatives.

The main design question of this stage is also treated after its sub-questions are answered.

3.1 *What are possible design deviations from a regular conservative lock which can be applied to help against the uncertainty of sea level rise?*

Two adaptive alternatives are studied which can be constructed instead of a conservative lock. The first is a one with a base (foundation, lock heads, and chamber walls) which is strong enough to withstand the weight and other forces accompanied by a conservative retention height, but without a conservative retention height. Thus, a heightening of the structure can be executed without having to strengthen the lock. The other adaptive alternative is one with no such stronger base. Thus, in case sea level rises severely, and the structure has to be heightened, the base also has to be strengthened. The lock has to be designed in a way to allow such a strengthening.

For both alternatives, a choice is still to be made on what sea level rise to implement in the initial design. In this thesis, two options are applied to both alternatives, i.e. 0.5 m and 1.0 m sea level rise. In the conservative alternative, a sea level rise of 2.0 m is implemented.

3.2 *How do the costs and value of these alternatives compare in their initial state, and their adapted state which is applied in a severe sea level rise scenario?*

Obviously, the costs of the adaptive alternatives are lower than those of the conservative lock in case no adaptation is required. The margins are however not large. In the most severe climate scenario, the locks do however have to be adapted. The total costs required over the lifetime of the alternatives then become larger than those of the conservative lock. The costs which can eventually be made additionally are similar to the costs which can initially be saved for all alternatives. However, by first saving costs and later spending them, the total expenses of the structure are partially spread out over the life cycle of the lock. On the other hand, the application of the adaptation does temporarily lower the capacity of the waterway, temporarily obstructs the crossing bridge(s), and brings nuisance to neighboring residents. Also, it can be of value to know that a conservative construction does not require any adaptation over a long stretch of time.

3. *Is it efficient to replace a lock with an adaptive alternative, rather than a conservative one, when facing an uncertain sea level rise?*

Whether an adaptive design is better than a conservative one unfortunately still strongly depends on the specifics of a lock, and the timing of the replacement. If not much money is available at the time, the small margin which can be saved can be of great value. On the other hand, the obstructions which arise when adapting a lock may be decisive not to apply an adaptive lock, depending on the arrangement of the lock, the waterway, and the occupation of the neighboring area. Although the method of the adaptive pathways map does help in understanding the applicability of the alternatives, the conclusions which can eventually be drawn are no different than they would be without the adaptive pathways map. Thus, the method only helps with illustrating the issue, and not with quantifying the it.

8.2. Recommendations

Throughout this thesis, there are several loose ends and assumptions of which a more thorough analysis could benefit the confidence of the conclusion, or could make for an interesting follow up study. Here, some recommendations are made on the topics which could be studied forward.

Variety of case studies
In this thesis, only the case study of the Western lock in Terneuzen has been applied. Obviously the results are therefore restricted in their representation of the large variety in properties and dimensions which a lock can possess. To provide a more complete overview of how the developments of sea level rise affect the integrity of a marine lock, and how these effects can be resolved, a larger range of case studies should be applied. By studying multiple locks with different properties, the influence of those properties on the sufficiency of the current locks and lock adaptations can be better analysed.

Deduction points
To evaluate the pathways, a system of deduction points is applied in the thesis. These can be used properly to relate one pathway to another, but still hold a rather vague significance in decision making. For instance, it is unclear how much worse it is for a pathway to have 5 deduction points rather than 0. The deduction points are based on construction time, construction nuisance, and familiarity of the procedure. These elements could all be reworked to a cost item. For the construction time, the costs lost by the reduction of the capacity of the waterway can be derived. For the nuisance, a societal cost indication can be calculated. For the familiarity of the procedure, a risk analysis can be applied, resulting in a cost increment due to unfamiliarity of a procedure. These cost items can simply be added upon the regular construction costs of the adaptations, resulting in a single comparable value.

Filling and emptying system
In the thesis, the filling and emptying system has not been regarded, as this does not concern the flood safety of the hinterland. However, when the operability of a lock is desired to remain equal when the sea level rises, the lock has to be filled and emptied at larger head differences than originally planned. It has to be assessed whether the Hawser forces (forces applied to vessels by water jets created by filling and emptying a lock) do not complicate the stabilising of the vessels in the lock chamber (Molenaar, 2020). If these forces become too large, it might become relevant to also consider different adaptive designs for the filling and emptying system. One example of such a system is worked out in the thesis of Dorrepaal (2016).

Crossing bridge
At the case study, and many other locks, one or two bridges are present to allow "dry" traffic to cross the waterway. The structures have here been regarded as rigid elements which require to be elevated along with the heightening of the lock structure. It is however possible that there are more elegant, and maybe adaptive solutions to this lock component.

Adaptability per component
Adaptability has in this thesis been regarded as a property which a lock can possess in its entirety. However, in reality, it should be analysed per component whether it can and should be applied adaptive. A different conclusion may be drawn per component regarding the efficiency of an adaptive design.

River water levels
As discussed in Section 2.1.2, the yearly fluctuation of the river levels is also expected to change due to climate change. In winter, water levels are expected to rise, while in summer draughts cause lower river levels. Obviously, this will also have an effect on the failure mechanisms which are dependent on the head difference. The case study in Terneuzen is connected to a regulated channel, but for locks in an unregulated river, analysing the combined effects of sea level rise and river level fluctuations could result in a much more urgent failure probability development.

Hydraulic boundary conditions
Climate change can have even more effect on the hydraulic boundary conditions of a marine lock. Increased severe storm conditions can cause extreme storm surges and wave heights. Combined with the sea level rise, this aggravates the design loading conditions of the locks even further. Whether these conditions form a danger for the integrity of a lock obviously depends on the geography of its surroundings.

Maintenance costs
In the development of the cost progression of the adaptation pathways in Section 6.6.2, the maintenance costs have been regarded as a yearly cost item. However, this yearly cost has been derived from maintenance which has to be executed roughly each 5, 10, or 20 years, depending on the component (De Pelsmaeker, 2010). To get a more realistic cost progression, the maintenance costs should not be regarded as yearly costs, but as a cost item which is to be paid at the moment of relevancy. This does also require knowledge on the moment at which the components of a case have undergone maintenance most recently.

Economical model
In the economical model used to analyse the development of the benefits of the pathways in Section 6.6.2, many assumptions have been made on economical developments, like the altering of the value of the euro, and case specific aspects, such as the loan interest. A variation to these assumption can be applied to study how the benefits of the pathways alter in case these aspects develop differently.

Wearing of timber
For the case study, steel roller gates are installed. For such gates, the wearing of the material is not a very important factor. For timber gates, however, the deterioration of the material can have a grave influence on the structural capacity of the structure. The wearing of material is case specific, and should be evaluated per project.

Modular extension
It has been assumed in this thesis that a modular extension can be applied to a lock, without any derivation of the structural capacity of such elements. Retention walls have been made out of modules prior, but this does not mean that the application to a lock works regardless. In addition, it could be interesting to study whether a lock can be constructed out of modules completely, or at least a large section of it. Regarding Rijkswaterstaat's ambition to construct fully circular by the year 2030, this might offer a well applicable solution.

A

River discharge extremes

In Section 2.1.2, the projections of the average discharge of the rivers Rhine and Meuse are provided for 5 different climate scenarios. These provide a sufficient insight in the change that can be expected in case of those scenarios, but as structures are designed to be able to withstand extreme loading conditions, it is also important to view the development of the extreme high and low discharges. These have been analysed in the same study by Deltares as the average discharge, and are discussed throughout this Appendix (Klijn et al., 2015).

A.1. Extreme high discharges

The monthly averaged discharge provides a clear general presentation of the severity of the discharge concerns. However, regarding the flood safety of the Netherlands, the frequency of extreme discharge events is much more important. As the average discharge is likely to increase over the forthcoming years, and the rainfall is expected to be more intense, such events will be triggered more often, and increase in severity. Regarding the flood protection of a lock, the storage capacity is decreased significantly in case of a high river level.

Figure A.1 shows the projections made by Deltares of the return period of discharges predicted per scenario. In addition to the coloured lines for each scenario, grey lines are drawn for the expected discharge in case the dyke system surrounding the Rhine would be infinitely high. However, as the standards of the German dykes are much lower than in the Netherlands, overflow is expected to occur at lower water levels. Therefore flood planes are created, decreasing the flow of water reaching Lobith. Although this phenomenon considerably limits the discharge estimation, the 16.000 m^3/s on which the Dutch dyke system has been designed will have a return period of 1:500 years instead of the current 1:30.000 years for scenarios WL and WH in 2085.

In the projections of the Meuse discharge return periods (presented in Figure A.2), no grey lines are plotted, as no flood planes have been taken into account. The Meuse mostly runs through a gorge in Belgium, and therefore no dykes can be flooded over a large section of the river. The dyked section of the Meuse in the Netherlands has been designed for an event with an occurrence of 1:10.000 years. The discharge at this return period could potentially increase from 4350 m^3/s to 5200 m^3/s in scenario WL in 2085.

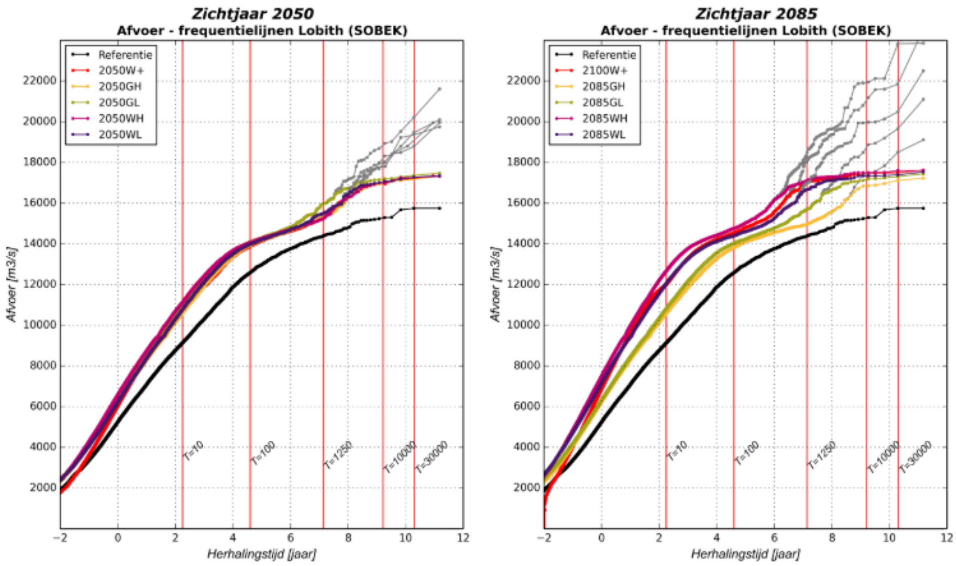


Figure A.1: Projections return periods (in years) of discharges (in m^3/s) of Rhine at Lobith in 2050 and 2085 per climate change scenario. Retrieved from Klijn et al. (2015).

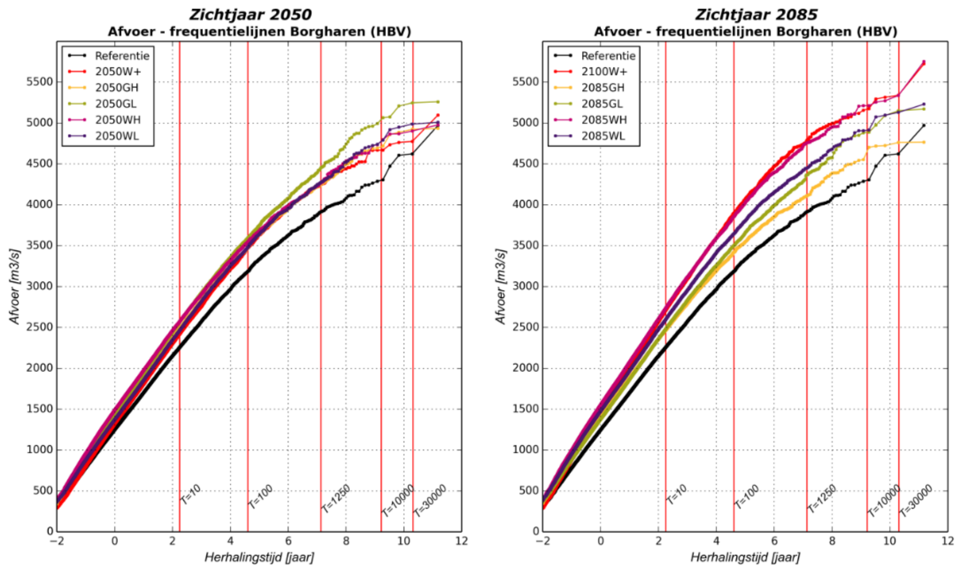


Figure A.2: Projections return periods (in years) of discharges (in m^3/s) of Meuse at Borgharen in 2050 and 2085 per climate change scenario. Retrieved from Klijn et al. (2015).

A.2. Extreme low discharges

As temperatures will rise, evaporation rates will increase in the future, and droughts will become more severe and long lasting during summer periods. These will result in low discharges, and thus low water levels. As this causes the head at a marine lock to increase, failure mechanisms can be triggered. Deltares has calculated the discharge at the seven consecutive days of the year at which it is expected to be lowest for the five given scenarios.

The percentage of change in discharge during the before mentioned dry period is presented in Figure A.3 for the Rhine and Moselle. For the Netherlands, the left section (Lobith) of the graphs is important. The figure shows a significant reduction of even 30% by the year 2085 in scenario WH$_{\mathrm{dry}}$, but a positive increment of 5% in scenario GL.

The estimations for the droughts of the Meuse are presented in Figure A.4. As was to be expected from the averaged monthly discharge projection in Figure 2.9, the consequences are more severe for the Meuse then for the Rhine. Predictions vary between 0% (GL) change to 60% (WH_{dry}) by the year 2085. The latter could have grave consequences for the Dutch lock complexes. This again shows both the relevance and unpredictability of the consequences of climate change for the Dutch rivers and flood protection.

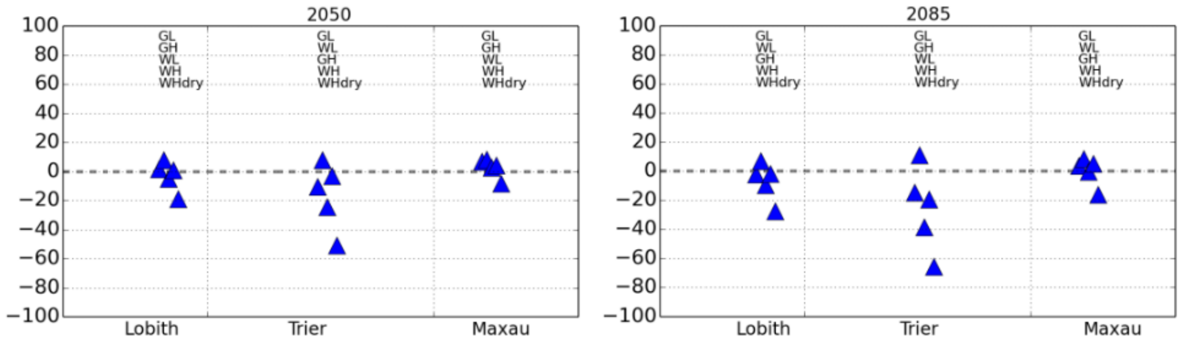


Figure A.3: Prediction change (in percentage) of the lowest yearly discharge per scenario of the Rhine in Lobith (left), the Moselle in Trier (middle), and the Rhine in Maxau (right) in 2050 and 2085. Retrieved from Klijn et al. (2015) (edited).

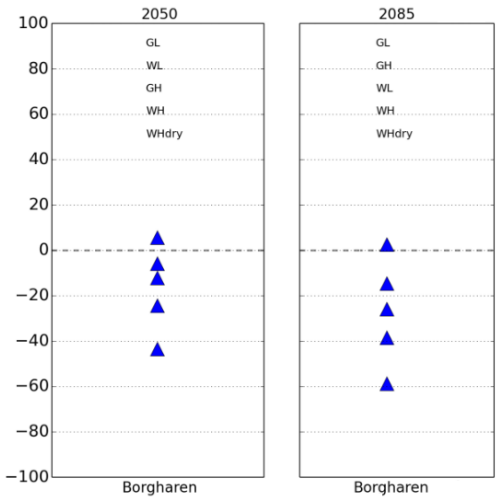


Figure A.4: Prediction change (in percentage) of the lowest yearly discharge per scenario of the Meuse in Borghoren in 2050 and 2085. Retrieved from Klijn et al. (2015) (edited).

B

Retention failure fault tree

Section 4.2 describes the relation between the different failure mechanisms of a marine lock. To illustrate how these relations can be described mathematically, fault trees are drawn up throughout this Appendix.

A division is made between the consequences of the failure of the outer gate due to the four mechanisms as listed in Section 4.2: closure failure, failure due to head difference, failure due to vibration, and failure due to collision.

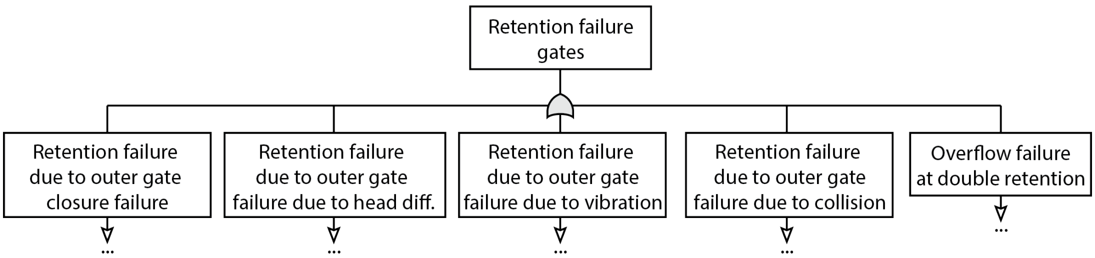


Figure B.1: Fault tree for "Failure retention elements". Down pointing arrows indicate the fault tree can be expanded.

B.1. Closure failure

Closure failure of the outer gates can coincide with many other failure mechanisms: the inner gate can fail due to the head difference, vibration, and a collision, or otherwise water can still pass the gate by means of overflow. This is presented in the fault tree in Figure B.2. Important to note is that mechanisms "Failure due to head difference" and "Failure due to vibration" depend on water levels, and so do their consequences (bed protection erosion and storage capacity surpassage). Thus, for these mechanisms, the consequence of "Inflow failure" is correlated to the cause. The probability of interest is therefore only the inflow failure *given* failure due to head difference or vibration. For "Failure due to collision" this is not the case.

The probabilities calculated through the Monte Carlo analysis are probabilities of occurrence per year. If these would be implemented in the fault tree in Figure B.2, the probability would be derived of "Closure failure" occurring in the same year as any of the other mechanisms. Of course this is not what is sought after, but rather the chance of a simultaneous occurrence, i.e. the probability of failure of the inner gate whilst the closure failure of the outer gate has not yet been resolved. In the assessment of the IJmuiden lock complex, a span of 12 hours is assumed for the failure to be mitigated (Van den Berg et al., 2019). The 12 hourly probability can be derived from the yearly probability as described in Equation B.1.

$$P_{yearly} = 1 - (1 - P_{12\ hourly})^{(365 \cdot 2)} \quad \rightarrow \quad P_{12\ hourly} = 1 - \sqrt[365 \cdot 2]{1 - P_{yearly}} \tag{B.1}$$

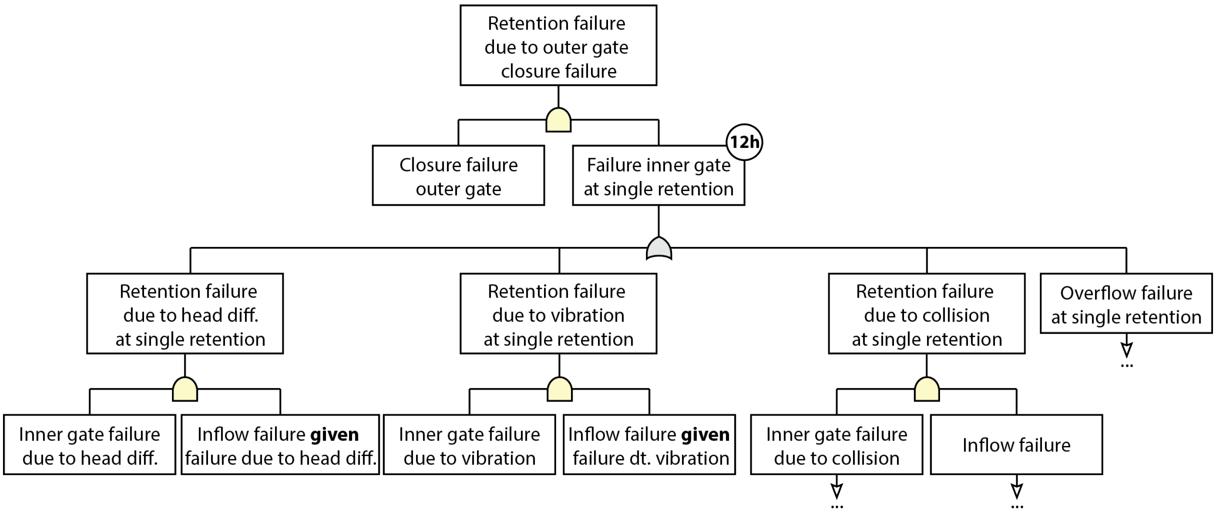


Figure B.2: Fault tree for "Closure failure". "Failure inner gate at single retention" concerns the probability of occurrence within the 12 hour time span between closure failure of the outer gate and its mitigation. Down pointing arrows indicate the fault tree can be expanded.

B.2. Failure due to head difference

For a lock with the policy to close all gates during extreme conditions, the head difference to which the outer gate is subjected will change when closure failure occurs at the inner gate. In case the gate is intact, the water levels can be managed to cascade (decreasing the loads). This is impossible when the inner gate is unable to close. This is presented in the fault tree in Figure B.3. On the left, the water level is cascaded, whereas on the right it is not. The top events in this fault tree are mutually exclusive, as closure failure of the inner gate either does, or does not occur. In case extreme conditions arise for which the gates are to be closed, and the outer gate might be in danger of failure due to the unaccounted head difference, the gates have to be closed only once. Thus, the closure failure in this fault tree does not concern the yearly probability, but the the probability of failure at a single attempt. This probability is, as explained in Section C.7, a value which is specifically acquired per navigation lock, and therefore needs no further derivation.

When the policy is applied to open the inner gate during extreme conditions, the outer gate is subjected to the entire head difference regardless of a closure failure. This provides a much simpler fault tree, as presented in Figure B.4.

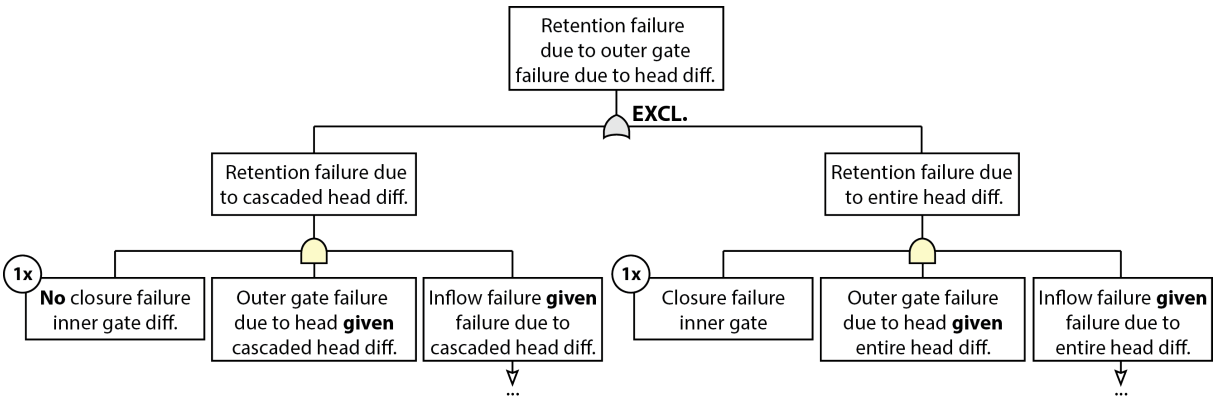


Figure B.3: Fault tree for "Failure due to head difference" for the policy to close both gates during extreme conditions. "(No) closure failure inner gate" only concerns the success or failure at a single attempt to close the gate. Down pointing arrows indicate the fault tree can be expanded.

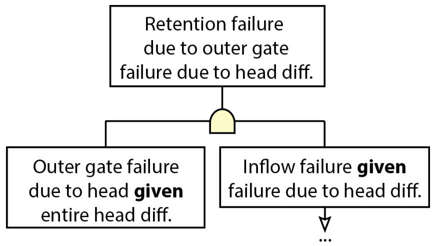


Figure B.4: Fault tree for "Failure due to head difference" for the policy to open the inner gate during extreme conditions. Down pointing arrows indicate the fault tree can be expanded.

B.3. Failure due to vibration

The fault tree for "Failure due to vibration" is very similar to the previous one. The limit state function of this mechanism is a simple critical value of the distributed discharge. Thus, in case the discharge is too high for the outer gate, the inner gate will sequentially fail.

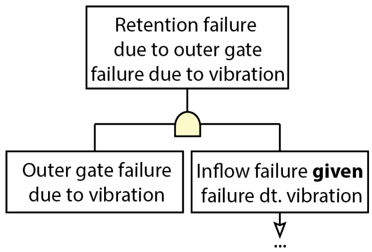


Figure B.5: Fault tree for "Failure due to vibration". Down pointing arrows indicate the fault tree can be expanded.

B.4. Failure due to collision

In case only "Failure due to collision" occurs at the outer gate, overflow can be initiated over the inner gate in case this is shorter than the outer gate. The mechanism is only expected to occur simultaneously with "Closure failure". The other failure mechanisms are related to extreme conditions, in which vessels are assumed not to enter the lock. The closure failure probability that needs to be implemented is again not the yearly probability of occurrence, but the failure probability at a single instance. The gate only needs to be closed once in case a vessel collision has occurred at the opposing gate.

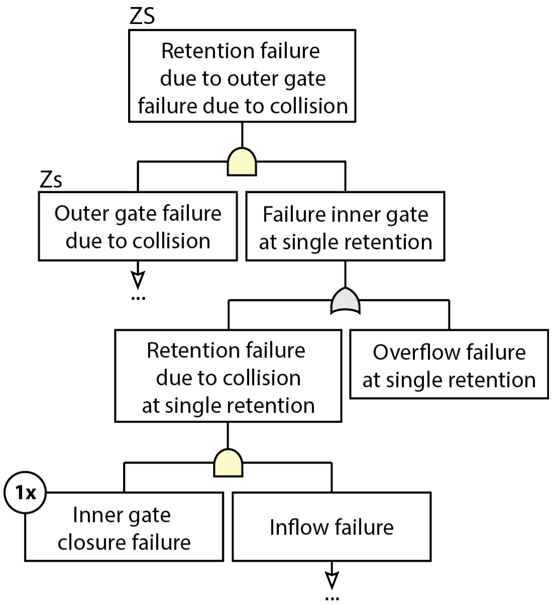


Figure B.6: Fault tree for "Failure due to collision". "Inner gate closure failure" only concerns the success or failure at a single attempt to close the gate. Down pointing arrows indicate the fault tree can be expanded.

B.5. Overflow and inflow failure

Overflow can occur regardless of any gate failure, but its probability increases in case the outer gate has failed, and the inner gate has a smaller retention height. Inflow occurs in case both the inner and outer gates have failed.

As a result of water flowing over or through a lock, the bed protection can erode either by the flow velocity or the pulse of a spilling jet, or the storage capacity of the water system can be surpassed due to the increase in water volume. The overflow and inflow discharge are implemented in the derivation of the probabilities of the sub-mechanisms, and are therefore inherently correlated.

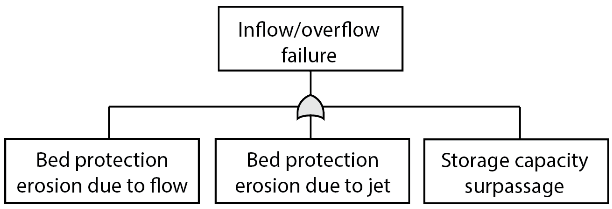


Figure B.7: Fault tree for "Overflow/inflow failure".

C

Limit state functions of failure mechanisms

Section 4.2 has provided a general overview of the failure mechanisms of a marine lock, and depicts how the occurrence of one can influence the probability of another. This Appendix expands on that knowledge by presenting how each mechanism can be calculated using the dimensions and other properties of a lock. Each failure mechanism is separately explored throughout the following sections.

C.1. Inflow, overflow and overtopping

Three mechanisms can cause water to enter the lock chamber, or the water system behind it, i.e. free flow, overflow and overtopping. Inflow occurs in case the lock is completely open, and no object (vessel, gate or temporary barrier) obstructs the water from flowing freely through the structure. Overflow and overtopping take place in case a gates are closed, and water can only pass the structure at its top. These phenomena establish varying discharges, and have to be analysed separately (Rijkswaterstaat, 2019b).

Inflow

The velocity can be approximated by regarding the lock as a submerged broad crested weir, i.e. a weir for which the lowest water level is located above the crest of the weir. The distributed discharge is determined using Equation C.1 (Rijkswaterstaat, 2019b).

$$q_{inflow} = m_{sub} \frac{\sqrt{2g(h_1 - h_2)}}{0.9} h_2 \tag{C.1}$$

Where:

Symbol	Description	Unit	Distribution	Value	
q_{inflow}	Inflow distributed discharge	$m^3/s/m$	Deterministic	Output	
m_{sub}	Flow model factor	-	Normal*	μ=1	σ=0.1
g	Gravitational acceleration	m/s	Deterministic	9.81	
h_1	High water level	m	Conditional	Input	
h_2	Low water level	m	Conditional	Input	

* Retrieved from Rijkswaterstaat (2019b).

Overtopping

In case the lock is not open, i.e. one or two gates are closed, overflow and overtopping can occur. Overflow is initiated when the water level is higher than the gate height, whereas this does not have to be the case for wave overtopping. Thus, two separate situations can be regarded, assuming waves are always present: only overtopping and overtopping combined with overflow. In the former situation, Equation C.2 is applied

(Rijkswaterstaat, 2019b). Mind that overtopped water will initially be caught in the lock chamber, and will only enter the water system in case one of the gates has failed its retention function.

$$q_{ot} = m_{ot} \cdot \sqrt{gH^3} \cdot e^{\left(-3.0\frac{h_{cr}-h_1}{H}/(\gamma_\beta \gamma_n)\right)} \tag{C.2}$$

Where:

Symbol	Description	Unit	Distribution	Value	
q_{ot}	Overtopping distributed discharge	$m^3/s/m$	Deterministic	Output	
m_{ot}	Overtopping model factor	-	Lognormal*	μ=0.09	σ=0.06
H	Wave height	m	Conditional	Input	
h_{cr}	Crest height	m	Normal*	μ=Input	σ=0.05
γ_β	Incidence factor	-	Deterministic	Input	
γ_n	Nose construction factor	-	Deterministic	1	

* Retrieved from Rijkswaterstaat (2019b).

Nose construction factor γ_n is equal to 1 for all navigation locks, as nose constructions are not applied to lock gates. Incidence factor γ_β does have to be applied. It is dependent on the angle at which waves collide with the structure, β, as presented in Equations C.3 and C.4.

$$0 \leq \beta \leq 20 \rightarrow \gamma_\beta = 0 \tag{C.3}$$

$$20 < \beta \leq 180 \rightarrow \gamma_\beta = max\left(cos(\beta - 20)\ ;\ 0.7\right) \tag{C.4}$$

Overflow and overtopping
In case of both overflow and overtopping, the discharges provided by both processes are added upon one another. The overtopping behaviour does change, as waves no longer endure much obstruction from the gate. Therefore, the overtopping Equation also changes to a simpler form (Rijkswaterstaat, 2019b).

$$q_{of+ot} = m_{of} \cdot 0.55\sqrt{-g(h_{cr} - h_1)^3} + m_{ot}\sqrt{gH^3} \tag{C.5}$$

Where:

Symbol	Description	Unit	Distribution	Value	
q_{of+ot}	Sum of distributed discharges	$m^3/s/m$	Deterministic	Output	
m_{ot}	Overflow model factor	-	Normal*	μ=1.1	σ=0.05

* Retrieved from Rijkswaterstaat (2019b).

C.2. Failure storage capacity

If water passes a navigation lock unintentionally, this does not directly lead to flooding of the hinterland. Only when the storage capacity of the water system is not sufficiently large enough, will water flow over the adjacent dykes. Thus, the limit state function consists of the available storage capacity volume, and the volume of water discharged over the lock, represented by the first and second term in Equation C.7 respectively (Rijkswaterstaat, 2019b).

$$Z = V_{stor} - V_{ot/of} \tag{C.6}$$

Storage capacity

The available storage capacity is a function of the storage area A_{stor}, storage height Δh_{stor}, and model factor m_{stor}.

$$V_{stor} = m_{stor} A_{stor} \Delta h_{stor} \tag{C.7}$$

Where:

Symbol	Description	Unit	Distribution	Value	
m_{stor}	Storage volume model factor	-	Lognormal*	μ=1	σ=0.20
A_{stor}	Storage area	m^2	Lognormal*	μ=Input	V_R=0.10
Δh_{stor}	Storage height	m	Lognormal*	μ=Input	σ=0.10

* Retrieved from Rijkswaterstaat (2019b).

Depending on the water system, the storage area can be hard to define. For a canal with constant profile and a regulated water level, the parameter is rather constant, and simple to derive. For a river, however, the inconsistent bathymetry and shape of a river make it much harder to estimate the available volume, let alone the inclination determining how far the water will flow into the system. The storage area is additionally dependent on the water level, which is constantly changing. Determining this parameter must be done with care, and should rather be done conservatively (Rijkswaterstaat, 2019b).

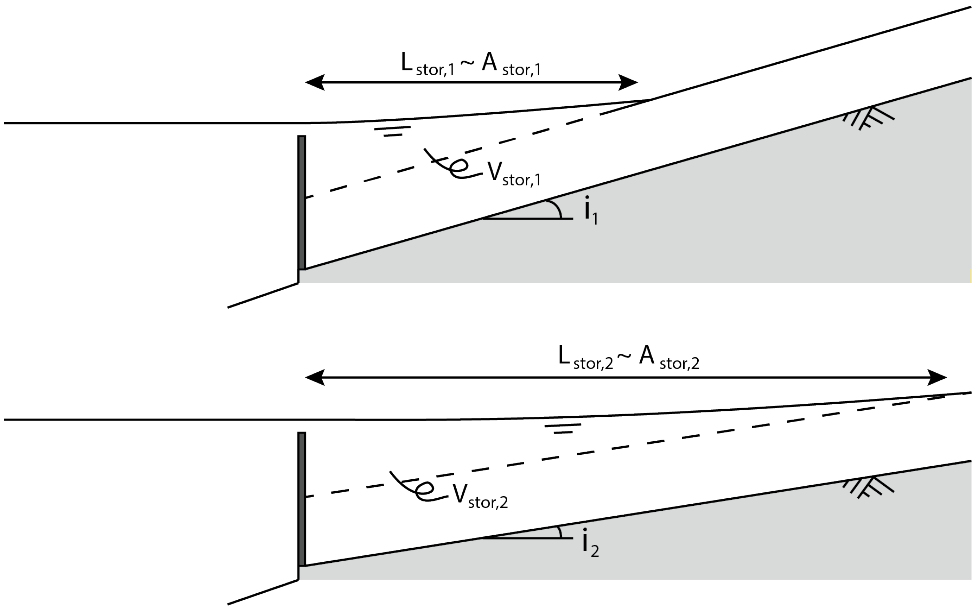


Figure C.1: Depiction of dependency storage area on river inclination

Just as the storage area, ease with which the storage height can be derived is dependent on the system. The storage height is the differnence between the water level and the crest of the dyke. For a canal or river with an insignificant inclination, this parameter can be assumed to be constant over the length of the channel. However, as can be seen in Figure C.1, in case of a stronger inclination, this difference increases over the length when inflow is occurring (Rijkswaterstaat, 2019b).

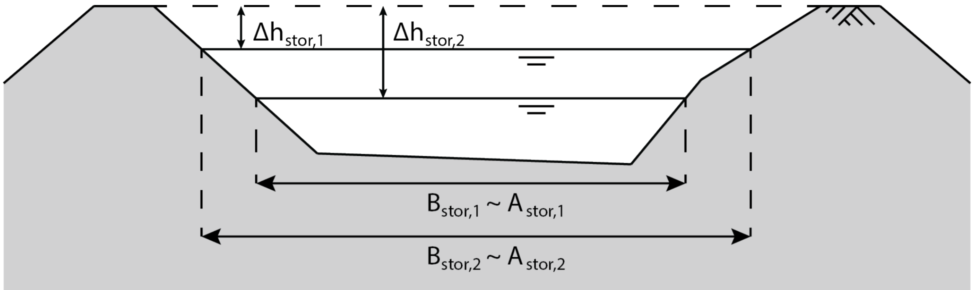


Figure C.2: Depiction of dependency storage area and storage height on water level

Inflow volume
The storage capacity of a water system is only insufficient if it cannot store the inflowing water volume. The inflowing volume is regarded which will occur during a storm (Rijkswaterstaat, 2019b), and can derived using Equation C.8.

$$V_{ot/of} = m_{in} t_s q_{ot/of} B_{chamber} \tag{C.8}$$

Where:

Symbol	Description	Unit	Distribution	Value	
m_{in}	Inflow model factor	-	Deterministic*	1	
t_s	Storm duration	h	Lognormal*	μ=6	V_R=0.25

* Retrieved from Rijkswaterstaat (2019b).

As this failure mechanism regards a volume, apart from the distributed discharge $q_{ot/of}$ and lock chamber width B, a time element is required. This is present in Equation C.7 as the storm duration t_s. This parameter is dependent on many natural phenomena and is therefore not easily defined. In absence of relevant data, a lognormal distribution with a mean of 6 hours is advised as a conservative assumption (Van Bree, Delhez, Jongejan, & Castelijn, 2018).

It can be expected that at the time when the sea level has risen close to the actual height of the gate, inflow can last much longer. It can at some point even become regular that overflow occurs. The assumption will however be made that at this point the norm for the failure probability must have been exceeded before. This has to be checked. Mind that the frequency of storm occurrence does increase due to the climate change driven conditions, as well as its intensity.

At last, the model factor m_{in} accounts for the uncertainty of inflow. It is however fixed at a deterministic value of 1, as enough uncertainty is regarded in the other factors of this limit state function (Rijkswaterstaat, 2019b).

C.3. Failure bed protection

The bottom protection regularly consists of multiple layers of differently graded rocks, and possibly a filter layer (Schierink & Verhagen, 2016). Without these layers, sediment would be lifted up by an occurring flow, creating a scour hole and jeopardising the stability of the soil. By applying a correct grading in the bed protection, the particles can no longer be lifted through the different layers, and no scour hole can develop beneath the protection. The size of the grains at the top layer should be selected such that no expected flow velocities can cause (significant) transportation of these grains. An important note is that the bed protection does not prevent a scour hole from forming at all, as it is still likely to develop at the toe of the protection. The goal of the bed protection is merely to assure that the scour hole will develop at a long enough distance from the structure to maintain its stability.

The bed protection will be loaded in two manners: by a regular water flow parallel to the bed, and by a jet caused by spilling water. Both have to be analysed separately, but share common grounds. For both loading conditions, it has to be derived whether the occurring discharge becomes larger than the critical discharge for which the bed protection is regarded as insufficient. For both derivations, a value is required which represents the variety of grain sizes which are found inside the top layer. As a bed protection often consists of natural rocks, the individual grains are bound to vary in shape and size. The distribution is often represented by either D_{50} or D_{n50}. The former presents the grain diameter for which 50% of the grains in a mixture are smaller. This can be derived by sieving a portion of the mixture. However, in case large rocks are to be used, sieving is rather impossible. Instead, rocks are weighed individually, from which the nominal diameter D_{n50} can be derived. This presents the grain diameter for which 50% of the grains are lighter.

As one is related to the size, and the other to the weight of the grains, D_{50} or D_{n50} are not interchangeable. They are however related, and can be derived from one another using the approximation presented in Equation C.9. It is assumed both have a normal distribution and a standard deviation of 5%.

$$D_{n50} \approx 0.84 D_{50} \tag{C.9}$$

Where:

Symbol	Description	Unit	Distribution	Value	
D_{50}	Median grain size	m	Normal*	μ=Input	V_R=0.05
D_{n50}	Nominal diameter	m	Normal*	μ=Input	V_R=0.05

* Self-made assumption.

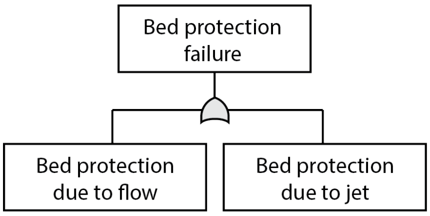


Figure C.3: Fault tree for "Bed protection failure"

Failure due to flow
Bed protection failure occurs in case the discharge above the protection becomes larger than the critical discharge of the system, i.e. the discharge at which (significant) transportation of the rocks follows (Rijkswaterstaat, 2019b). For a flow parallel to the bed, the occurring discharge is derived from the specific overtopping and overflow multiplied by the width of the lock chamber. The critical discharge however, is derived using not the width of the lock chamber, but B_{sv}, i.e. the width of the flow pattern at a specified distance from the lock. This is multiplied by the critical velocity and the river water level H_2.

$$Z = Q_c - Q_{ot/of} = u_c \cdot h_2 \cdot B_{sv} - q_{ot/of} \cdot B_{chamber} \tag{C.10}$$

To determine the critical velocity of a bed protection, the Pilarczyk formula is used, as presented in Equation C.11. An important parameter in this equation is the Critical Shield's parameter ψ_{cr}. This determines the threshold of motion which is allowed. For a conservative value of 0.035, no movement of any grains is permitted. A more suitable value is however in the order of 0.05, allowing some movement of individual rocks, but an overall steady bed protection. The events for which the sufficiency of the bed protection is regarded, such as overflow and structural damage, do not occur on a regular basis. It is therefore unnecessary to demand a complete lack of movement.

$$u_c = \sqrt{\frac{2g\Delta D_{n50}\psi_{cr}k_{sl}}{\phi_{sc}0.035k_h k_t^2}} \tag{C.11}$$

For which:

Δ	=	Relative density	$\Delta = (\rho_s - \rho_w)/\rho_w$ [-]
k_{sl}	=	Slope factor	1 for lack of slope
ϕ_{sc}	=	Stability parameter	0.75 [-]
k_h	=	Depth parameter	$k_h = (1 + \Delta h/D)^{-0.2}$ [-]
k_t	=	Turbulence factor	Conservatively: $k_t^2 = 2$ [-]

Where:

Symbol	Description	Unit	Distribution	Value	
ρ_s	Rock density	kg/m^3	Normal*	μ=Input	σ=0.05

* Self-made assumption.

The width of the flow pattern B_{sv} is determined making the assumption that the flow develops under an angle of 1/5. A distance x should be specified at which the bed protection is to be assessed. At least the bed protection immediately behind the lock should be checked, as the distributed discharge is largest here. Whether any other distances should be assessed depends on the composition of the bed protection. In case a transition between grain distributions is present at a certain distance, the possibility of transport has to be analysed here as well.

$$B_{sv} = min(B_{chamber} + 2 \cdot 0.2 \cdot x \ ; \ B_{river}) \tag{C.12}$$

Where:

Symbol	Description	Unit	Distribution	Value	
$B_{chamber}$	Flow width	m	Lognormal*	μ=Output	σ=0.05
$B_{chamber}$	Lock chamber width	m	Normal*	μ=Input	σ=0.05
B_{river}	River width	m	Normal*	μ=Input	σ=0.1

* Retrieved from Rijkswaterstaat (2019b).

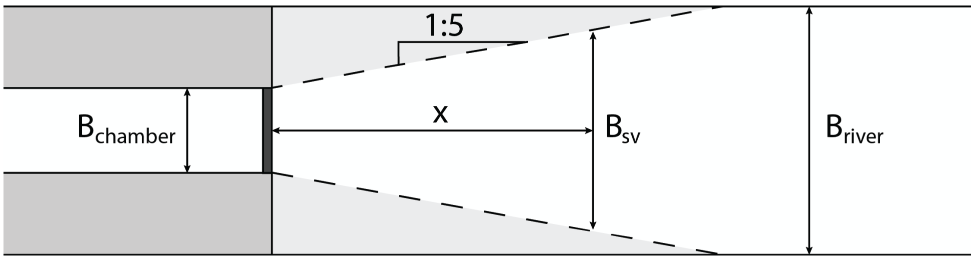


Figure C.4: Flow development behind lock

Failure due to jet

The other manner in which bed protection can be damaged is by the jet caused by spilling water (Rijkswaterstaat, 2019b). This is accompanied by a completely different limit state function, and even a different critical specific discharge. The limit state is derived from the allowable size of the scour hole that will develop, y_s. This is in continuity with the assessment of complex IJmuiden conservatively fixed at 0 m, i.e. a complete lack of scour (Van den Berg et al., 2019).

$$y_s = 0.4 \cdot q_c^{0.6} \Delta H^{0.4} D_{50}^{-0.3} - 0.5 H_2 \tag{C.13}$$

$$Z = \frac{y_s + 0.5H_2}{D_{50}^{-0.3}} - 0.4 \cdot q_c^{0.6} \Delta H^{0.4} \tag{C.14}$$

What is left to determine for this limit state function is the critical specific discharge, q_c. This is simply a function of the overflow height h_0, as presented in Equation C.15.

$$q_c = 1.705 \sqrt{h_0^3} \tag{C.15}$$

Where:

Symbol	Description	Unit	Distribution	Value	
q_c	Critical distributed discharge	$m^3/s/m$	Lognormal*	Input	V_R=0.15

* Retrieved from Rijkswaterstaat (2019b).

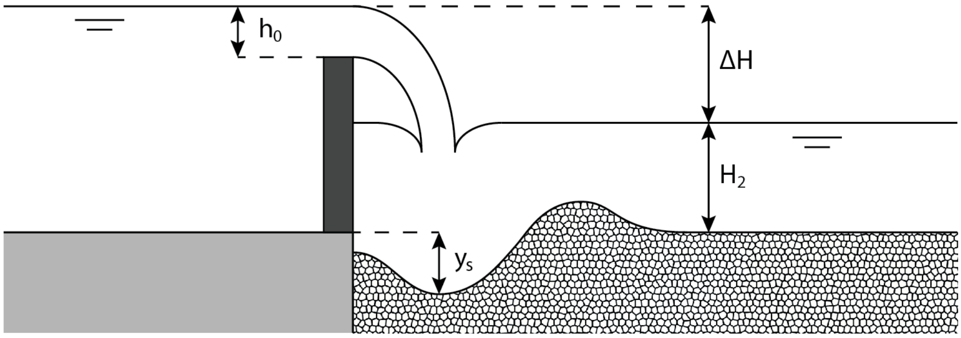


Figure C.5: Scour hole created by spilling

Failure instability due to erosion
The loss of integrity of the bed protection does not necessarily immediately lead to loss of stability of the subsoil or structure. As everything, the loss of stability is a probability. Unlikely the mechanisms described in the sections before, this failure is given by a fixed probability in the WBI (although the fixed value may vary for different structures): $P_{f,\text{stability loss}}$

Conservatively, this probability is assumed to be equal to 1. There is a number of factors which can influence this assumption:

- Complete surrounding of the foundation by sheet piles (which could be present due to certain construction methods), will realistically provide a $P_{f,\text{stability loss}}$ of 0.01.
- Based on knowledge of the present scour holes, a $P_{f,\text{stability loss}}$ between 1 and 0.01 can be expected, whereas shallower holes lead to a lower probability.
- In case of a relatively long sheet pile wall at the border of concrete and bottom protection, perpendicular to the structure, loss of stability is less likely to occur, providing a $P_{f,\text{stability loss}}$ between 1 and 0.01.

C.4. Failure due to head

To determine the load and resistance of a structure, two methods are described in the WBI manual, i.e. a linear or quadratic model. The former regards a distributed load over the height of the gate, whereas the latter considers the integral of this distributed load as a single load. Before assessing what method is preferable, it must be understood what loads act on the gate, how they are distributed, and how different loads compare to one another.

Load distributions over gate height
The hydraulic load regularly consists of a hydrostatic pressure and a wave pressure. Other forces (like those related to collisions) may additionally act on the gates, but are disregarded for this particular limit state, as it solely concerns failure caused by the head.

The gates of a lock are subjected to a hydrostatic pressure from two sides. For the outer gate this concerns the sea level and the water level in the lock chamber. For the inner gate this concerns the latter and the river level. The pressures from opposing sides counter-act one another. Per side, the hydrostatic pressure is determined by Equation C.16. As is presented in Figure C.6, the similar increase in hydrostatic pressure on both sides of the gates results in a constant pressure below the water level on the downstream end.

$$p_h = \rho_w \cdot g \cdot d \tag{C.16}$$

Where:

Symbol	Description	Unit	Distribution	Value	
ρ_w	Water density	kg/m^3	Normal*	Sea: μ=1.025	σ=0.005
				River: μ=1.0	σ=0.005

* Self-made assumption.

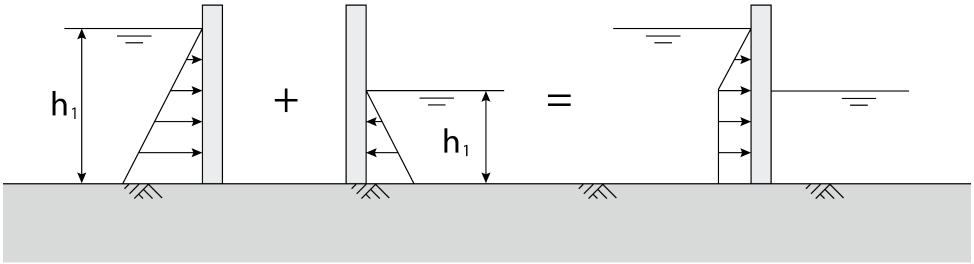


Figure C.6: Superposition of hydrostatic pressures

Although the determination of the hydrostatic pressure is rather simplistic, the wave pressure is more complex. Many methods have been derived describing the derivation of the wave pressure, from which the Sainflou and Goda-Takahashi model are currently most accepted (TAW, 2003). The most prominent difference between these methods is that Goda-Takahashi takes wave breaking into account, whereas Sainflou does not. The Sainflou model requires less input and computation, but provides a conservative output. Wave breaking is dependent on the relation between the wave height and water depth, and will therefore be relevant for some locks, and irrelevant for others. Thus, to create a model which functions regardless of the input parameters, the Goda-Takahashi model is implemented.

The Goda-Takahashi model has been derived using data of wave pressures on a vertical breakwater with a berm at its toe, but is applicable for any other vertical structure, regardless of the presence of a berm. The horizontal pressure is distributed as presented in Figure C.7, and is characterised by pressures p_1, p_3, and p_4 in the case of overtopping (TAW, 2003).

$$p_1 = 0.5 \cdot (1 + cos(\beta)) \cdot (\lambda_1 \alpha_1 + \lambda_2 \alpha_2 cos^2(\beta)) \rho \cdot g H_d \tag{C.17}$$

$$p_3 = \alpha_3 p_1 \tag{C.18}$$

$$p_4 = \alpha_4 p_1 \tag{C.19}$$

For which:

β	$=$	Angle of incidence	
η^*	$=$	Reflected wave amplitude	$\eta^* = 0.75(1 + cos(\beta))\lambda_1 H_d$
α_1	$=$	Coefficient for wave frequency	$\alpha_1 = 0.6 + 0.5\left(\frac{4\pi h/L_d}{sinh(4\pi h/L_d)}\right)^2$
α_2	$=$	Coefficient for impulse load of wave	$\alpha_2 = min\left(\frac{(1 - D/h_b)(H_d/D)^2}{3}, \frac{2D}{H_d}\right)$
α_3	$=$	Coefficient for pressure reduction over depth	$\alpha_3 = 1 - (h'/h)\left(1 - \frac{1}{cosh(2\pi h/L_d)}\right)$
α_4	$=$	Coefficient for overtopping	$\alpha_4 = 1 - h_c^*/\eta^*$
h_c^*	$=$	Overtopping relevant height	$min(\eta^*; h_c)$
h_b	$=$	Water depth at distance 5D from the wall	
H_d	$=$	Wave height	
L_d	$=$	Wave lenght	
D	$=$	Water depth above sill	
h'	$=$	Water depth above foundation plane	
h	$=$	Water depth in front of sill	
h_c	$=$	Difference between water level and top wall	
$\lambda_{1,2,3}$	$=$	Modification factors dependent on geometry	Equal to 1 for vertical walls and non-breaking waves; for breaking waves λ_2 alters as described below

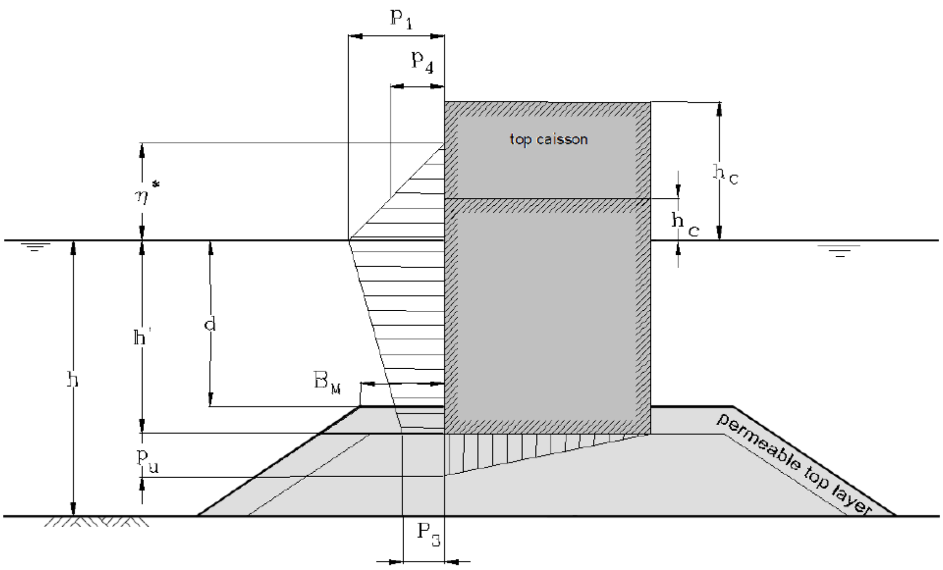


Figure C.7: Depiction of wave pressure distribution by the Goda method. Retrieved from Molenaar and Voorendt (2018).

In case a sill is present in front of a structure, this can cause the wave to break. This is accounted for in the impulse coefficient α_2 and modification factor λ_2. In the absence of a sill, these factors are equal to 0 and 1 respectively, cancelling out the impulse term in Equation C.17. λ_2 is determined using Equation C.20. Note that this will be equal to 1 in the absence of a berm or in case of a non-breaking wave.

$$\lambda_2 = max\left(1; \frac{\alpha_l}{\alpha_2}\right) \tag{C.20}$$

For which:

$$
\begin{aligned}
\alpha_{\mathrm{l}} &= \alpha_n \alpha_m \\
\alpha_{\mathrm{m}} &= min\left(\frac{H_d}{D};2\right) \\
\alpha_{\mathrm{n}} &= \frac{cos(\delta_2)}{cosh(\delta_1)} && \text{if } \delta_2 \leq 0 \\
\alpha_{\mathrm{n}} &= \frac{1}{cos(\delta_1)\sqrt{cosh(\delta_2}} && \text{if } \delta_2 > 0 \\
\delta_1 &= 20\delta_{11} && \text{if } \delta_{11} \leq 0 \\
\delta_1 &= 15\delta_{11} && \text{if } \delta_{11} > 0 \\
\delta_2 &= 4.9\delta_{22} && \text{if } \delta_{22} \leq 0 \\
\delta_2 &= 3.0\delta_{22} && \text{if } \delta_{22} > 0 \\
\delta_{11} &= 0.93\left(\frac{B_m}{L_d} - 0.12\right) + 0.36\left(\frac{h-D}{h} - 0.6\right) \\
\delta_{22} &= -0.36\left(\frac{B_m}{L_d} - 0.12\right) + 0.93\left(\frac{h-D}{h} - 0.6\right) \\
\mathrm{B_m} &= \text{Width of berm in front of wall}
\end{aligned}
$$

Model derivation
Understanding how the hydrostatic- and wave pressure act on the gate, a model can be derived of to determine the strength of the gates. As mentioned, the WBI manual provides two options: a linear model in which the strength and load are regarded as continuous variables over the gate height, and a quadratic model in which the strength and load are regarded as a single parameter, i.e. the integral of the continuous variable. Both approaches are however flawed.

The quadratic method can result in a very optimistic strength calculation, as a high pressure at a certain height of the gate can be cancelled out by low pressures at a different height. The method regards the gate as a single component, which it is obviously not. The linear model on the other hand can provide a pessimistic strength. The components of the gate retain a the pressure subjected to a certain section of the structure, meaning the maximum pressure on the gate should to some extent be moderated by its neighboring pressures. Thus, a hybrid model is applied, in which the gate is segregated into sections, for which the integral of the continuous pressure is taken over the height of those sections. This is illustrated on the right in Figure C.8.

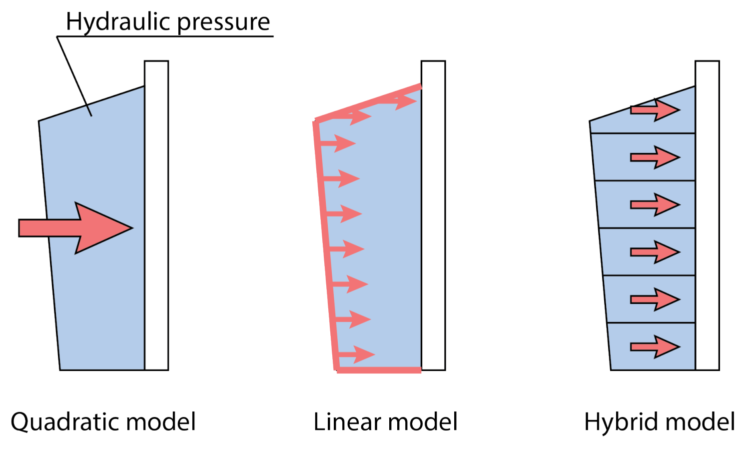


Figure C.8: Gate resistance model depictions

By means of this hybrid model, the force applied on each segment can be derived using the equations for the hydrostatic pressure and wave pressure. To decrease the amount of calculations that need to be performed, only the critical segment will be examined, i.e. the segment subjected to the largest load. Segments with a height of 1.0 m are considered.

What segment will be critical will alter as the loads on the gate change. Theoretically it can be expected that the hydraulic pressure is largest at the downstream water level. The hydrostatic pressure reaches a maximum at this point, whereas the wave pressure decreases further to the bottom. This is confirmed by the graphs provided in Figures C.9 and C.10, where the hydraulic pressure distribution over the gate height of a hypothetical lock arrangement is given for a varying upstream water level and wave height.

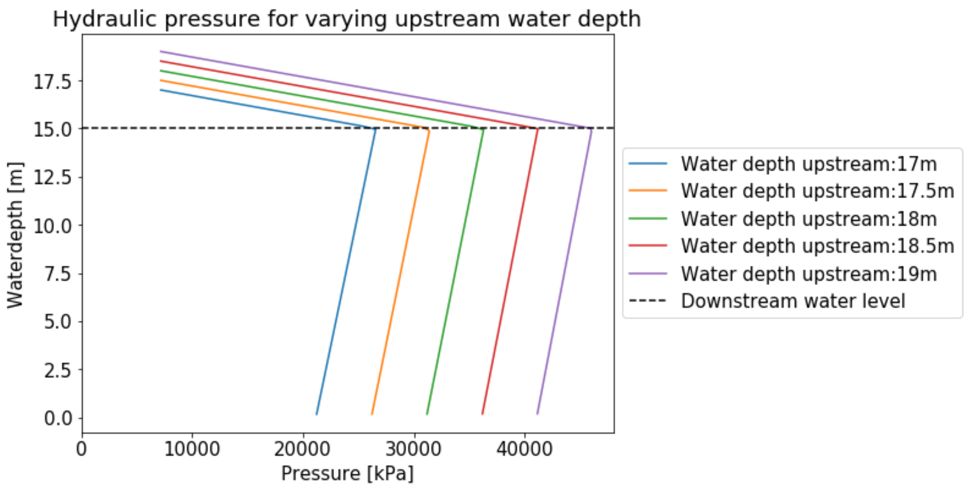


Figure C.9: Hydraulic pressure distribution over gate height for a varying water depth

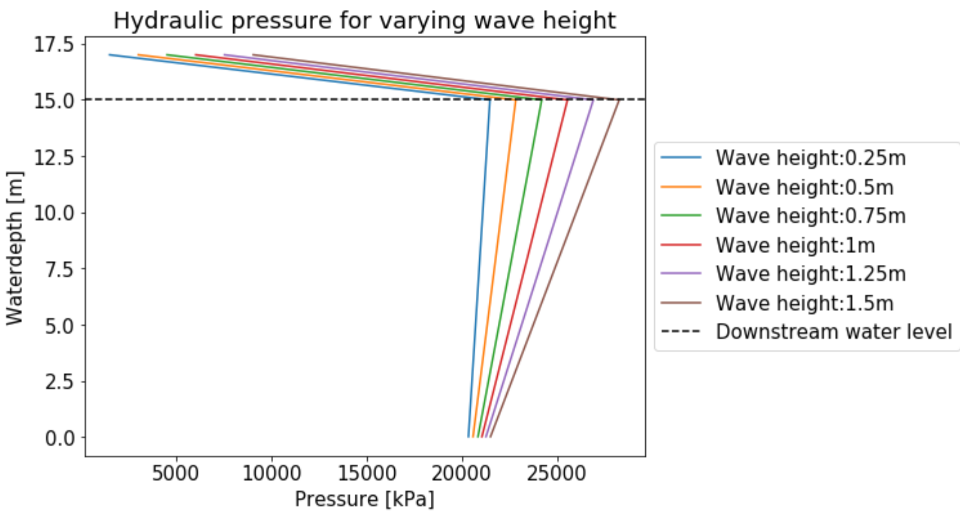


Figure C.10: Hydraulic pressure distribution over gate height for a varying wave height

Thus, the largest hydraulic pressure is located at the height of the downstream water level. However, what can also be derived from the graphs in Figures C.9 and C.10 is that the pressure decreases quicker above this waterline than it does below. Therefore, it can not be assumed that the segment at which the downstream water level is located is necessarily the segment which is subjected to the largest total pressure. If the waterline is located at the upper region of the segment (as on the left in Figure C.11), the segment does bear the largest load. However, if the waterline is located at the lower region of the segment (as on the right in Figure C.11, the segment below it will bear a larger load.

To simplify the problem, it is assumed that in case the downstream water level is located between the middle of a segment and the middle of the segment above it, this segment is critical in terms of hydraulic pressures.

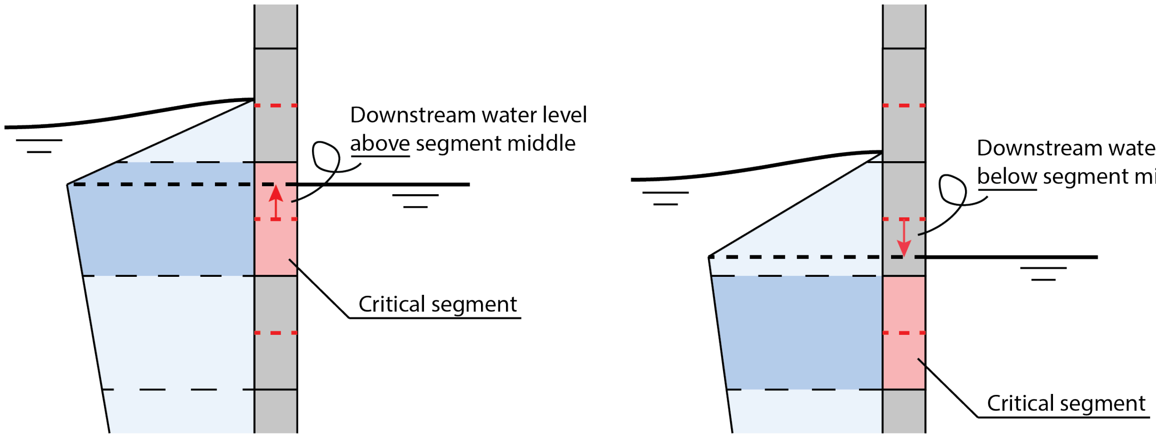


Figure C.11: Determination critical gate segment

Depending on whether the critical segment is located at or below the downstream water level, a different equation has to be used to determine the hydrostatic load on the segment. This can be seen in Figure C.12, in which the relevant hydrostatic- and wave pressures are illustrated. Equations C.21 and C.22 are used to calculate the hydrostatic load on a segment *at* the downstream water level and *below* the downstream water level respectively. The definition of the parameters is presented in Figure C.12.

$$P_h = \frac{\Delta h_1 + \Delta H}{2} \cdot \rho_w g \Delta h_2 + \Delta H \rho_w g \cdot (h_s - \Delta h_2) \tag{C.21}$$

$$P_h = \Delta H \rho_w g \cdot h_s \tag{C.22}$$

The wave pressure can be derived using the same equation, regardless of the location of the critical segment relative to the downstream water level:

$$P_w = \left(\frac{(N - 0.5) \cdot h_s}{h_1} \cdot (p_1 - p_3) + p_3 \right) \cdot h_s \tag{C.23}$$

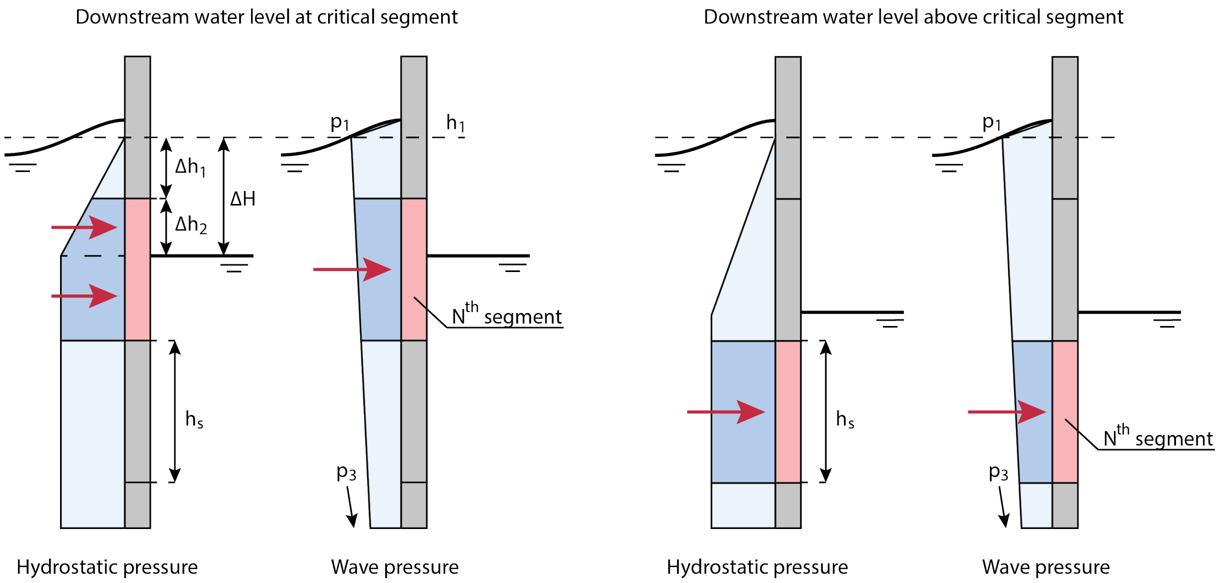


Figure C.12: Hydrostatic- and wave pressure on critical segment

Sequentially the total load on the critical segment can be derived by added the hydrostatic- and wave load, concluding the hybrid model. Both the load and resistance of a gate can be acquired using these calculations, as will be explored in the following section.

Relation load and strength

The limit state of "Failure due to head" is presented in Equation C.24, in which $S_{segment}$ can be directly derived using the hybrid model. For the strength of the gate segment $R_{segment}$, a number of additional steps are required.

$$Z = R_{segment} - m_s \cdot S_{segment} \tag{C.24}$$

Where:

Symbol	Description	Unit	Distribution	Value	
m_s	Load distribution model factor	-	Normal*	μ=1	σ=0.05

* Retrieved from Rijkswaterstaat (2019c).

The strength of the gate is derived using the design water levels and design wave height. A structure is designed such that its design strength is equal to the design load. This is described in Equation C.25. The subscript "old" is used to refer to the time of design. $S_{d,old}$ will be derived with the design conditions, but only for the critical segment according to the current environmental conditions, as this is the segment for which the current load is critical.

$$R_{d,old} = S_{d,old} \tag{C.25}$$

From the design value, the characteristic value can be derived using the design factor $\gamma_{S,old}$ and material factor $\gamma_{m,old}$. The former is used to account for uncertainties in the determination of load cases, whereas the latter regards the inaccuracy of material production. For pressure differences caused by water levels and waves, a $\gamma_{S,old}$ of 1.25 is used (TAW, 2003). For a steel or timber structure, a $\gamma_{m,old}$ of respectively 1.0 and 1.3 would have been used in case the lock was constructed after 1990 (NEN-EN 1995-2, 2005).

$$R_{k,old} = S_{k,old} \cdot \gamma_{S,old} \cdot \gamma_{m,old} \tag{C.26}$$

Before the year 1990, however, no use was made of two separate partial factors. Instead, a single factor was used, based on the material type. For steel, this factor was equal to 1.5 (Rijkswaterstaat, 2019c). In the WBI it is stated that it is unknown what this factor was for timber structures, and advised to use the current partial factors in combination with a reduction factor for degeneration (presented in Equation C.28).

Table C.1: Partial factors after and before 1990. Retrieved from Rijkswaterstaat (2019b).

Material	$\gamma_S \cdot \gamma_m$ after 1990	$\gamma_S \cdot \gamma_m$ before 1990
Steel	$1.25 \cdot 1.0 = 1.25$	1.5
Concrete	$1.25 \cdot 1.5 = 1.875$	1.7
Timber	$1.25 \cdot 1.3 = 1.625$	1.625

Thus the characteristic value of the resistance is determined from the design load. The significant parameter for the strength of the entire structure is however the mean resistance. This is determined using the variance coefficient of the construction's material, V_R, as presented in Equation C.27. For a steel or wooden construction, V_R is equal to 0.1 or 0.25 respectively.

$$R_{m,old} = \frac{S_{k,old} \cdot \gamma_{S,old} \cdot \gamma_{m,old}}{1 - 1.64 \cdot V_R} \tag{C.27}$$

Finally a factor for the state of the structure can be applied, as its resistance may have decreased over the years due to creep and usage. This factor will usually be smaller than 1, but can even be larger in case the structure has been reinforced over the years. The final product is the mean current resistance, which serves as input for the limit state function of this failure mechanism. Because degradation of material develops differently in each specific case, this factor has to be determined per case by means of observational judgement.

This factor is especially important for timber structures, as deterioration of wood is harder to monitor. However, the Western lock in Terneuzen, which is regarded in this thesis, has steel gates. Therefore, this factor is assumed to be equal to 1.

$$R_{hybr} = R_{m,new} = \gamma_{\Delta R} \cdot R_{m,old} \tag{C.28}$$

Where:

Symbol	Description	Unit	Distribution	Value	
R_{hybr}	Hybrid resistance	kN/m	Logormal	μ=Input	Steel: V_R=0.10 Wood: V_R=0.25

C.5. Failure due to vibration

In case a constant flow of water occurs over one of the gates, the turbulent behaviour will cause the gate to vibrate at a certain frequency. Vibrations can cause damage to a structure in case the deflection per oscillation become sufficiently large. The current level of research towards determining at what flow velocities gates are likely to fail under their vibrating motion is limited. A general assumption is made that a distributed discharge of 1 $m^3/s/m$ can cause harmful deformations (Van den Berg et al., 2019). This statement provides the limit state function presented in Equation C.29.

$$Z = q_{overflow} - 1 \tag{C.29}$$

C.6. Failure closure after structural damage

The limit state functions presented in Equations C.24 and C.29 describe how the gates of a navigation lock can be damaged due to hydraulic loads. This does however not necessarily lead to failure of the retaining elements. There is a probability that the occurring flow can be blocked, for instance by using retaining elements such as sand bags. This probability has been fixed to a deterministic probability $P_{f,\text{damaged closure}}$ for a variety of structures (Rijkswaterstaat, 2019c).

For navigation locks, $P_{f,\text{damaged closure}}$ is equal to 1, as the dimensions of the flow channel are often large and flow velocities become too high to provide sufficient working conditions. Failure due to head or vibrations will thus always lead to failure of the retention function of a gate.

C.7. Closure failure

The closure of the gates is a process which relies on the performance on many different components, which can potentially all fail their tasks, resulting in an unintentionally opened gate. The probability of closure failure is regularly acquired from the empirically derived failure probabilities of these individual components, and not by means of a limit state function (Van den Berg et al., 2019). Sequentially, this failure probability, P_{ns}, is multiplied with the probability of a gate being open initially, P_{open}, and the number of openings (1 in case of a navigation lock), to find the failure probability of the closure process, $P_{f,\text{closure process}}$.

$$P_{f,closure\ process} = P_{ns} \cdot P_{open} \cdot n \tag{C.30}$$

In case of absence of data, P_{ns} can be assumed to be equal to 10^{-4} for a navigation lock (Van Bree et al., 2018). A conservative assumption is for each gate to be open for half of the time, resulting in a P_{open} of 0.5 (Van den Berg et al., 2019).

If a gate fails to close initially, there is still a probability of recovery. A broken component might be repaired in time, preventing large consequences to arise. Deriving this probability requires detailed knowledge on the individual components, accessibility of the lock, the cause of the initial closure failure and the environmental conditions at that time. In general, a conservative probability of 1 can be applied (Rijkswaterstaat, 2019a).

C.8. Failure due to collision

The gate of a lock can be damaged when a vessel collides with it. This does not necessarily lead to failure of the retention function of the structure. Three elements are of importance, as presented in the fault tree in Figure C.13. At first, obviously a vessel has to collide with the gate. Secondly, the energy of the collision has to be larger than the critical energy which is bearable by the gate. If both occur, the gate is damaged, but in some occasions, a gate can still be closed sufficiently after it has been damaged. Only if this is not the case, will water flow past the gate.

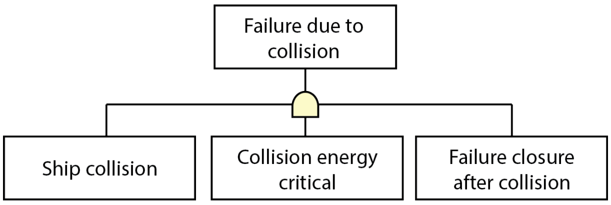


Figure C.13: Fault tree for "Failure due to collision"

Probability of ship collision
When a vessel enters a lock chamber, it has a probability of not decelerating quickly enough, and sailing into one of the gates. This impact load can obviously cause damage to the structure, and lead to flow of water into the system. The probability of a vessel collision is derived using data on the traffic through the lock, and is not captured in a limit state function.

The probability of collision failure is determined by the number of risky levelings which are expected to occur on a yearly basis, and the probability of collision per leveling (Rijkswaterstaat, 2019c). The term "risky" is explained below.

$$P_{f,collision} = f_{risky\ leveling} \cdot P_{collision\ at\ enter} \tag{C.31}$$

Not each occurring leveling is regarded as equally risky. A distinction is made between commercial- and recreational shipping. Vessels of the latter category are presumed not to be able to damage the gates sufficiently. Thus, in case a recreational vessel is sailing at the front into the lock chamber, this is not counted as a risky leveling.

A second distinction is made between inward and outward transport. Not only is inward transport related to the inner gate(s), and outward transport to the outer gate(s), but their frequency of occurrence also differs. The total number of yearly risky levelings can be derived using Equations C.32 and Equation C.33 for the inner gate and outer gate respectively.

$$f_{risky\ leveling\ inner} = f_{leveling} \cdot P_{commercial} \cdot P_{inward} \tag{C.32}$$

$$f_{risky\ leveling\ outer} = f_{leveling} \cdot P_{commercial} \cdot P_{outward} \tag{C.33}$$

For mitre gates, a last distinction is made between high tide and low tide, i.e. a sea level higher or lower than the river level. As for marine locks with mitre gates, both the inner and outer head are often equipped with two sets of gates (for high- and low tide). These gates are positioned at an angle from one another, which causes them to be either pushed closed by the pressure of the head, or pushed open, depending on which side of the gates has a higher water level. For most marine locks, the sea level can be either higher or lower than the river level, for which often two sets of gates are positioned at each lock head. One set retains water during high tide, while the other set is closed during low tide.

When a collision occurs during low tide, water flows from the river into the lock, which causes no harm. By the time the tide has turned, the high tide gates are closed, and no flooding will occur. There is however a probability that the collided vessel and damaged gates collectively form a clog, obstructing the high tide gates from closing, but not obstructing the water from flowing out of the lock (see Figure C.14). The probability of such a clog forming has been estimated to be 1/100 per collision, and is equal for inward and outward traffic.

If an outward sailing vessel collides with the gates during high tide, the gates will be pushed back by the head. It is likely that the gates and collided vessel will create a clog, and mostly prevent water from flowing into the system. Therefore only half of the outward levelings during high tide are considered to be risky.

Table C.2: Probability of a leveling being "risky" for mitre gates

	High tide	Low tide
Inward traffic	1	1/100
Outward traffic	1/2	1/100

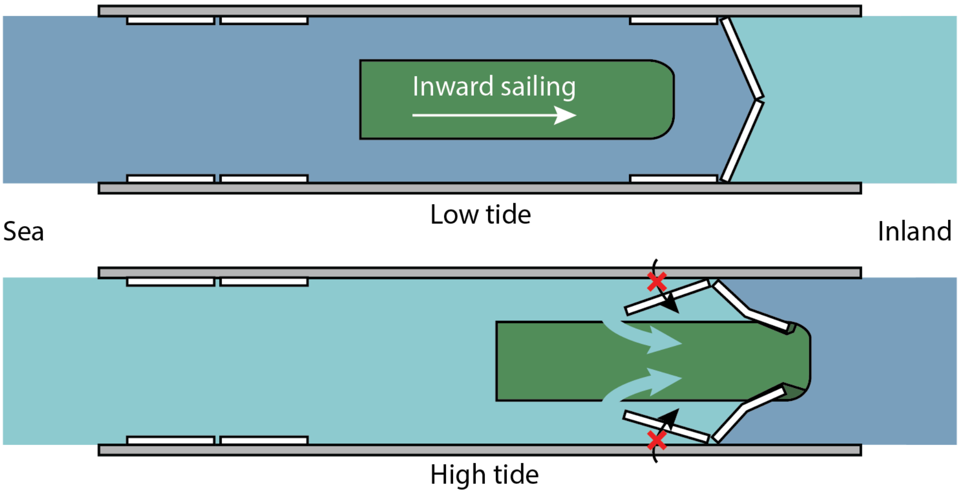


Figure C.14: TOP: Vessel sailing into the lock chamber at low tide. BOTTOM: Vessel has collided with low tide gates and got stuck, forming a clog. The clog prevents the high tide gates from closing once the tide has changed, but the clog does not suffice to obstruct water from flowing out of the lock. This has a probability of occurring of 1/100.

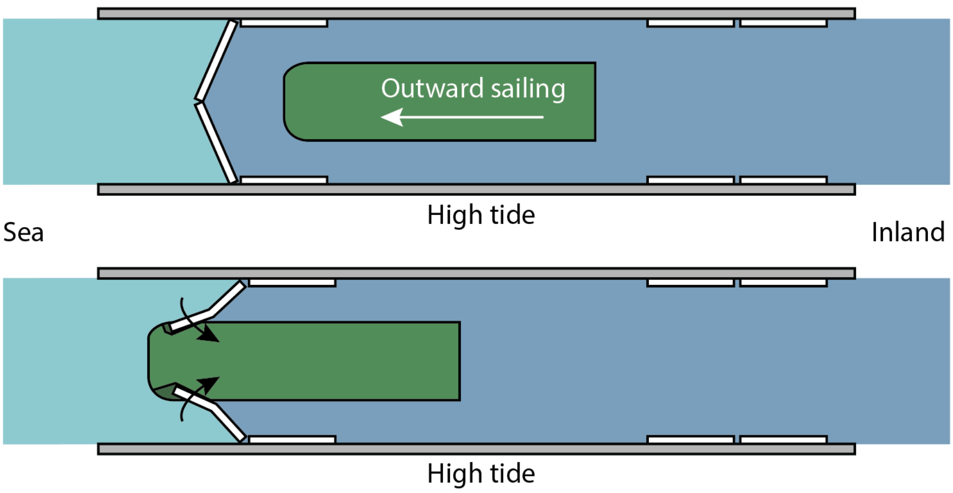


Figure C.15: TOP: Vessel sailing into the lock chamber at low tide. BOTTOM: Vessel has collided with high tide gates, which are being pushed into the vessel due to the head, forming a clog. This has a probability of occurring of 1/2.

Given the probabilities presented in Table C.2, the total yearly number of risky levelings for a lock with mitre gates can be drawn using Equations C.34 and C.35 for the inner gates and outer gates respectively.

$$f_{risky\ leveling\ inner} = f_{leveling} \cdot P_{commercial} \cdot (P_{inward} \cdot (P_{high} + 0.01 P_{low})) \tag{C.34}$$

$$f_{risky\ leveling\ outer} = f_{leveling} \cdot P_{commercial} \cdot (P_{outward} \cdot (0.5P_{high} + 0.01P_{low})) \tag{C.35}$$

This concludes the first term of Equation C.31. The second term is the probability of a vessel colliding with a gate when sailing into the lock chamber, $P_{\text{collision at enter}}$. This is estimated to be equal to 1/33.000 per leveling.

Collision energy critical
A collision is not evidently critical for the integrity of a lock gate. This is dependent on both the strength of the gate and the graveness of the collision (Rijkswaterstaat, 2019c). For the strength of the gate, specific information is required about the gate elements at the height of the collision. For the collision, information is required about the types of vessel that pass, their size, mass, and sailing velocity. To determine these factors for a non-existing navigation lock would lead to an ambiguous process. The other option is to take the value from a performed assessment. This also does not provide a sound estimation of the probability, but is a less dubious process and requires fewer assumptions. Thus, the available assessment of the IJmuiden lock complex is used as reference. Here, a probability of 1/100 was found for the exceedance of the critical collision energy (Van den Berg et al., 2019).

Closure failure after collision
Even after a fatal collision, there is still a possibility that the damaged gate can close. This is highly dependent on the flow velocity and the type of gate. All marine locks destined for commercial shipping are equipped with either rolling gates or mitre gates (see Section 2.3). Both types are very limited in motion under flow. Rolling gates are estimated not to be able to close under a flow velocity higher than 2 m/s, whereas the limit for mitre gates is estimated at 1 m/s.

The flow velocity can be approximated using theory of a submerged weir, as discussed in Section C.1. This is presented in the second term of the limit state function in Equation C.36.

$$Z = v_{c,close} - m_{sub}\frac{\sqrt{2g\Delta h}}{0.9} \tag{C.36}$$

Where:

Symbol	Description	Unit	Distribution	Value	
$v_{c,close}$	Critical closing velocity	m/s	Normal*	μ=1 or 2	V_R=0.2

* Retrieved from Rijkswaterstaat (2019c).

In addition, a section of the flow area might still be blocked by the gates and the vessel. Most likely the free area is defined by the width of the vessel and the height difference between the draught and bottom of the lock chamber (Rijkswaterstaat, 2019c). The critical velocity can then be converted by means of Equation C.37.

$$v_{c,close,adapted} = v_{c,close} \cdot \frac{A_{chamber}}{A_{free}} = v_{c,close} \cdot \frac{A_{chamber}}{B_{vessel} \cdot (h - draught)} \tag{C.37}$$

How to derive the percentage of collisions for which the opening is partly clogged is not included in the WBI manual. Because the collision failure is not included in the mechanisms which are relevant given the sea level rise scenarios (see Section 4.3), no further investigation is performed on the matter.

C.9. Instability due to head

Instability of the structure and its subsoil can occur in three degrees of freedom: horizontal-, vertical-, and rotational instability (Rijkswaterstaat, 2019c). All are initiated by either or both horizontal and vertical loads. These mechanisms do not necessarily need to be taken into consideration. Depending on the characteristics and dimensions of a construction, they can pose an insignificant threat.

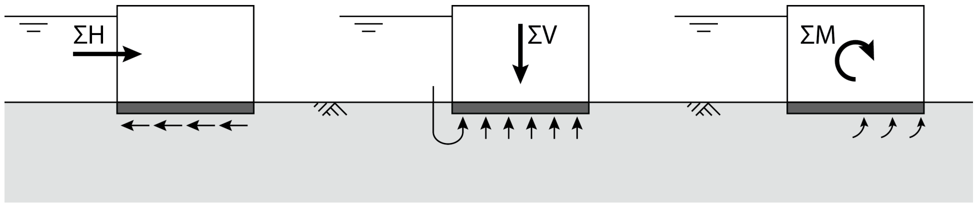


Figure C.16: Horizontal-, vertical-, and rotational stability of structure

Following the criteria of the WBI 2017 manual, it can be concluded that non of these degrees of freedom have to be taken into account for a navigation lock. This is based on the following statements (Rijkswaterstaat, 2019c):

- **Horizontal**: In case the dimensions of the surface which is subjected to the head are small in comparison to the length of the structure, horizontal instability is highly unlikely to occur.

- **Vertical**: Complete vertical instability of a structure practically does not occur. Especially uplift is an important factor for vertical stability, which is only prominent during construction and maintenance, when the lock chamber is emptied. This does not happen unexpectedly, meaning preventive measures can be taken to avoid instability.

- **Rotational**: Rotational instability can occur in case a section of the structure looses its bearing due to uplift, or a moment caused by a head which lifts one side of the structure Rotational stability only poses a threat to short structures with a large head. As determined for the horizontal stability, a navigation lock qualifies as a long structure, and is thus unlikely to fail due to rotational instability.

C.10. Piping

Piping is the mechanism of water seeping underneath or alongside a structure. This is caused by a difference in water level over the structure, as water tends to flow to body with the lowest pressure. The mechanism is largely dependent on the hydraulic gradient, i.e. the difference in water level over the length of the seepage path. Therefore, a large head will not necessarily lead to piping in case the seepage length is sufficiently long enough.

To determine whether piping will occur, two methods have been developed: Bligh (Equation C.38) and Lane (Equation C.39) (Molenaar & Voorendt, 2018). Both methods are based on the study of different hydraulic structures. Lane found that the vertical sections of a seepage path add less to the flow potential, as vertical structure elements have a higher probability of collapsing and thus obstructing the seepage. An important note is that for structures on a pile-foundation, the horizontal seepage length is none, as the narrow void that arises beneath the structure offers no resistance to seepage.

$$Z_B = \sum L_{vert} + \sum L_{hor} - \gamma C_B \Delta h \tag{C.38}$$

$$Z_L = \sum L_{vert} + \sum \frac{1}{3} L_{hor} - \gamma C_L \Delta h \tag{C.39}$$

Where:

Symbol	Description	Unit	Distribution	Value	
L_{vert}	Vertical seepage length	m	Normal*	μ=Input	σ=0.05
L_{vert}	Horizontal seepage length	m	Normal*	μ=Input	σ=0.05
γ	Safety factor	-	Normal*	μ=1.5	σ=0.05
C_B	Bligh's constant	-	Deterministic	Soil dependant	
C_L	Lane's constant	-	Deterministic	Soil dependant	

* Self-made assumption.

The constants in both methods, C_B and C_L, have been empirically derived, and depend on the type of subsoil.

Table C.3: Values for C_B and C_L for varying soil types. Retrieved from Molenaar and Voorendt (2018).

Soil type	C_B	C_L
Very fine sand / silt / sludge	18	8.5
Fine sand	15	7.0
Middle fine sand	-	6
Coarse sand	12	5
Gravel	6-9	4

To assure no piping will occur, both methods are used simultaneously. In case either one of the limit state functions in Equations C.38 and C.39 returns below zero, piping is assumed to occur. This is presented in Equation C.40.

$$Z = min(Z_B\ ;\ Z_L) \tag{C.40}$$

D

Computational model

In this Appendix, the computational model used to derive the correlation between the sea level rise and failure mechanisms of a lock is presented. An overview of the structure of the model is presented in Section 5.2. The script is written in the Python language, using the platform Spyder. A graphic depiction of the script is presented in Figure 5.10. Here sections of the script are presented in order, with text below the section explaining what cannot directly be implied from the script itself.

D.1. Input parameters

```
# original design conditions
originalWaterdepthUp1 = 5.489 #m
originalWaterdepthUp2 = 5.489 #m
originalWaterdepthDown = 2.1 #m
originalWaveHeight = 2.32 #m
originalWaveLength = 42.4 #m
constructionYear = 1962

# strength calculation
loadFactor = 1.25 #Leidraad
materialFactorSteel = 1.0 #EN1993
materialFactorWood = 1.3 #NEN-EN 1995-1-1
oldFactorSteel,oldFactorWood = 1.5,1.62 #WBI
```

Here at the top the hydraulic boundary conditions used to calculate the strength of the gate are implemented, i.e. the “original” design conditions. The values are derived as presented in Section 5.4.3. Below the new and old partial factors are given. These have been drawn from different guidelines as indicated.

```
# boundary conditions
def waveHeightFunction(Y):
    X = -1/5.739*np.log(Y/602.53)
    return (X)

def waterLevelFunction(Y):
    X = -1/3.453*np.log(Y/186143)
    return (X)

def wavePeriodFunction(Y):
    if Y>0.0033333333333:
        X = -1/4.21*np.log(Y/7292977)
    else:
        X = -1/6.54*np.log(Y/631384496671)
    return (X)

waterdepthDownMean, waterdepthDownStd = 2.13, 0.1 #m
rho_wMean,rho_wStd=1.025,0.005
durationStormMean, durationStormStd = 6.0, 6.0*0.25 #hours
g = 9.81 #m/s2
```

In the three definitions, the retention lines representing approximating the distribution of the hydraulic boundary conditions are implemented. These regression lines are derived externally in Excel, as the process requires some visual feedback, which does not fit within the structure of this script. After the definitions, a number of other parameters related to the boundary conditions are provided. For some, a mean and standard deviation is given, as this is later required for the Monte Carlo simulations (see Section 5.1.3).

```
# dimensions and properties lock
heightGate1Mean, heightGate1Std = 6, 0.05 #m NAP
heightGate2Mean, heightGate2Std = 6, 0.05 #m NAP
sillHeightMean, sillHeightStd = -12.8, 0.05 #m NAP
widthChamberMean,widthChamberStd = 38, 0.05 #m NAP
gateType = "roll" #"mitre", "roll","lift","rotary"
gateMaterial = "steel" #"wood", "steel"

# dimensions water system
bottomUpMean,bottomUpStd = -14,0.05 #m NAP
bottomDownMean,bottomDownStd = -10.4,0.05 #m NAP
widthRiverMean,widthRiverStd = 140,0.05 #m NAP
storageAreaMean, storageAreaStd = 4480000, 4480000*0.1 #m2
heightDykeMean, heightDykeStd = 4.2, 0.05 #m NAP
m_komMean, m_komStd = 1, 0.2  #fixed
```

Here a number of properties of the lock and water system are provided. Throughout the script, the outer gate is referred to as "Gate1", whereas the inner gate is referred to as "Gate2". All heights in the script are given in m NAP. For the gate type and gate material, the input options are provided. The parameter m_{kom} is identical for all locks, and thus fixed.

```
# specific for overflow/-topping
m_OlMean, m_OlStd = 1.1, 0.05 #fixed
m_OsMean, m_OsStd = 0.09, 0.06 #fixed
m_In = 1 #fixed

# specific for erosion
ys = 0 #m
psi_cr = 0.55
distance = 0 #m
rho_sMean, rho_sStd = 1.650, 0.05 ##kg/l

# specific for collision
yearlyLevelings = 5050
percentageCommercialShipping = 0.95
percentageInwardTransport = 0.67

# specific for closure
P_open = 0.5
P_closureFail = 0.0001

# specific for piping
LhMean, LhStd = 439, 0.05 #m
LvMean, LvStd = 27, 0.05 #m
CLMean, CLStd = 8.5, 0.05 #-
CBMean, CBStd = 18, 0.05 #-
safetyFMean, safetyFStd = 1.5, 0.05 #fixed
```

Here a number of parameters are given specific to the failure mechanisms of a lock. If names of these parameters seem obscure, like "m_Ol", these are mostly symbols for model factors used in the equations provided in Appendix C.

```
Dn50Mean = 0.4
phi_sc = 0.75
k_sl = 1
k_t = (2)**0.5
Delta = (rho_sMean-rho_wMean)/rho_wMean
psi_cr = 0.035
u = ((2*g*(originalWaterdepthUp1-originalWaterdepthDown))**0.5/0.9*
     (originalWaterdepthDown-bottomDownMean))/widthChamberMean
for i in range(10):
    k_h = (1+(originalWaterdepthDown-bottomDownMean)/Dn50Mean)**(-0.2)
    Dn50Mean = u**2*phi_sc*0.035*k_h*k_t**2/(2*g*Delta*psi_cr*k_sl)
Dn50Std = 0.05*Dn50Mean
```

For the study case of Terneuzen, no information on the bed protection was directly available. Therefore, a calculation is performed to approximate this. The D_{n50} is found iteratively, because the parameter is required to determine k_h, whereas k_h is required to determine the D_{n50}. It was found that using 10 iterations provides a sufficient precision of the diameter. The equations used here can be found in Section C.3.

```
def lognorm(logMean,logStd):
    std = np.sqrt(np.log(1+logStd**2/logMean**2))
    mean = np.log(logMean)-0.5*std**2
    return(mean,std)
```

In the WBI2017, the mean and standard deviation of Lognormal distributions are related to the distribution itself, whereas in Python, these are related to the underlying Normal distribution. This definition is required to rewrite the Python function to work with the WBI2017 parameters (Jonkman et al., 2017).

D.2. Failure mechanisms

```python
def Overflow(waterdepthUp,heightGate,waveHeight):
    m_OlLOG=lognorm(m_OlMean, m_OlStd)
    m_Ol = np.random.lognormal(m_OlLOG[0],m_OlLOG[1])
    m_OsLOG=lognorm(m_OsMean, m_OsStd)
    m_Os = np.random.lognormal(m_OsLOG[0],m_OsLOG[1])
    if waterdepthUp > heightGate:
        qOl = m_Ol * 0.55 * (-g * (heightGate - waterdepthUp)**3)**0.5
        qOs = m_Os * (g * waveHeight**3)**0.5
    else:
        qOl = 0
        qOs = m_Os *(g*waveHeight**3)**0.5*np.exp(-3*(heightGate-waterdepthUp)/
                             waveHeight)
    q_In = qOl + qOs
    return(q_In)
```

This function derives the distributed discharge during overflow (if any at all), given some hydraulic boundary conditions. These conditions are later provided in the Monte Carlo simulation. Because the factors m_{Ol} and m_{Os} are only relevant for this particular function, these can be prescribed here, out of the Monte Carlo function.

Depending on the height of the waterline relative to the crest of the gate, overflow “qOl” occurs, and overtopping “qOs” is defined by a different equation. This is defined within the if-statement. The equations used can be found in Section C.1.

```python
def Inflow(waterdepthUp,waterdepthDown,bottomDown):
    if waterdepthUp > waterdepthDown:
        q_In = (2*g*(waterdepthUp-waterdepthDown))**0.5*(waterdepthDown-
                      bottomDown)
    else: q_In = 0
    return(q_In)
```

This functions derived the distributed discharge during inflow, given some hydraulic boundary conditions. In case the water level in the river is higher than the sea level, inflow is registered equal to 0. The equation can be found in Section C.1.

```python
def BedProtectionFlow(q_In,Dn50,waterdepthDown,widthChamber,widthRiver,Delta,
                      bottomDown,psi_cr,distance):
    k_h = (1+(waterdepthDown-bottomDown)/Dn50)**(-0.2)
    uc = ((2*g*Delta*Dn50*psi_cr*k_sl)/(phi_sc*0.035*k_h*k_t**2))**0.5
    qcMean = uc * (waterdepthDown-bottomDown)
    qcStd = 0.15*qcMean
    qc = np.random.lognormal(lognorm(qcMean,qcStd)[0],lognorm(qcMean,qcStd)[1])
    widthFlow = min(widthChamber+2*0.2*distance,widthRiver)
    Z = qc * widthFlow - q_In * widthChamber
    return(Z)
```

Here the limit state function of bed protection failure due to flow is provided. The equations used can be found in Section C.3.

```
def BedProtectionJet(waterdepthUp,waterdepthDown,heightGate,Dn50,bottomDown,ys):
    h0 = waterdepthUp - heightGate
    H = waterdepthUp - waterdepthDown
    if h0 > 0 and H>0:
        qcMean = 1.705 * (h0**3)**0.5
        qcStd = 0.15*qcMean
        qcLOG = lognorm(qcMean,qcStd)
        qc = np.random.lognormal(qcLOG[0],qcLOG[1])
        Z = (ys+0.5*(waterdepthDown-bottomDown))/(Dn50**(-0.3))-0.4*(qc**0.6)\
        *(H**0.4)
    else:
        Z = 1
    return(Z)
```

Here the limit state function of bed protection failure due to a jet is provided. This jet only occurs when water is spilling, and thus the sea level needs to be higher than the crest of the gate, and the river level. This is provided in the if-statement, given as the value for Z is otherwise equal to 1 (no failure). The equation can be found in Section C.3.

```
def StorageCapacity(waterdepthDown,widthRiver,q_In):
    heightDyke = np.random.normal(heightDykeMean,heightDykeStd)
    storageLOG = lognorm(storageAreaMean, storageAreaStd)
    storageArea = np.random.lognormal(storageLOG[0],storageLOG[1])
    m_komLOG = lognorm(m_komMean, m_komStd)
    m_kom = np.random.lognormal(m_komLOG[0],m_komLOG[1])
    durationLOG = lognorm(durationStormMean,durationStormStd)
    durationStorm = np.random.lognormal(durationLOG[0],durationLOG[1])

    deltaH = heightDyke - waterdepthDown
    Z = deltaH*storageArea*m_kom-q_In*m_In*widthRiver*durationStorm*3600
    return(Z)
```

Here the limit state function of the surpassage of the storage capacity is provided. Some parameters are provided as input from the Monte Carlo simulation, as these are also used in other limit state functions. For the parameters specific for this failure mechanism, a random value is drawn from their distribution here. The equation can be found in Section C.2.

```
def sections(gateHeight,waterdepthDown,bottomUp):
    gateH = gateHeight - bottomUp
    if gateH - int(gateH) < 0.5:
        sectionH = 1 + (gateH - int(gateH))/int(gateH)
        sectionN = int(gateH)
    else:
        sectionH = 1 - (int(gateH)+1 - gateH)/(int(gateH)+1)
        sectionN = int(gateH)+1
    for i in range(sectionN):
        if (i+0.5)*sectionH > (waterdepthDown-bottomUp):
            sectionCr = max(i-1,0)
            break
    return(sectionH,sectionN,sectionCr)
```

To derive the relation between the load on a gate and its strength, the gate is divided into sections, and only the one to which the largest load is applied is regarded. The gate height, “gateH”, is divided into sections as close as possible to 1 m. By means of the first if-statement, a gate with a height of 8.4 m would be divided into 8 sections of 1.05 m, whereas a gate of 8.6 m would be divided into 9 sections of 0.96 m. The height of each section is here “sectionH”, whereas the amount of sections is “sectionN”. Use is made of the fact that by taking the integer of a float (the gate height in this situation), the float is always rounded down.

A section is critical if the downstream water level is located at its top half, or at the bottom half of the section above it (see Section C.4 for an explanation). This is here derived by means of a loop through all sections from the bottom up, and finding the first section for which the downstream water level is above (i+0.5) times the section height.

```
def GodaTakahashi(heightGates,waterdepthUp,waveLength,waveHeight,rho_w,
                  incidence,bottomUp):
    sillHeight = np.random.normal(sillHeightMean,sillHeightStd)
    Bm = bottomUp - sillHeight
    hc=heightGates-waterdepthUp
    d=waterdepthUp-sillHeight
    h1=waterdepthUp-sillHeight
    h=waterdepthUp-bottomUp
    Ld=waveLength
    hb=waterdepthUp-bottomUp
    Hd=waveHeight

    l1=1
    n=0.75*(1+np.cos(incidence))*l1*Hd
    hc1=min(n,hc)
    a1=0.6+0.5*((4*np.pi*h/Ld)/np.sinh(4*np.pi*h/Ld))**2
    a2=min((1-d/hb)*(Hd/d)**2/3,2*d/Hd) #=0 when no berm present
    a3=1-(h1/h)*(1-1/(np.cosh(2*np.pi*h/Ld)))
    a4=1-hc1/n
    d11=0.93*(Bm/Ld-0.12)+0.36*((h-d)/h-0.6)
    d22=-0.36*(Bm/Ld-0.12)+0.93*((h-d)/h-0.6)
    if d11>0:
        d1=15*d11
    else:
        d1=20*d11
    if d22>0:
        d2=3*d22
        an=1/(np.cos(d1)*np.sqrt(np.cosh(d2)))
    else:
        d2=4.9*d22
        an=np.cos(d2)/np.cosh(d1)
    am=min(Hd/d,2)
    al=an*am
    if a2!=0:
        l2=max(1,al/a2)
    else:
        l2=1
    p1=0.5*(1+np.cos(incidence))*(l1*a1+l2*a2*(np.cos(incidence))**2)*\
    rho_w*g*Hd
    p3=a3*p1
    p4=a4*p1
    return(p1,p3,p4)
```

Here the wave force on a gate is derived. The equations used can be found in Section C.4.

```
def HydrostaticPressure(waterdepthUp,waterdepthDown,rho_w,bottomUp,casc):
    h1 = waterdepthUp-bottomUp
    h2 = waterdepthDown-bottomUp
    S = ((h1)-(h2+(waterdepthUp-waterdepthDown)/2*casc))*g*rho_w
    return(S)
```

Here the hydrostatic force on a gate is derived. Factor “casc” is equal to 1 if the policy of a lock is to provide a cascading water level of the lock (water level inside the chamber in the middle of the upstream and downstream water level). If the water level inside the lock is equal to the downstream water level, “casc” is equal to 0. The equation used can be found in Section C.4.

```
def StructuralResistance(heightGates,originalWaterdepthUp,waterdepthDown,
                         waterdepthUp,sectionH,sectionCr,incidence,rho_w,
                         bottomUp,bottomDown,casc):
    if constructionYear > 1990:
        if gateMaterial=="wood":
            materialFactor = materialFactorWood
        if gateMaterial=="steel":
            materialFactor = materialFactorSteel
    else:
        if gateMaterial=="wood":
            designFactor = oldFactorWood
        if gateMaterial=="steel":
            designFactor = oldFactorSteel
    if gateMaterial == "wood":
        RVr = 0.25
    if gateMaterial == "steel":
        RVr = 0.1
    Sk_old_w = GodaTakahashi(heightGates,originalWaterdepthUp,
                             originalWaveLength,originalWaveHeight,rho_w,
                             incidence,bottomUp)
    Sk_old_h = HydrostaticPressure(originalWaterdepthUp,originalWaterdepthDown
                                   ,rho_w,bottomUp,casc)
    if waterdepthUp - bottomUp > (sectionCr+1)*sectionH:
        Sk_old_W = (Sk_old_w[1]+(Sk_old_w[0]-Sk_old_w[1])*(sectionCr+0.5)*\
                sectionH/(originalWaterdepthUp-bottomUp))*sectionH
    else:
        Sk_old = 10000
    if originalWaterdepthDown - bottomUp > (sectionCr+1)*sectionH:
        Sk_old_H = Sk_old_h * sectionH
    else:
        Sk_old_H = (originalWaterdepthUp-originalWaterdepthDown-((sectionCr+1)\
                   *sectionH-(originalWaterdepthDown-bottomUp))/2)/\
                   (originalWaterdepthUp-originalWaterdepthDown)*Sk_old_h*\
                   ((sectionCr+1)*sectionH-(originalWaterdepthDown-bottomUp))\
                   + ((originalWaterdepthDown-bottomUp)-sectionCr*sectionH)*\
                   Sk_old_h
    Sk_old=abs(Sk_old_W+Sk_old_H)
    if constructionYear > 1990:
        Rm_old = (materialFactor * loadFactor * Sk_old)/(1-1.64*RVr)
    else:
        Rm_old = (designFactor * Sk_old)/(1-1.64*RVr)
    RmLOG=lognorm(Rm_old, 0.10*Rm_old)
    Rm_new = np.random.lognormal(RmLOG[0],RmLOG[1])
    return(Rm_new)
```

In this function the strength of a gate is derived. Use is made of the two previous functions to determine the wave pressure and hydrostatic pressure on the structure. Initially, the partial factors with which the gate would have been designed are determined based on the construction year and material of the gate. “Sk_old_w” and “Sk_old_h” are the wave pressure and hydrostatic pressure that would be used to design the gate, making use of the “original” hydraulic boundary conditions, i.e. the design conditions. From these pressures, the wave force “Sk_old_W” and hydrostatic force “Sk_old_H” on the critical section are derived. The equation used to derive the wave force falls short when the upstream water level is below the critical section. However, in this scenario one can assume that the gate will not fail due to the head difference, because of the small or even negative head difference. Therefore, a large design load is provided in this case, assuring the gate does not fail. For the hydrostatic pressure, it matters whether this section is located below or above the downstream waterline. This is defined within an if-statement. The used equations can be found in Section C.4.

Finally the two forces are added as “Sk_old”, i.e. the load on the section according to the original design criteria. From this, the resistance can be derived, depending on the construction year and gate material.

“RVr” is the coefficient of variance of the used material.

```
def StructuralFailureFall(heightGates,waterdepthUp,waterdepthDown,sectionH,
                          sectionCr,waveLength,waveHeight,rho_w,
                          originalWaterdepthUp,incidence,bottomUp,
                          bottomDown,casc):
    ms = np.random.normal(1,0.05)
    Sk_new_w = GodaTakahashi(heightGates,waterdepthUp,waveLength,waveHeight,
                             rho_w,incidence,bottomUp)
    Sk_new_h = HydrostaticPressure(waterdepthUp,waterdepthDown,rho_w,bottomUp,
                                   casc)
    if waterdepthUp - bottomUp > (sectionCr+1)*sectionH:
        Sk_new_W = (Sk_new_w[1]+(Sk_new_w[0]-Sk_new_w[1])*(sectionCr+0.5)*\
                    sectionH/(waterdepthUp-bottomUp))*sectionH
    else:
        Sk_new_W = 0
    if waterdepthDown - bottomUp > (sectionCr+1)*sectionH:
        Sk_new_H = Sk_new_h * sectionH
    else:
        Sk_new_H = (waterdepthUp-waterdepthDown-((sectionCr+1)*sectionH-\
                   (waterdepthDown-bottomUp))/2)/(waterdepthUp-waterdepthDown)\
                   *Sk_new_h*((sectionCr+1)*sectionH-(waterdepthDown-bottomUp)\
                   ) + ((waterdepthDown-bottomUp)-sectionCr*sectionH)*Sk_new_h
    Sk_new = abs(Sk_new_W+Sk_new_H)*loadFactor
    Rm_new = StructuralResistance(heightGates,originalWaterdepthUp,
                                  waterdepthDown,waterdepthUp,sectionH,
                                  sectionCr,incidence,rho_w,bottomUp,
                                  bottomDown,casc)
    Z = Rm_new - Sk_new * ms
    return(Z)
```

Finally the limit state function of “Failure due to head” is provided here. The new wave pressure “Sk_new_w” and hydrostatic pressure “Sk_new_h” are determined using the functions previously described. In a similar manner as before, the wave force “Sk_new_W” and “Sk_new_H” are calculated from these stresses. The wave force is set equal to 0 in case the upstream water level is below the critical section, because in this scenario (as explained before), no failure can be assumed. The two forces are added as “Sk_new”, and the strength of the gate is derived using the previous function. The limit state function is thereby complete. The used equations can be found in Section C.4.

```
def Vibration(q_In):
    Z = 1 - q_In
    return (Z)
```

Here the limit state function of vibration failure is provided. The equation can be found in Section C.5.

```
def Piping(waterdepthUp,waterdepthDown):
    Lh = np.random.normal(LhMean,LhStd)
    Lv = np.random.normal(LvMean,LvStd)
    CL = np.random.normal(CLMean,CLStd)
    CB = np.random.normal(CBMean,CLStd)
    safetyF = np.random.normal(safetyFMean,safetyFStd)
    ZL = Lh/3 + Lv - safetyF * CL * abs(waterdepthUp-waterdepthDown)
    ZB = Lh + Lv - safetyF * CB * abs(waterdepthUp-waterdepthDown)
    Z = min(ZL,ZB)
    return (Z)
```

Here the *two* limit state functions for piping are given. Because both functions have to suffice to be sure piping failure does not occur, the minimum of the two Z-values is returned by the function. The equations can be found in Section C.10.

```
def ClosureFailure():
    ClosureFailure_y = 1 - (1- P_open * P_closureFail) ** yearlyLevelings
    ClosureFailure_n = P_open * P_closureFail
    return (ClosureFailure_y,ClosureFailure_n)
```

Closure failure is not derived through a limit state function. Instead, the probability can directly be derived. Two probabilities have to be calculated: the yearly probability, and the probability per closure. The equations can be found in Section C.7.

The functions mentioned before concern all failure mechanisms regarded in this thesis with respect to sea level rise. These are sequentially used in the Monte Carlo simulations, as presented hereafter.

```
def se(name):
    for l in range(len(Z)):
        if Z[l] == name:
            ind = l
    return(ind)
```

Throughout the remainder of the script, two lists will be updated simultaneously: one containing abbreviations for the failure mechanisms, called "Z", and one containing the probabilities of those failure mechanisms, called "P". Thus, when a probability has to be used to calculate another probability (for instance within the fault tree), the probability can be found in "P", as it has the same index within the list as its counterpart within "Z".

This function, "se" (short for search), takes in the abbreviation of a failure mechanism, and returns the index of that mechanism in list "Z".

D.3. Monte Carlo analysis

The following function is the actual Monte Carlo simulation. Because this is a substantial section of the script, the function is split into segments.

```
def MonteCarlo(it,SLR):
    run = 0
    for j in range(it):
        if int(j/(it/10))>int((j-1)/(it/10)):
            run+=1
            print (run,"/ 10")

        bc_random = np.random.uniform(0,1)
        waterdepthUp = waterLevelFunction(bc_random)+SLR
        waveHeight = waveHeightFunction(bc_random)
        waveI = np.random.uniform(0,1)
        if waveI > 0.015:
            waveHeight = 0
        wavePeriod = wavePeriodFunction(bc_random)
        waveLength = 50
        for i in range(20):
            waveLength = g*wavePeriod**2/(2*np.pi)*np.tanh(2*np.pi*\
                        (waterdepthUp+bottomUpMean)/waveLength)
        heightGate1 = np.random.normal(heightGate1Mean,heightGate1Std)
        heightGate2 = np.random.normal(heightGate2Mean,heightGate2Std)
        bottomUp = np.random.normal(bottomUpMean,bottomUpStd)
        bottomDown = np.random.normal(bottomDownMean,bottomDownStd)
        incidence = 0#np.random.normal(incidenceMean,incidenceStd)
        waterdepthDown = np.random.normal(waterdepthDownMean,waterdepthDownStd)
        widthChamber = np.random.normal(widthChamberMean,widthChamberStd)
        widthRiver = np.random.normal(widthRiverMean,widthRiverStd)
        Dn50 = np.random.normal(Dn50Mean,Dn50Std)
        rho_w = np.random.normal(rho_wMean,rho_wStd)
        rho_s = np.random.normal(rho_sMean,rho_sStd)
        Delta = (rho_s-rho_w)/rho_w
        sectionH,sectionCr = sections(heightGate1,waterdepthDown,bottomUp)
```

The Monte Carlo function takes in an amount of iterations “it”, and a sea level rise “SLR”. The amount of iterations determines the preciseness of the outcome of the analysis. The sea level rise will be increased each time the Monte Carlo analysis is repeated, as will be presented later in the script.

At the top of the function, “run” is assigned. This merely functions as a manner to monitor how far along the script is while running, as it is printed each time a tenth of the iterations is executed.

Sequentially, a loop is started of “it” amount of iterations. Then, for each variable which is defined by a distribution, a random value is drawn, using either the regression lines, or the built in “numpy”-function “random.normal”. An exception is the wave length. This has to be iteratively found using the wave period, as the wave length is a function of itself. It was found that 20 iterations provide a value precise enough for this matter. For the water depth, wave height, and wave period, it is assumed that their probability of occurrence is dependent. Thus, a single random value “bc_random” is drawn from a uniform distribution, and applied to each of their regression line functions.

```
        q_In1 = Overflow(waterdepthUp,heightGate1,waveHeight)
        q_In2 = Overflow(waterdepthUp,heightGate2,waveHeight)
        q_In3 = Inflow(waterdepthUp,waterdepthDown,bottomDown)

        ZP = Piping(waterdepthUp,waterdepthDown)

        ZoF = BedProtectionFlow(q_In1,Dn50,waterdepthDown,widthChamber,widthRiv
        ZoJ = BedProtectionJet(waterdepthUp,waterdepthDown,heightGate1,Dn50,bot
        ZoS = StorageCapacity(waterdepthDown,widthRiver,q_In1)
        ZiF = BedProtectionFlow(q_In3,Dn50,waterdepthDown,widthChamber,widthRiv
        ZiS = StorageCapacity(waterdepthDown,widthRiver,q_In3)
        Z_f = StructuralFailureFall(heightGate2,waterdepthUp,waterdepthDown,sec
        Z_fiF,Z_fiS=max(Z_f,ZiF),max(Z_f,ZiS)
        Z_v = Vibration(q_In2)
        Z_viF,Z_viS=max(Z_v,ZiF),max(Z_v,ZiS)

        Z_oF = BedProtectionFlow(q_In2,Dn50,waterdepthDown,widthChamber,widthRi
        Z_oJ = BedProtectionJet(waterdepthUp,waterdepthDown,heightGate2,Dn50,bo
        Z_oS = StorageCapacity(waterdepthDown,widthRiver,q_In2)
        Zf1 = StructuralFailureFall(heightGate1,waterdepthUp,waterdepthDown,sec
        Zf2 = StructuralFailureFall(heightGate1,waterdepthUp,waterdepthDown,sec
        Zf1iF,Zf1iS=max(Zf1,ZiF),max(Zf1,ZiS)
        Zf2iF,Zf2iS=max(Zf2,ZiF),max(Zf2,ZiS)
        Zv = Vibration(q_In1)
        ZviF,ZviS=max(Zv,ZiF),max(Zv,ZiS)

        listZ = [ZP,ZoF,ZoJ,ZoS,ZiF,ZiS,Z_f,Z_fiF,Z_fiS,Z_v,Z_viF,Z_viS,Z_oF,
                 Z_oJ,Z_oS,Zf1,Zf2,Zf1iF,Zf1iS,Zf2iF,Zf2iS,Zv,ZviF,ZviS]
```

The next step is to implement all these random values into the functions of the failure mechanisms. The functions all return a Z-value, and these are brought together in a list "listZ". This list thus does not yet contain any probabilities, merely values below or above 0, implying a failure or success respectively, based on the random values drawn from their distributions.

For some failure mechanisms, the probability of one mechanism *given* the probability of another mechanism is required. In such an arrangement, a failure occurs only in case *both* Z-values are below 0. Thus, the maximum of both Z-values is drawn, because if the larger value of the two is below 0, both are.

```
        if j == 0:
            listF = np.zeros(len(listZ))
        for jj in range(len(listZ)):
            if listZ[jj] < 0:
                listF[jj] += 1
```

At the first iteration, a list of zero's, "listF" is created. Sequentially, the script loops through "listZ", and for each value below 0 (i.e. failure), a 1 is added to the spot in list "listF" coinciding with that failure mechanism. Thus, after all iterations are performed, the total number of failures per failure mechanism are stored in this list.

```
    listP = [0]*len(Z)
    for j in range(len(listF)):
        if Z[j] == "Z_fiF" or Z[j] == "Z_fiS":
            listP[j]=listF[j]/listF[se("Z_f")]
        elif Z[j] == "Z_viF" or Z[j] == "Z_viS":
            listP[j]=listF[j]/listF[se("Z_v")]
        elif Z[j] == "Zf1iF" or Z[j] == "Zf1iS":
            listP[j]=listF[j]/listF[se("Zf1")]
        elif Z[j] == "Zf2iF" or Z[j] == "Zf2iS":
            listP[j]=listF[j]/listF[se("Zf2")]
        elif Z[j] == "ZviF" or Z[j] == "ZviS":
            listP[j]=listF[j]/listF[se("Zv")]
        else:
            listP[j]=listF[j]/it
```

Here finally the probabilities of the failure mechanisms are derived. A list "listP" is created, and then the script loops through the failures of "lsitF", followed by a number of if-statements. In the last statement, "else", for all failure mechanisms which are not mentioned in the if-statements above it, the number of failures in "listF" is divided by the number of iterations performed, providing the failure probability.

The other if-statements concern failure mechanisms which require the probability of occurrence *given* the probability of another mechanism. For these probabilities, the amount of failure should not be divided by the total number of iterations, but by the amount of failures of the other mechanism which has to be given. These amounts are found using the "se"-function.

```
    for j in range(len(listP)):
        if math.isnan(listP[j]) == True:
            listP[j] = 0

    for j in range(len(Z)):
        if Z[j] == "Zc":
            listP[j]=ClosureFailure()[0]
        if Z[j] == "Zcn":
            listP[j]=ClosureFailure()[1]

    return(listF,listP)
```

To conclude the Monte Carlo function, all Nan-values are replaced by zeros. These Nan-values are created when calculating a probability *given* another probability which equal to 0, because a division is made over 0.

At last, the values of "Closure failure" are added to "listP". These were derived directly without the use of a Monte Carlo simulation.

D.4. Fault tree

```
def OR(setP):
    P = (1-setP[0])
    for i in range(1,len(setP)):
        P = P*(1-setP[i])
    P = 1 - P
    return(P)

def AND(setP):
    P = setP[0]
    for i in range(1,len(setP)):
        P = P*setP[i]
    return(P)
```

The next step is to implement all probabilities into the fault tree. In the fault tree, two types of gate are used: an OR-gate and an AND-gate. These are given here as functions taking a list of probabilities as input. The equations used can be found in Section 5.1.4.

```python
def FaultTree(Z,P):
    P.insert(se("ZoS")+1,OR([P[se("ZoF")],P[se("ZoJ")],P[se("ZoS")]]))
    Z.insert(se("ZoS")+1,"ZO")
    P.insert(se("ZiS")+1,OR([P[se("ZiF")],P[se("ZiS")]]))
    Z.insert(se("ZiS")+1,"ZI")
    P.insert(se("Z_fiS")+1,OR([P[se("Z_fiF")],P[se("Z_fiS")]]))
    Z.insert(se("Z_fiS")+1,"Z_fi")
    P.insert(se("Z_fi")+1,AND([P[se("Z_f")],P[se("Z_fi")]]))
    Z.insert(se("Z_fi")+1,"Z_F")
    P.insert(se("Z_viS")+1,OR([P[se("Z_viF")],P[se("Z_viS")]]))
    Z.insert(se("Z_viS")+1,"Z_vi")
    P.insert(se("Z_vi")+1,AND([P[se("Z_v")],P[se("Z_vi")]]))
    Z.insert(se("Z_vi")+1,"Z_V")
    P.insert(se("Z_oS")+1,OR([P[se("Z_oF")],P[se("Z_oJ")],P[se("Z_oS")]]))
    Z.insert(se("Z_oS")+1,"Z_O")
    P.insert(se("Zc")+1,OR([P[se("Z_F")],P[se("Z_V")],P[se("Z_O")]]))
    Z.insert(se("Zc")+1,"Zcx")
    P.insert(se("Zcx")+1,1-(1-P[se("Zcx")])**(1/(2*365)))
    Z.insert(se("Zcx")+1,"Zcx12")
    P.insert(se("Zcx12")+1,AND([P[se("Zc")],P[se("Zcx12")]]))
    Z.insert(se("Zcx12")+1,"ZC")
    P.insert(se("Zf1iS")+1,OR([P[se("Zf1iF")],P[se("Zf1iS")]]))
    Z.insert(se("Zf1iS")+1,"Zf1I")
    P.insert(se("Zf2iS")+1,OR([P[se("Zf2iF")],P[se("Zf2iS")]]))
    Z.insert(se("Zf2iS")+1,"Zf2I")
    P.insert(se("Zf1I")+1,AND([1-P[se("Zcn")],P[se("Zf1")],P[se("Zf1I")]]))
    Z.insert(se("Zf1I")+1,"ZF1")
    P.insert(se("Zf2I")+1,AND([P[se("Zcn")],P[se("Zf2")],P[se("Zf2I")]]))
    Z.insert(se("Zf2I")+1,"ZF2")
    P.insert(se("ZF2")+1,P[se("ZF1")]+P[se("ZF2")])
    Z.insert(se("ZF2")+1,"ZF")
    P.insert(se("ZviS")+1,OR([P[se("ZviS")],P[se("ZviF")]]))
    Z.insert(se("ZviS")+1,"Zvi")
    P.insert(se("Zvi")+1,AND([P[se("Zvi")],P[se("Zv")]]))
    Z.insert(se("Zvi")+1,"ZV")
    P.insert(se("ZV")+1,OR([P[se("ZC")],P[se("ZF")],P[se("ZV")],P[se("ZO")]]))
    Z.insert(se("ZV")+1,"ZR")
    return(Z,P)
```

In this function the fault tree is completely worked out. It takes as input "Z", a list of abbreviations, and "P", a list of probabilities. By means of the OR- and AND-functions, the layers of the fault tree are derived and added to "P". To assure that the "se"-function works correctly, and to be able to use the newly added probabilities in the fault tree, an abbreviation must also be added to "Z" for each derived failure probability. The fault tree can be found in Appendix B.

D.5. Sea level rise development

```python
SLR_list=np.arange(0,3,0.5)
it=100000

results = []
for i in range(len(SLR_list)):
    Z = ["ZP","ZoF","ZoJ","ZoS","ZiF","ZiS","Z_f","Z_fiF","Z_fiS","Z_v",
     "Z_viF","Z_viS","Z_oF","Z_oJ","Z_oS","Zf1","Zf2",
     "Zf1iF","Zf1iS","Zf2iF","Zf2iS","Zv","ZviF","ZviS","Zc","Zcn"]
    P = MonteCarlo(it,SLR_list[i])
    Z,P = FaultTree(Z,P)
    results.append(P)
    print ("Executed", i+1, "/", len(SLR_list))
```

The process of deriving the yearly probability of each mechanism through the Monte Carlo analysis, and extracting the total failure probability of the flood protection function from the fault tree has to be iterated for a series of values for the sea level rise, here "SLR_list". A value of required iterations has to be provided as "it". An empty list called "results" is created, in which a list "P" will be stored for each value of sea level rise.

This list "P" is obtained within a loop, in which first the initial list "Z" is given (this has to be renewed each time because the abbreviations added in the fault tree function should not be included initially). The probabilities are derived in the Monte Carlo function, which are then put into the fault tree function, and the final list of probabilities is added to "results".

```python
interest = ["ZF","ZV","ZO"]
labels = ["Head failure","Vibration failure","Overflow failure"]
marks=["-","-","-"]

plots = []
for i in range(len(interest)):
    plots.append([])
    for ii in range(len(results)):
        plots[i].append(results[ii][se(interest[i])])
```

All results are now derived, and can be plotted. In list "interest", one can add the abbreviations of the failure mechanisms which are desired to be presented. The lists "labels" and "marks" concern the names in the legend, and the type of line used in the graph respectively.

Through a loop, the correct values are drawn from "results" for the failure mechanisms indicated in "interest".

```
plt.figure(figsize=(14,6))
norm = 1/1000
normSignal = 1/3000
for i in range(len(plots)):
    plt.plot(SLR_list,plots[i],linestyle=marks[i],label=labels[i])
plt.axhline(normSignal,color="orange",linestyle="--",label="Signaling norm")
plt.axhline(norm,color="red",linestyle="--",label="Final norm")

ax = plt.subplot(111)
box = ax.get_position()
ax.set_position([box.x0, box.y0, box.width*0.65, box.height])
legend_x = 1
legend_y = 0.5
plt.legend(loc='center left', bbox_to_anchor=(legend_x, legend_y))
plt.xlim(0,max(SLR_list))
plt.title("Development failure mechanisms with sea level rise")
plt.xlabel("Sea level rise [m]")
plt.ylabel("Failure probability [-]")
plt.yscale("log")
```

At last, the desired failure mechanisms are plotted against the sea level rise. Additionally, the signalling norm and regular norm are plotted as dotted horizontal lines. The legend is positioned outside the graph, and the scale of the vertical axis is made logarithmic.

E

Verification computational model

To use the computational failure probability model with confidence in its results, the script has to be verified. The script, presented and explained in Appendix D, consists of many different functions, which are all used to derive the failure probability development. Because of the size of the script, it is impossible to verify it by means of its final output. However, these functions can be verified individually. If each individual function can be trusted to work on its own, so can the model in its entirety. In this Appendix, the functions are verified by means of example calculations presented in the WBI manuals (De Waal, 2019), or hand calculations.

Overflow/overtopping
To test the equations for overflow and overtopping presented in the WBI (Rijkswaterstaat, 2019b), a graph is made for which all variables except the water depth on the sea side are constant, for three different wave heights. The graph is presented in Figure E.1. Obviously correct is that the discharge increases with an increasing water depth. A small kink is present at the point where the water depth surpasses the gate height. This occurs because at this point a different equation is used.

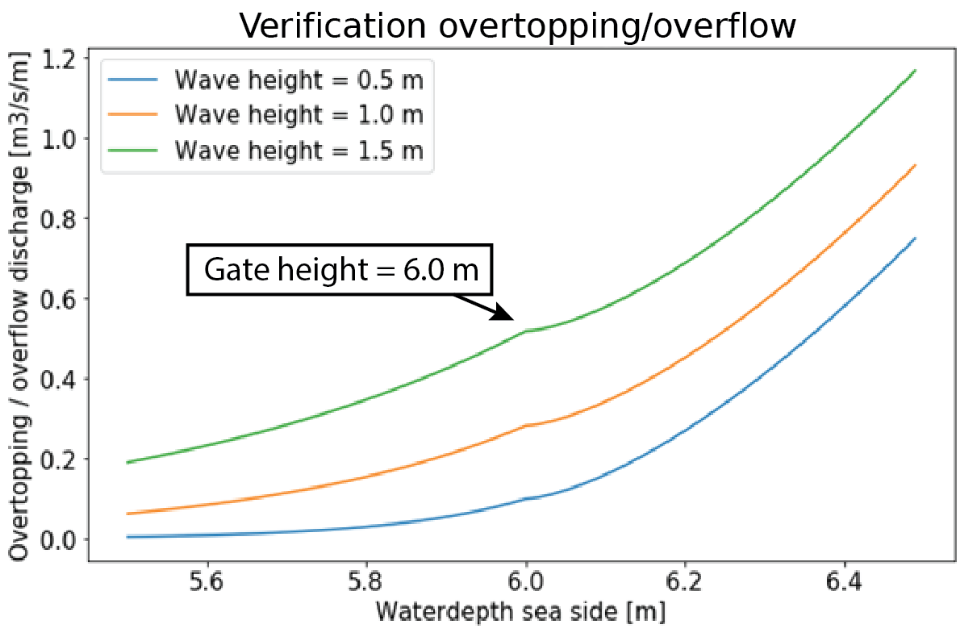


Figure E.1: Verification of overflow/overtopping function

In addition to the analysis of the discharge development, the function is tested by means of an example provided in the WBI manual, presented in Figure E.2. In the example, concerning a lock head, an overtopping discharge of 0.284 $m^3/s/m$ is found for a number of input parameters. An identical value is derived by implementing these parameters into the script.

Voorbeeld
Stel een sluis heeft keermiddelen met een kerende hoogte h_{kr} van NAP+4,0 m en een breedte van 12 meter (bron: ontwerptekening). Het sluishoofd is aan weerszijden 7,5 meter breed en heeft een kerende hoogte van NAP+4,5 m Het sluishoofd heeft een verticaal front en het overslag-/overloopdebiet van het sluishoofd komt uiteindelijk ook in de sluiskolk terecht. In dat geval wordt eerst een berekening gemaakt waarin alleen de breedte van de keermiddelen wordt meegenomen. Stel nu dat in het ontwerppunt van de overslagberekening de waterstand h NAP+3,7 m bedraagt en de significante golfhoogte H_{m0} 1,5 m is bij een loodrechte golfinval (γ_β=1). Het totale overslagdebiet bedraagt dan 0,284 $m^3/s/m$ (zie paragraaf 7.2.6)

Figure E.2: Example of overflow calculation. Retrieved from Rijkswaterstaat (2019b) (edited).

This example only concerns the overtopping discharge. In addition, a situation where overtopping and overflow are combined is checked. For this, a self made example is used. From the example, a combined overflow and overtopping discharge is found of 1.188 $m^3/s/m$. This is identical to the value found by implementing the parameters into the script function.

$$q_{ol+os} = m_{ol} \cdot 0.55 \cdot \sqrt{-g \cdot (h_{kr} - h)^3} + m_{os} \cdot \sqrt{gH_{m0}^3} \tag{E.1}$$

$$q_{ol+os} = 1.1 \cdot 0.55 \cdot \sqrt{-9.81 \cdot (4 - 4.5)^3} + 0.09 \cdot \sqrt{9.81 \cdot 1.5^3} = 1.188 m^3/s/m \tag{E.2}$$

Bed protection failure

Failure of bed protection can be initiated in two manners: by free flow, and by a jet created by a spilling flow. Both mechanisms are implemented in the function which is used to derive the failure probability of the bed protection, and therefore both have to be verified. For the former, an example is presented in the WBI manual (Rijkswaterstaat, 2019b). In the example, a flow velocity of 2.8 m/s and critical discharge of 4.2 $m^3/s/m$ are found by implementing the presented parameters (see Figure E.3). The same values are found by filling in these parameters in the script.

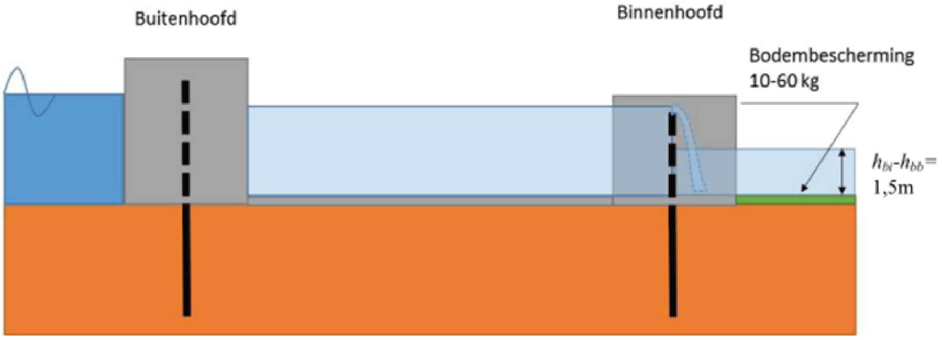
For the bed protection failure caused by a spilling jet, no example is provided in the WBI manuals. Therefore, a self made hand calculation is performed to check whether the script provides an identical answer. The function in the script finds the same values as the sequence of equations presented below.

$$q_c = 1.705 \cdot \sqrt{h_0^3} = 1.705 \cdot \sqrt{0.25^3} = 0.213 m^3/s/m \tag{E.3}$$

$$Z = \frac{y_s + 0.5h}{d_{50}^{-0.3}} - 0.4 \cdot q_c^{0.6} \cdot H^{0.4} \tag{E.4}$$

$$Z = \frac{0.1 + 0.5 \cdot 1.5}{0.24^{-0.3}} - 0.4 \cdot 0.213^{0.6} \cdot 0.5^{0.4} = 0.434 \tag{E.5}$$

Voorbeeld 1
Beschouw de volgende situatie: achter een recreatiesluis is een bodembescherming aanwezig bestaande uit stortsteen met sortering 10-60 kg. Beide deuren zijn gesloten. Door het optredende overslag-/overloopdebiet is de kolk volgelopen tot aan het niveau van de binnendeuren. Hierover treedt als gevolg van het overslag-/overloopdebiet een overstortende straal op die de bodembescherming belast.



Figuur 7-2 Sluis met bodembescherming belast door overtrekkende stroming

De overstortende straal komt terecht binnen de contouren van het betonwerk. Voor het voorbeeld wordt aangenomen dat de straal zich over de volledige waterdiepte spreidt voordat de bodembescherming wordt bereikt.
Invullen van formule 7-8 levert een kritieke stroomsnelheid op van $\boxed{2{,}8 \text{ m/s.}}$ Vermenigvuldiging met de waterdiepte van 1,5 m levert een waarde voor het in te vullen kritiek debiet q_c op van $\boxed{4{,}2 \text{ m}^3\text{/s/m.}}$

Figure E.3: Verification of bed protection failure function. Retrieved from Rijkswaterstaat (2019b) (edited).

Failure due to head difference

In the script, the wave force on a gate is derived using the Goda-Takanashi function. An example of this is presented in the Leidraad Kunstwerken. Because this example is spread out over several pages, not the entire derivation is presented here. Instead, only the final results are presented in Figure E.4. By implementing the parameters of the example into the script, identical answers are found.

Horizontale Golfdruk:

$$
\begin{aligned}
p_1 &= 0{,}5(1+cos(\beta))(\lambda_1\alpha_1+\lambda_2\alpha^* cos^2(\beta))\rho g H_d \\
&= 0{,}5\cdot 2\cdot(0{,}61+0{,}08)\cdot 1000.9{,}81\cdot 1{,}16\ N/m^2 \\
&= 7{,}85\ \ kN/m^2 \\
p_3 &= \alpha_3 p_1 \\
&= 0{,}71\cdot 7{,}85\ kN/m^2 \\
&= 5{,}57\ \ kN/m^2 \\
p_4 &= \alpha_4 p_1 \\
&= 0{,}32\cdot 7{,}85\ kN/m^2 \\
&= 2{,}51\ \ kN/m^2
\end{aligned}
$$

Resulterende horizontale belasting per meter breedte: **:**

$$
\begin{aligned}
P_{Goda} &= \frac{p_1+p_4}{2}\cdot h_c^* + \frac{p_3+p_1}{2}\cdot h' \\
&= \frac{7{,}85+2{,}51}{2}\cdot 1{,}19 + \frac{5{,}57+7{,}85}{2}\cdot 1{,}81 \\
&= 18{,}3\, kN/m^1
\end{aligned}
$$

Figure E.4: Verification of wave load function. Retrieved from Rijkswaterstaat (2019b).

In addition, the wave force is plotted against the water level to assure that the behaviour of the model is still correct in a situation where the water level is above the crest of the gate. This is presented in Figure E.5. Here it is clear that if the water level surpasses the gate height, the wave force decreases (eventually to 0, and no further), as is to be expected.

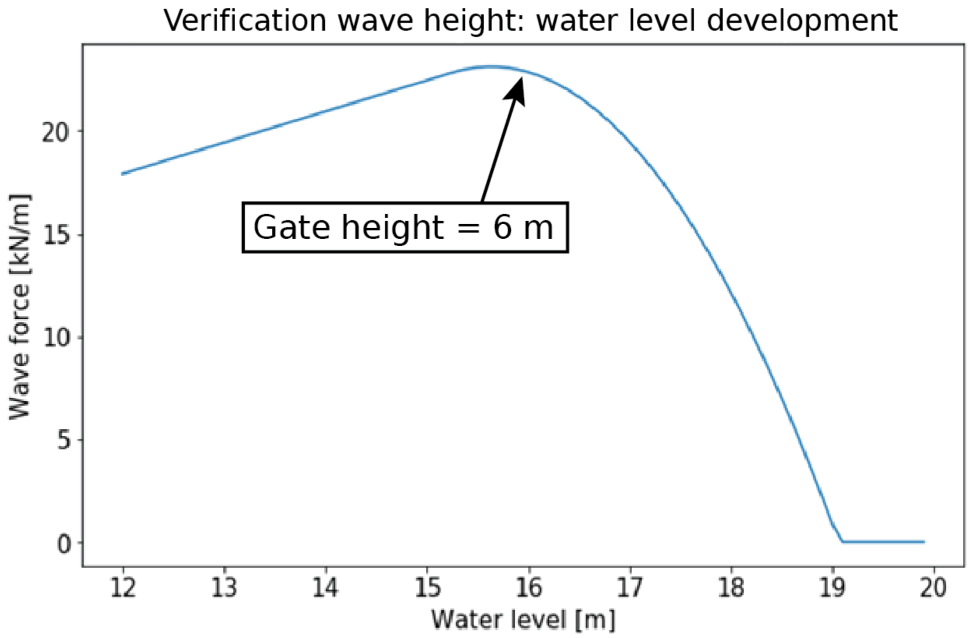


Figure E.5: Verification of wave load over water level development.

The remainder of the function of this failure mechanism consists of rather simple equations, and is therefore not presented here.

Piping

For the piping mechanism, no example is presented in the WBI manuals. Therefore, a self made set of input parameters is drafted, which provide the Z-values presented in the equations below. The script derives identical values when these parameters are implemented.

$$Z_L = \frac{L_h}{3} + L_v - \gamma \cdot C_L \cdot (h_1 - h_2) = 130 - 1.5 \cdot 8.5 \cdot 5 = 66.25 \tag{E.6}$$

$$Z_B = L_h + L_v - \gamma \cdot C_B \cdot (h_1 - h_2) = 330 - 1.5 \cdot 18 \cdot 5 = 195 \tag{E.7}$$

Monte Carlo

Because the Monte Carlo analysis contains the element of randomisation, it cannot be checked by means of a hand calculation. What *can* be checked is whether the derivation of a failure probability from certain Z-values works correctly. From a run of the Monte Carlo function with 50,000 iterations, 6 Z-values below 0 were found for failure mechanism "Failure due to vibration". This coincides with a failure probability of 0.00012, also derived by the script. Failure mechanism "Failure bed protection *given* vibration failure" also had 6 Z-values below 0. Because this is a failure probability *given* another probability, and the amount of Z-values below 0 are equal, the probability should be 1. The script also gives a correct output for this calculation. Thus, the Monte Carlo function is verified.

Fault tree

The fault tree is a rather intricate system of causes and consequences. The entire system is presented and explained in Appendix B. It has to be verified whether the calculations (which are simple individually, but more complex on the large scale) are performed correctly. The script is run to find the failure probabilities at the bottom of the fault trees. With these values, and the standard probabilistic Equations presented in Table 5.1, the fault trees are worked out from bottom to top, as presented in Figure E.6. The top failure probability in the fault trees is derived to be equal to 0.126, which is also the value derived by the fault tree function in the script. Thus, the entire script is verified.

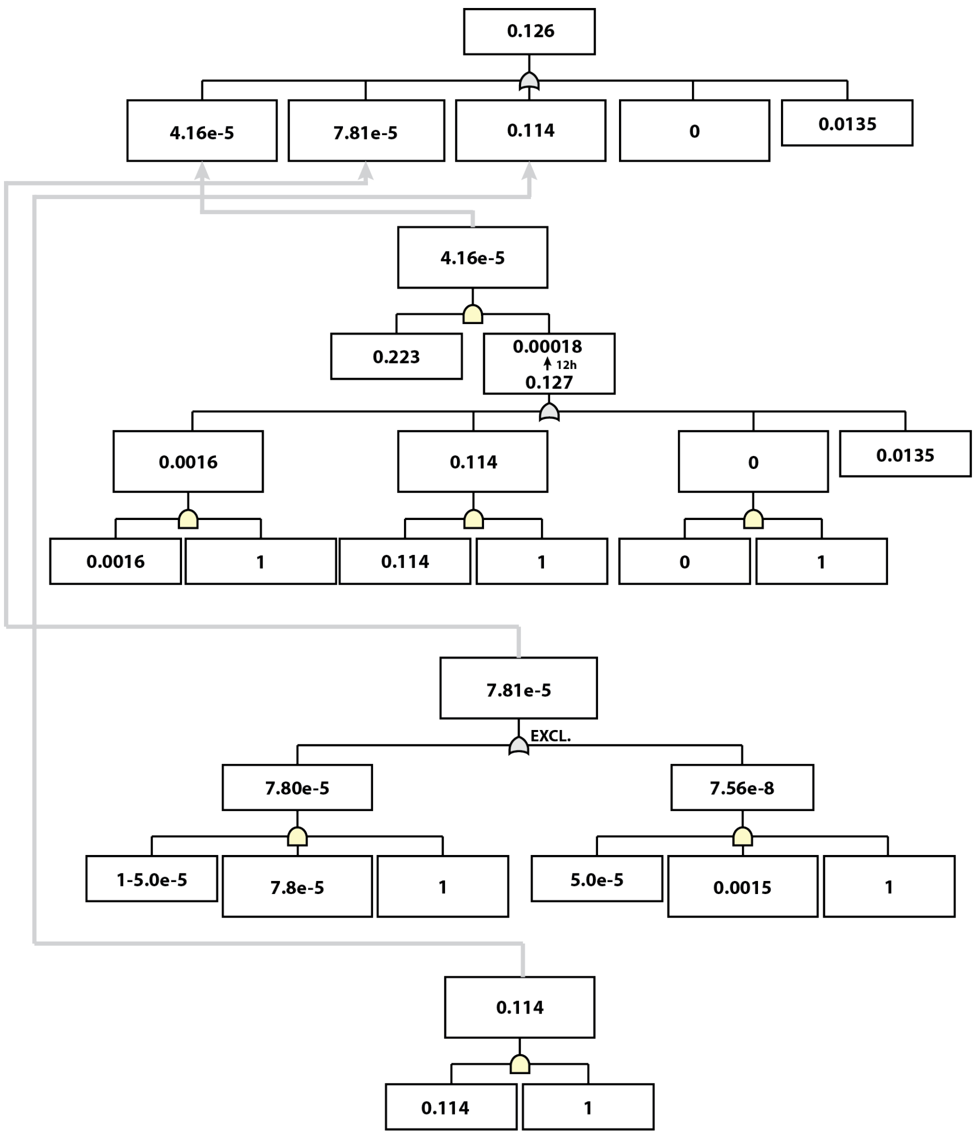


Figure E.6: Filled in and worked out fault trees

F

Adaptation concept application to case study

In this Appendix, the adaptation concepts developed in Section 6.2 are applied to the case study, the Western lock in Terneuzen. Because exact material and element quantities are required to derive cost estimations of the adaptation applications, estimations are made of the dimensions of the components. It should beforehand be noted that these dimensions merely function to make very rough approximations, and are therefore not always well substantiated with detailed calculations.

The Appendix is divided in sections per critical lock characteristic.

F.1. Retention height increment and gate strengthening

The retention height has to be improved for two elements: the lock structure and the gate. The former can be heightened by means of a wall, a fixed extension, or a modular extension. The latter can be heightened by means of a gate extension, gate replacement, or gate module.

F.1.1. Wall

The wall can be applied at the edge of the lock head. Because of its insignificant weight, the addition of this element requires no further strengthening of any lock component. The integration of adaptation at the Western lock in Terneuzen is presented in Figure F.1. The path has a length of approximately 200 m. The wall has a height of 0.5 m, a thickness of 25 cm, and a reinforcement ratio of 1% is implied.

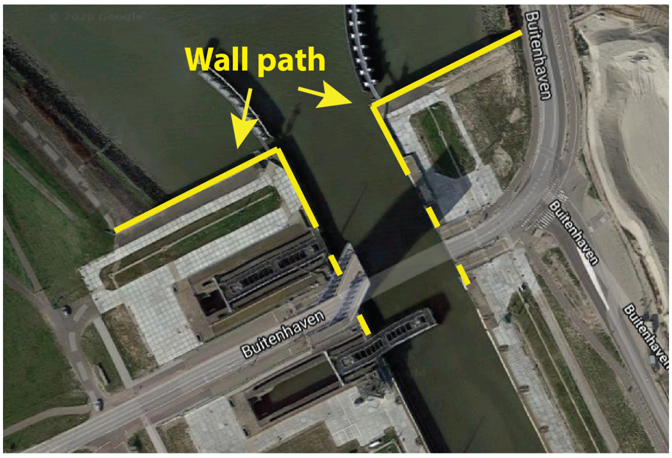


Figure F.1: Wall path at the Western lock in Terneuzen. Retrieved from https://www.google.com/maps (Accessed on October 23, 2020) (edited).

Table F.1: Properties of concept "Wall" applied to the Western lock in Terneuzen

Wall properties	
Length	200 m
Width	0.25 m
Height	0.50 m
Reinforcement ratio	1%

F.1.2. Fixed lock extension

The heightening of the entire lock can be executed either by means of a fixed extension or a modular extension. For each specific case, a choice is to be made: whether only the outer head is heightened, or the entire lock (including the chamber walls and inner head). By means of the former, only the retention height of the lock is improved. However, if only the outer lock head is heightened, and the sea level becomes higher than the original height of the lock (and thus still higher than the chamber walls and inner head), the lock is inoperable. If the outer gates would be opened to let a vessel through, water would flow over the chamber walls, as its height would be too low. To avoid this, the lock should not let any vessels pass if the sea level is higher than the inner elements of the lock. Thus, the operability of the lock would be lowered. However, for the Western lock in Terneuzen, the situation is different. The maximum sea level at which currently leveling is allowed for this lock is NAP+3.5 m (Kemper, 2019). The height of the entire lock (outer head, chamber walls, and inner head) is equal to NAP+6.0 m. This implies that, if the maximum water level at which leveling is allowed is wished to be increased when the sea level rises to attain an equal operability, this requires no heightening of the entire lock structure. It is already tall enough for this purpose. Only at a sea level rise of 2.5 m would heightening of the entire structure become relevant. However, the case study is designed with very high requirements. It even has two roller gates in both the outer lock head, and the inner lock head. It seems that the inner structure has to be deployable as retention element in case the outer head has failed. It is therefore assumed for this thesis, that the entire lock has to be heightened in order to fulfill the requirements.

The concept "Fixed extension" consists of a vertical extension of the current lock heads and walls, an land fill to connect the existing dyke to the new crest of the lock. The integration of the concept onto the site in Terneuzen is presented in Figure F.2. This concept does not include the heightening of the existing dyke, as this is left out of the scope of this project, and when the sea level rises the sea dykes of the Netherlands will have to be heightened regardless of any lock structure. The lock heightening does however have to be connected to the sea dyke. All properties of the application are presented in Table F.2 for a heightening of 1.0 m. For other extension heights, the same values can be used, except of course the height.

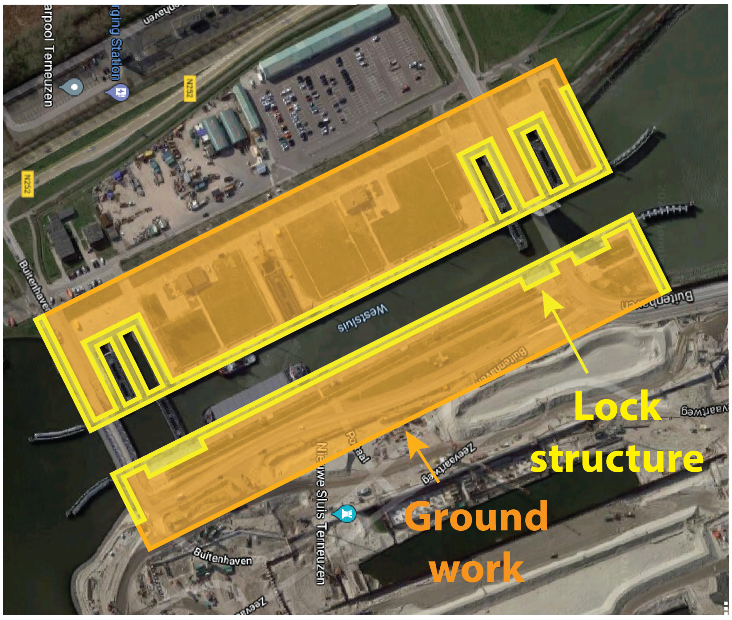


Figure F.2: Fixed extension at the Western lock in Terneuzen. Retrieved from https://www.google.com/maps (Accessed on October 23, 2020) (edited).

Table F.2: Properties of concept "Fixed extension" applied to the Western lock in Terneuzen

Lock extension	
Area extension	5855 m^2
Circumference extension	2930 m
Height extension	1.0 m
Reinforcement ratio	1%
Landfill	
Area	65100 m
Height	1

F.1.3. Modular lock extension

As mentioned in the previous section, the design of the modular lock extension is based on identical principles as the fixed lock extension. Thus, the entire lock is to be heightened, and not just the outer head. The concept consists of an array of modules along the edge of the structure, enclosed by landfill, connected to the existing dykes. Such modules are manufactured by many different companies, and therefore come in a large variety of dimensions. Here is chosen for a length of 1.5 m, a width of 1.0 m, and a height of 0.6 m, because the costs of a module with such dimensions are publicly available (Mholf Bestrating, n.d.). As for the fixed extension, the dyke supplement does not include the existing sea dyke. The properties of the application of this concept to the Western lock in Terneuzen are presented in Table F.3. In case a heightening of for instance 1.0 m is required, an extension of 1.2 m has to be applied, as the modules are 0.6 m tall. Because of the same principle, for a required heightening of 1.5 m, an extension of 1.8 m has to be applied.

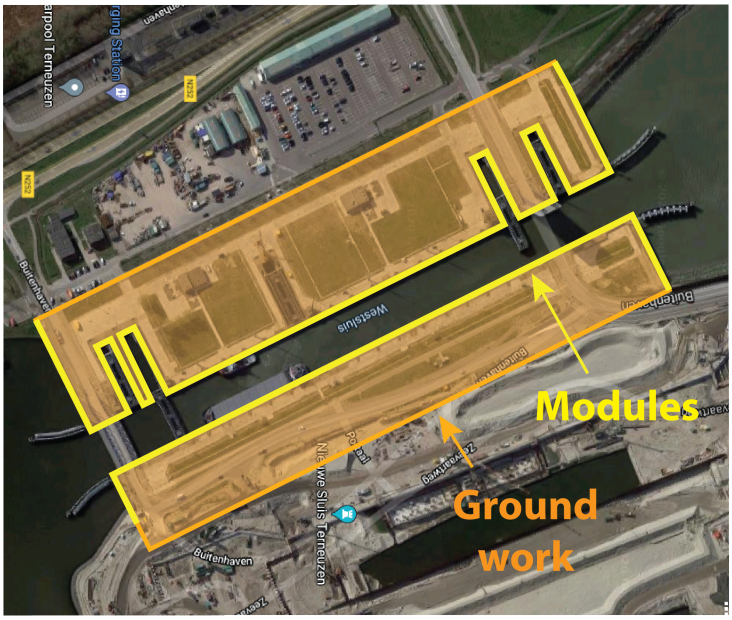


Figure F.3: Modular extension at the Western lock in Terneuzen. Retrieved from https://www.google.com/maps (Accessed on October 23, 2020) (edited).

Table F.3: Properties of concept "Modular extension" applied to the Western lock in Terneuzen

Lock extension	
Length module array	1600 m
Module length	1.5 m
Module height	0.6 m
Landfill	
Landfill area	65120 m^2
Landfill height	1.2 m

F.1.4. Crossing bridge elevation

At the Western lock in Terneuzen, two bridges are located, i.e. one at each lock head. The bridge at the outer lock head is placed in between the two roller gates. Therefore, if the lock is heightened, this bridge also has to be placed at a higher level. For a small heightening, i.e. the 0.5 m tall wall and gate extension, the bridge could be closed off with a temporary barrier in case of high water. However, for the more severe heightening adaptations, this becomes insufficient. Thus, the bridge has to be levelled along with the rest of the lock. At the inner lock head, the bridge is not placed in between the roller gates, but outside the general lock structure, at the side of Canal Ghent-Terneuzen. It therefore does not necessarily require to be heightened along with the structure.

To level the bridge, it is not required to fabricate a complete new one. The original bridge can be lifted out of the construction, temporarily located at a near site, and later placed back onto position after the lock and infrastructure have been heightened. It is assumed that the operating mechanism can also be reused, but a new chamber has to be constructed at a higher level, in which the mechanism can be seated. Obviously two new abutments are to be made for the bridge to rest on, a new road has to be constructed to connect the new elevation of the bridge to the existing infrastructure. It is assumed that the road can be constructed with a slope of 8%. An elevation of 1 m thus requires a new road with a length of 1250 m. On the west side of the lock this is possible, but on the east side, the newly constructed lock is far closer than 1250 m. However, if the Western lock requires a heightening, than so will a large portion of the area between these two locks, as it functions as a dyke. Therefore, at the eastern side, instead of 1250 m, an estimation is made of the length of the road that will be placed between the Western lock and the new lock: 250 m. For the entire road, a width of 8 m is assumed. In addition, a roundabout is currently an element of the infrastructure on the west side of the lock, which is well within 1250 m of the bridge, and therefore also requires a replacement.

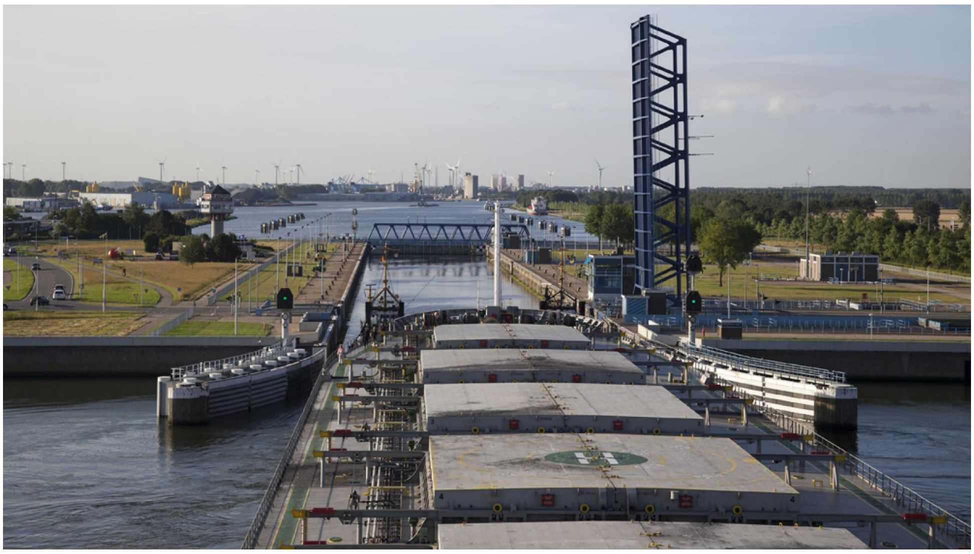


Figure F.4: Picture of opened bridge, taken from vessel entering the Western lock in Terneuzen. Retrieved from https://nieuwesluisterneuzen.eu/nieuws/2017-09/akkoord-over-nieuwe-sluis-terneuzen (Accessed on November 20, 2020).

Table F.4: Properties of concept "Crossing bridge elevation" applied to the Western lock in Terneuzen

Crossing bridge	
Length bridge	40 m
Elevation	1 m
Road	
Maximum slope road	8%
Length new road	1500 m
Width road	8 m
Roundabout	1

F.1.5. Lock structure strengthening

Because increasing the lock height with a fixed or modular extension increases both the weight, and the possible horizontal stress on the structure, the lock head, chamber walls, and foundation may have to be strengthened. Determining whether a construction has surplus in its structural capacity to withstand these additional forces requires extensive information on the construction. Because such details are not readily available, it is assumed that the case study, the Western lock in Terneuzen, has no such surplus in its design. However, this does not necessarily entail that all the before mentioned components require a strengthening adaptation.

Because for this lock both the lock heads and the chamber walls are heightened (as explained in Section F.1.2), these components also require support. The foundation of this case study does not have to be improved. The lock is installed with a shallow foundation. For this type of foundation, the critical load is often not the downward weight during its lifetime, but the upward directed pore pressure during the construction period, when the construction pit is emptied and contains no water (Molenaar & Voorendt, 2018).

To strengthen the lock heads, a sheet pile wall is to be installed alongside the outer lock head, and diagonal bearing piles are to be driven and connected to the head. From the reference project, the Southern lock in IJmuiden, dimensions of the piles and their spacing is taken (Beem et al., 2000). The structure is presented in Figure F.5. It is assumed that the strongest sheet piles available at GWWkosten.nl Cobouw (n.d.) is required (this website is used in Appendix I to derive adaptation costs), i.e. type PU32E, with a length of

15 m, stretching to the bottom of the lock structure. Using the sketch of the strengthening of the lock in IJmuiden, presented in Figure 6.10, and the available pile dimensions at GWWkosten, it is assumed that piles of a length of 23 m, a diameter of 1220 mm, and a thickness of 12.7 mm can be used (the largest available, and comparable to those used in the reference), with a spacing of 3 m. For the strengthening of the chamber walls, identical sheet piles are used alongside the structure.

It should be noted that, because these material indications are merely based on a reference, and not on calculations, the actually required elements may differ considerably when applied in reality. These assumptions function mostly as indication to be able to estimate the costs of the adaptation in Appendix I.

Figure F.5: Strengthening of lock head of the Southern lock in IJmuiden. Retrieved from (Beem et al., 2000).

Table F.5: Properties of concept "Lock structure strengthening" applied to the Western lock in Terneuze

Lock head (2x)	
Sheet pile width	122 m
Sheet pile depth	15 m
Sheet pile type	PU32E
Spacing bearing piles	3 m
Length bearing piles	23.0 m
Diameter bearing piles	1220 mm
Thickness bearing piles	12.7 mm
Chamber walls	
Sheet pile width	320 m
Sheet pile depth	15 m
Sheet pile type	PU32E

F.1.6. Gate extension, gate replacement, and gate module

In addition to the vertical extension of the lock, the roller gates located in the outer and inner head have to be heightened, and the operating mechanism has to be adjusted if necessary. The gates can be heightened in three manners, by means of a simple gate extension with a height of only 0.5 m, the replacement of the entire gate, or the addition of a gate module, for which the gates would formerly have to be designed. The last option therefore can only be applied to a specifically designed gate (which may also be entirely modular). The gate extension can only be applied for a sea level rise up to 0.5 m, as a taller extension would allow for relatively large forces on the component, for which it has no proper structural capacity. The last two options can be applied for a larger sea level rise than 0.5 m. For the gate replacement and gate module, the width and depth of the components are equal to those of the gates currently in place.

If a taller and stronger gate is installed with a larger weight, the gate rails have to carry a larger load. This causes the steel to wear down faster over the years (Beem et al., 2000). Whether this shortens the lifetime of the rails so much that the rails would have to be replaced before the end of the lifetime of the entire structure, requires detailed knowledge on these elements. For this thesis, it is assumed that the installation of a taller gate also requires a better support system, with the exception of the gate extension, of which the weight is assumed to be negligible.

F.2. Elongation piping length

The piping path of the Western lock in Terneuzen seems to be designed for a head difference of over 13 m, as derived in Section 5.4.4. This coincides with an emptied, or nearly emptied channel, depending on the sea level. This may have been required during the construction of the lock. It is unnecessary to further elongate the piping length, as it is already longer than required for any head difference occurring under regular circumstances. This component is therefore henceforward disregarded for the case study.

F.3. Improvement bed protection

The bed protection can be improved in two manners: by penetrating the top layer of the composition with colloidal concrete, or by replacing the entire bed protection. Both options are valid for the case study.

F.3.1. Replacement bed protection

For this concept, the existing bed protection has to be removed, after which a new composition with a top layer with a larger nominal diameter is placed on the same location. To understand the extensiveness of this operation, an estimation of the dimensions of the composition has to be made, i.e. the bed protection length, height, and density. In an alternative study for the adaptation of the bed protection of the Southern lock in IJmuiden, a method is described to derive the required length of a bed protection (Ministry of Transport and Water Management, 1992). The length is based on the maximum scour hole depth which is expected to occur, and the slope of the soil slide which might be caused by that hole. As long as the scour hole will develop at a distance far enough from the lock, an occurring soil slide will not endanger the stability of the lock.

$$L = H_{max} \cdot (n_a + n_u) \tag{F.1}$$

Where:

Symbol	Description	Value	Unit
L	Bed protection length	output	m
H_{max}	Maximum scour hole depth	input	m
n_a	Leading slope	4	m/m
n_u	Trailing slope	4	m/m

The maximum expected depth of a scour hole is based on the maximum flow velocity which is accounted for in the bed protection design, and the effective time of this flow. After a high flow velocity has occurred, it can be checked whether a large scour hole has developed, and mitigation can be executed. Therefore, only the effective time of a single extreme event has to be regarded.

Here, as extreme event, a situation is taken into account at which water is able to flow freely through the lock at a head difference occurring with a yearly probability of 1:1000, i.e. the norm of the Western lock in Terneuzen (Dutch Ministry of Transport, Public Works and Water Management, 2017c). A sea level with this occurance is NAP+5.5 m (see Figure 5.18), and the channel level is kept approximately stationary at a level of NAP+2.1 m. Because of the backup gates of the lock, it can be assumed that such a situation will be managed within short period of time. An effective time of 0.25 hours is considered. In Equation F.2, the critical flow velocity concerns the subsoil, and not the top layer of the bed protection.

$$H_{max} = (\alpha_1 \cdot U - u_{cr})^{1.7} \cdot h_0^{0.2} \cdot T_{eff}^{0.4} / (10 \cdot \Delta^{0.7}) \tag{F.2}$$

Where:

Symbol	Description	Unit	Value
H_{max}	Maximum scour hole depth	output	m
α_1	Local factor	4	-
U	Flow velocity	input	m/s
u_{cr}	Critical flow velocity	0.5	m/s
h_0	Water depth channel	12.1	m
T_{eff}	Effective scour time	0.25	h
Δ	Relative rock density	1.65	-

To derive the flow velocity at the end of the bed protection, where the scour hole develops, the water levels as discussed above are applied to Equation F.3. The flow velocity at the end of the bed protection is not equal at the flow velocity at the outlet, as the flow diverges with an angle of approximately 1:5 (see Figure C.4). This is captured in the fraction of Equation F.3. The distance from the outflow at which the velocity is to be calculated, is equal to the bed protection length. Thus, deriving this parameter is an iterative process.

$$U = \sqrt{2g\Delta H} \cdot \frac{W_{lock}}{min(W_{channel}\ ;\ W_{lock} + x \cdot 0.2)} \tag{F.3}$$

Where:

Symbol	Description	Unit	Value
U	Flow velocity	output	m/s
ΔH	Head difference	3.4	m
x	Distance from outflow	L	m
W_{lock}	Lock chamber width	40*	m
$W_{channel}$	Channel width	140**	m

* Retrieved from Kemper (2019).
** Retrieved from *www.rijkswaterstaat.nl/water/vaarwegenoverzicht/kanaal-van-gent-naar-terneuzen* (last accessed on 23-10-2020).

Following these steps, one finds a bed protection length of 138 m, here conservatively rounded up to 140 m.

Although there might be a granular filter layer below the top layer of the bed protection, it can be assumed that the height of the bed protection is mainly determined by the diameter of its top layer. It was derived that for the Western lock in Terneuzen, a standard sorting of 15-300 kg should be in place with a D_{n50} of 38 cm. The thickness of a layer should be at least twice its D_{n50}, i.e. 76 cm (Capel, 2015). In addition, it can be assumed that the bed protection has a porosity of 0.38, meaning that with a rock density of 2650 kg/m^3, the weight of the layer is 1643 kg/m^3. The denominator of the fraction in Equation F.3 concerns the width of the flow at the end of the bed protection, and is equal to 68 m in the regarded extreme event. It is conservatively assumed that the width of the bed protection is equal to 75 m over the entire length of the composition.

The bed protection has to be replaced by a new one when the sea level rises. In case a sea level rise of 1.0 m is regarded, a grain diameter of at least 53 cm is required according to Equation C.10. The smallest standard sorting with a diameter larger than 53 cm is 300-1000 kg, with a minimal D_{n50} of 62 cm (Capel, 2015). The thickness of this layer should be 114 cm. Following the same steps as before, the required bed protection length can be calculated to be equal to 156 m, here rounded up to 160 m, whilst the original width of 75 m remains sufficient.

Table F.6: Properties of concept "Replacement bed protection" applied to the Western lock in Terneuze

	Original bed protection	**New bed protection**
Bed protection length	140 m	160 m
Bed protection width	75 m	1.14 m
Bed protection height	0.76 m	0.76 m
Layer density	1643 kg/m^3	1643 kg/m^3

F.3.2. Colloidal concrete

In this concept, the top layer of the bottom protection is penetrated with colloidal concrete. The density of this concrete is equal to 2300 kg/m^3 (Betonhuis, n.d.). The volume of the concrete can be expected to be roughly equal to the volume of the pores in between the rocks of the top layer, i.e. 38% of the total volume (Capel, 2015). It can additionally be assumed that roughly 20% of the concrete will be lost due to a present water flow during poring (Ministry of Transport and Water Management, 1992).

Because the flow velocity decreases with the distance from the outlet, the penetration with colloidal concrete is not required over the full length of the bed protection. By rewriting Equation F.3, one can derive the distance x from the outlet at which the flow velocity caused by a new design head difference has decreased to the critical flow velocity of the original bed protection.

$$x_{(U=u_{cr})} = 5 \cdot W_{lock} \left(\frac{\sqrt{2g\Delta H}}{u_{cr}} - 1 \right) \tag{F.4}$$

Filling in a lock width of 40 m, a head difference of 4.4 m (1.0 m larger than for the derivation of the original bed protection), and a critical velocity of 4.55 m/s, Equation F.4 provides a distance x of 28 m. Thus, the colloidal concrete would only have to be applied to the initial 28 m behind the lock in order to prevent erosion. To be conservative, a distance of 50 m is applied here.

Table F.7: Properties of concept "Colloidal concrete" applied to the Western lock in Terneuze

	Colloidal concrete
Colloidal concrete length	50 m
Colloidal concrete width	75 m
Height bed protection top layer	0.76 m
Porosity bed protection top layer	0.38
Density colloidal concrete	2300 kg/m^3
Concrete loss	20%

G

Grading derivation of adaptation criteria evaluation

In this Appendix, the grading of the adaptation concepts applied to the case study in Terneuzen, presented in Section 6.3.2 is thoroughly explained. The evaluation is performed based on five criteria: the construction time, construction nuisance, familiarity of the procedure, sustainability, and circularity. In each Section of this Appendix, a single criterion is treated. In the last Section, G.6, the evaluation system which is applied to compare the individual adaptation concepts is reworked to a system which can be applied to the adaptive pathways maps. This system is used in Sections 6.6.1 and 7.4.1.

G.1. Construction time

The construction time is a very important aspect for a project regarding an existing lock, as the lock will (often) not be operable during this period. The loss of costs due to inoperability adds upon the regular construction costs. For this thesis, only the construction time at the site is regarded, as time spent elsewhere is not of hindrance to the operability of the lock, and sea level rise is a slow process for which fast handling is not required. Among elements of the construction time are preparation of the site, fabrication and installation of components (in case fabrication is executed on site), finishing off the structure and making the site user-ready.

A rating system has to be implemented, based on the properties of a specific project. As mentioned, the Western lock in Terneuzen is part of a larger complex with two other locks. Because during the adaptation of the Western lock, these other locks are still available for transportation. On the other hand, the lock complex forms the main entrance for the harbour of Antwerpen, and is therefore of great significance for its economy. Inoperability should thus not continue for a long stretch of time. A construction period longer than a year is deemed unacceptable.

- **Very good**: T $\leq$ 3 months
- **Good**: 3 months $<$ T $\leq$ 6 months
- **Bad**: 6 months $<$ T $\leq$ 1 year
- **Very bad**: 1 year $<$ T $\leq$ 2 years
- **Unacceptable**: T$>$2 year

Retention height

The first heightening concept is the short wall. The fabrication and installation of this concrete construction do not require much time. Additionally, for this evaluation only the time spent at the site is of importance. The installation and inspection of quality of these elements can be assumed not to take longer than a single month, according to F. Verschoor (personal communication, October 6, 2020).

The fixed extension of the lock head and chamber walls, inclusing an elevation of the outer bridge, is obviously a much more intensive and time consuming adaptation than the wall. For the entire construction time, including preparation of the site, fabrication, clearance and inspection, an estimated construction time of 1 year is suggested (Verschoor, 2020). This does not include the design period and the arrangement of permits.

For the modular extension, a similar period can be expected to be necessary. The main difference between the two methods is the fact that the hardening of the concrete of the modules takes place off site, while other tasks can take place. In addition this method includes the placement of more landfill, which is not required for the fixed extension. Overall it can be assumed that a 1 year construction period is also appropriate for the modular extension (Verschoor, 2020).

In case the adapted lock is a weak adaptive one, the lock head will have to be strengthened by means of sheet piles and diagonal piles. This would have to be applied both for the fixed and modular extension. The construction period of this procedure is estimated to be 6 months (Verschoor, 2020), prolonging both periods to 1.5 years in total.

Three manners to improve the retention height of the gate are analysed: a small gate extension, a large gate module, or a full gate replacement. All these elements can be fabricated off-site. The only construction period which is accounted for here is the attachment of the element, or the replacement of the gate. This is all estimated to take less than a month to execute.

Bed protection composition

To improve the bed protection composition, it can either be completely replaced, or the top layer can be penetrated with colloidal concrete. For the former method, the entire bed protection has to be removed and placed again. It is expected that this will take 2 months. The colloidal concrete can be applied much faster, and is estimated to have a construction period of 1 month (Verschoor, 2020).

Table G.1: Grading of construction period. A green cell indicates “very good”. Yellow indicates “good”. Orange indicates “bad”. Red indicates “very bad”. Purple indicates “unacceptable”.

Characteristic	Retention height						Bed pr.	
	Lock height			Gate				
Concept	Wall	Fixed ext.	Modular ext.	Gate extension	Gate replacement	Gate module	Colloidal concrete	Replacement
Construction time	<1 m	1 y	1 y	<1 m	<1 m	<1 m	1 m	2 m
Including lock support:		1.5 y	1.5 y					

G.2. Construction nuisance

The activities at a construction sight can cause nuisance to the residents of its neighborhood and users of the lock. This can come in the following forms:

- **“Dry” traffic** can be **obstructed** if the construction site has to be expanded onto existing roads, or if an important traffic way crosses over the lock.
- **Inoperability** of a locks follows when the construction procedures make it dangerous for vessels to pass the lock, and in case the construction has to be performed from and in the water.
- **Noise pollution** is caused by machinery used at a construction site.
- **Vibration disturbance** is caused by pile driving.
- **Light pollution** concerns artificial light used during the night, which may cause nuisance to people, but especially to wildlife.

The more of these nuisance criteria apply to a construction concept, the lower its score in the evaluation. Thus, the grading system is adopted:

- **Very good**: 1 nuisance criterion
- **Good**: 2 nuisance criteria
- **Bad**: 3 nuisance criteria
- **Very bad**: 4 nuisance criteria
- **Unacceptable**: 5 nuisance criteria

In the following section, it is discussed per adaptation concept in what manner these cause nuisance to the area and its residents.

Retention height

The installation of the wall is a very simple and quick operation, and only causes nuisance to passing vessels, as the lock will be inoperable during the construction. Although the lock technically does not have to be closed for this procedure, it is deemed unsafe to have vessels pass while construction is taking place.

The fixed and modular extension, however, can cause nuisance in multiple manners. Roads cross the Western lock in Terneuzen, which have to be closed off during construction of these elements. Also during the construction period, the lock will have to be closed for vessels for at least a period of time in which construction is executed close to the waterway. Obviously hazards could occur in case materials or machinery are dropped over the edge of the lock head while vessels are present. In addition, for a weak adaptive lock, sheet piles and diagonal piles have to be installed to strengthen the lock head and chamber walls. The driving of sheet piles causes both noise pollution and vibration disturbance. Light pollution can always be avoided if regulations are followed strictly.

For the replacement or modification of the gate, it is assumed that during construction, "dry" traffic cannot cross the bridge, as the adaptation is executed from the water. This obviously also means that the lock is inoperable during that time.

Bed protection composition

Penetrating a bed protection with colloidal concrete gives barely any nuisance. Only the waterway is obstructed during the construction period, as the bed protection is positioned beneath it. The replacement of the bed protection also obstructs the waterway, causing inoperability, but does not cause any other type of nuisance.

Table G.2: Grading of construction nuisance. A green cell indicates "very good". Yellow indicates "good". Orange indicates "bad". Red indicates "very bad". Purple indicates "unacceptable".

	Retention height							
Characteristic	Lock height			Gate			Bed pr.	
Concept	Wall	Fixed ext.	Modular ext.	Gate extension	Gate replacement	Gate module	Colloidal concrete	Replacement
"Dry" traffic obstruction		x	x	x	x	x		
Inoperability lock	x	x	x	x	x	x	x	x
Noise pollution		x*	x*					
Vibration disturbance		x*	x*					
Light pollution								
Grading								
Including lock support:								

* These nuisances will only occur during the execution of a fixed extension or modular extension for a weak adaptive lock, because sheet piles and diagonal piles are required to strengthen the structure.

G.3. Familiarity procedure

The familiarity of a procedure can be an important aspect to a construction period. If unknown techniques are used, this will cost more time both in the design phase and the construction phase (and maybe even during the lifetime of a structure due to unforeseen or unknown maintenance operations). Initially it costs more time to determine how new concepts can be applied to a design, and develop the method of execution. Sequentially, when executing an unfamiliar procedure, the probability of mistakes is higher. Thus, it is more likely that time has to be spent mitigating or fixing those errors.

The following grading system is applied:

- **Very good**: Frequently applied procedure
- **Good**: Common procedure
- **Bad**: Uncommon application of common procedure
- **Very bad**: Unproven application of common procedure
- **Unacceptable**: Unknown procedure

In principle all drafted concepts are based on existing procedures. The wall, fixed extension, gate replacement, colloidal concrete and bed protection replacement are even widely used and very common. The modular extension of the lock structure is based on the common procedure of stacking interlocking blocks, but this technique has not been applied to a lock before. The gate extension is common use, but not applied too often, while the modification of the gate with an entire additional module has never been applied, and is therefore not proven. The module can however be designed using the same techniques as an ordinary gate, and is therefore not a completely unknown procedure. The method used to install the lock strengthening, i.e. driven of sheet piles and diagonal piles, is frequently used, but because this is executed directly next to the existing lock structure, caution has to be taken with the accompanied vibrations. For screens of the required height, pressing the sheet piles into the ground is however not optional, so this cannot be avoided (Verschoor, 2020).

Table G.3: Grading of familiarity procedure. A green cell indicates "very good". Yellow indicates "good". Orange indicates "bad". Red indicates "very bad". Purple indicates "unacceptable".

Characteristic	Retention height						Bed pr.	
	Lock height			Gate				
Concept	Wall	Fixed ext.	Modular ext.	Gate extension	Gate replacement	Gate module	Colloidal concrete	Replacement
Familiarity procedure	very good	very good	bad	good	very good	very bad	very good	very good
Including lock support:		good	bad					

G.4. Sustainability

Sustainability is a broad term concerning many different facets. Harm can be done to nature and to ourselves in a lot of manners, which makes it difficult to judge a concept's sustainability with quick reasoning. To sufficiently evaluate sustainability, all different aspects must be made comparable to one another. Therefore, methods have been developed to quantify the influence of a construction on the various sustainability facets. For this thesis, the program DuboCalc is used, which has been created by Rijkswaterstaat to derive environmental effects of constructions, taking the entire life cycle into consideration (Rijkswaterstaat, n.d.-b).

DuboCalc regards the following criteria in its derivations:

- Smog
- Acidification
- Eutrophication
- Ozone depletion
- Climate change
- Depletion of abiotic resources

- Human toxicological effects
- Ecotoxicological - terrestrial
- Ecotoxicological - aquatic fresh water
- Ecotoxicological - aquatic marine
- Depletion of fossil energy carriers

The magnitude of each criterion is measured in an equivalent unit. For instance, climate change is expressed in kg CO_2-eq. Within this single unit, all greenhouse gasses are expressed, as the effect of each gas is scaled to its CO_2 equivalent.

DuboCalc holds an inventory of the quantities of all damaging substances that are produced or released when mining, transporting, and manufacturing a large variety of materials and construction elements. By entering the volume or tonnage of the different components of a construction into the program, DuboCalc can calculate the equivalent units that are emitted. As the entire life cycle of a product is taken into consideration, the emissions are split up in costs made during construction, usage, maintenance, and the end of the lifetime.

Finally, the tool derives the societal costs of each different criterion by means of characteristic values implemented in the program. Thereby, the total environmental impact of each concept can be expressed in a single number, i.e. the Environmental Cost Indicator (ECI).

A higher ECI is obviously rated with a lower score. Obviously the client has to indicate how much societal impact they allow from their construction. For this case study, an ECI of over a million euros is regarded as unacceptable. Thus, the following grading system is derived:

- **Very good**: ECI < €50M
- **Good**: €50M < ECI < €100M
- **Bad**: €100M < ECI < €500M
- **Very bad**: €500M < ECI < €1MM
- **Unacceptable**: ECI > €1MM

The properties and volumes of materials derived in Appendix F are consecutively implemented in DuboCalc. Figure G.1 provides an overview of the costs that have been derived. The contents of this graph are explicated in Sections G.4.1 through G.4.9.

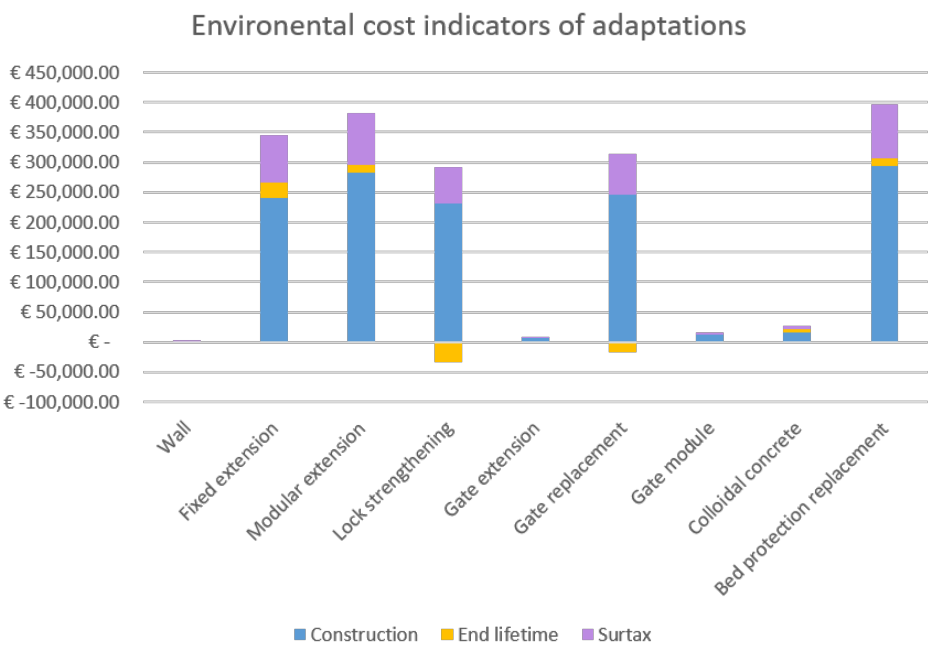


Figure G.1: Overview of environmental cost indicators of adaptations at the Western lock in Terneuzen. The data is derived using DuboCalc.

By combining the ECI values presented in Figure G.1 with the grading system derived above, the sustainability of the concepts can be evaluated:

Table G.4: Grading of sustainability. A green cell indicates "very good". Yellow indicates "good". Orange indicates "bad". Red indicates "very bad". Purple indicates "unacceptable".

Characteristic	Retention height						Bed pr.	
	Lock height			Gate				
Concept	Wall	Fixed ext.	Modular ext.	Gate extension	Gate replacement	Gate module	Colloidal concrete	Replacement
Sustainability								
Including lock support:								

G.4.1. Wall

The total environmental cost indicator of concept "Wall" is equal to €830. From this, €411 is produced during construction, €228 is produced at the end of the lifetime, as the potential of the materials is then fully used up, and €192 is surtax. Figure G.2 presents from what ecological criteria the costs originate.

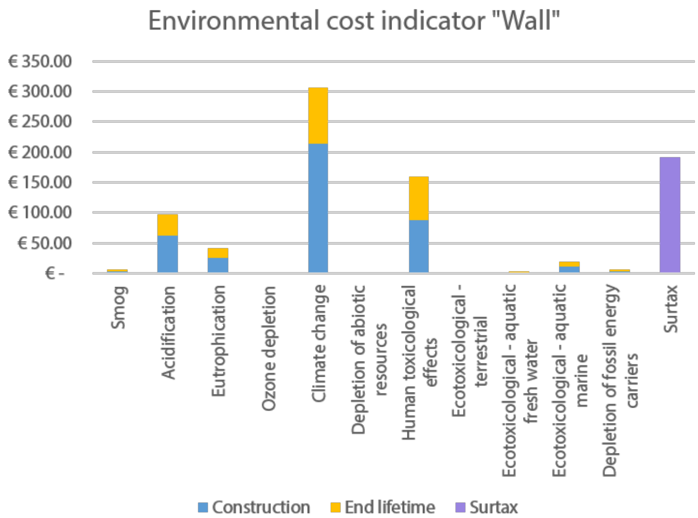


Figure G.2: Environmental cost indicator of concept "Wall" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.2. Fixed extension

The total environmental cost indicator of concept "Fixed extension" is equal to €345,395. From this, €240,285 is produced during construction, €25,402 is produced at the end of the lifetime, as the potential of the materials is then fully used up, and €79,708 is surtax. Figure G.3 presents from what ecological criteria the costs originate.

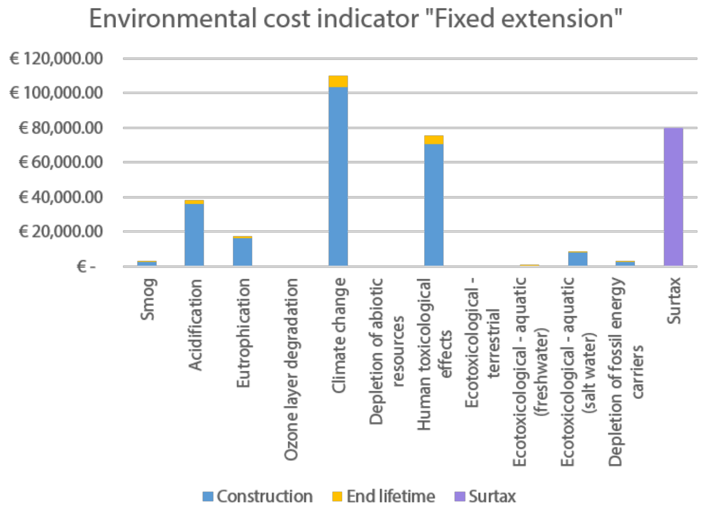


Figure G.3: Environmental cost indicator of concept "Fixed extension" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.3. Modular extension

The total environmental cost indicator of concept "Fixed extension" is equal to €381,414. From this, €283,252 is produced during construction, €12,279 is produced at the end of the lifetime, as the potential of the materials is then fully used up, and €85,882 is surtax. Figure G.4 presents from what ecological criteria the costs originate.

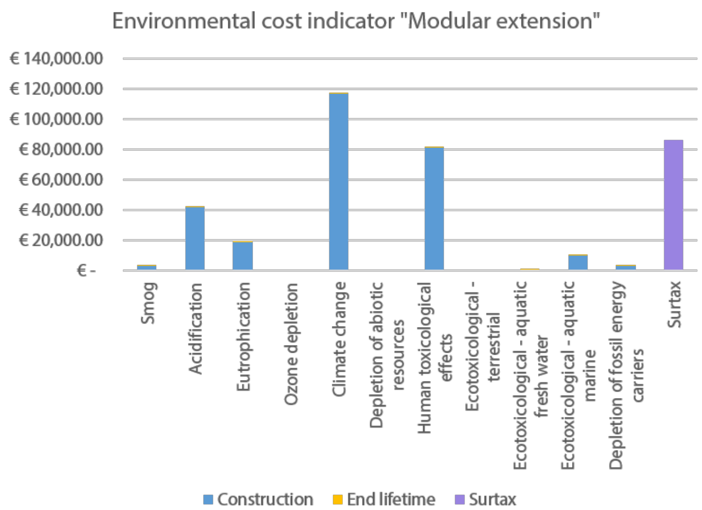


Figure G.4: Environmental cost indicator of concept "Modular extension" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.4. Gate extension

The total environmental cost indicator of concept "Gate extension" is equal to €7,693. From this, €6,348 is produced during construction, €-430 is produced at the end of the lifetime, as some of the steel can be recycled, and €1,775 is surtax. Figure G.5 presents from what ecological criteria the costs originate.

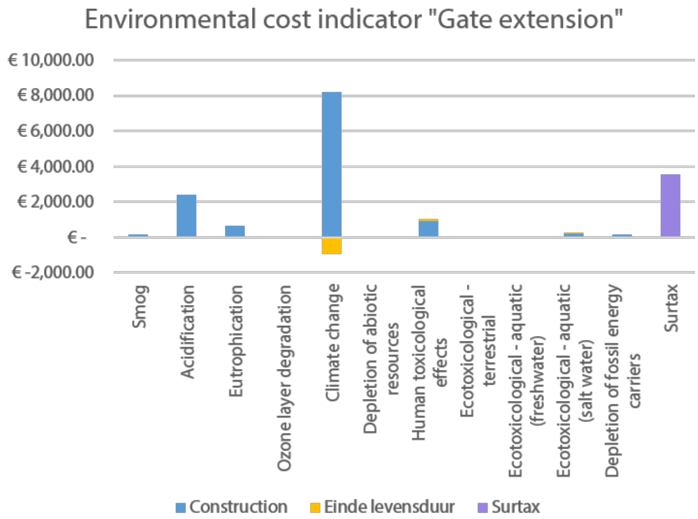


Figure G.5: Environmental cost indicator of concept "Gate extension" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.5. Gate replacement

The total environmental cost indicator of concept "Gate replacement" is equal to €296,967. From this, €245,021 is produced during construction, -€16,584 is produced at the end of the lifetime, as some of the steel can be recycled, and €68,531 is surtax. Figure G.6 presents from what ecological criteria the costs originate.

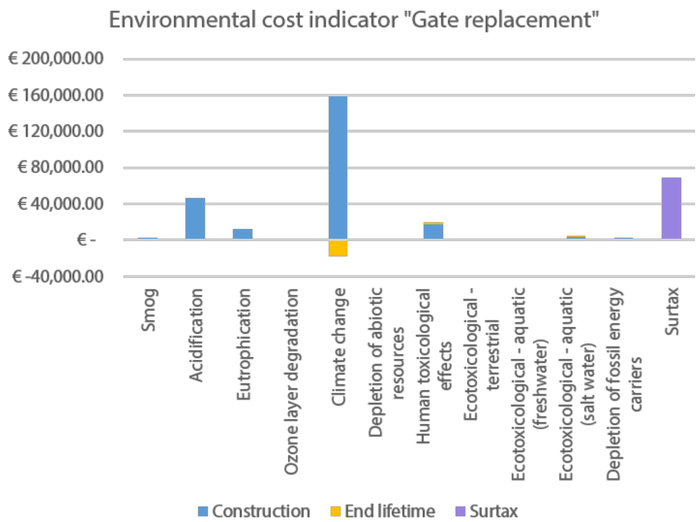


Figure G.6: Environmental cost indicator of concept "Gate replacement" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.6. Gate module

The total environmental cost indicator of concept "Gate module" is equal to €15,387. From this, €12,695 is produced during construction, €-859 is produced at the end of the lifetime, as some of the steel can be recycled, and €3,551 is surtax. Figure G.7 presents from what ecological criteria the costs originate.

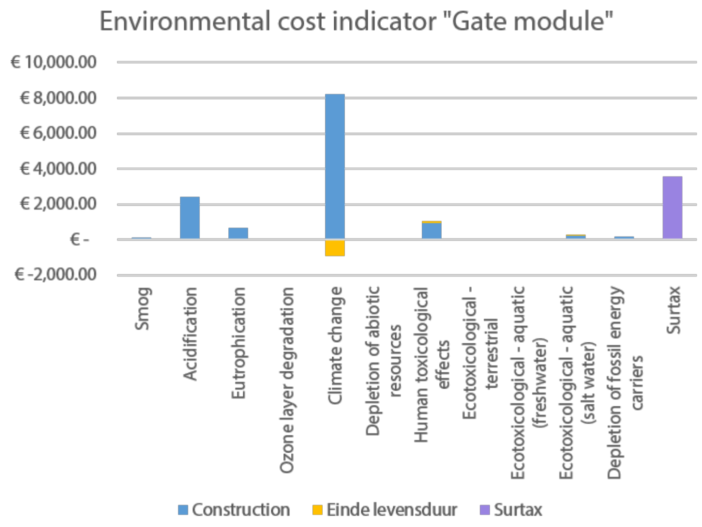


Figure G.7: Environmental cost indicator of concept "Gate module" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.7. Lock strengthening

The total environmental cost indicator of concept "Lock strengthening" is equal to €257,632. From this, €231,672 is produced during construction, €-33,492 is produced at the end of the lifetime, as some of the steel can be recycled, and €59,452 is surtax. Figure G.8 presents from what ecological criteria the costs originate.

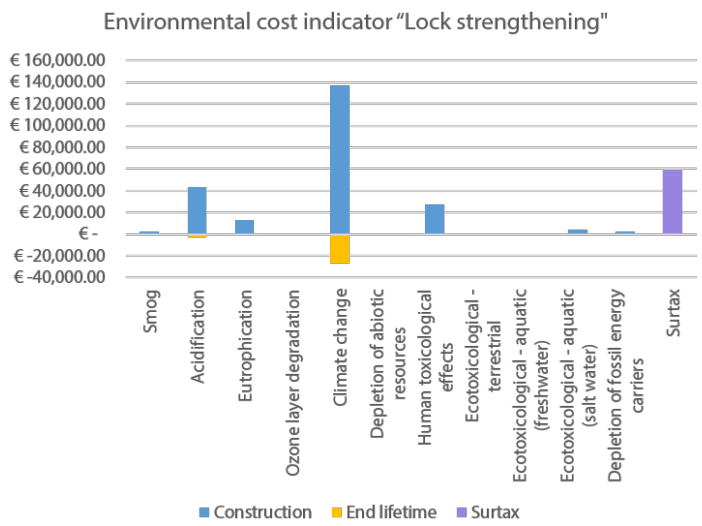


Figure G.8: Environmental cost indicator of concept "Lock strengthening" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.8. Colloidal concrete

The total environmental cost indicator of concept "Colloidal concrete" is equal to €27,415. From this, €16,442 is produced during construction, whereas only €4,646 is produced at the end of the lifetime, as the potential of the materials is then fully used up, and €6,326 is surtax. Figure G.9 presents from what ecological criteria the costs originate.

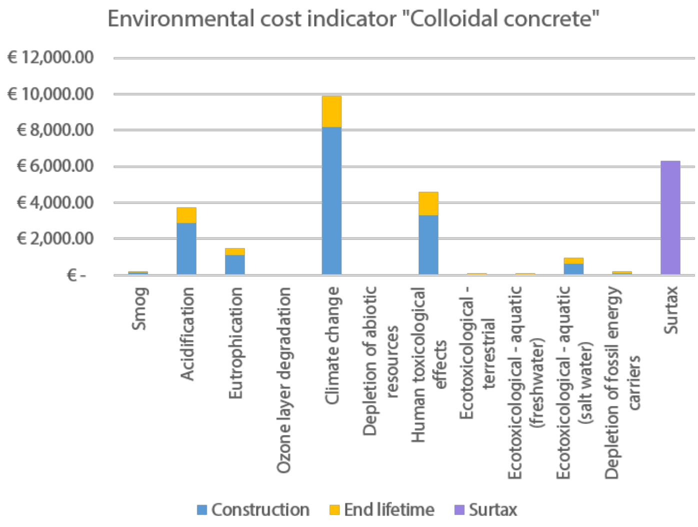


Figure G.9: Environmental cost indicator of concept "Colloidal concrete" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.4.9. Bed protection replacement

The total environmental cost indicator of concept "Bed protection replacement" is equal to €397,494. From this, €293,335 is produced during construction, whereas only €12,382 is produced at the end of the lifetime, as some of the material can be reused, and €91,777 is surtax. Figure G.10 presents from what ecological criteria the costs originate.

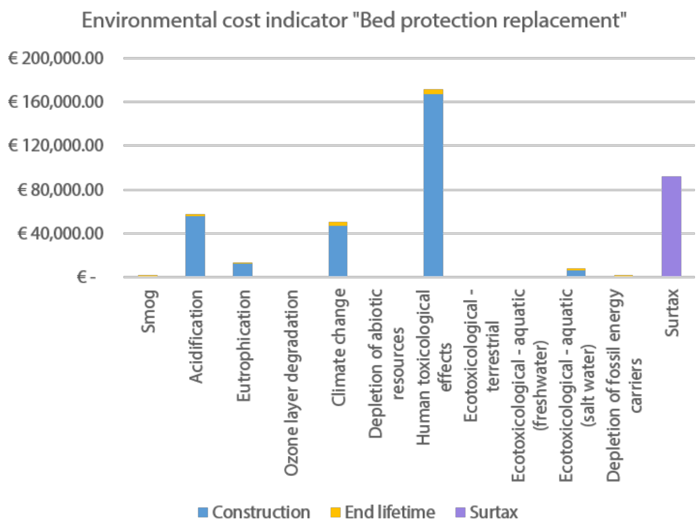


Figure G.10: Environmental cost indicator of concept "Bed protection replacement" at the Western lock in Terneuzen. The data is derived using DuboCalc.

G.5. Circularity

The Dutch Rijkswaterstaat wants to work completely circular by the year 2030, meaning no waste products are created from the industry (Rijkswaterstaat, n.d.-a). By reusing all elements from a demolished construction, eventually no new materials have to be extracted. One argument for a circular economy is the halt of depletion of materials. The results of the sustainability study however show that barely any societal costs are assigned to the depletion of abiotic resources. A more prominent reason for a circular life cycle is its economic value.

Migrating towards a fully circular economy can provide a yearly profit of €7.3 billion and 54,000 jobs, while increasing export opportunities for the Netherlands (Bastein et al., 2013).

Some methods of reusing materials are better than others. For instance concrete can be pulverised and used to create new concrete or asphalt. This does however require addition of other substances, and the recycled concrete does not have the same compressive and tensile strength as its predecessor (Xiao, 2018). A fully circular element or material is something that can be used in a different build, in its original form, and with its initial structural capacity.

- **Very good**: Concept includes mostly perfectly circular components.
- **Good**: Concept includes significant components which are merely reusable.
- **Bad**: Concept includes significant components which are merely recyclable.
- **Very bad**: Concept includes significant components which are neither circular, reusable, or recyclable.
- **Unacceptable**: Because policies concerning circularity are not yet effective, no structure is currently unacceptable.

One concept in which completely circular elements are implemented, is the modular lock extension. The interlocking blocks can be used for many other purposes, and require no modification between the use in this build and a different construction. Also the rocks in a bed protection can be reused after the lifetime of the structure. After sieving the mixture, the grains can be used for any of their purposes. Penetrating the top layer of the bed protection with colloidal concrete on the other hand makes this process more difficult, as concrete has to be removed in order to regain the material in its original state. The colloidal concrete itself could be recycled, but is not perfectly circular, as it looses some of its structural potential. The lock support, consisting of sheet piles and diagonal piles, can be reused, but it has to be noted that their performance does decrease over time due to corrosion (Benamar & Habib, 2016). These elements are therefore not perfectly circular, but *can* be used in a design requiring less strength. For all other concepts, components are either not removable as a single element, like concrete, or are specifically dimensioned for this very specific structure, like a gate, and are therefore in no sense circular. For mitre gates, standard module sizes can be used for locks with varying dimensions. The Western lock in Terneuzen is however fitted with roller gates, which are too much dependent on the specific width of the lock to be reused in its original shape. The dimensions of locks vary too much in the Netherlands for circular use of roller gate elements (see Table 2.1).

Table G.5: Grading of sustainability. A green cell indicates "very good". Yellow indicates "good". Orange indicates "bad". Red indicates "very bad". Purple indicates "unacceptable".

Characteristic	Retention height						Bed pr.	
	Lock height			Gate				
Concept	Wall	Fixed ext.	Modular ext.	Gate extension	Gate replacement	Gate module	Colloidal concrete	Replacement
Circularity								
Including lock support:								

G.6. Reworking criteria grading to system of deduction points for pathway evaluation

The criteria evaluation method presented in this Appendix can be well applied to compare the value of the adaptations to one another. However, for the evaluation of the pathways, several adjustments have to be made in order to make the grading useful.

For the evaluation of the adaptation concepts, five criteria were analysed: construction time, construction nuisance, familiarity of the procedure, sustainability, and circularity. In Chapter 6, the adaptations are compared to a replacement. The replacement has however not been regarded before in the criteria evaluation. However, the replacement has to occur eventually, regardless of the initial choice of adaptation. It is therefore deemed unnecessary to consider it in the comparison of the pathways. In Chapter 7, multiple lock replacement alternatives are compared to one another. The only differences between these alternatives are mere small dimensional variations, which would not prompt a different evaluation score at the grand scale of the entire lock. The construction time is roughly identical, and much more influenced by other factors than minor differences in dimensions of the locks. The grading of the construction nuisance, familiarity, and circularity of the alternatives would be equal, as no different techniques are required for them to be constructed. At last, the sustainability would also vary negligibly due to the small differences in dimensions. Thus, for both Chapter 6 and 7, no evaluation of the replacement alternatives is required.

If the replacement is disregarded, the pathways can solely be evaluated on the executed adaptations during the lifetime of the lock. That is to say, a lock alternative which requires no adaptations during its lifetime, receives a better score than an alternative which *does* require adaptations. The scores derived in the adaptation criteria evaluation in Section 6.3.2 can be used again here, but have to be applied differently.

For the evaluation of the adaptations, also not all criteria have to be regarded. Because the concepts are now placed in a larger picture, i.e. the lifetime of a lock, some elements become less important. For instance, the incorporation of the sustainability and circularity in the evaluation is no longer valid. In comparison to the entire lock, the addition of the adaptations does not contribute much to these criteria, as they are rather insignificant in size. The footprint and regained costs of an adaptation do not contribute much regarding the total sustainability and circularity of the lock. The other criteria, construction time, construction nuisance, and familiarity of the procedure, all concern the effectiveness of an adaptation, the duration of operability loss, and the nuisance to the residents. Due to these aspects, the addition of the adaptations provides complications which are not relevant at a replacement. When a lock is replaced, measures are taken to assure that the capacity of the waterway only decreases marginally. Because the construction of the entire lock takes several years, such measures are profitable. For instance when a lock is replaced, the new lock is preferably not constructed at the exact location of the original one. Instead, as is currently happening in Terneuzen, the lock is constructed to the side, so that the original lock can remain operable during the construction period (Pfaff-Wagenaar & De Wit, 2015). For an adaptation, such measures are not possible, or become very expensive for the magnitude of the operation. Thus, the capacity of a waterway is lowered during an adaptation.

Not only can some criteria be occluded from the pathway evaluation, the application of the evaluation also has to be altered. The path which starts with an immediate replacement has no adaptations in it, and therefore no grading can be assigned to this based on the method applied in Section 6.3.2. Instead, points should be deducted for each adaptation executed during the lifetime of a lock. In that manner, a lock undergoing no adaptations during its entire lifetime is granted a perfect score. This deduction system can conveniently be derived from the grading system of the adaptation evaluation. The weight of the criteria remains equal, but instead of multiplying the weight with the score of that criterion, it is multiplied with the difference between the maximum achievable score (4 in this case) and the score that was granted, i.e. the points it did not get, but could have gotten. If for instance a criterion has a weight of 2, and a concept scores a 3 out of 4 on this criterion, the concept misses 1 point, which, multiplied with the weight of 2, amounts to 2 deduction points.

By applying this system, in case a pathway includes multiple adaptations, the deduction points of those adaptations can simply be added upon another to derive a final score of the pathway. Table G.6 presents the scores of the adaptation concepts. Not all concepts which were regarded throughout the previous sections of this Appendix are presented in this table, because in Section 6.4, a selection is made as to what adaptation concepts are to be considered for the pathways map.

Table G.6: Grading on criteria of adaptation concepts applied to the Western lock in Terneuzen

	Weight	Wall	Modular extension S	Modular extension W	Gate extension	Gate replacement	Gate module	Colloidal concrete
Construction time	2	0	-2	-3	0	0	0	0
Construction nuisance	3	0	-1	-3	-1	-1	-1	0
Familiarity procedure	2	0	-2	-2	-1	0	-3	0
Total deduction points		0	-11	-19	-5	-3	-9	0

H

Compatibility and applicability adaptation alternatives

The adaptation concepts presented in Chapter 6.2 do not all improve identical characteristics of the lock, and therefore have to be combined with one another to assure that the lock complies to all requirements. It is however possible that not all concepts are compatible with one another. This concerns both the combination of two adaptation concepts (compatibility), and the application of an adaptation concept to a replacement concept (applicability).

H.1. Adaptation compatibility

The adaptation concepts concern four critical characteristics, and therefore have to be combined with one another to assure that the lock suffices to all requirements.

When the wall attachment is firstly installed, no other heightening of the structure can be applied without having to demolish the wall. As stated in Section 6.2.1.1, this concept only works up to a height of 0.5 m, and therefore mounting a second wall upon the original one is not considered feasible. It is however optional to first apply a wall, and than later apply an extension of the lock head (either fixed or modular), but this would require removing the wall. It may not seem to be the most sensible route, but optional.

After installing an extension of the original construction, it is possible (in case the substructure can bear it) to construct yet another heightening, either by means of a wall or a second extension. Applying a modular extension upon a fixed extension would however make little sense. It is assumed that one either makes the choice to construct modular or not. The combination of the two is not considered. Thus, for the modular extension, identical additional heightening is possible: either by means of a wall, or by means of the first heightening, e.g. a modular extension.

The gate extension is meant to be combined with the wall. Both are very minimal measures, and can be applied to any lock, regardless of its structural capacity. The extension can however not be combined with the fixed and modular lock extensions. The maximum height of the gate extension is assumed to be 0.5 m, whereas the lock extensions are only applied for a heightening larger than that. Both the gate replacement and modular gate can be combined with the fixed and modular lock extensions.

All height increasing adaptations can be combined with the improvement of the piping length and bed protection composition. These are applied at completely different sections of a lock, and therefore do not obstruct the construction of one another.

As stated in Section 6.2.3, the concepts regarding the length of the piping path are to be designed conservatively, as it is undesirable to perform these adaptations multiple instances. It is therefore not needed to analyse the compatibility of these concepts with one another. The possible combination of these concepts with the bed protection improvements does however need to be assessed. The cut-off screen would not obstruct any of the bed protection improvement concepts, as would be located at its toe. The sandtight filter, however, would be illogical to apply in combination with a penetration with colloidal concrete. The sandtight filter requires the excavation of the bed protection, which is exactly what this concept is meant to avoid. On the other hand, when replacing the entire bed protection, this provides a perfect opportunity to place a sandtight filter.

Table H.1: Compatibility of adaptations. Green indicates compatible, and red indicates incompatible. Only for the concepts concerning the retention height is assessment required of compatibility with concepts of the same characteristic.

Characteristic	Lock height			Gate height			Piping		Bed pr.	
Concept	Wall	Fixed ext.	Modular ext.	Gate extension	Gate replacement	Gate module	Cut-off screen	Sandtight filter	Colloidal concrete	Replacement
Wall attachment	Red	Green	Green	Green	Red	Red	Green	Green	Green	Green
Fixed ext.	Green	Green	Red	Red	Green	Green	Red	Green	Green	Green
Modular ext.	Green	Red	Green	Red	Green	Green	Green	Green	Green	Green
Cut-off screen							Green	Green		
Sandtight filter							Red	Green		

H.2. Adaptation applicability

Both the weak and strong adaptive lock alternatives are meant to be combined with adaptation concepts. This does however not indicate that they can be combined with *all* drafted adaptations. The conservative lock replacement is not specifically designed to be adapted. It is therefore important to analyse the applicability of the adaptations after a lock has been replaced.

Table H.2 presents the applicability of the adaptation concepts to the replacement concepts. The wall adaptation can be applied at all locks, as the additional weight and accompanied moment on the structure are minimal. The fixed extension of the original lock design and the modular extension do provide a significant enlargement of the forces. For the weak adaptive lock alternatives, additional adaptation of the lock head is required to support the structure. For the adaptive lock alternatives, additional forces can be borne, but the structures are still limited. The concepts are designed to withstand a heightening of the structure of a certain degree (as indicated in Section 7.2.2), and cannot bear any larger addition.

The gate alternatives are applicable to all lock designs. The same accounts for the piping and bed protection improvements. These concepts are all applied outside of the lock head and chamber, and impose no additional forces on the structure, and therefore do not jeopardise the structural integrity of the locks.

Table H.2: Applicability of adaptations to lock replacement alternatives. Green indicates unconditionally applicable, yellow indicates conditionally applicable, orange indicates conditionally applicable under additional adaptation, and red indicates inapplicable.

Characteristic	Lock height			Gate height			Piping		Bed pr.	
Concept	Wall	Fixed ext.	Modular ext.	Gate extension	Gate replacement	Gate module	Cut-off screen	Sandtight filter	Colloidal concrete	Replacement
Robust	Green	Orange	Orange	Green	Green	Green	Green	Green	Green	Green
Adaptive	Green	Yellow	Yellow	Green	Green	Green	Green	Green	Green	Green

I

Cost derivations

In this Appendix, indications of the costs of the adaptation concepts and replacement concepts are derived. Because material quantities are required to calculate the costs of a construction, the concepts first have to be applied to the case study of the Western lock in Terneuzen. For all applications, only the direct costs are taken into consideration. Estimations of other cost items are usually made as a percentage of the direct costs (Cobouw, n.d.). Therefore, the comparison between only the direct costs, or the entire costs of a project is identical. To reduce complexity of the cost derivations, only direct costs are determined.

I.1. Adaptations

The costs of the adaptation concepts are mostly derived using characteristic cost values acquired from GWWkosten.nl (Cobouw, n.d.). This is a website which can be used to make cost and duration estimations and of civil constructions. As mentioned, only direct costs are taken into consideration.

I.1.1. Wall

The properties and dimensions of concept "Wall" were derived in Appendix F, and presented in Table F.2. With these parameters, and characteristic cost values from GWWkosten.nl (Cobouw, n.d.), an estimation of the construction costs can be made, as presented in Table I.1.

Table I.1: Cost indication of concept "Wall" applied to the Western lock in Terneuzen. Characteristic cost values are derived from GWWkosten.nl (Cobouw, n.d.).

Cost item	Cost indicator	Value	Cost per unit	Costs
Concrete wall	Concrete volume	25 m^3	€158.- per m^3	€3,950
Reinforcement	Steel weight*	1962.5 kg	€2.20 per kg	€4,300
Formwork	Formwork area	89.6 m^2	€89.50 per m^2	€17,900
			Direct costs	**€26,200**

* Reinforcement steel density of 7850 kg/m^3 assumed.

I.1.2. Fixed lock extension

The application of this concept was worked out in Appendix F. The dimensions of the adaptation, summarised in Table F.2, are used in combination with characteristic cost values from GWWkosten.nl (Cobouw, n.d.) to derive the direct costs presented in Table I.2.

These costs concern a lock heightening of 1.0 m. For altering required heights, this price can be scaled linearly with the height.

Table I.2: Cost indication of concept "Fixed extension" applied to the Western lock in Terneuzen. Characteristic cost values are derived from GWWkosten.nl (Cobouw, n.d.).

Cost item	Cost indicator	Value	Cost per unit	Costs
Concrete extension	Concrete volume	5855 m^3	€140.- per m^3	€ 819,700
Reinforcement	Steel weight	459,600 kg	€2.20 per kg	€1,011,200
Formwork	Formwork area	2930 m^2	€27.25 per m^2	€79,900
Landfill	Equalising ground	65,120 m^2	€13.- per m^2	€ 846,600
	Delivery soil	65,120 m^3	€8.70 per m^3	€566,500
	Process soil	65,120 m^3	€0.82 per m^3	€ 53,400
			Direct costs	**€3,377,200**

* Reinforcement steel density of 7850 kg/m^3 assumed.

I.1.3. Modular lock extension

In Table I.3, the direct costs are derived for concept "Modular extension", applied to the case study as presented in Appendix F. The properties of the adaptation with a height of 1.0 m are presented in Table F.3. Modules like the ones used in this design can be purchased for €160.- (Mholf Bestrating, n.d.). Also taking transport into consideration, it is estimated that the cost of a single module is around €200.-. Also using the characteristic cost values from GWWkosten.nl, the total direct costs are derived in Table I.3.

Table I.3: Cost indication of concept "Modular extension" applied to the Western lock in Terneuzen. Except for the module, the characteristic cost values are derived from GWWkosten.nl (Cobouw, n.d.).

Cost item	Cost indicator	Value	Cost per unit	Costs
Modules	Module units	2134	€200.- per unit	€426,700
Landfill	Equalising ground	65120 m^2	€13.- per m^2	€846,600
	Delivery soil	78144 m^3	€8.70 per m^3	€679,900
	Process soil	78144 m^3	€0.82 per m^3	€53,400
			Direct costs	**€3,473,000**

I.1.4. Crossing bridge elevation

The dimensions and properties of the crossing bridge and additional road work are presented in Table F.4. For the derivation of the costs of the procedure of leveling the bridge, more than just material prices found on GWWkosten.nl are required. For instance, the removal and relocation of the bridge does not require any new material, but does require expenses. These costs are gathered from the cost estimation of a reference project, i.e. the construction of the New lock in Terneuzen (KGT2008, 2008).

Because the Old lock in the Terneuzen lock complex is going to be removed, a cost estimate has been made for the removal of its bridges, equal to €15,000 per bridge. For this case study, those costs are scaled up with the ratio of the length of the bridge. It is also assumed that the costs of placing the gate back in position are similar to the costs of removal. To be conservative, the total replacement costs are estimated to be a factor 2.5 times larger than the removal costs. The replacement of the E&M installations of the bridge is also taken into account. For the New lock in Terneuzen, these costs were estimated to be €750,000 per bridge. These costs are also scaled down by the ratio of the lengths of the bridges. The costs of the bascule basement of the Western lock in Terneuzen (the chamber in which the operating mechanism of the bridge is located), are actually referenced in the cost estimation of the New lock. The costs are estimated to be €509,000, and can be directly used without scaling. The costs of the abutments (one on each side of the bridge) are taken from a cost indication at GWWkosten.nl (Cobouw, n.d.).

The construction of a road consists of many different procedures and elements. Instead of estimating the costs using material prices, an example project found on GWWkosten.nl is used, in which the costs of a very similar road have been derived. The costs of the construction of the road at the lock is scaled up by means of the surface area of these projects. Likewise, an example project of a roundabout found on GWWkosten.nl is used to derive the costs of the replacement of the one located near the case study.

Table I.4: Cost indication of concept "Lock strengthening" applied to the Western lock in Terneuzen. Reference costs are derived from KGT2008 (2008), and characteristic cost values are derived from GWWkosten.nl (Cobouw, n.d.).

Cost item	Costs example	Proportion	Ratio	Costs
Bridge removal	€15,000	L	42 m/33 m	€19,100
E&M replacement	€750,000	L	42 m/55 m	€572,700
Road	€1,061,500	LxW	1500 m^2/9299 m^2	€1,369,800
Bridge replacement			Costs bridge removal x 2.5	€47,700
Bascule basement			Specific for case study	€509,000
Roundabout (1x)			Costs from GWWkosten.nl	€548,300
Abutment (2x)			Costs from GWWkosten.nl	€80,100
			Direct costs	**€3,127,700**

I.1.5. Lock head and walls support

The dimensions and properties of the lock support were derived in Appendix F, and summarised in Table F.5. Using characteristic cost values of GWWkosten.nl (Cobouw, n.d.), the total direct costs of this adaptation are derived in Table I.5.

Table I.5: Cost indication of concept "Lock support" applied to the Western lock in Terneuzen. Characteristic cost values are derived from GWWkosten.nl (Cobouw, n.d.).

Cost item	Cost indicator	Value	Cost per unit	Costs
Driving sheet piles	Sheet pile width	880 m	€2,890.- per module	€2,543,200
Driving bearing piles	Pile units	108	€21,365.- per module	€2,307,400
			Direct costs	**€4,850,600**

I.1.6. Gate extension, gate replacement, and gate module

Because these components require intricate fabrication, the costs depend on much more than only the material price. Therefore, the costs are determined with a reference project. In Section I.2, the costs of the replacement concepts are derived in this manner, including specific costs for the gate and operating mechanism. Thus, for these concepts, the costs of these items can be directly taken from the cost derivations in Section I.2. All concepts have to be applied to four gates: two in each lock head. The direct costs for four gates with an integrated sea level rise of 1.5 m and 2.0 m is respectively €8.0 million and €8.4 million.

Extrapolating these costs towards four modules with a height of 1.0 m and 1.5 m gives a price of respectively €400,000 and €610,000. The gate extension is only 0.5 m tall, and is estimated to cost €200,000 from extrapolation. Although it can be expected that because the gate extension requires less structural potential than the module, the element would cost even less, but to be conservative, this price is regarded in this thesis.

I.1.7. Replacement bed protection

For this adaptation, the original bed protection has to be removed, and a new composition has to be placed. Both processes have a cost. The dimensions of both the original and new bed protection were derived in Appendix F, and presented in Table F.6. The total direct costs of the concept are derived in Table I.6.

Table I.6: Cost indication of concept "Replacement bed protection" applied to the Western lock in Terneuzen. Characteristic cost values are derived from GWWkosten.nl (Cobouw, n.d.).

Cost item	Cost indicator	Value	Cost per unit	Costs
Removal bed protection	Tonnage rocks	12766 mT	€4.24	€54,100
Placement filter mat	Filter area	10500 m^2	€6.95	€73,000
Placement bed protection	Tonnage rocks	22476 mT	€33.25	€747,300
			Direct costs	**€874,400**

I.1.8. Colloidal concrete

The properties of the colloidal concrete were derived in Appendix F, and summarised in Table F.7. Using the charachteristic cost value for colloidal concrete from GWWkosten.nl (Cobouw, n.d.), the total direct costs of the adaptation are derived in Table I.7.

Table I.7: Cost indication of concept "Colloidal concrete" applied to the Western lock in Terneuzen. Characteristic cost values are derived from GWWkosten.nl (Cobouw, n.d.).

Cost item	Cost indicator	Value	Cost per unit	Costs
Penetration with colloidal concrete	Tonnage concrete	2990 mT	€115.-	€343,700
			Direct costs	**€343,700**

I.2. Replacement concept costs

The direct costs of the replacement alternatives are derived differently than those of the adaptation concepts. The method is explained in Section I.2.1. Again, only the direct costs of the projects are taken into consideration.

I.2.1. Cost derivation method

For entire lock constructions, determining costs by means of characteristic values for materials and procedures is an intensive task, requiring many derivations and assumptions of dimensions and properties. A less excessive method is to instead scale the costs of other projects to this case study based on the dimensions.

Thus, reference locks with available cost indications are required to be able to scale these down or up to the case study. By using multiple references, an average can be taken from the scaled costs, providing a solid estimation for the case study, the Western lock in Terneuzen. In an alternative study for the new lock in Terneuzen, the costs of three lock variants have been derived and compared to one another (De Pelsmaeker, 2010). Because of the similar location, and the significant variation in dimensions between the lock variants, these serve as a reliable reference.

In the alternative study, the three locks listed below were analysed. In addition to these general dimensions, the locks also have different cross sectional designs for the lock heads and walls. Because the costs of the replacement of the Western lock will not be derived from a single lock, but scaled and averaged from three different locks with different main dimensions and different designs, the indication is less influenced by the specifications of a single lock.

- KGT1: 552 x 58 x 24.18 m
- KGT2: 424 x 40 x 20.5 m
- KGT3: 480 x 28 x 18 m

The costs of the weak adaptive alternatives can be directly determined by scaling and averaging the costs of these reference locks, but for the strong adaptive locks some additional calculations are required.

I.2.2. Conservative and weak adaptive lock alternatives

For the weak adaptive lock, three alternatives are analysed. In those three designs, either a sea level rise of 0.5 m, 1.0 m, or 2.0 m is integrated. These are henceforth referenced as W+0.5, W+1.0, and C+2.0. The dimensions of these locks are presented in Table I.8.

Table I.8: Dimensions of reference cases, the conservative alternative, and weak adaptive alternatives

	KGT1	KGT2	KGT3	TWL	W+0.5	W+1.0	C+2.0
Length [m]	552	424	480	440	440	440	440
Width [m]	587	40	28	38	38	38	38
Height [m]	24.18	20.5	18	18.8	19.3	19.8	20.8

In the alternative study for the New lock in Terneuzen, a number of cost items has been regarded, as listed below (De Pelsmaeker, 2010).

- Lock construction
- Roller gate and operating system
- Ground work
- Other direct costs
- Indirect costs
- Additional construction costs
- Unforeseen costs

All direct costs, i.e. the lock construction, roller gate and operating system, ground work, and other direct costs, have been calculated from characteristic cost values for required volumes of material. All other cost items have been derived as percentages from the total direct costs. To be able to compare the costs of the adaptations to those of the replacements alternatives, it is therefore easier to simply only consider the direct costs of the constructions.

Not all direct costs should be scaled by the same dimensions. The costs for the lock construction, ground work, and other direct costs are dependent on the length, width, and height of the lock, whereas the costs of the roller gate and operating system only depend on the width and height.

At last, the replacement costs have to be comparable to those of the adaptations. Because the costs are derived in different manners, this is not necessarily the case. Therefore, the costs are scaled. A good reference is the fixed lock extension. The costs of this adaptation should approximately be equal to 1.0 m of the lock construction. As presented in Table I.9, the costs of the lock construction of W+0.5, with a height of 19.3 m, are equal to €93.7 million when following the before mentioned steps. This amounts to €4.9 million per meter lock construction. The fixed adaptation was estimated to cost around €3.4 million. This is a factor 1.44 smaller than would follow from the reference projects. Therefore, all costs are divided by 1.44 to assure comparability.

The direct costs of W+0.5, W+1.0, and C+2.0 can sequentially be derived by following the steps listed below. This is worked out in Tables I.9, I.10 and I.11. Based on another cost estimation of the New lock in Terneuzen, it is assumed that of the combined costs of the roller gate and operating mechanism, 60% is the roller gate, and 40% is the operating mechanism (KGT2008, 2008).

1. Obtain from all reference locks the costs per cost item
2. Determine per cost item the ratio using the relevant dimensions:
   $\mathrm{ratio} = \dfrac{L_1 \cdot W_1 \cdot H_1}{L_2 \cdot W_2 \cdot H_2}$ or $\mathrm{ratio} = \dfrac{W_1 \cdot H_1}{W_2 \cdot H_2}$
3. Derive per reference lock the scaled costs per cost item by multiplying its costs with the acquired ratio
4. Per cost item, average the scaled costs of the reference locks
5. To be able to compare the costs of the replacement to those of the adaptations, the costs are scaled down with a factor 1.44
6. Sum up the scaled averaged costs to derive the total direct costs of the lock alternative

Table I.9: Direct costs scaling for W+0.5. Costs are given in millions.

Cost item	**Proportion**		**KGT1**	**KGT2**	**KGT3**	Averaged	**W+0.5** Scaled
Lock construction	LxWxH	Costs	€ 167	€ 120	€ 75		
		Ratio	0.42	0.93	1.33		
		Scaled cost	€ 70	€ 111	€ 100	€ 93.7	**€ 65.2**
Roller gate and operating mechanism	BxH	Costs	€ 39	€ 24	€ 10		
		Ratio	0.52	0.89	1.46		
		Scaled cost	€ 20	€ 21	€ 14	€ 18.7	**€ 13.0**
Roller gate	60%						**€ 7.8**
Operating mechanism	40%						**€ 5.2**
Ground work	LxWxH	Costs	€ 137	€ 120	€ 16		
		Ratio	0.42	0.93	1.33		
		Scaled cost	€ 57	€ 111	€ 21	€ 63.3	**€ 44.1**
Other direct costs	LxWxH	Costs	€ 206	€ 184	€ 78		
		Ratio	0.42	0.93	1.33		
		Scaled cost	€ 86	€ 171	€ 104	€ 120.2	**€ 83.7**
						Total direct costs	**€ 206.0**

Table I.10: Direct cost scaling for W+1.0. Costs are given in millions.

Cost item	**Proportion**		**KGT1**	**KGT2**	**KGT3**	Averaged	**W+1.0** Scaled
Lock construction	LxWxH	Costs	€ 167	€ 120	€ 75		
		Ratio	0.43	0.95	1.37		
		Scaled cost	€ 71	€ 114	€ 103	€ 96.1	**€ 66.9**
Roller gate and operating mechanism	BxH	Costs	€ 39	€ 24	€ 10		
		Ratio	0.54	0.92	1.49		
		Scaled cost	€ 21	€ 22	€ 14	€ 19.1	**€ 13.3**
Roller gate	60%						**€ 8.0**
Operating mechanism	40%						**€ 5.3**
Ground work	LxWxH	Costs	€ 137	€ 120	€ 16		
		Ratio	0.43	0.95	1.37		
		Scaled cost	€ 59	€ 114	€ 22	€ 64.9	**€ 45.2**
Other direct costs	LxWxH	Costs	€ 206	€ 184	€ 78		
		Ratio	0.43	0.95	1.37		
		Scaled cost	€ 88	€ 175	€ 107	€ 123.3	€ 85.9
						Total direct costs	**€ 211.4**

Table I.11: Direct cost scaling for C+2.0. Costs are given in millions.

Cost item	Proportion		KGT1	KGT2	KGT3	Averaged	C+2.0 Scaled
Lock construction	LxWxH	Costs	€ 167	€ 120	€ 75		
		Ratio	0.45	1.00	1.44		
		Scaled cost	€ 75	€ 120	€ 108	€ 101.0	**€ 70.3**
Roller gate and operating mechanism	BxH	Costs	€ 39	€ 24	€ 10		
		Ratio	0.56	0.96	1.57		
		Scaled cost	€ 22	€ 23	€ 15	€ 20.1	**€ 14.0**
Roller gate	60%						**€ 8.4**
Operating mechanism	40%						**€ 5.6**
Ground work	LxWxH	Costs	€ 137	€ 120	€ 16		
		Ratio	0.45	1.00	1.44		
		Scaled cost	€ 62	€ 120	€ 23	€ 68.2	**€ 47.5**
Other direct costs	LxWxH	Costs	€ 206	€ 184	€ 78		
		Ratio	0.45	1.00	1.44		
		Scaled cost	€ 93	€ 184	€ 112	€ 129.6	**€ 90.2**
						Total direct costs	**€ 222.0**

I.2.3. Strong adaptive lock alternatives

Two strong adaptive lock alternatives are analysed: one with an initial retention height for 0.5 m sea level rise and one with an initial retention height for 1.0 m sea level rise. Both alternatives have the possibility of adaptation for up to 2.0 m sea level rise. The alternatives are henceforth referenced as S+0.5 and S+1.0.

The costs of the strong adaptive lock alternatives are derived using the costs of the conservative and weak adaptive locks, as they are comprised of the same elements. It however depends on the cost item from which lock alternative the cost has to be extracted.

- **Lock construction**: Although the initial retention height of the strong adaptive lock alternatives is smaller than for C+2.0, the foundation, lock head, and chamber walls are designed to be able to withstand the additional forces accompanied by a retention height fitted for 2.0 m sea level rise. Therefore, the costs of these components should not be scaled down to initial retention height. This would imply a weaker construction. Contrarily, the costs of the lock construction are equal to those of C+2.0, with a subtraction of the costs of the missing top of the lock heads and chamber walls. Thus, the strength of the components is taken into consideration, but the material costs are spared.
- **Roller gate and operating mechanism**: The operating mechanism is installed which can carry the load of a conservative gate, but a non-conservative gate is fabricated. The costs of the operating mechanism can be taken from C+2.0, whereas the costs of the gate can be taken from the weak adaptive lock alternative with an identical retention height as the initial height of the strong adaptive lock alternative, i.e. W+0.5 or W+1.0. These gate would be replaced in case the lock is adapted, but not the operating mechanism.
- **Ground work**: It is assumed that an identical amount of ground work would have to be performed for an strong adaptive lock as for C+2.0, because the an identical retention height should eventually be available.
- **Other direct costs**: Under this cost item fall many sub-items, such as electrical work, the bed protection, infrastructure, architectural costs and detailing. It is assumed that these costs are identical to those of W+0.5 or W+1.0, dependent on the initial retention height of the strong adaptive lock alternative.

Thus, all costs can be taken directly from the conservative and weak adaptive lock alternatives, except for the subtraction of the costs of the missing retention height in the lock structure. These costs are estimated to be roughly equal to those of the fixed lock extension. For S+0.5, this concerns a fixed extension with a height of 1.5 m, which was retrieved to cost €5.1 million. For S+1.0, this only a fixed extension of 1.0 m is extracted. This was determined to cost €3.8 million. Knowing the saved costs for both alternatives, the total direct costs can be derived, as discussed, from the cost items of the conservative and weak adaptive lock alternatives.

The derivation of the costs of the strong adaptive lock alternatives is presented in Table I.12.

Table I.12: Direct cost derivation of S+0.5 and S+1.0. Costs are given in millions.

	S+0.5		**S+1.0**	
Cost item	**Ref. Design**	**Cost**	**Ref. Design**	**Costs**
Lock construction	C+2.0	€ 70.3	C+2.0	€ 70.3
Roller gate	W+0.5	€ 7.8	W+1.0	€ 8.0
Operating mechanism	C+2.0	€ 5.6	C+2.0	€ 5.6
Ground work	C+2.0	€ 47.5	C+2.0	€ 47.5
Other direct costs	W+0.5	€ 83.7	W+1.0	€ 85.9
Saved costs	-	-€5.1	-	-€3.4
	Total direct costs	**€ 209.8**		**€ 213.9**

J

Estimation long term sea level rise projection RCP6

The pathways maps created in Sections 6.5 and 7.3 are rather meaningless without any insight of the probability of the sea level rise presented in these maps. Thus, the expected sea level rise at the end of the lifetime of a lock is depicted per for three scenarios: RCP8.5, RCP6, and RCP2.6. Long term projections are required, because a lock has a desired lifetime of at least 100 years (Rijkswaterstaat, 2010). Such projections of the sea level rise at the Dutch coast are available for RCP8.5 and RCP2.6 (Le Bars, 2019), but not for RCP6. RCP8.5 and RCP2.6 are extreme cases, i.e. the most pessimistic and optimistic respectively. These are very useful, but it is deemed important to also consider a moderate scenario, as this might be more probable to occur.

For the global sea level rise, long term predictions have been made for RCP6 as well (Nauels, Meinshausen, Mengel, Lorbacher, & Wigley, 2017). By scaling the projections of the global sea level rise in RCP8.5 and RCP2.6 to fit the sea level rise at the Dutch coast in those scenarios, a very rough estimation of the Dutch sea level rise in RCP6 is made. This is presented in Figure J.1. On the left, sea level rise predictions of the Dutch coast made by Le Bars (2019) are presented of RCP8.5 and RCP2.6. On the right, projections for the global sea level rise are presented for RCP8.5, RCP6, RCP4.5 and RCP2.6 (Nauels et al., 2017). The latter has been scaled down so that the height of the average sea level rise in 2300 of projections RCP8.5 and RCP2.6 is located at an equal elevation as those in the graph on the left (for RCP8.5 depicted by means of a red dashed line).

Consecutively, one can draw straight lines from certain degrees of sea level rise from the left graph to the right, to translate the global sea level rise projection to one for the Dutch coast. In Figure J.1, this process is presented for discrete sea level rise values with a 0.5 m gap. Those values are used in the derivation of the cost progression of the different pathways in Sections 6.6.2 and 7.4.2.

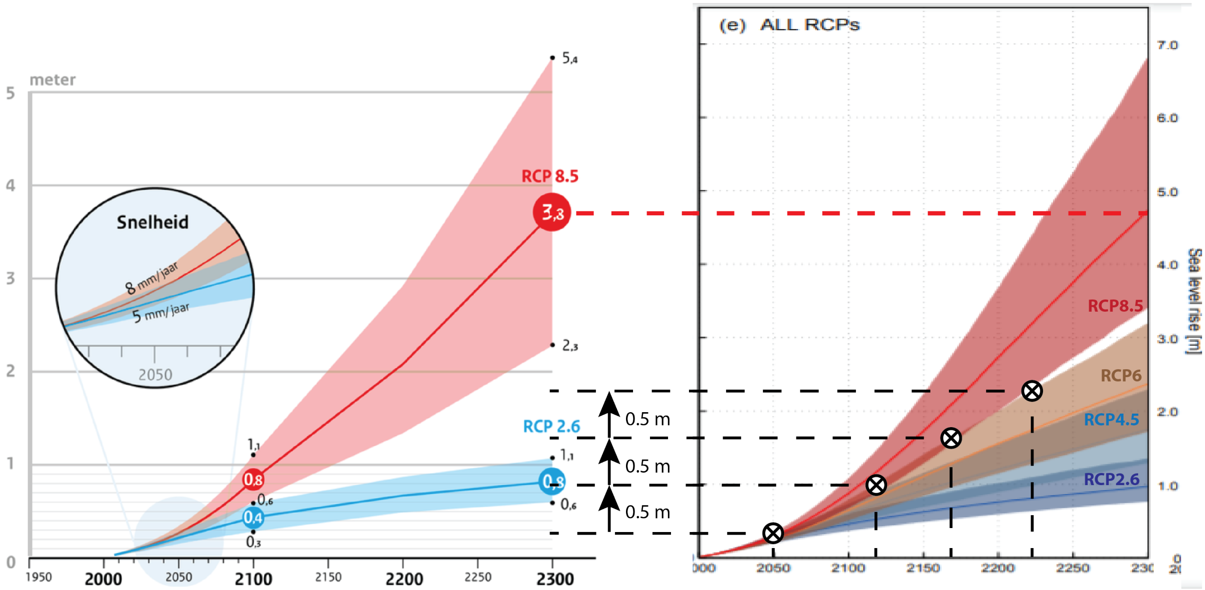


Figure J.1: Derivation of years in which sea level rise stages with a difference of 0.5 m are reached in RCP6. On the left is the projection of RCP8.5 and RCP2.6 for the Dutch coast, and on the right the projection of the global sea level rise RCP8.5, RCP6, RCP4.5 and RCP2.6. This has been scaled down to fit the Dutch predictions. Retrieved from Le Bars (2019) and Nauels et al. (2017) (edited).

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