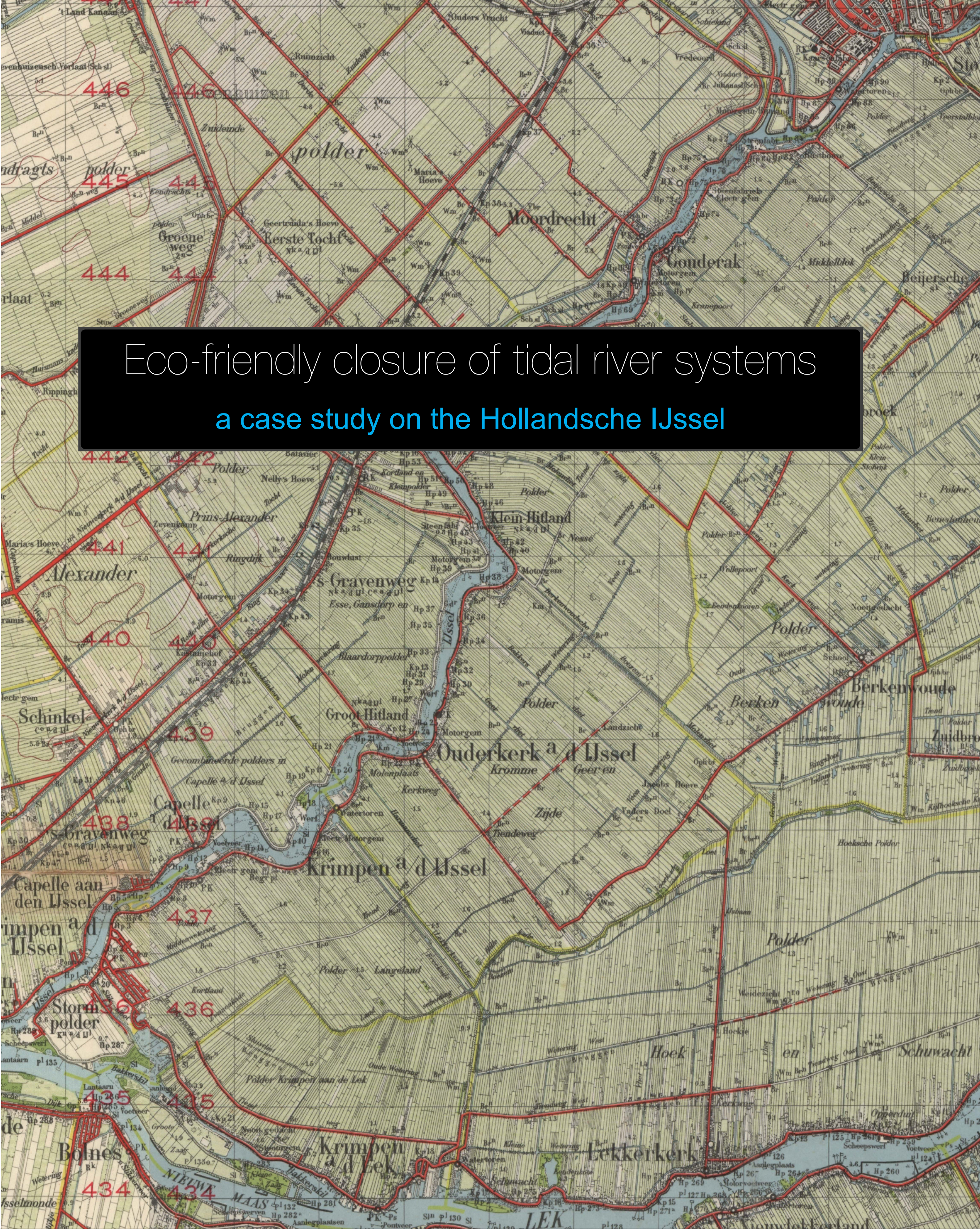
Eco-friendly closure of tidal river systems

a case study on the Hollandsche IJssel



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Eco-friendly closure of tidal river systems

a case study on the Hollandsche IJssel

by

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“Oh let the river in, the will of men can’t hold it in.”

- Dotan

Acknowledgement

This thesis concludes my Master of Science in Civil Engineering and therefore my study period in Delft. I would like to take the opportunity to thank the people who contributed towards completion of this thesis. First of all, I would like to thank my thesis committee for assessing my work. Furthermore, I want to give my word of gratitude to Royal HaskoningDHV who gave me the opportunity to write and research at their offices. I am certain that the open mindset of the employees and the various discussions I had with engineers from different backgrounds, ranging from ecologists to pump experts, allowed me to gain valuable (tacit) information which has been incorporated in this research.

The moral support I received throughout my studies is probably just as important as the academic and professional support. I would like to give my word of gratitude to my mother for her unconditional support throughout my entire study period. Finally, I would like to thank my friends who I know through my study, student and sport associations, my room mates and all other friends. They enabled me to enjoy student life to the fullest.

C.J. van Zaal
Delft, March 2017

Summary

To protect the hinterland, flood defences are often located on the border between water and land. This same border however is also an attractive place to live, which may hamper improvement of flood protections. Closing off a tidal (river) system, especially in case of little discharge, may in this case be a logical consideration. This would however result in the disappearance of tidal action, while ecological value in urban areas is already under pressure. Environmental organisations are representing the increasing voice and power of society and its cry for nature conservation. The Hollandsche IJssel is one of the side branches of the Dutch delta. It is a perfect case of the conflicting interests mentioned. The aim in this research was to design a hybrid barrier for this case that provides (1) the security of a close-off to the hinterland and (2) the openness of a surge barrier (tide and flow are possible) to nature and thus conserves this unique system.

The core of this research is to come up with a preliminary design of a hybrid barrier that is appropriate from an ecological point of view. To do so, boundary conditions were distilled that ensure conservation of ecological value of the freshwater tidal river system. Focus was laid on the unique aspects of the ecosystem at hand. Ecological value is considered to be conserved if the hybrid barrier conserves the tidal range as much as possible (90%) and if fish migration is still possible. The line of thought on how to take into account ecological value presented in this research could be used by others seeking to fully incorporate ecological value in their design.

Several concepts and locations for the eco-friendly barrier are discussed. Furthermore, in order to increase the available intertidal area, a proposition is made to narrow the river. The proposed strategy consists of three aspects, namely (1) an eco-friendly barrier with pumping capacity, (2) river narrowing and (3) no dike reinforcements *after* implementation of the new barrier. Until the proposed strategy is implemented, dike reinforcements may still be necessary however. The proposed strategy was compared to the current or reference strategy. The reference strategy consists of three aspects, namely (1) improvement of the current barrier, (2) replacement of the barrier with a similar barrier in 2050, and (3) continuation of dike reinforcements. In both strategies the current barrier is replaced.

It was found that for the proposed strategy, a single gate solution is not an option, hence multiple small gates are required. A single gate solution would lead to too high water levels in the Hollandsche IJssel. The final design (figures 1 & 2) consists of 31 square culverts with sides of 2.7 m. To maintain maximum tidal range pumps are required between 2105 and 2140 (depending on the severity of sea level rise). Therefore, a pump structure with 6 square inlets with sides of 3.7 m is included in the design. Approximately 90% of the original tidal amplitude is maintained and the available intertidal area almost triples. Furthermore, fish are able to migrate the majority of the time. The slight reduction of the tidal range is compensated by creating extra intertidal area.

The cost of implementation of the proposed strategy in 2030, 2050 and 2070 was compared to the reference strategy. Three points of comparison were taken into account, namely: (1) societal cost of shipping delay (2) cost of construction and (3) dike reinforcement cost. It was found that the proposed barrier including the creation of new intertidal areas is significantly cheaper than the reference strategy in two cases considered (implementation in either 2030 or 2050) or just as expensive as the reference strategy (implementation

in 2070). In total, societal cost ranging from 400 M€ up to 700 M€ can be saved when implementing the new barrier complex in 2030. Although the societal cost due to shipping delay are higher, construction cost of the barrier itself are lower. More importantly, expensive dike reinforcements can be averted in the proposed strategy. Earlier implementation results in lower investment cost, mainly because less dike reinforcements need to be carried out.

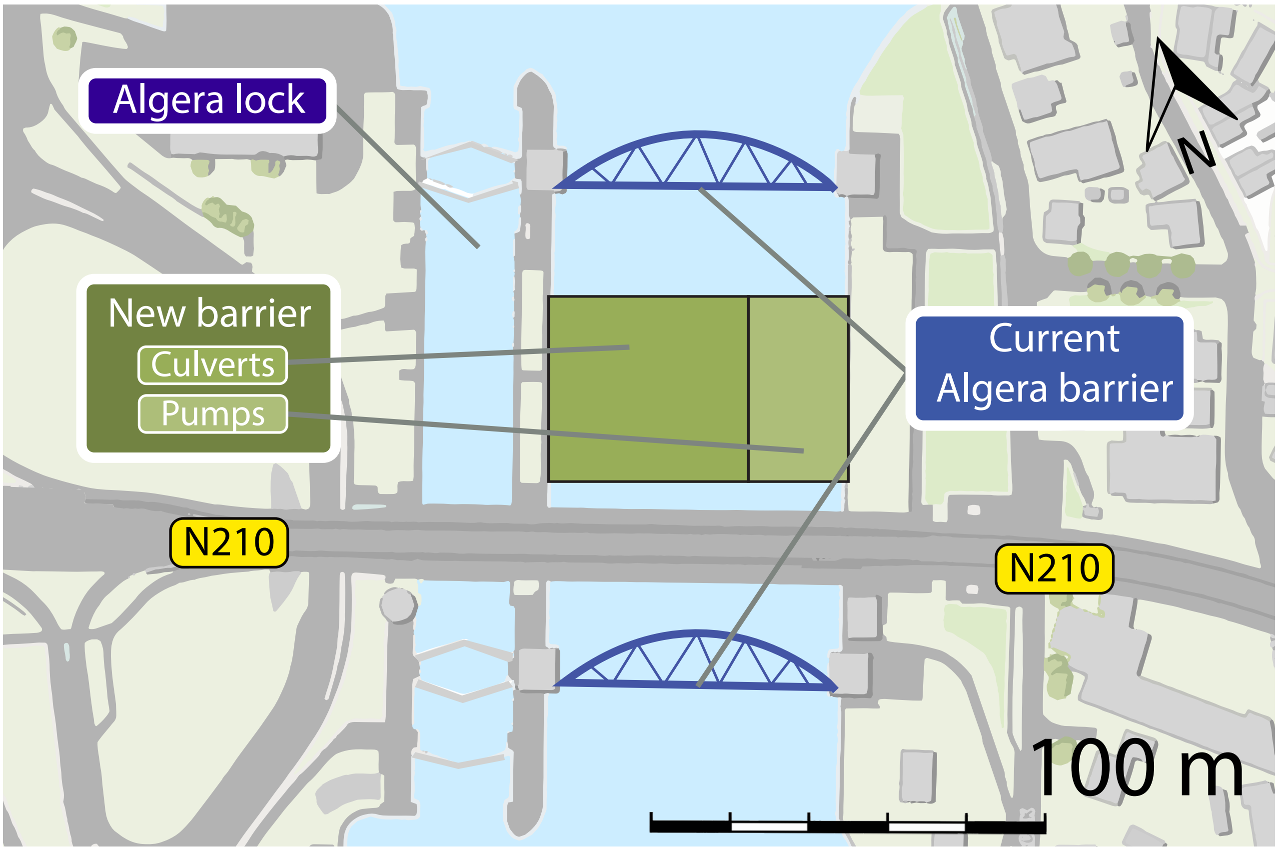

Figure 1: Top view of the barrier complex.

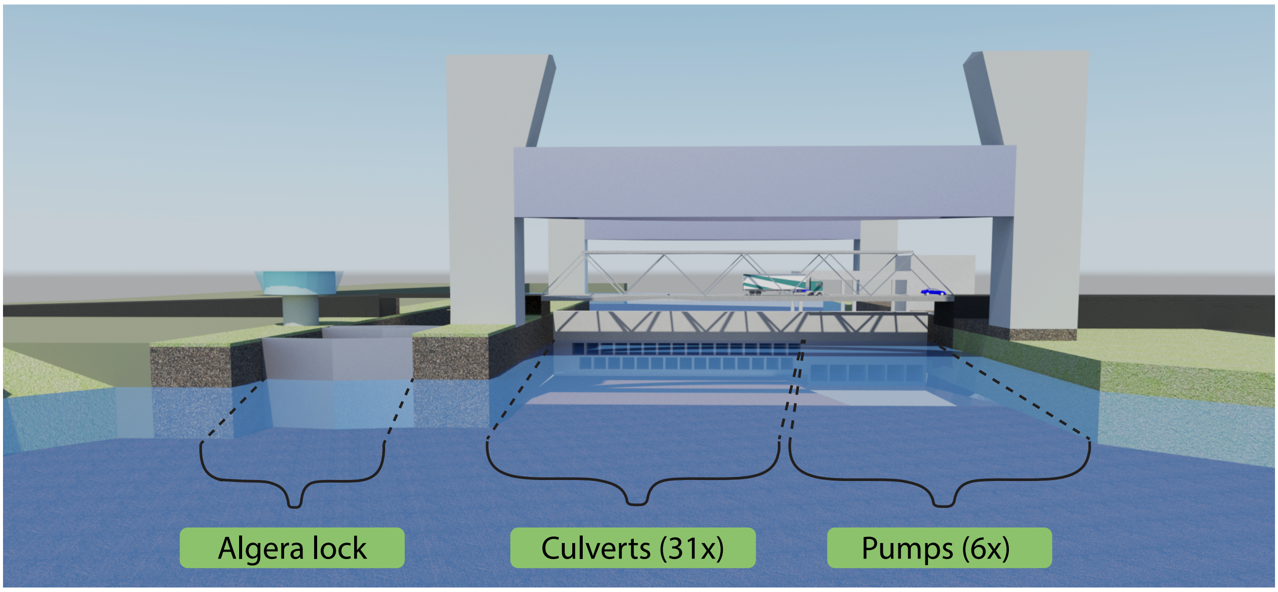

Figure 2: Visualisation of the new barrier complex.

Contents

1 Introduction

1.1 Introduction to the topic

The combination of sea level rise and high density populations in river deltas has led and will continue to lead to protection of this economic valuable land. To protect the hinterland, flood defences are often located on the border between water and land. This same border however is also an attractive place to live which may hamper improvement of flood protections. A close-off, especially in case of low discharges, may in this case be a logical consideration. This would however result in the disappearance of tidal action, while the natural value in urban areas is already under pressure. Environmental organisations are representing the increasing voice and power of society and its cry for nature conservation.

It is the combination of the need and appreciation for nature, its intrinsic value and the uniqueness of fresh-water tidal river systems that has led to this thesis, which has the aim to design a hybrid barrier that provides the security of a close-off to the hinterland (1) and the openness of a surge barrier to nature (2) and thus conserves this unique system.

1.2 Introduction to the case: The Hollandsche IJssel

The Hollandsche IJssel is one of the side branches of the Dutch delta. It is a perfect case of the conflicting interests presented in the previous section. The river connects to the New Meuse or *Nieuwe Maas* near Cappelle a/d IJssel, a suburb of Rotterdam. It has a length of 46 km of which the last 17 km are influenced by the tide. Near Gouda, the Waaier lock and Juliana locks block any further upstream tidal influence. Figure 1.1 shows the tidal part of the river (Yellow). The upstream connection to the Lek river was closed off in 1285, as described by Rijkswaterstaat (ndc). The main natural phenomenon that is able to stir the water, is the tide, with a maximum tidal amplitude of approximately 1.8 m (Rijkswaterstaat, nda). Mean High Water (MHW) varies between NAP +1.2 m and +1.4 m and Mean Low Water (MLW) varies between NAP -0.27 m and -0.34 m for respectively Krimpen a/d IJssel and Gouda (Rijkswaterstaat, ndd). The river only has a minor discharge, that is mainly due to pumps discharging into the Hollandsche IJssel (van Balen et al., 2010).

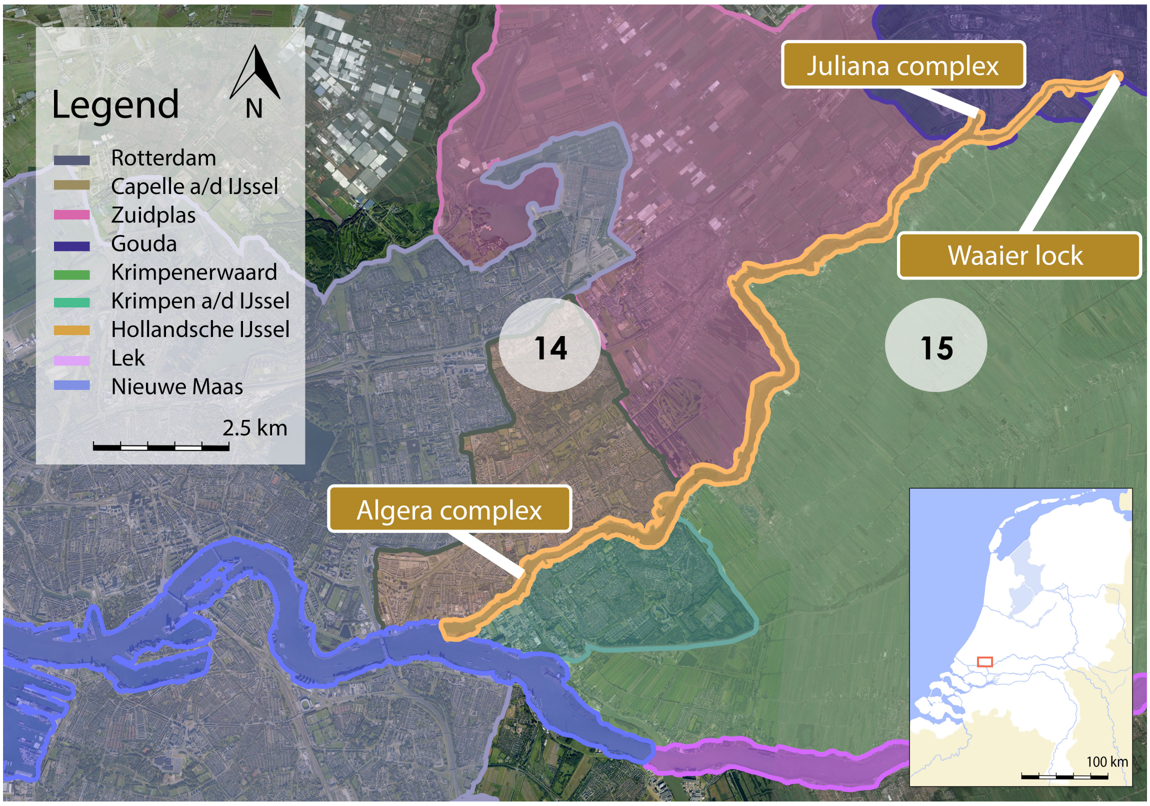

Figure 1.1: Hollandsche IJssel and its location in The Netherlands.

The Hollandsche IJssel (HIJ) provides several issues to be dealt with. The high probability of failure (≈1:100, (Schoemaker, 2016)) of the primary storm surge barrier and the failure of the dikes to pass the assessment test, in combination with sea level rise and significant subsidence, pose challenges to protect the area from flooding. Furthermore, the river provides challenges in terms of environmental health. Dumping of waste and industry along the banks have deposited unwanted chemicals. Recent cleaning operations have significantly improved the situation, but partial restoration of the river is still needed to comply with new European regulation. Chapter 4 discusses the ecology. The river is also of importance for economic activities. It is used as shipping route and as fresh water inlet to supply other areas for agricultural purposes and to counter subsidence. The issues have been further explained in chapter 2. Responsibilities to deal with the abovementioned issues are scattered over various organisations and have different problem owners. Besides stakeholders with responsibilities, there are various stakeholders without responsibility but who are affected by interference in the Hollandsche IJssel. Chapter 3 provides further analysis of the stakeholders involved.

1.2.1 Flood protection

The area around the Hollandsche IJssel is subjected to the Deltaprogramma Rijnmond-Drechtsteden (Deltaprogramma, 2013, p.5). The north western banks form the border of dike ring 14 (DR14, Central Holland), the south eastern banks form the borders of dike ring 15 (DR15, Lopiker- en Krimpenerwaard), see figure 1.1. Current regulations prescribe a probability of exceedance for normative load combinations of 1:10,000 for DR14 and 1:2,000 for DR15 (Rijksoverheid, 2009). The new norms for C-barriers in dike ring 14 and 15 are a probability of flooding of 1:30,000 and 1:10,000 respectively (Deltaprogramma, 2014, p.64). Figure 1.2 illustrates what a C-barrier is.

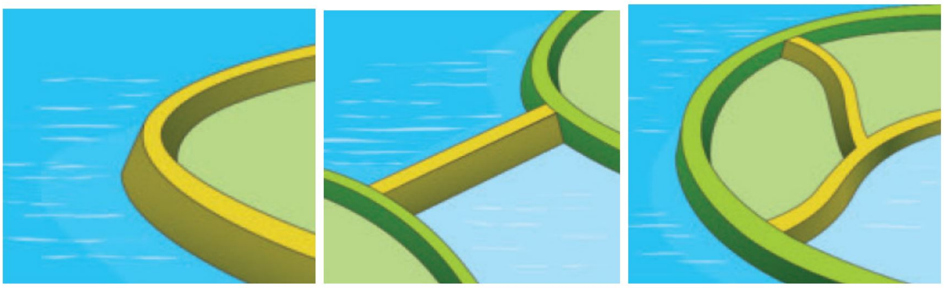

Figure 1.2: Category A (left), B (middle) and C (right) barriers. Source: Rijkswaterstaat (2009).

At the moment, the tidal part of the Hollandsche IJssel is bordered by four elements protecting the area from flooding, being:

1. Algera barrier complex;
2. Juliana locks;
3. Waaier lock;
4. Dikes along the banks.

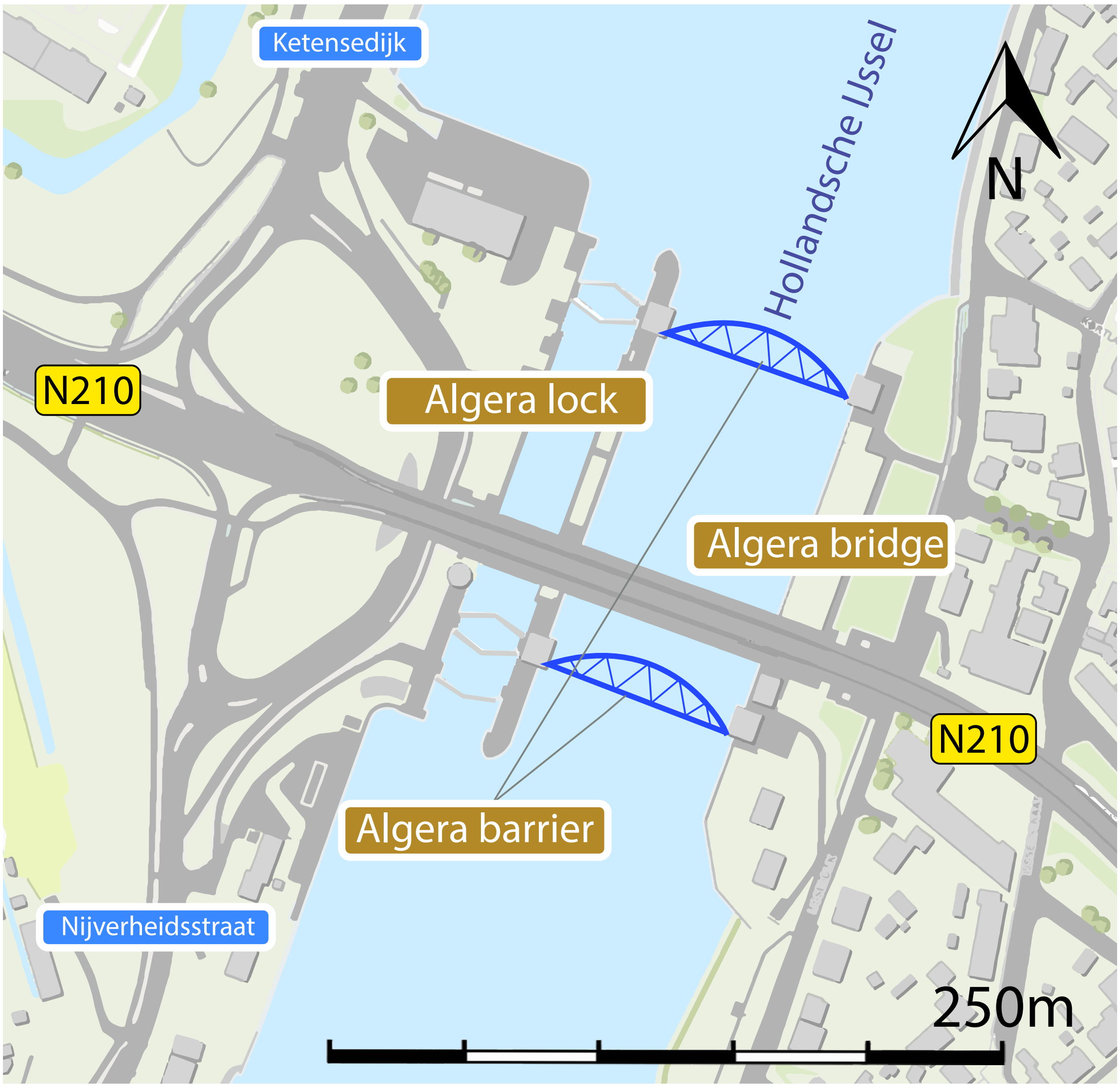

Figure 1.3: The Algera complex, with Capelle at the west side and Krimpen at the east side of the barriers. Source: World Topographic Map.

The Algera or Hollandsche IJssel barrier complex was constructed in 1958 and was the first structure to be completed as part of the Delta Works. It is located between Capelle a/d IJssel and Krimpen a/d IJssel, see figure 1.1. The barrier has been classified as a B-barrier (figure 1.2). The complex, as illustrated in figure 1.3, consists of two barriers, a lock and a bridge connecting Krimpen a/d IJssel and Capelle a/d IJssel. To increase safety, a second barrier was constructed and completed in 1977 (Schoemaker, 2016, p.5). Still, the barrier has low closure reliability (p_f ≈1:100 (Schoemaker, 2016, p.14)) and is expected to reach the end of its technical lifetime by 2050 (Deltaprogramma, 2013, p.8) and needs to be replaced (but not necessarily demolished, it could become an industrial heritage site). Under normal conditions, ships can pass the complex without delay by passing underneath the two barriers. When the barriers are closed or when a vessel does not fit underneath the barrier, the adjacent lock can be used. The road connection is a bottleneck for cross-river traffic (Deltaprogramma, 2014, p.88).

The Juliana lock complex (see figure 1.1) separates the Hollandsche IJssel from the Gouwe and consists of two locks and the mr. Pijnacker Hordijk pumping station. A second lock has been in use since 2014 (van de Sandt, 2014). The complex is classified as a category C barrier, see figure 1.2.

The Waaier lock separates the canalised part of the Hollandsche IJssel from the river that does have tidal influence (see figure 1.1). The lock is classified as a category C barrier, see figure 1.2.

The dikes along the banks of the Hollandsche IJssel border the river and link the different structures mentioned. The dikes are classified as C barrier, see figure 1.2. The dikes have steep slopes, which have led to (macro)stability issues. Besides macro stability issues, shortage of height is also considered to be a problem in some cases. Piping does not play a role (Krol, 2014).

1.3 Previous studies

Previous studies made the Deltaprogramma to decide that the Hollandsche IJssel should be kept open and continue its policy. The current policy and time path of the national government is as follows:

- **2015 - 2030** Improve Hollandsche IJsselkering;
- **2021 -** Improve dikes along Hollandsche IJssel;
- **2050 - 2100** Replace Hollandsche IJsselkering.

A full closure has been investigated by the Deltaprogramma. A qualitative cost benefit analysis showed that no significant savings could be made. This, in combination with the following reasons, has led to the advice to keep the river open and continue with the current policy (Deltaprogramma, 2014, p.88-89):

- Shipping connection;
- Ecology;
- Water quality;
- Dike improvements have to be done anyway;
- Freshwater inlet.

Recently however, Schoemaker (2016) analysed the Hollandsche IJssel and concluded that from an economical perspective, canalisation of the Hollandsche IJssel by closing it off is the best solution. The basic assumption in this research was that at a certain moment in time, the raising of dikes will no longer be possible due to the limited space available. Due to subsidence, the construction of a new storm surge barrier in combination with an *open* sea connection merely postpones the need to raise the dike height. To avert raising dikes, pumping capacity is required. Still, even when canalisation is the economically preferred solution, its impact on the current ecology in and along the river is serious and has to be considered. The freshwater tidal river system is a rarely seen ecotope. Canalisation would imply disappearance of the tidal regime and the related nature. The uniqueness of this type of nature makes this resource valuable. Additionally, European regulation demands removed ecotopes to be compensated elsewhere. A first cost estimate puts a 10 M€ - 1 B€ price tag on such compensation measures (Schoemaker, 2016).

1.4 Research question

The objective of this research is to conduct a feasibility study on a further closure of this river system (the Hollandsche IJssel) that takes into account relevant functions in this river system. Integration of these functions and cost drivers could provide to the most relevant stakeholders with a study that enables them to make well-informed decisions. A closure could significantly impact:

1. Flood protection;
2. Ecological value;
3. Shipping;
4. Cross river transport capacity;
5. Fresh water inlet;
6. Value of property.

Although all six topics are of great importance in further closing the Hollandsche IJssel, this thesis focuses on the first four aspects mentioned. The primary objective is to design a storm surge barrier (topic 1) from an ecological point of view (topic 2): an effort is made to conserve ecological boundary conditions as far as deemed possible. The transport function (shipping, topic 3) of the river is acknowledged and taken into account, however on a secondary level. The same is valid for the road connection (topic 4). Summarising, the main research question is formulated as follows:

> *What is an appropriate conceptual design of a storm surge barrier for the Hollandsche IJssel that takes into account ecological aspects?*

Subquestions regarding **Flood protection**:

1. What are the hydraulic loads the structure needs to be able to withstand?
2. How do soil conditions influence possible degrees of freedom in design?
3. Is pumping capacity needed to maintain a lower water level?

Subquestions regarding **Ecological value**:

4. How can ecological desires and goals be translated into measurable hydraulic boundary conditions?
5. What tidal opening is needed to maintain ecological value?

Subquestions regarding **Shipping**:

6. What are the societal costs of the extra waiting time due to an extra lock which ships need to pass?
7. Is a lock-free passage possible?
8. Is the current Algera lock sufficient for the predicted shipping?

Subquestions regarding **Cross river transport capacity**:

9. What are the waiting/congestion costs?
10. What are the opportunities in creating a better cross river connection?

1.5 Scoping

From the size of the river and the complexity of problems related to the system, it follows that not all related aspects can be researched. Only the topics that are regarded as most important are taken into account and thus a certain scoping is created. Topics that are taken into consideration are however not always researched in the full detail; the bandwidth is reduced as far as deemed necessary for this stage of design. The listing below shows which aspects have been left out of scope and where emphasis was put:

- This thesis only considers replacement of the current Algera complex. A combination of a new barrier together with further dike reinforcements behind the barrier *after* the barrier is realised, is out of scope. *Before* the new barrier is realised, dike reinforcements may be needed to resolve current stability issues (and sometimes insufficient height) (Krol, 2014, p.17).
- Emphasis will be placed on creating maximum value for ecology.
- The focus in this thesis is placed at a design of a barrier. An in-depth analysis of current and future strength of the dikes bordering the tidal part of the Hollandsche IJssel is beyond the scope.
- The barrier is designed on a conceptual level. Therefore, closing reliability in case of storm surge is only assessed on a conceptual level. A full-scale analysis on the reliability of the proposed barrier including expected water levels lays beyond the scope of this thesis.
- For replacement of the current barrier, a decision on its location has to be made. Only locations in the vicinity (within ±3 km) of the current barrier are considered.
- Only the uncanalised part of HIJ is taken into account. Problems with height of dikes along the canalised part, as mentioned in e.g. ARCADIS (2010) and Deltaprogramma (2013), are out of the scope. The functioning of other flood protecting measures, that could have an influence on the HIJ, but are positioned in different river branches, is not altered. The Maeslantkering, located downstream in the Nieuwe Waterweg, is an example of this. Here official documents on strategies and closure reliability, such as Deltaprogramma Rijnmond-Drechtsteden (Deltaprogramma, 2013) are leading.

1.6 Relevance

The relevance of this thesis can be assessed from both a local and an international perspective. The recent findings of Schoemaker (2016), which showed that canalisation is economically the best solution, justify further research into this option. Schoemaker's study however did not take into account ecological costs and benefits. This research could show whether it is possible to create extra safety by canalisation while conserving the tidal climate. Globally, a quick scan (Appendix A) has shown a myriad of metropoles, such as New York, Hamburg and Taizhou, that could potentially be helped by a solution that combines safety and ecology in the abovementioned way. Four cases (of which one is the Rotterdam/Hollandsche IJssel case) were described in further detail in Appendix B.

1.7 Methodology

In this thesis, a case study has been performed. Reason for this is to show the concept's applicability in real life, which is an aspect of importance in applied sciences such as civil engineering. In order to present a design, data needs to be collected. Data is collected by extensive literature research and consultation of experts to capture tacit knowledge on e.g. ecological aspects. The collected data serves as input for models. Where possible, validated models have been used. The design process is iterative; insights could lead to reassessment of previously rejected or approved variants. Emphasis has been put on the aspects that have significant impact on the cost of the project. Here, the work of Schoemaker (2016) serves as a first indication of the relative size of each cost benefit.

The Hollandsche IJssel case has been chosen based on its location and the various benefits clinging to that. Firstly, my presence in The Netherlands allows me to visit the river, consult local experts, interview stakeholders and access data that is often written in the mother tongue. Secondly, the host company showed interest in researching this case. The publication of this thesis in English allows a wider public to research the topic into further detail.

1.8 Report structure

Although a design process is iterative and therefore difficult to capture in a report structure, an attempt has been made to do so. Firstly, the problem at hand and the problem statement have already been introduced in this very chapter. Subsequently, chapter 2 and 3 provide an analysis of the river system, describing the various issues and stakeholders that play a role in the area. Thirdly, the ecology is described (chapter 4). Chapter 5 describes future scenarios, used for comparison of the proposed design to the current trend/strategy. Where possible, the scenarios are quantified in chapter 6. After this, steps have been made towards design. Chapters 7, 8 and 9 present the results of the design process. Chapters 4 - 9 will serve as input for assessment of the proposed design on cost and natural value, chapter 10. Furthermore, a closer look will be taken at whether postponement or earlier implementation has influence on the total cost. The scheme below, figure 1.4, provides an overview of the report's structure.

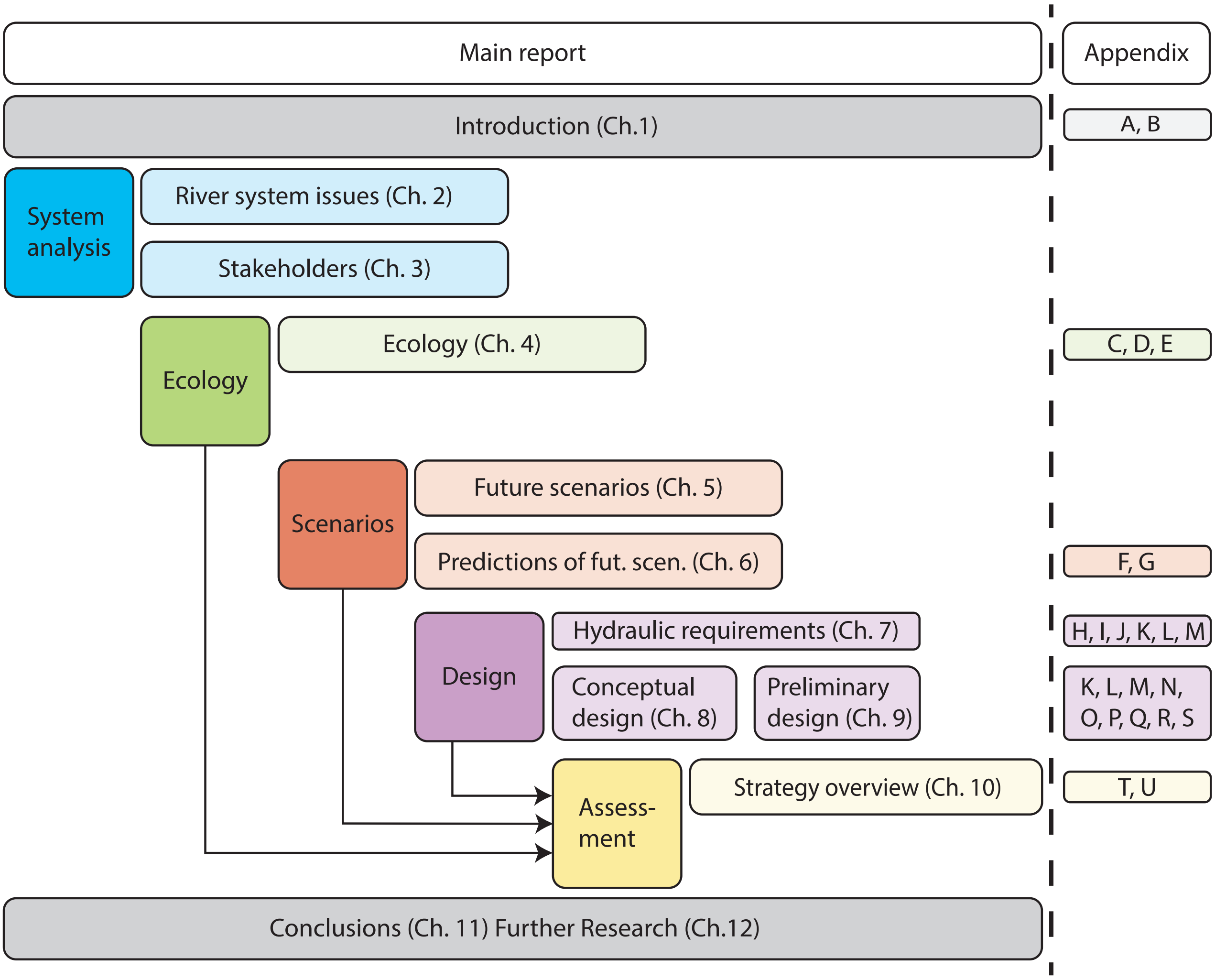

Figure 1.4: Structure of the report.

2

River system issues

This chapter provides a short introduction to the area and the issues that play a role here. It gives insight on the complexity of the river system. Issues that play a role in the river system are:

- Flooding (section 2.1);
- Health (section 2.2);
- Ecology (section 2.3;
- Economic activities (section 2.4).

2.1 Flooding

2.1.1 Subsidence

The area around the Hollandsche IJssel is subject to subsidence. The subsidence is a combination of a soil profile largely consisting of peat and clay and the continuous drainage of the polders (Pieterse et al., 2015). The predicted subsidence in the coming century is depicted in Figure 2.1. Subsidence of the region not only increases the difference in water level inside and outside the river dikes, but also causes their slopes to increase and therefore reduces stability. Subsidence therefore decreases the resistance to flooding.

2.1.2 Climate Change

In addition to the subsidence, sea levels are expected to rise due to climate change (KNMI, 2015, p.14). The open connection of the Hollandsche IJssel to the sea, results in rising water levels in the river as well. Climate change not only influences the height of water levels but also the occurrence of extreme water levels due to storms. Winds from northern direction however, that cause set-up along the Dutch coast, are not expected to increase in frequency and strength (KNMI, 2015, p.16). Furthermore, the expected increase in precipitation will result in higher river discharges (KNMI, 2015, p.22) which in turn lead to higher water levels in the Hollandsche IJssel.

2.1.3 State of the dikes and loads

During the third assessment round in 2011, the majority of the dikes along the Hollandsche IJssel have been assessed as insufficient, illustrated in figure 2.2. The entire east side has been tested as insufficient and also

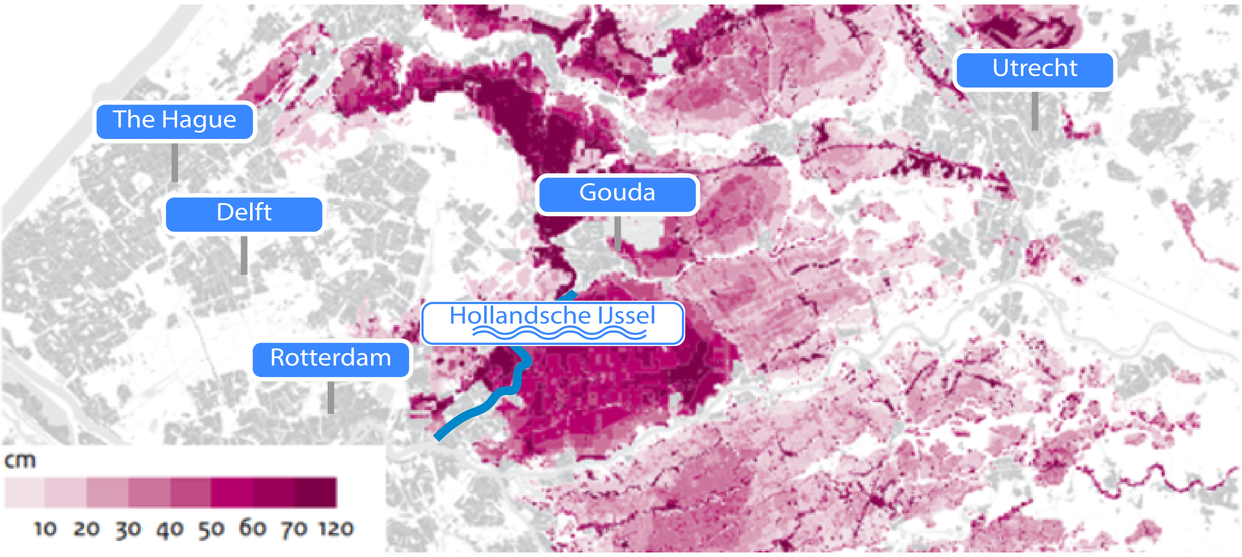

Figure 2.1: Predicted subsicence in 'Het Groene Hart' until 2050. Source: Pieterse et al. (2015).

a major part of the west side lacks safety. Both lack in height and lack of stability are problematic. Steep slopes leading to instability are however the major failure mechanism of current dikes (Krol, 2014).

Taking foreland into account would mean that not all sections marked red in figure 2.2 need to be improved. The cost reduction is estimated to be 1/3. When forelands are included, estimated costs for improving the dikes (mostly at the east side) are still €334M (Krol, 2014). The dike reinforcement measures are costly because a lot of property is present close to the dike (see chapter 6).

The relatively high probability of failure of the storm surge barrier is the major reason why the dikes are insufficiently high. Besides surge level, wind waves also have a significant influence on the required dike height. Reducing water levels are therefore not directly reducing the required dike height one-on-one (Krol, 2014).

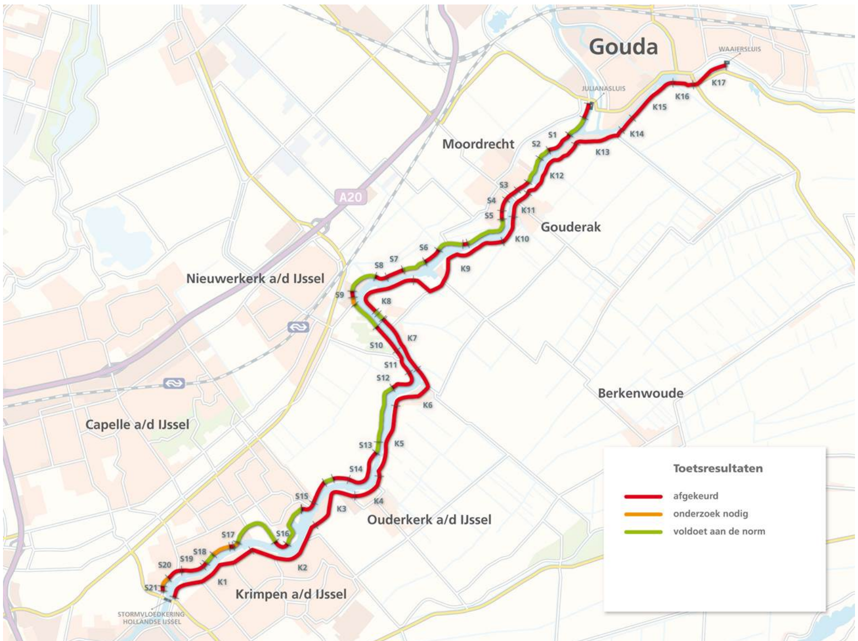

Figure 2.2: Results of third test round 2011. Insufficient dikes are coloured red. Source: Kraaijenbrink (2010).

Intermezzo: Current and new safety standards

Current (2016) regulations are based on probability of exceedance. Dikes and structures should be able to resist all load combinations that have a probability of exceedance smaller than the regulations prescribe. The prescribed standard for each dike ring is based on economic value (van der Most et al., 2014). For DR14 (central Holland) and DR15 (Krimpener- and Lopikerwaard) the probabilities of exceedance are 1 in 10,000 and 1 in 2,000 years respectively (Rijksoverheid, 2009).

New regulations published in 2014 and expected to replace current regulations early 2017, are based on probability of flooding. Dikes and structures should be able to resist various load scenarios that all contribute to a probability of flooding. This probability of flooding is dependent on the resistance of the dike or structure. The maximum probability of flooding is not only based on economical risk, individual and societal risk are also taken into consideration. Individual or local risk is the probability of mortality of an unprotected person constantly present at one location. Societal risk is the risk of an event with multiple fatalities. In general, societies tend to be risk averse: small probability events with large loss in life have a large impact on society (Jonkman et al., 2016, p.83-85). The concept of dike rings is replaced by dike trajectories. As a breach in one trajectory could have more severe consequences than in another, the desired probability of flooding is determined per trajectory instead of per ring (van der Most et al., 2014). The new regulations require the east (DR14) and west (DR15) dike trajectories along the Hollandsche IJssel to have a probability of flooding of 1 in 30,000 and 10,000 years respectively (Deltaprogramma, 2014, p.64). Probability of flooding and exceedance probability are not the same. A flood probability combines exceedance probability with probability of failure (Schoemaker, 2016, p.13).

2.2 Health

Dumping of waste and industry along the banks have resulted in deposits of various unwanted chemicals in the ’50s and 60s. Lekkerkerk, a village in the Krimpenerwaard municipality, was subject to one of the biggest chemical waste dumping scandals in the Netherlands (Aarden, 2005). Demolition of 100 houses in the Zellingwijk in 1985 due to contaminated soil illustrate the severity of the problem along the Hollandsche IJssel (Rijksoverheid, 2011). More recently, contaminated soil has been dredged (Informatiehuis Water, 2015, p.530-543) and forelands or *zellingen* such as the Geitenwei (Gemeente Krimpenerwaard, 2015) have been remediated. The presence of chemicals in the soil could impose health risks to the population. At the time of the scandal in Lekkerkerk, fear of intrusion of harmful chemicals into drinking water led to national debates about remediation of contaminated soil.

2.3 Ecology

The presence of the abovementioned harmful chemical substances in the soil and the strongly altered character of the river influence its ecological value negatively. The recent improvements (sealing and cleaning), partly caused by KRW regulations, have improved the ecological value. In periods with very low discharge of the Rhine, salt water may enter the Hollandsche IJssel upto Gouda (Rijkswaterstaat, 2015). This is a potential threat for the flora and fauna that has adapted to fresh water.

2.4 Economic activities

Along the Hollandsche IJssel various fresh water inlets are positioned of which 'gemaal mr. Pijnacker Hordijk' is the largest (Rijkswaterstaat, 2005). The fresh water is used for agricultural purposes, as well as prevention of intrusion of salt water and subsidence of peat layers that dry out (Hoogheemraadschap van Rijnland, 2011). The intrusion of salt water, also mentioned in section 2.3, is a potential threat to the fresh water inlets.

2.4.1 Shipping

In the past, stone factories were situated along the Hollandsche IJssel (HV Ouderkerck op d'IJssel, 2016). Nowadays, industry along the river is still present and mostly in need of a river connection. Besides serving as a direct connection for industry along the river, vessels also use the river as a transport connection to reach facilities further upstream. The Hollandsche IJssel is navigable for CEMT class Va ships (Rijkswaterstaat, DVS, 2009). The majority of the commercial ships continue to the Gouwe canal, which is a CEMT class IV waterway (Provincie Zuid-Holland, ndb). Approximately 31,000 ships pass the Juliana locks per year, of which 16,000 are commercial (Rijksoverheid, 2016). As ship passages are recorded at locks and not all ships pass the Algera lock (only the vessels that are too high to sail under the Algera barrier), the exact number of ship movements through the Algera complex remains unknown. The number of passages through the Juliana lock however serves as a good estimate. The Hollandsche IJssel is part of the *Staande mastroute*, which means that ships with high masts can sail from the North to the South of The Netherlands without height restrictions (Rijkswaterstaat, 2014).

2.4.2 Traffic

The Algera bridge and the road connection over the Juliana locks are the only bridge connections over tidal part of the Hollandsche IJssel (Gouda - Krimpen a/d IJssel). The Algera bridge corridor is notorious for its congestion (Rotterdam Vooruit, 2010).

2.4.3 Residential function

Living along the banks of a river is popular in The Netherlands. Currently, large parts of the dikes are covered with ribbon development. This makes dike improvements more costly to implement and is a major driver to investigate a new Algera complex instead. New houses and apartment buildings have been built outside the dikes, and also at the inside of the dikes Gouda and greater Rotterdam wish to expand (Projectbureau Westergouwe, nd). This means that the north western side of the Hollandsche IJssel will be more and more a residential area at the cost of agricultural land. The increased value of the area that comes along with the development plans amplify the consequences of flooding. Various recreation areas and marinas are present along the river. The river however is not considered to be swimming water (RWS Waterdienst, 2012).

3

Stakeholder analysis

This chapter introduces the stakeholders that are present and their relation to the issues described in the previous chapter. Only the most important organisations are discussed. It is assumed that the interests of individuals with a significant stake (such as home owners or farmers) are represented by the governmental bodies.

3.1 Rijkswaterstaat

Rijkswaterstaat is the national governmental organisation responsible for the navigability of the Hollandsche IJssel and maintenance of the storm surge barrier. Their dual responsibility results in high interest in this project. If executed, Rijkswaterstaat will probably initiate it and the funds for a new barrier will possibly come from this organisation, making it a powerful player.

3.2 Province Zuid-Holland

The province Zuid-Holland is a regional governmental power that could act as a facilitator between the different parties with different objectives. The province took part in a previous project that involved removal of sources of contamination in the river (Provincie Zuid-Holland, nda).

3.3 Waterboards

The waterboards are responsible for water quality, availability, flood protection and waste water treatment (Rijksoverheid, ndb). They collect their own taxes and therefore have their own funds to execute projects. Three waterboards are present along the Hollandsche IJssel, which is further explained in the paragraphs below.

3.3.1 Hoogheemraadschap Schieland en Krimpenerwaard (HHSK)

As can be seen in figure 3.1, HHSK borders most of the Hollandsche IJssel. A recent assessment of the dikes along the HIJ showed that the majority has insufficient capacity to protect the surrounding area from flooding (Deltaprogramma, 2015). This has led to initiation of the project KIJK, that envisions the improvement of dikes along a large part of the south east banks of HIJ.

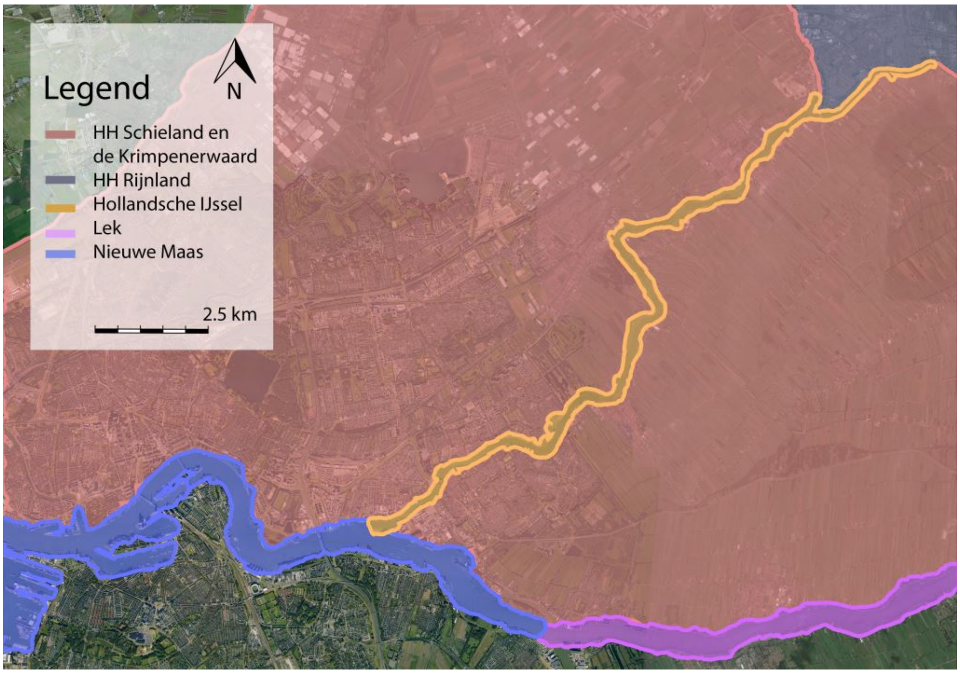

Figure 3.1: Region of influence of relevant waterboards. HHSK is the red region, HHR is the grey region. Source: Google Inc. (2016).

3.3.2 Hoogheemraadschap van Rijnland (HHR)

HHR is responsible for only a minor part (in terms of length) of the flood protection (see figure 3.1). Their stretch however protects Gouda, meaning that a breach in their part would also lead to significant damage. A major part, 2.1 km, is insufficient in terms of strength (Deltaprogramma, 2015). The area that HHR is responsible for, runs in northern and western direction and encompasses large polders with organic soil that need to be kept wet to reduce subsidence. Additionally polders in the west of the Netherlands are subject to salt intrusion if the water table drops too much, harming agricultural activities. HHR uses river water to maintain the water table in arid periods. A significant intake of water is situated along the Hollandsche IJssel (mr. Pijnacker Hordijk Gemaal).

3.3.3 Hoogheemraadschap de Stichtse Rijnlanden (HdSR)

HdSR is responsible for maintenance of the Waaiersluis, the eastern boundary of tidal influence in the Hollandsche IJssel. Their main concern is the ability to discharge water into the river and take water in when needed. The Waaiersluis is also the endpoint of the 'Kleinschalige Wateraanvoer' (KWA), which can be used to keep the intake at the mr. Pijnacker Hordijk pumping station fresh in arid periods (Krol, 2014, p.26).

3.3.4 Interests and power

In general, one could say that the waterboards have similar interests. Since the boards are co-responsible for flood protection and fresh water supply, their interest in a solution is considered to be high. Their formal authority and financial capabilities make them players to take into account seriously.

3.4 Municipalities and their inhabitants

As can be seen in figure B.7, there are five municipalities that border the Hollandsche IJssel. The municipalities are expected to act in the interest of their inhabitants. The municipalities of Capelle a/d IJssel, Krimpen a/d IJssel and Gouda have an urban character, while Zuidplas and Krimpenerwaard have a rural character. Water quality is important for both types. Citizens need clean water for recreation, while farmers need fresh water (not salt) for their crops. Contaminants can have both a human (medicines etc.) or an agricultural origin (phosphates from fertiliser). In terms of flood risk, both types of municipalities have the same goal: reduce the risk as much as possible. While urban areas have no storage capacity, rural municipalities do. Raising dikes is an unpopular intervention in urban areas. Historically, towns and villages have grown around the dikes and raising them would not only lead to reduced value for their owners but also make the area less attractive for tourism and recreation. The rural character of the other municipalities has not led to complete absence of houses along the dikes. Here, raising is problematic as well.

The level of interest is considered to be high: the risk of flooding of the municipalities is directly influenced by alteration of the flood protection infrastructure. The power of the municipalities is considered to be medium as the municipalities and their inhabitants do own land which might be built upon, but do not have a direct responsibility for maintenance of one of the river's functions or flood protection.

Table 3.1: The municipalities, their character and their specific interests

Municipality	Character	Water quality - contaminants	Salt water intrusion
Capelle aan den IJssel	Urban	++	0
Zuidplas	Rural	+	++
Gouda	Urban	++	0
Krimpenerwaard	Rural	+	++
Krimpen aan den IJssel	Urban	++	0

3.5 Freighters/vessel owners

This group of stakeholders can be divided into two groups: professional shipping (1) and recreational shipping (2). Freighters transport goods and materials from the Rotterdam harbour to terminals in e.g. Alphen a/d Rijn. It is the only connection of Alphen a/d Rijn southwards navigable for CEMT IV class ships (Provincie Zuid-Holland, ndb). While freight ships are often constricted to the width and depth of waterways, the height above the water of bridges etc. is a limiting factor for recreational shipping. The Hollandsche IJssel is part of the *staande mastroute*, which means that ships with high masts can sail from north to south NL relatively easy (Rijkswaterstaat, 2014). A waterway with dimensions in which CEMT IV ships can sail and that has no height restriction is therefore required. The government policy to move freight from the road to the waterways (Rijksoverheid, nda) and the expected growth of a local container terminal (Gemeente Alphen aan den Rijn, 2012) stress the importance of the waterway for nautical purposes. The power of this stakeholder is expected to be medium because they have no formal power, but can potentially mobilise a larger public and have various lobby organisations. Since any adjustments will directly influence their business (or leisure possibilities), their interest in adjustments is high.

3.6 Local industry

Local industry along the banks uses the transport possibilities, similar to freighters. Not all industry is directly in need of the waterway and transport could be organised over land. This however requires investments. The industry is mostly located outside the dikes and is only protected by the storm surge barrier. Additionally, owners have increased the height of the forelands (*zellingen*) to reduce their individual flood risk and make berthing possible. Their interest is considered to be equal to the freighters. Due to their ownership rights and possible lobby organisations, but their absence of 'real' influence, their power is considered to be of medium level.

3.7 Environmental organisations

Environmental organisations try to preserve or improve e.g. ecological value. The organisations give nature a voice. Based on their activities, organisations that are interested in the HIJ and potential changes in this river are e.g. WNF and Zuid-Hollands Landschap. The nature organisations have a high level of interest since tidal influence may be altered which in turn can influence natural value. Their power is considered to be of medium level because the organisations have no formal power. They can however can mobilise the crowds. The introduction of European legislation, which forces national governments to restore water bodies as much as possible (KRW, see chapter 4), has increased the power of the nature organisations.

3.8 Summary

Figure 3.2 summarises the interests and power of different stakeholders. It shows that the larger governmental organisations (the waterboards, RWS and the province Z-H) have more power and should be managed closely. The other organisations have less power. Informing these stakeholders is sufficient, at least for the scope of this thesis.

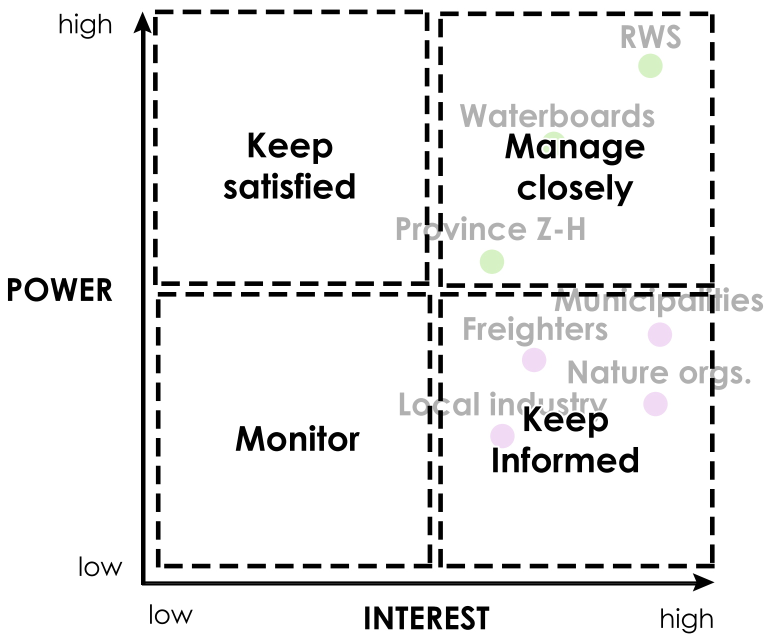

Figure 3.2: Power and interest of different stakeholders and the action that should be taken. Source: Gardner et al. (1986)

4

Ecology

4.1 Introduction

Natural value is an aspect that is taken into account increasingly when it comes to flood protection of estuaries. In The Netherlands, the first closures as part of the Dutch Delta Works were solely meant to reduce the shoreline and improve the safety against flooding. A logical step, given the fact that the Dutch Delta was just struck by disaster (van Wesenbeeck et al., 2014, p.5). Direct economic benefits for shipping were also taken into account leading to the open characteristics of the Algera Barrier and the decision to keep the Western Scheldt open. Later on, projected (Eastern Scheldt barrier) or already realised (Haringvliet) closures were adjusted due to the increased societal pressure to take into account ecological value as well. The introduction of legislation protecting natural habitats, such as Natura2000 and Water Framework Directive is a logical result of the societal desire to protect these ecosystems. Appendix C provides a full overview of the development of closures in The Netherlands.

Outside The Netherlands, valuing the natural component of estuaries has led to ‘green’ rather than ‘grey’ flood protection plans as well. One example is New York. The city presented a vision for 2020 regarding its waterfront in which ecology is explicitly mentioned (Dept. of city planning, city of New York, 2011). Furthermore, as a result of societal desires, recreation forms an important part of the vision. It should be mentioned that this vision was published before hurricane Sandy struck. One could expect that after such a disaster, ideas to more drastically close off the estuary would surface. The contrary seems to be true, illustrated by the mayor at that time (Bloomberg): “I don’t think there’s any practical way to build barriers in the oceans. Even if you spent a fortune, it’s not clear to me that you would get much value for it” (Feuer, 2012). Plans presented after Sandy do not include large closures and do not neglect ecological value. Several ideas have been presented, including multifunctional barriers (Big U, see figure 4.1) and urban wetlands (ARO and dandlstudio, see figure 4.2).

Figure 4.1: Multifunctional barrier (yellow) in New York. source: www.tribecatrib.com

Figure 4.2: Urban wetlands in New York. source: www.nytimes.com

4.1.1 Structure of this chapter

The remainder of this chapter can roughly be split up into two parts, being (1) a more general part on how to take ecological value into consideration (sections 4.2 - 4.5), followed by (2) an elaboration on how to take ecological value into consideration in the case at hand (sections 4.6 - 4.8). Both parts consist of three similar sections, namely (1) a description of economical, societal and legislative pressures, (2) a description of aspects that are considered to be important and (3) which aspects to take into account. Additionally, in the general part, the approach on natural value is explained (section 4.3). This chapter is wrapped up by a summary (section 4.9) and the boundary conditions that have resulted from the analysis in this chapter to be used for design purposes.

4.2 Types of pressure

Based on the process towards incorporating natural value in a design, described in the introduction, three forces were distilled that result in an 'eco-friendly' design, which will be explained in more detail in the following sections:

1. **Economic presure**;
2. **Societal presure**;
3. **Legislative pressure**.

4.2.1 Economic pressure

People can yield economic benefits (both direct and indirect) from an ecosystem. Aquaculture, fisheries and coastal agriculture in estuaries provide food and a source of income (Wetlands International, nd). An open connection to the sea could be an economic driver for ships to berth in nearby ports, thus leading to economic value. Also, houses bordering water are highly valued. Indirect sources of income encompass for example tourism or recreation in the estuary. Economic forces are relatively easy to translate into €-value.

4.2.2 Societal pressure

Societal pressure represent the less direct benefits that can be yielded from ecosystems. Naturally, as members of society seek economic benefits as well, societal pressure overlaps with economic pressure. As mentioned in the introduction of this chapter, at least in developed countries, a trend can be seen towards conservation of estuaries. Different aspects play a role, shortly introduced in this section. One could think of the landscape that can be enjoyed or the desire for recreation close to urban areas. Another example is the increased concern in society in well-being of species, exemplified by election of the 'Partij voor de Dieren,' (a political party focusing on well-being of animals) in Dutch parliament. More indirect benefits include preservation of ecosystems for posterity to enjoy.

A major difference between societal benefits and economic benefits is that economic benefits can be measured more directly and therefore more objectively in comparison with societal pressure. One could express e.g. the economic benefits of aquaculture, by calculating the yearly yield. However, if one needs to calculate the benefits a landscape that can be enjoyed, things become more difficult, for example because not everybody likes the same landscape.

4.2.3 Legislative pressure

Rules and regulations are meant to protect the weak, in this case an ecosystem. Various laws are in force to protect natural value, such as Natura 2000, Ecological Framework (Ecologische Hoofdstructuur) and Water Framework Directive (Kaderrichtlijn Water). Legislation also envisions compensation elsewhere in case of damaged or lost ecosystems. One could say that nature laws embody the indirect societal pressures such as preservation for posterity. With legislation in place, one cannot only look at direct economic benefits and is forced to comply with rules and regulations.

Legislation focusing on chemical composition is relatively easy to take into account. Laws on water quality are an example. Concentrations of contaminants in the water should be below the norm. Imagine a decision to create a constriction in an estuary with a contaminated soil layer. The structure increases flow velocities and could therefore stir up contaminated sediment. In order to check whether the norms are exceeded, one could model the created scour hole(s) and the released contaminated sediment.

Other legislation focuses more on the ecosystem of a water body. Target species or *doelsoorten* give an indication of the ecological composition and therefore the quality of of a certain ecotope in a waterbody (Bal et al., 2001; Nijboer et al., 2000). Legislation often requires a certain amount of target species in an area. Since direct relationships between altered conditions and occurrence of (target) species are less available, the influence of a structure on ecological composition is harder to assess.

4.3 Approach of ecological value in this thesis

This section will explain how ecological value is taken into account in this thesis. It will shortly introduce the current attempts already made to fully take into account ecological value and explain what is missing.

In engineering, value is mostly accredited to direct, and to a lesser extent to indirect, benefits to mankind. As mentioned in the previous section, it is easiest to take direct economic benefits into account, such as accessibility for shipping and economic gains from fisheries. It becomes more difficult to take into account indirect (societal) benefits, such as a landscape that can be enjoyed, the value of well-being of animals or the preservation of an ecosystem for posterity. The information on the value of these constructs is incomplete

(Ruijgrok, 1991). Whether direct or indirect, the benefits obtained are benefits to mankind (anthropocentric view). A more complete approach on value of an ecosystem would be to value it according to the benefits yielded by all species (biocentric view). In other words, is nature only of value when it has instrumental (anthropocentric) value, or does nature have intrinsic (biocentric) value (TEEB, 2010, Ch. 5) as well? Answering this question forms the basis of ongoing discussion between nature conservationists and preservationists (Cunningham, nd). It is tempting to have an anthropocentric view and only look at quantifiable benefits to mankind. The next section provides a thought experiment that might prove differently.

4.3.1 The last man

To illustrate that human ethics and with it the biocentric/intrinsic/preservational view should be taken into consideration, Routley (1973) published a thought experiment, called ‘the last man’:

> *Imagine that except for one, all humans are dead. This last man/woman decides to destroy all species and has the proper equipment to do so. Is it then morally right for this last person left to extinguish all life?*

Many will answer this experiment with a *no*, implying that besides having instrumental value, nature has intrinsic value as well. This intrinsic value cannot be quantified in terms of benefits to mankind because there are none (all humans are dead). Because of the lack of instrumental value, it is however difficult to preserve species or ecosystems just based on their intrinsic value.

4.3.2 Ecosystem services

The thought experiment by Routley caused quite a change in the view of accrediting value to nature. Although intrinsic aspects of natural value are difficult to be taken into account, at least attempts should be made to appreciate the more indirect aspects of natural value. “Ecosystem services” is such an attempt.

Ecosystem services can be defined as “the aspects of ecosystems utilized (actively or passively) to produce human well-being” (Fisher, 2009, p. 646). Indeed, as the definition mentions human well-being, only the instrumental benefits are taken into account. This said, it is a good attempt to take at least all possible instrumental benefits into account. Of course the question remains what value to attach to each service delivered. The services that can be provided by an ecosystem are (Millennium Ecosystem Assessment, 2005, p. v-vi),(de Boer, 2015, p.44):

- Provisioning: tangible products that can be obtained, such as food, water and clay;
- Regulating: benefits that affect e.g. the climate, floods and water quality;
- Cultural: non-tangible benefits e.g. recreational, aesthetic, educational and spiritual benefits;
- Supporting: indirect benefits e.g. nursery provision, soil formation and nutrient cycling.

4.3.3 Conclusion

The previous sections have shown that current ways of valuing an ecosystem are not flawless. Firstly, there is no complete information on the value of indirect benefits to mankind (antropocentric view). Secondly, intrinsic value of nature is not taken into account. The *last man argument* however implied a necessity to do so. Although ecosystem services are a good attempt to take all instrumental value into account, it can however be concluded that direct value attribution to nature is imperfect and (intrinsic) value is left on the table. This raises the question whether attempts should even be made to translate the created or conserved ecotopes into €-value. An analogy can be seen with how human life is taken into account in risk calculations. Loss of life is not expressed in monetary terms either, but in terms of individual risk (at least in The Netherlands). The individual or local risk in a new situation should e.g. be at most 10^{-6} per year (Jonkman et al., 2016).

Based on the analysis above, it is decided that in this research ecosystems will not be rated on their instrumental value. Focus will be placed on the unique aspects that the current system has and how these can be conserved. E.g. an estuary could be unique for its large tidal range in combination with alterations in fresh and salt water. In that case, the tidal range and the shifting locations of fresh, brackish and salt water should be conserved. Section 4.4 shows what aspects one should think of in general. In section 4.7 and 4.8 the aspects are specified for the case at hand.

4.4 Important ecological aspects

Several aspects define the ecological value of an ecosystem. Factors that influence an ecosystem can roughly be divided in biotic and abiotic factors. Strictly speaking, the natural dynamics of an ecosystem are an interplay between abiotic and biotic factors. However, the matter is treated separately in this section. The following aspects will be discussed:

1. **Biotic factors**;
2. **Abiotic factors**;
3. **Natural dynamics**.

4.4.1 Biotic factors

Biotic factors are factors of biological origin that affect or influence ecosystems (Reference.com, nd); e.g. the lack of prey leads to a less suitable environment for a predator. Another example is a beaver that can alter an environment by changing water levels. Next to beavers, humans are even more capable of altering an ecosystem. In line with the beaver, human intervention is also a biotic factor. Examples of human intervention are (over) fishing, altering abiotic factors such as releasing or removing contaminants, regulating the water levels or limiting access for migratory fish. To check whether a good biotic situation is created, the actual ecotope should be examined on site.

4.4.2 Abiotic factors

Abiotic factors are non-biological aspects that influence an ecosystem. Abiotic factors include temperature, water and soil chemistry, water clarity and tidal range (Reference.com, nd). The presence of contaminants such as PAKs, drins and heavy metals influence the quality of an area negatively. The effect of other abiotic factors is however not straightforward. A rich supply of nutrients is beneficial for certain species, while in nutrient-poor environments other organisms thrive. The clarity of water can also have an ambiguous outcome. While clearer water allows submersive plants to grow at larger water depths, it may have negative effects on prey that cannot hide for predator fish. Presence of tide is not beneficial or disadvantageous in all cases either. While some plants have adapted to fluctuating water levels, others cannot deal with it. In conclusion, one can say that the presence and occurrence of abiotic factors leads to a certain ecosystem. Depending on which ecosystem is aimed for, presence of an abiotic factor can either improve or reduce the quality of an ecosystem.

4.4.3 Natural dynamics

An ecosystem is hardly ever stable. At least in dynamic environments, such as estuaries, a more stable environment leads to different conditions and could thus result in different ecotopes (van den Broek, 2016; Braakhekke, 2016). The development of an ecosystem into another ecosystem over time is called succession. The attempt to create natural value does not have a binary outcome in a sense that natural value either will or will not be there. In the Netherlands, a forest is the end stage or steady state of natural development or succession. Figure 4.3 shows the different stages of succession in a fresh water environment.

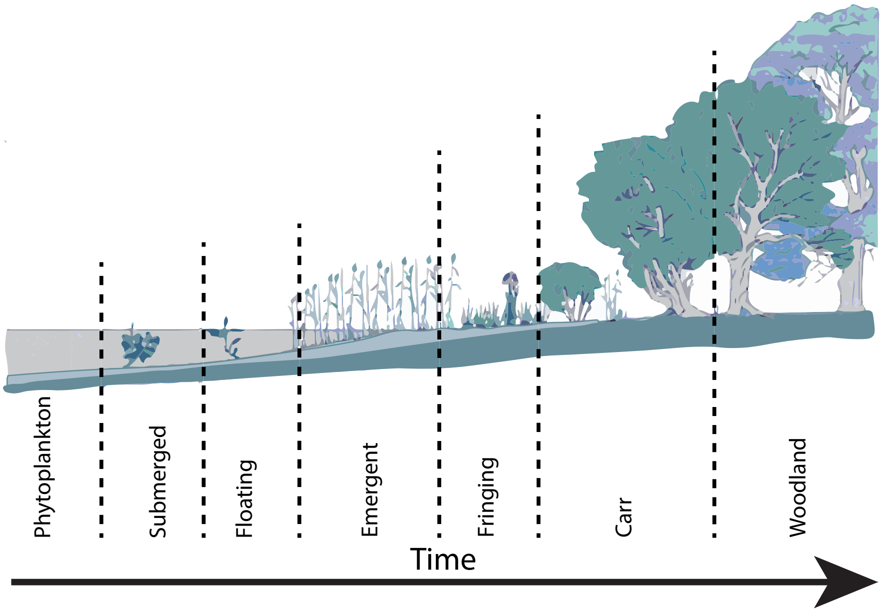

Figure 4.3: Succession in a fresh water environment. Source: biogeography.weebly.com

4.4.4 Note: from ecological conditions to ecological value

One can say that the right abiotic environment forms a boundary condition for development of a suitable biotic environment. If one wants to maintain a certain state of an environment, a solid maintenance plan should be in place to keep an environment in such way. Realising and maintaining the right ecological conditions is a prerequisite to create ecological value, however it is not necessarily the result. In order to actually create natural value, valuable ecotopes should be present which in turn lead to ecological value. Whether this occurs, should be verified on site after realisation of the construction/measure.

4.5 Taking ecological aspects into account

After the relevant (a)biotic and dynamic aspects are identified, one needs to incorporate them in further design steps. This section will first show that boundary conditions can be created, followed by a short introduction of software that can be used as aid. This is followed by an explanation of when in the lifetime of a structure each of the aspects is important.

4.5.1 Boundary conditions

As mentioned in section 4.3, focus in this thesis is placed on the unique aspects of an ecosystem. Target species or *doelsoorten* can serve as an identification tool on whether an ecosystem can be considered of sufficient quality. Creating or conserving the right conditions for target species of a desired ecosystem are indications of a good ecosystem. In order to define what *right conditions* are, one could use dose-effect relationships. The dose-effect relationships show for example at which water depths certain species can thrive and thus what *right conditions* for that type of species are. Deltares provides dose-effect relations of different abiotic factors for different (target) species in an ecological knowledge base (Deltares, nd). Furthermore, laws and regulations provide insights on maximum allowable concentrations of contaminants.

4.5.2 Software

Using computer programs to predict the ecological value of measures is relatively new in The Netherlands. STOWA (2009) provides an overview of available programs which can be used for ecology in Dutch water management. The first version of KRW Verkenner for example was released in 2005 (Deltares, 2006). Consultation of different ecologists made it clear that no other programs have been introduced recently. The programs mentioned in STOWA (2009) have however been improved. The KRW Verkenner is one of the most sophisticated programs to predict ecological value. Another useful program is HABITAT. HABITAT is a spatial analysis tool in which the quality and availability of habitats can be assessed (Deltares, nd). It should be mentioned that there are more programs, such as PCLake and PCDitch, that have predicting capabilities. A program such as PCDitch focuses on ditches while in this study a tidal river system is analysed. It is therefore not relevant for this study.

4.5.3 Phases in construction lifetime

This section provides a hands-on guideline on how to take into account ecological value during the lifetime of a construction.

Design & Construct
During the design process, one should carefully think of what the desired ecological situation is. One should ask the question: ‘What kind of ecosystem do we want to create or maintain?’ The follow-up question is how to reach that goal. Different biotic and abiotic factors have to be taken into account. Given the desired ecological situation, one can define target species, which in return desire certain boundary conditions, such as a minimum required tidal range or water depth (see e.g. the paragraphs above or section 4.8). Dose-effect relations describe how the suitability for species changes by changing the boundary conditions. Different dose-effect relations are provided by Deltares (nd), see also figure 4.6. Abiotic conditions such as water

quality are specified in laws. During construction, one should be aware that this temporary phase does not permanently damage the ecosystem it is trying to preserve. For example, one can imagine that cutting off an area from nutrients for a long period of time during construction can seriously harm an existing ecosystem.

Maintain
Once a desired situation is reached, it will not remain that way. In the Netherlands, a forest is the end stage or steady state of natural development or succession. Figure 4.3 shows the different stages of succession in a fresh water environment. Nature around rivers and estuaries is however not often reintroduced to create a forest. Therefore, a certain amount of maintenance needs to be done to keep an ecotope in a certain state. This raises the question to what extent and at which cost a certain ecotope needs to be maintained (STOWA, 2011). Maintenance related questions are e.g.: 'do we let cattle graze to keep trees from colonising the tidal areas? Do we actively prevent salt water to enter even though it would be more natural to slowly make the area saline?' The questions raised are to be answered by decision-makers, it is the task of an engineer to deliver the right initial conditions.

4.6 Pressure in the case

This section shortly describes a few economic and societal forces in the case. After that, more attention is given on Water Framework Directive legislation.

4.6.1 Economic pressure

The river delivers economic value in several ways. As already introduced in chapter 2, the river is used for navigation and people like to live along the water. Although important to be maintained, the quality of the ecosystem in the river is of little influence to these forces. Of course, if the water quality of the river drastically drops, living along the water has no added value. Drastically reducing the water quality in the river is however not in question.

4.6.2 Societal pressure

Several societal forces press towards conservation of the freshwater tidal river system. The societal pressure can be divided into local pressure and general trends. As mentioned in chapter 2, contaminants in the soil led to serious health issues with demolition of houses and sanitation of forelands as a consequence. It is expected that these sentiments will lean towards keeping the river in open connection as to further dissolve the contaminant concentrations. Furthermore, as mentioned in the introduction of this chapter and in Appendix C, it is a more general trend to keep estuaries open.

4.6.3 Legislative forces: KRW

Due to their cross-national character, pollution and ecological decay of rivers and waterways is an issue with international dimensions. In order to tackle this supra-national problem, the European Commission introduced Water Framework Directive or *de Europese Kaderrichtlijn Water* (KRW) in Dutch. In the KRW, different types of water have been distinguished. For each type of water body, an original physical state has been described. Besides the physical state, the KRW describes different types of algae, plants, animals and fish that characterise the type of water (STOWA, 2005). The original state of each water body is presented as the ideal situation.

Rivers, estuaries, lakes and waterways have been altered by mankind in the past centuries. Especially a country like The Netherlands is famous for its creation of land, mostly at the expense of natural habitat. Seas have been turned in to lakes, lakes have been turned into polders and rivers have been tamed and constricted by surrounding dikes. In the last decades however, projects such as Room for the River (Ruimte voor de Rivier, 2016) show an opposing trend of nature compensation and restoration in combination with increasing safety standards. Building *against* nature is replaced by building *with* nature.

It should be stressed that the KRW has not been initiated to restore all water bodies to their original state (STOWA, 2005). From an economical and/or political perspective, full restoration is often considered to be infeasible and unwanted: it would probably lead to a drastic reduction of flood safety in large parts of the surrounding areas. In terms of regulations, this has led to separate regulations for natural and strongly altered water bodies.

Assessment water body with KRW

For every type of water body, standards have been introduced for both ecological and chemical compositions. The reduction of tidal action could however lead to a decrease in the refreshment rate of water and a build-up of e.g. phosphates, nitrogen, drins and PAKs (RWS Waterdienst, 2012). However, point sources of contaminants along the Hollandsche IJssel (side inflows of e.g. contaminated forelands) have been removed or isolated. The contaminated river bed (diffuse source) is sealed off with a layer of sediment (de Haan, 2016). Water quality standards are therefore less relevant for this research. Influence on the tidal amplitude and in and out flux of water could however be of influence to organisms, making ecological standards of importance.

Natural water bodies in the KRW are assessed with a five-point scale, ranging from bad or *slecht* (score =0) to very good or *zeer goed* (score=1). Assessment of natural water bodies should a least result in a 'goed', or a good ecological condition (GET), which is four out of five on the five-point scale (score =0.8). For man-made or strongly altered water bodies, an adjusted scale that is based on the assessment of natural water bodies is used. This is a four-point scale ranging from bad or *slecht* to good or *goed*. Man-made or strongly influenced water bodies have to score a 'goed' or good ecological potential (GEP). The GEP differs per water body, with a maximum of 0.6. The ecological potential is assessed on four types of organisms, being:

- Phytoplankton;
- Vegetation;
- Macrofauna;
- Fish.

It should be noted that both the variety of different species and abundance of each individual species is part of the KRW assessment. For rivers, phytoplankton is not assessed in the KRW.

Hollandsche IJssel in KRW framework

As mentioned above, different standards govern for different types of water bodies. This is logical as different biotopes house different organisms. Presence of an organism may indicate improvement of the ecological condition in one biotope, while it could signal deterioration in another biotope. For the Netherlands, four categories of water bodies can be distinguished, which in turn can be subdivided into 42 types. The Hollandsche

IJssel is classified as a fresh tidal water (R8 in KRW) (RWS Waterdienst, 2012). RWS Waterdienst (2012) concluded that the Hollandsche IJssel is strongly altered and that *hydromorphological* adjustments with a substantial beneficial effect on the ecological value would be significantly harmful to its current functions (p.18-22). Improvements that are possible, are however not enough to reach good ecological conditions (GET). Therefore, adjusted scales have been introduced, on which the Hollandsche IJssel has to reach good ecological potential (GEP). A good ecological potential in the Hollandsche IJssel has been defined for each type of organism, shown in table 4.1. In this table current and future scores are presented as well. It can be concluded that the river does not have enough potential yet to comply with the adjusted KRW scales (e.g. for 2015, a score of 0.49 for macrophytes/phytobenthos is expected in 2015, while GEP is 0.53).

Table 4.1: KRW Score of the Hollandsche IJssel. The scores in 2015 and 2027 are expectations.

Element	GEP (RWS, 2016)	Current (2006 - 2008) (RWS Waterdienst, 2012)	2015 (Buijse et al., 2008)	2027
Macrophytes / Phytobenthos	0.53	0.38	0.49	0.52
Macrofauna	0.42	0.27	0.42	0.42
Fish	0.19	0.32	0.32	0.32

4.7 Important ecological aspects in the case

When designing a further closure of a tidal river system, two abiotic aspects are of major importance, namely:

1. **Tidal range behind the barrier**;
2. **Fish migration through the barrier**.

4.7.1 Tidal range behind the barrier

Tidal action in the Hollandsche IJssel is the most unique aspect of this freshwater tidal river system. Further constriction of the river by means of a new barrier could lead to reduction of the tidal range. This could lead to loss of value of this freshwater tidal river system, because a wider tidal range is considered to be better (Braakhekke, 2016; van den Broek, 2016). Spring tide is less important, although it could lead to a reset of succession (van den Broek, 2016). Areas that are only subject to spring tide could be inhabited by willows (Nijboer et al., 2000). In Nijboer et al. (2000), tidal waters are classified into three groups:

1. **Intertidal zone:** the area between average high water (GHW) and average low water (GLW). This area is the most unique zone of the tidal river system. Several species that appear here cannot be found elsewhere.
2. **Shallow tidal waters:** The region between GLW and one meter below GLW. Although permanently submersed, tidal fluctuation is important.
3. **Deep tidal waters and channels:** areas with water depths larger than 1 meter below GLW. Macrophytes are not found here. Fish dependent on tidal areas such as smelt (spiering) need the deeper parts to reach intertidal flats to spawn their eggs.

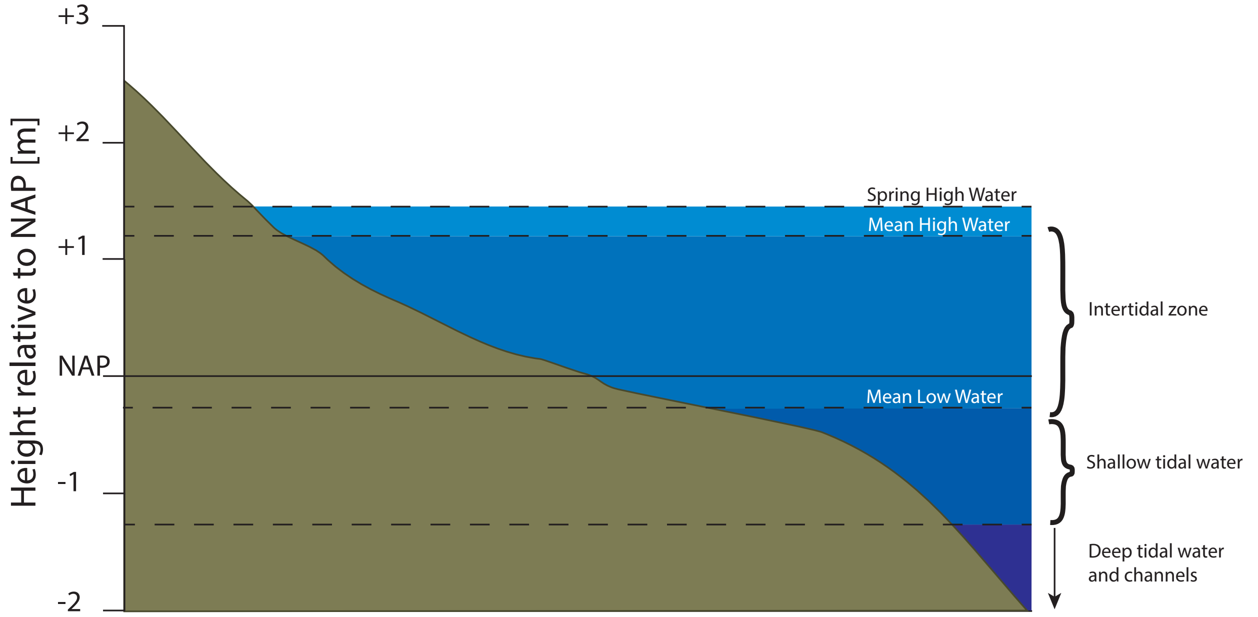

Figure 4.4: Schematisation of a river bank and the tidal zones.

4.7.2 Fish migration through the barrier

Fish are an important component of a river system. A barrier could hamper fish migration into and out of the Hollandsche IJssel. Different aspects could hamper fish migration:

- Fast flowing water;
- Very small opening;
- In case of pumps: blades.

4.7.3 Other aspects

Besides fish migration and tidal range, other aspects are of importance for the quality of the freshwater tidal river system. Other aspects include:

- Salinity;
- Water depth;
- Contaminant concentrations;
- Flow velocities;
- Nutrient supply;
- Sediment supply.

4.8 Taking ecological aspects into account in the case

This section shows the boundary conditions by which the tidal range and fish migration are taken into account in this thesis. Furthermore, two types of software are discussed that could be used for prediction and analysis of the suitability of the ecosystem for certain species. The section is ended by a short guideline on how to use the found data in this case.

4.8.1 Tidal range

The number of daily tidal cycles (approx. 2 per day) should be similar to the number of natural cycles. A semi-diurnal tide is therefore required (Nijboer et al., 2000; Bal et al., 2001; van den Broek, 2016; Braakhekke, 2016). Most literature on ecology, e.g. Bal et al. (2001); Nijboer et al. (2000) mention target species or *doelsoorten* that give an indication on the quality of a certain ecotope. For the KRW (European regulation, see section 4.6.3 and onwards) a large list has been published with flora and fauna that indicate a beneficial ecological situation. The only *unique* species in freshwater tidal areas is the Scirpus triqueter (Driekantige bies), see figure 4.5. Furthermore, the Caltha palustris (Spindotterbloem) is almost solely occurring in freshwater tidal systems. An analysis of all target species mentioned in relevant literature is beyond the scope of this thesis. Focus is placed on the unique Scirpus triqueter. The connection of this plant to intertidal areas can be found in the name for the largest freshwater tidal system in The Netherlands: The Biesbosch. For the Scirpus triqueter, a minimal tidal amplitude of 50 cm is required, while a tidal amplitude of more than 80 cm is optimal, see figure 4.6 (Deltares, nd).

Figure 4.5: The Scirpus triqueter or Driekantige bies. source: www.waddenzeeschool.nl

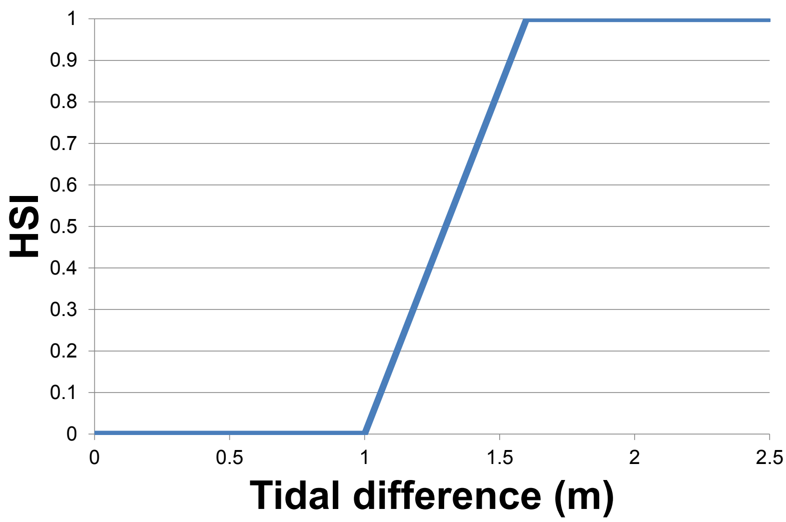

Figure 4.6: Habitat Suitability Index (HSI) for tidal range Scirpus triqueter or Driekantige bies (Deltares, nd).

In summary, the following boundary conditions regarding tidal range should be taken into account:

- A semi-diurnal tide is required;
- A minimal tidal range of 1 m is required, tidal ranges above 1.6 m are optimal.

4.8.2 Fish migration

Currently, the KRW scores regarding fish are sufficient (table 4.1). However, as a barrier could negatively influence accessibility, attention should be paid to fish. An example of a migratory fish is the *Osmerus esperlanus* or Spiering/Smelt in Dutch and English. Smelt uses intertidal areas to spawn their eggs. As mentioned in section 4.7.1, the fish need deeper parts to reach the intertidal flats (Nijboer et al., 2000). The smelt used to be common in The Netherlands, but has become rare due to the closure of estuaries (Zuiderzee and Haringvliet) (RAVON, nd).

Velocity and dimensions
The maximum flow velocity for fish to be able to migrate is 1.0 m/s. However, if juvenile fish need to pass as well, a maximum flow velocity of 0.8 m/s is recommended. (Kroes and Monden, 2005, p.84). Furthermore, Kroes and Monden (2005) mention minimum widths of between 0.2 m and 1.0 m for different structures and minimum water depths of 0.5 m.

Mitigating measures
In case where free passage is not possible, e.g. due to pumps, different mitigating measures are available, such as fish friendly pumps, a fish ladder and a fish lock. In Appendix D different concepts are shown and explained in more detail. For now it is important to realise that problems with fish migration can be mitigated. Summarising, regarding fish, the following boundary conditions are taken into account:

- The barrier should be passable by migratory fish species (Bal et al., 2001);
- The length of the ‘shoreline’: the amount of deep water bordering shallow tidal and intertidal waters (Nijboer et al., 2000) is an indication for habitat suitability for Smelt. The more shoreline, the more space available for the fish to spawn their eggs.
- It is assumed that any deep water is just as suitable for fish species: no distinction is made;
- The maximum velocity for fish to pass is 0.8 m/s;
- The minimum dimensions for fish is WxH=1.0 x 0.5 m.

4.8.3 Other aspects

Other aspects that are important for the ecosystem such as salinity, water depth, contaminant concentrations and flow velocities (see section 4.7.3) are not expected to be influenced by a different barrier. Of course the local water depth could be altered due to locally increased flow velocities and subsequent scour. This could possibly result in the release of contaminants. The effects of locally increased flow velocity can however be mitigated by scour protection and are therefore not considered further. A number of interesting dose-effect relationships for rare (however not unique) species are added in Appendix E and could be used in future design.

The amount of nutrients and sediment that is able to pass the barrier could be influenced by a new barrier that constricts the river further. For this thesis however, it is assumed that with sufficient tidal action, sufficient sediment and nutrients will be available in the Hollandsche IJssel. The modelling of sediment and nutrients is beyond the scope of this thesis. It is advised to either confirm or falsify this assumption in future research.

4.8.4 Software

In the following paragraphs, software with predictive capabilities that could be relevant for a study on fresh water tidal rivers is discussed.

KRW Verkenner/WFD Explorer

The KRW Verkenner is one of the most sophisticated programs to predict ecological value. It has a so-called 'neural network' that compares a modelled situation with actual situations logged in a database. It has the ability to predict and interpolate between values found in the database to adjust to the modelled situation. Its strength lies in regional waters. National waters, such as the Hollandsche IJssel have only recently been added. Because only few tidal river systems are present in The Netherlands, little data has been gathered (Evers, 2016). Currently, the KRW Verkenner only takes realised tidal area into account and it has no predictive capabilities. It is therefore not considered to be of added value (unfortunately) in this case compared to other programs.

HABITAT

HABITAT is a spatial analysis tool in which the quality and availability of habitats can be assessed (Deltares, nd). Some documents refer to EMOE (Ecohydrologisch Model voor Oevervegetatie van Estuaria) which is its predecessor. HABITAT has predictive capabilities, however only when response curves (see e.g. figures 4.6 and E.1 - E.4) are added. As mentioned before, Deltares provides dose-effect relations of different species in an ecological knowledge base (Deltares, nd). From the relations in this database, the suitability of a habitat for a chosen type of species can be obtained: the Habitat Suitability Index (HSI).

4.8.5 Phases in construction lifetime

This section provides a short guideline on how to take into account ecological value during the lifetime of a construction in a fresh water tidal river system.

Design & Construct

The boundary conditions on tidal range and fish migration mentioned in the paragraphs above serve as input for the design of a new barrier. Unfortunately, the KRW Verkenner has insufficient predictive capabilities for freshwater tidal river systems. HABITAT, or at least the dose-effect relations (indicating the suitability of a habitat for certain species) could be used. During construction, one should not bring permanent damage to the governing ecosystem. Temporary blocking of the tide may be possible, although the maximum allowed duration should be researched in further detail.

Maintain

Once a desired situation is reached, it will not remain that way. Succession will eventually lead to a forest. If this is not desired a maintenance plan should be available. More information is found in section 4.5.3.

4.9 Summary

There is a tendency, at least in western countries, towards keeping estuaries open. Various forces play a role which can be categorised into three groups, being (1) Economic forces, (2) Societal forces and (3) Legislative forces. Economic forces are the most direct and easy to take into account. Societal forces partly overlap with economic forces, however a major difference is that the majority are indirect forces which makes them hard to quantify. The increasing protective legislative forces are possibly a result of the increasing societal desire to keep estuaries open. The difficulty and subjectivity that comes with the incorporation of all forces has led to the decision in this thesis to include only the aspects that make the analysed ecosystem unique and valuable.

Figure 4.7 summarises the proposed process of an eco-friendly design, specified for this case. The left column shows the general process steps on how to reach the ultimate goal in eco-friendly design: creating ecological value. Realising and maintaining the right ecological (abiotic/biotic) conditions is a prerequisite to create ecological value, however it is not necessarily the result. In order to actually create natural value, valuable ecotopes need to be present which in turn leads to ecological value. Whether this occurs, should be verified on site after realisation (section 4.4.4). The right column shows the steps that were taken in the process of incorporating ecology in this thesis. The goal in this research is design a barrier in which favourable abiotic conditions are created/maintained. Without the right abiotic conditions, ecological value cannot be created. The conditions are realised by conservation of unique aspects of the current ecosystem. To check whether the unique aspects are conserved well enough in this case, a closer look is taken at unique (target) species (Scirpus triqueter) and fish migration. A favourable environment for these species is translated in a set of abiotic requirements. The analysis of unique abiotic factors and unique species has led to a set of boundary conditions (see section 4.9.1) which can be used as input for the design of the barrier.

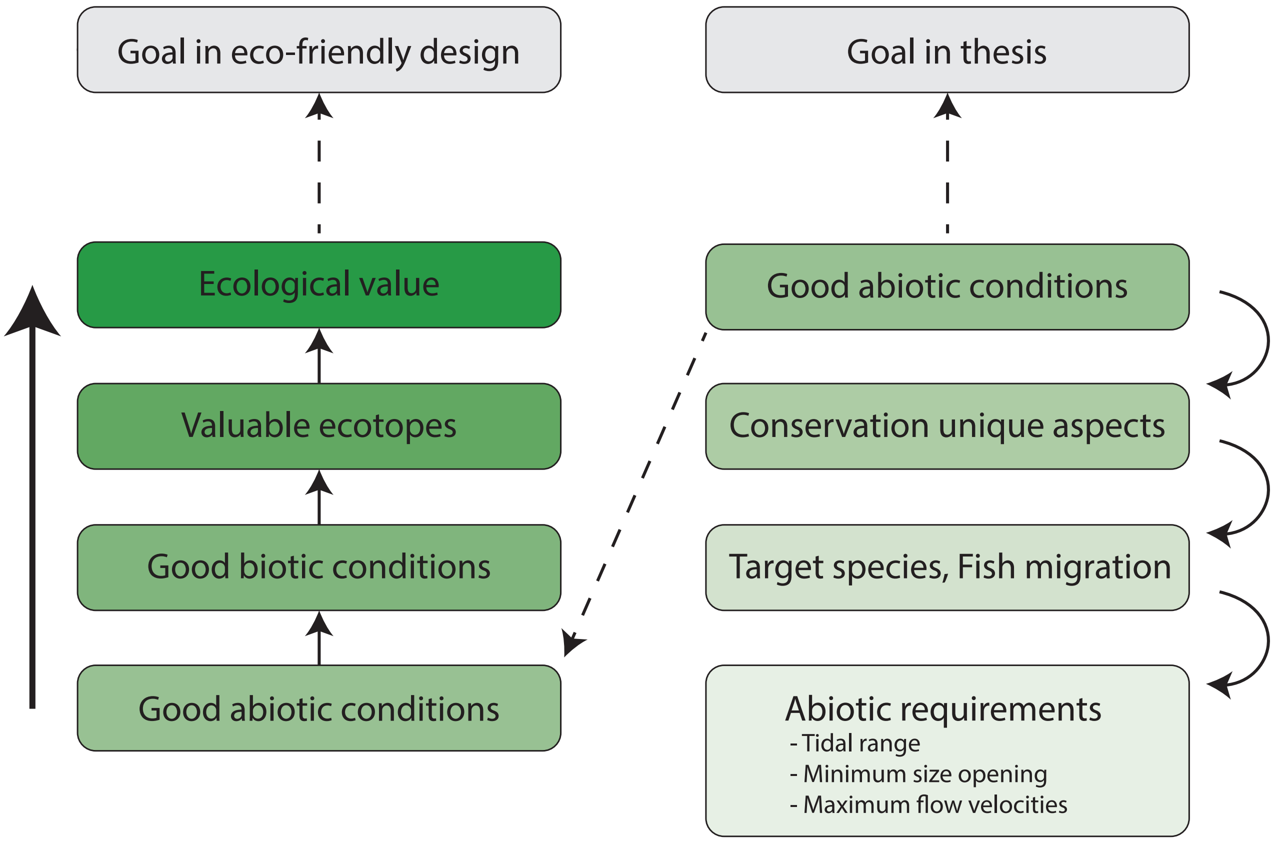

Figure 4.7: Process steps for an eco-friendly design.

4.9.1 Framework for creation ecological value

The analysis presented in the previous sections has led to a set of boundary conditions that can be used as input for the design of the barrier.

Regarding the **tide**, the following boundary conditions are taken into account:

- A semi-diurnal tide is required;
- A minimal tidal range of 1 m is required, tidal ranges above 1.6 m are optimal.

Regarding the **barrier**, the following boundary conditions are taken into account:

- The barrier should be passable by migratory fish species;
- The maximum velocity for fish to pass is 0.8 m/s;
- The minimum dimensions for fish is WxD = 1.0 x 0.5 m.

Regarding the **river** itself, the following assumptions are made:

- The length of the 'shoreline': the amount of deep water bordering shallow tidal and intertidal waters is an indication for habitat suitability for Smelt. The more shoreline, the more space available for the fish to spawn their eggs;
- It is assumed that any deep water is just as suitable for fish species: no distinction is made.

Further assumptions that have been made are:

- The level of the riverbed will not subside;
- The level of the riverbed cannot be lowered due to contamination (de Haan, 2016);
- The shape of the current intertidal areas will not change due to changes in water level. A morphological assessment, e.g. done by van Zanten (2016), is beyond the scope of this thesis;
- The surface level of current intertidal areas can not be lowered due to contamination (de Haan, 2016).

5

Future scenarios

5.1 Introduction

This chapter makes an attempt to globally describe the different scenarios in the coming 125 years (≈2150). The scenarios described by the Dutch government until 2100 will be used as guideline (section 5.2). In section 5.3 a qualitative model is introduced that describes the relationship between climate change and economic growth and various dependent variables. Next, the current policy/strategy is described (section 5.4), followed by a global time line of the proposed variant/strategy (section 5.5). This chapter forms the qualitative basis of further analysis on the points of difference of the proposed strategy in comparison with the current strategy or 0-variant.

5.2 Delta Scenarios

The furthest publicly available outlook of the Dutch government are the Delta Scenarios (Bruggeman et al., 2013). Two predictions in terms of economic growth and climate change lead to four scenarios (WARM, STOOM, RUST and DRUK). The scenarios are used to predict their influence on land usage. The effects in 2050 and 2100 are described in the Delta Scenarios. As the technical lifetime of a new construction is estimated to be 100 years, the presented predictions are extrapolated to 2150. The Delta Scenarios are summarised in table 5.1.

Table 5.1: Summary of the Delta Scenarios: expected sea level rise (SLR) and annual economic growth upto 2050, 2100 and 2150.

Scenario	Climate change	Sea level rise [m/y]	Socioeconomic growth	2050 [%]	2100 [%]	2150 [%]
DRUK	Moderate	0.004	Strong	2.5	2.5	2.5
STOOM	Fast	0.01	Strong	2.5	2.5	2.5
RUST	Moderate	0.004	Slow	1.0	0.5	0.5
WARM	Fast	0.01	Slow	1.0	0.5	0.5

5.3 Dependent variables

Figure 5.1 shows qualitatively how economic growth and climate change have an influence on various variables relevant for current and alternative policies. The following paragraphs will shortly explain the relationships.

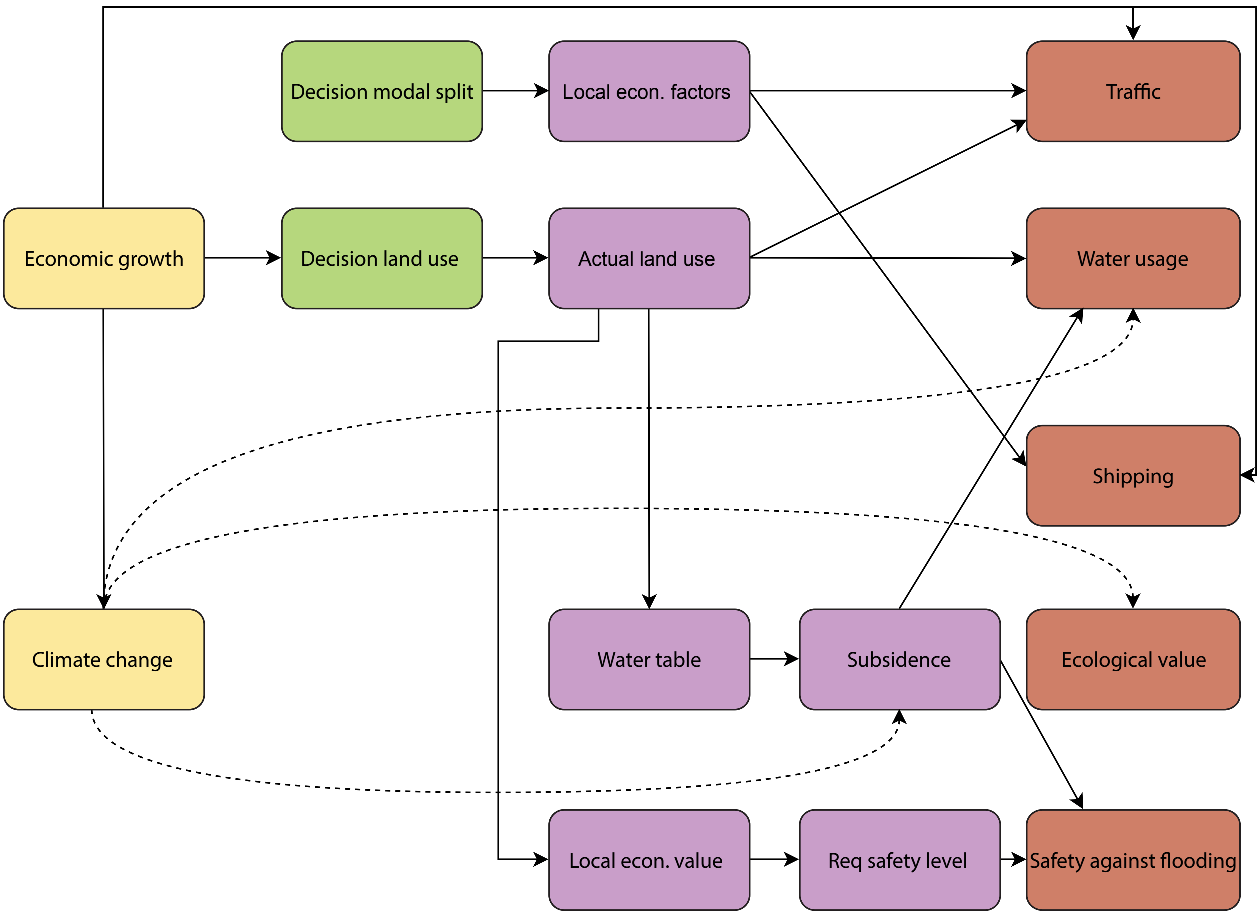

Figure 5.1: Relevant qualitative relationships.

Decision in modal split

A government can actively encourage sectors, firms and individuals to use certain means of transportation. An example would be to promote inland shipping instead of road transportation. A decision like this could positively influence the number of ship movements on the Hollandsche IJssel. Encouraging individuals to use public transport instead of travelling by car is another example.

Decision in land use

By deciding what type of land use is allowed in a certain area, policy makers have a large say in what way a region develops. Strong economic growth may influence the decision to change the function of an area.

Actual land use

A decision to alter the function of an area paves the way to turn e.g. pastures into urban areas and increase value per m^2. Economic growth alone can also lead to increasing value: the use of a piece of land can be exploited further. For example, higher value property can be built on a location.

Local economic factors

Local economic factors can play a significant role. The construction of an extra container terminal could have a significant impact on the number of ship movements on the Hollandsche IJssel. Construction of a terminal could in turn be the result of a government encouraging inland shipping over road transportation (decision in modal split). Another local economic factor is a change in industry along one of the banks. A decision of Royal IHC (owner of a large shipyard at Krimpen side of the river) to increase or decrease its local activities could significantly alter the amount of vehicles crossing the river.

Local economic value
A change in land use has an impact on local economic value. Increasing e.g. the number of households in the Krimpenerwaard would result in an increase in local economic value.

Water table (polders)
The water table is a trade-off between subsidence and use of land. Lowering the water table makes it accessible for large machinery for agricultural purposes. This however does lead to increased oxidation of surface peat layers. Policy makers could also decide to give up agricultural land and create more wetland areas, a scenario described by CE Delft and Oranjewoud (2008, p.27).

Subsidence
As mentioned in chapter 2, subsidence is influenced by alteration of the water table. Furthermore, arid periods could also lead to an increase in oxidation. Policy makers need to decide whether and to what extent subsidence should be countered. Agreements such as 'Veenweidepact Krimpenerwaard' (van Schie, 2014, p.11) can limit subsidence.

Required safety level
The required safety level (in terms of flood risk) is dependent on the local economic value and more generally on economic growth. As risk can be expressed as probability times the result (Jonkman et al., 2016), one can imagine that with increasing potential damage due to general economic growth, the probability of flooding has to be reduced to maintain current risk levels. The same effect is present when increase in economical value occurs locally (e.g. by change in land use).

Traffic
Traffic is influenced by local economic factors such as the amount of industry along a bank. Furthermore, an increase in the number of households at the Krimpenerwaard side would increase the amount of commuters towards the greater Rotterdam region.

Water usage and pumping capacity
The amount of water taken in depends on several factors. In the first place, climate change in the form of increased evaporation would lead to a higher intake. Secondly, land use plays an important role. Not the direct area around the Hollandsche IJssel is of importance, but the area of HHR. (Intensive) agricultural activities require water that is taken in near Gouda. A change in land use also influences the required intake capacity. Thirdly, subsidence also has an influence on water usage. A policy to counter subsidence would lead to a higher water table and increased water intake (or reduced outlet).

Climate change in the form of increased severity and duration of precipitation leads to an increase in required pumping capacity. Although currently water can be discharged freely into the Nieuwe Maas, pumping capacity may be needed in the future. Changes in land use could reduce the storage capacity of surrounding polders, making extra pumping capacity a necessity.

Shipping
As mentioned, shipping is influenced by local economic factors such as an extra container terminal. Furthermore, economical growth in general probably has a positive impact on the number of ship movements.

Ecological value
The ecological value of the river system may change due to climate change. Salt may intrude further upstream on a more regular basis which could lead to a gradual transition in a salt water tidal system. Increased number of ships could disturb nature due to noise increase and increased capability to stir up sediment which in turn decreases light permeability.

Safety against flooding
The actual safety against flooding is positively influenced by the desired/required level of safety. Subsidence decreases the level of safety.

5.4 Reference strategy

The reference strategy encompasses a continuation of current policy, namely to strengthen dikes and keep the Hollandsche IJssel open. It is primarily based on the 'Deltaprogramma' (Roeleveld et al., 2014). In times of storm surge a storm surge barrier will close off the river. The probability of failure of the Algera barrier is improved and has a probability of failure of 1:200 up to 2050/2058 after which the current barrier is replaced by a barrier with a probability of failure of 1:1,000. Furthermore, the Maeslant barrier is replaced in 2070 by a barrier with a probability of failure of 1:1,000. Dikes are improved using earthen solutions where possible. Removal of adjoining property is prevented as much as possible with structural measures during the first rounds of strengthening, but is likely to become inevitable in the second period (~2075-2125). When dike strengthening will occur depends on how fast climate changes and how severe subsidence will be and what the maximum probability of failure will be. Summarising, the strategy is as follows:

- Continuous dike improvements;
- 2015 - 2030: Improve current barrier to failure probability 1:200;
- 2015 - 2030: Improve macrostability dikes;
- 2040 - 2100: Improve height problems dikes;
- 2058: Replace current barrier with 1:1,000 barrier. When attainable, improve traffic situation;
- 2070: Replace Maeslant barrier with 1:1,000 barrier.

5.5 Proposed strategy

The proposed strategy encompasses a further close-off, however taking into account ecological aspects. As mentioned in chapter 4, there is an increasing pressure on the engineer to take ecological value into consideration. Therefore, aspects such as tidal range are maintained as far as deemed possible. In the proposed strategy, the Hollandsche IJssel is closed-off with a new barrier as soon as possible. It is considered most likely that the structure can be completed in 2050. This is based on:

- The time needed for policy makers to decide;
- The construction time;
- The decision by policy makers to postpone a decision on further closing of the river upto the moment that the current barrier has reached its technical life time (2050/2058) (Deltaprogramma, 2014, p.25).

After the new construction is realised, dikes along the Hollandsche IJssel will not be improved. The resistance of the dikes against flooding will reduce due to subsidence and sea level rise, resulting in decreasing allowable water levels over time. Tidal action takes place under natural conditions upto a point in time in which pumping capacity is required to guarantee lower water levels than the levels that the dikes are able to resist (shown in chapter 7). The proposed strategy is summarised as follows:

- Realisation of an 'eco-friendly' barrier;
- Until 2050: Improve dikes when necessary;
- 2017 - 2030: Improve current barrier to failure probability 1:200;
- 2050: Realisation new barrier. When attainable, improve traffic situation;
- 2050 - 2150: No further dike improvements along Hollandsche IJssel.

To check for sensitivity and whether opportunities are missed out upon, scenarios in which the close-off is realised in 2030 and 2070 are also considered in chapter 10. The following variants will be considered:

- **Early adopters**: The close-off is realised as early as possible (2030).
- **Current Policy**: The close-off is realised according to current policy and decisions (2050).
- **Lag behind**: Policy- and decision-makers start to think of a new barrier after the current barrier has reached its technical life time. The close-off is realised in 2070.

5.6 Qualitative comparison of strategies

In this section the reference and proposed strategies are briefly compared. The reference strategy focuses on an open 'lock-free' connection of the Hollandsche IJssel with the Nieuwe Maas. Shipping is not hampered and the tidal range and conditions for fish migration are not altered. The ecological value of the river is not altered, neither positively nor negatively. A consequence of the open connection of the river is that continuous dike reinforcements will be required, which may be costly (elaborated in chapters 6 & 10).

The proposed strategy focuses on a barrier that improves the safety against flooding behind it. The new barrier should be eco-friendly: fish migration and tidal range should be influenced as little as possible. Compared to the reference strategy, no dike reinforcements behind the barrier will be required after implementation, which could save cost (elaborated in chapters 6 & 10). The decision not to raise dikes after implementation, may however result in lowering of the water levels and installation of pumps to ensure sufficient tidal action (elaborated in chapter 7). The eco-friendly barrier may prevent ships to pass the barrier 'lock-free'. When this is the case, the societal cost due to shipping delay should be weighed against the possible savings due to averted dike reinforcements (chapter 10).

5.7 Summary

In this chapter, the Delta scenarios were introduced. Furthermore, a qualitative framework was presented, showing the relationship between the independent and dependent variables. The variables that form the basis of the Delta Scenarios (and are assumed independent) are economic growth and climate change. The variables that will be elaborated upon in further chapters are: Traffic, Water usage, Shipping, Ecological value and Safety against flooding. These variables (chapter 6) can be used to give an indication of cost (chapter 10) of the reference and proposed strategy, which were introduced in this chapter.

6

Predictions of future scenarios

6.1 Introduction

This chapter provides insight on the data used as input for the design and provides means to compare current policy with the proposed concept. Only a short summary is provided. When needed, a more extensive elaboration has been added as an appendix.

Per design variable a few realistic scenarios have been elaborated upon. The scenarios serve as a method to compare the proposed close-off with the current policy. Other scenarios and alterations on current scenarios are possible, but beyond the scope of this research.

To calculate the present value of societal cost, the following formula is used:

$$PV = \sum_{i=1}^{t} CF_i \cdot PVIF = CF_i \cdot \frac{1}{(1+r)^t} \tag{6.1}$$

where CF_i represents the cash flow in year *i* and *r* the inflation, which has been assumed equal to economic growth. For future cash flows, a real risk discount rate of 5.5% was used (Rienstra and Groot, 2012, p.4) .

6.2 Traffic

As can be seen in figure 5.1, traffic is dependent on: general economic development, local economic factors and actual land use. For simplicity reasons, a change in modal split is not taken into account. Below three situations have been distinguished. The option to change use of land in combination with slow economic growth is not considered to be realistic.

Table 6.1: Sub scenarios for number of crossings

Sub scenario	Land use	Economic growth	Reference scenario
1.	Similar	Strong	DRUK
2.	Similar	Slow	RUST/WARM
3.	Different	Strong	STOOM

Assumptions

The number of crossings is expected to be closely related to economic growth. Different land use (further urbanisation) in the Krimpenerwaard will result in extra growth. If such a policy decision is made, it remains unknown when this decision is taken. Based on land use predictions presented by Bruggeman et al. (2013, p.34), it is expected however that such a decision is implemented after 2050. In the case of different land use (Sub scenario 3) an extra annual growth of 0.5% is added to the economic growth. Furthermore, a Value of Time of 9 Euro per hour is assumed (Kennisinstituut voor Mobiliteitsbeleid, 2013).

6.2.1 Results capacity calculation

The predictions for the peak intensities for the different scenarios are depicted in figure 6.1. It was found that the capacity of the bridge is sufficient when closed. The capacity of the junction in Krimpen is a more severe bottle neck. Figure 6.2 shows the accumulation of societal cost of waiting with the current capacity of the junction until 2058. The full results can be found in Appendix F. Table 6.2 shows the total incurred societal costs up to each year.

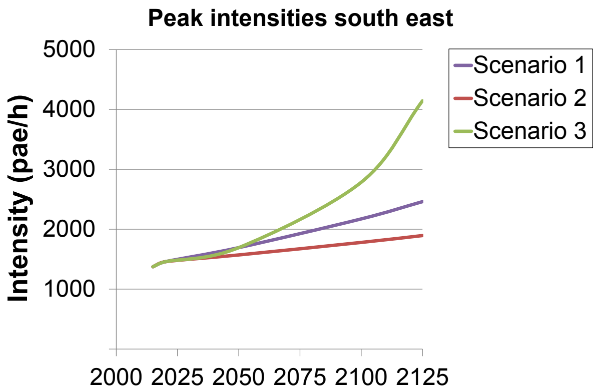

Figure 6.1: Development of peak intensities for different scenarios on the N210.

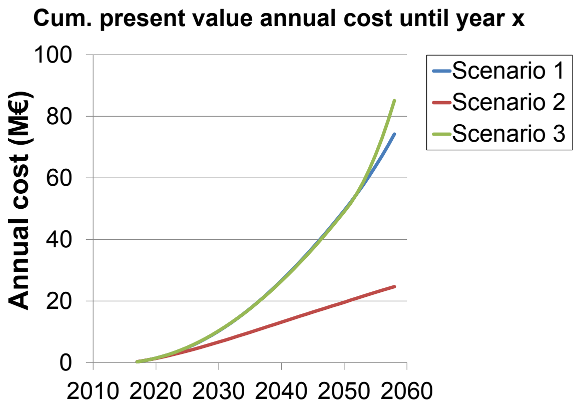

Figure 6.2: Cumulative societal cost for congestion.

Table 6.2: Societal costs congestion junction in million euro.

Scenario	2020 [M€]	2025 [M€]	2050 [M€]	2058 [M€]	2100 [M€]	2125 [M€]
1	2.0	4.9	49.4	74.2	520.4	1,291.1
2	1.8	3.8	19.6	24.7	45.3	53.5
3	2.0	5.0	49.0	85.1	1,583.7	5,381.5

6.2.2 Implications

The findings show that a significant amount of societal cost can be saved by tackling the congestion problem. Current policy plans to improve the situation in 2058, when the current barrier has reached its technical design life time. Until then, depending on the scenario, societal cost ranging from approximately 20 up to 85 million euro can be saved. The capacity of the junction is insufficient and space to improve the current situation is lacking. Therefore, a different trajectory of the road should be considered. An example of a new trajectory is provided in Appendix F.

6.3 Water usage & pumping capacity

As can be seen in figure 5.1, water usage is dependent on actual land use, policy decision on land use, economic growth and climate change. The amount of water that needs to be available for intake is dependent on the water usage. As water levels in the Hollandsche IJssel will remain the same or will be lowered, the potential influx will not be hampered. Therefore, water intake is secured similarly in both the proposed and the reference strategy.

The pumping capacity however is a point of difference. In the current situation and thus in the reference strategy, water can flow freely from the Hollandsche IJssel towards the Nieuwe Maas. When (partially) closed off, pumping capacity may be needed to pump out excess water. Further analysis should result in whether pumping capacity is actually needed or that excess water can be disposed off under free flow conditions. An assumption on the increase in required pumping capacity is based on the Delta Scenarios:

- **DRUK**: Increase in water usage due to intensified agricultural activities is countered by increased wetland area and innovation. Similar water usage.
  Result: moderate increase pumping capacity.
- **STOOM**: Increase in water usage due to salt water intrusion and mix of urban and rural areas. Increase water usage.
  Result: high increase pumping capacity.
- **RUST**: Decrease in water usage due to low economic growth, less amount of agricultural activities and limited climate change. Decreased water usage.
  Result: moderate increase pumping capacity.
- **WARM**: Increase in water usage due to salt water intrusion is countered by limited economic growth. Similar water usage.
  Result: high increase pumping capacity.

The different considered sub scenarios are tabulated in table 6.3. With ‘land usage’ a more regional or provincional change in land usage is meant compared to the locally change in land presented in figure 5.1.

Table 6.3: Sub scenarios water usage and pumping capacity.

Sub scenario	Climate change	Land usage	Water usage	Increase pump capacity	Reference scenario
1.	Moderate	Different	Similar	Moderate	DRUK
2.	Fast	Different	Increase	High	STOOM
3.	Fast	Similar	Decrease	Moderate	RUST
4.	Moderate	Similar	Similar	High	WARM

In order to quantify moderate and high increase in pumping capacity, figures presented in KNMI (2015) and KNMI (nd) have been used. Here the increase of pumping capacity is based on the increase in 1:10 year precipitation in 10 days in winter (highest precipitation). Local peaks are not taken into account as the proposed pumping station is situated at the end of the pumping chain. Table 6.4 summarises the required pumping capacity in m^3/s for both scenarios, assuming that the current (2016) pumping capacity ‘upstream’ (Appendix G) is sufficient.

Table 6.4: Sub scenarios increase pumping capacity [m^3/s].

Sub scenario	2025	2030	2050	2085	2100	2150	Reference scenario
1 & 3	95.5	96.7	96.8	98.5	99.2	101.8	DRUK/RUST
2 & 4	96.2	97.6	102.5	108.8	111.5	120.4	STOOM/WARM

6.4 Shipping

As can be seen in figure 5.1, shipping is dependent on: general economic development and local economic factors. In table 6.5, four sub scenarios have been distinguished. For simplicity reasons, a change in modal split is only taken into account implicitly, in the extra container terminal. The development in shipping is predicted based on (Bruggeman et al., 2013), and tabulated in table 6.6.

Table 6.5: Sub scenarios for shipping.

Sub scenario	Container capacity	Economic growth	Reference scenario
1.	Increased	Strong	DRUK
2.	Current	Strong	STOOM
3.	Increased	Slow	RUST/WARM
4.	Current	Slow	RUST/WARM

Table 6.6: Development professional shipping: growth factor compared to 2008-2012 (reference).

Sub scenario	2008-2012 (Reference)	2050	2100	2150
1.	1	1.6	2.2	2.6
2.	1	1.5	1.5	1.5
3.	1	0.9	0.7	0.7
4.	1	0.9	0.5	0.5

Double container capacity

Double container capacity leads to an addition of 0.2 to the different factors. This number was found based on the capacity of CEMT class IV ships, 96 TEU (n.a., 2008), an assumed average load of 75% of the maximum capacity and a terminal capacity of 116,000 TEU, similar to the capacity of the current terminal in Alpen a/d Rijn (n.a., 2014). For the future scenarios it is expected that construction of an extra terminal will take place after 2050.

Based on (1) the predictions presented above and IVS data (Rijksoverheid, 2016), (2) an assumed average waiting time of one hour and (3) average societal costs of 338 Euro per ship per hour for waiting for a lock (Kennisinstituut voor Mobiliteitsbeleid, 2013, p.17-19), an estimate on societal cost incurred due to ships having to sail through an extra lock can be made. Figure 6.3 shows the development of shipping on the Hollandsche IJssel for different scenarios. Figure 6.4 shows the cumulative cost incurred by society. Scenario 3 and 4 show similar values. Between 2050 and 2150, the total societal cost of sailing through an extra lock range from approximately 10 up to 64 million Euro.

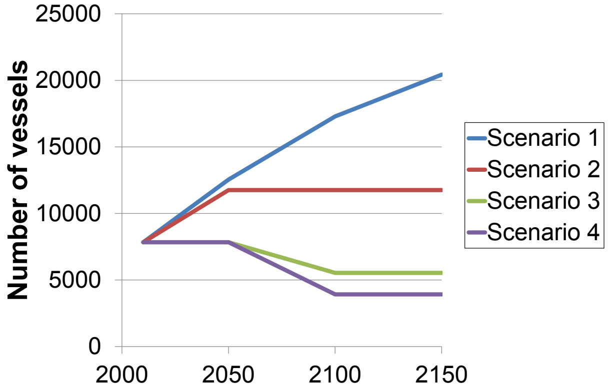

Figure 6.3: Development commercial shipping for different scenarios.

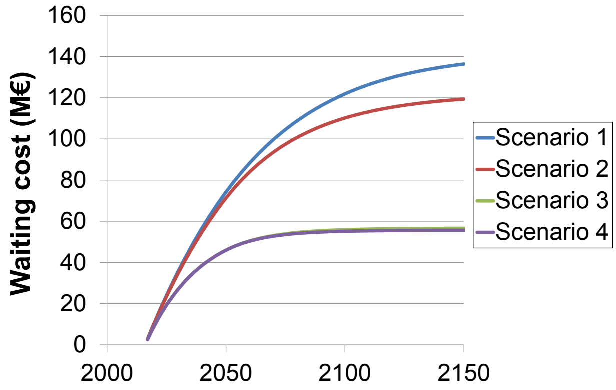

Figure 6.4: Cumulative societal cost of waiting hours commercial shipping for different scenarios.

6.4.1 Recreational shipping

In order to provide a complete picture of the situation, similar calculations as presented above have been performed for recreational shipping. Unfortunately, no estimates are made as part of the Delta Scenarios. Water board HdSR expects an increase of 1% per year until 2030 (Hoogheemraadschap De Stichtse Rijnlanden, 2014, p.11). However, the Dutch population ages, boat owners become older and less young people decide to own a boat (Rabobank, nd). Therefore, the growth in recreational shipping is expected to be considerably less than in the professional sector, see figure 6.5. For recreational shipping, a societal cost of 8.25 per ship per hour for has to be accounted for (Kennisinstituut voor Mobiliteitsbeleid, 2013, p.17-19). With cost ranging from 0.48 to 0.89 million Euro over a period from 2050-2150, the societal cost related to recreational shipping (figure 6.6) are very small compared to professional shipping (figure 6.4).

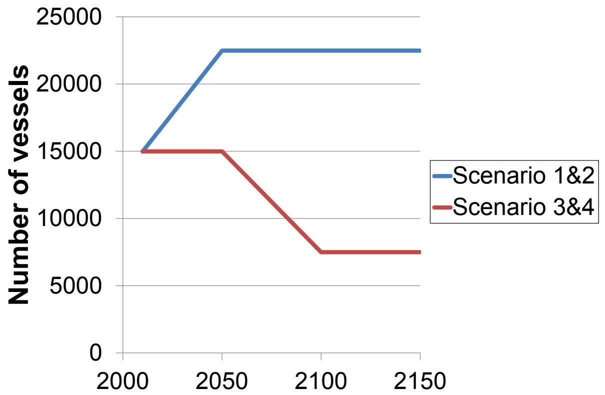

Figure 6.5: Development recreational shipping for different scenarios.

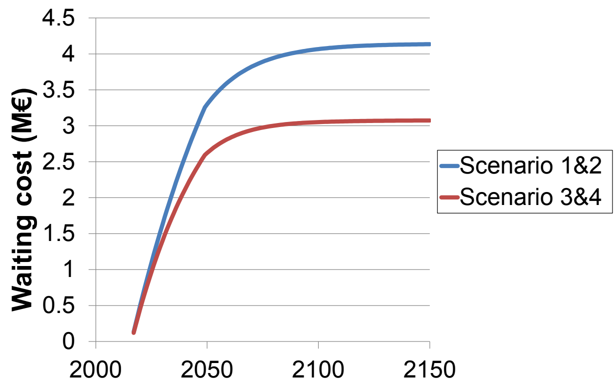

Figure 6.6: Cumulative societal cost of waiting hours recreational shipping for different scenarios.

6.4.2 Reference strategy

In the current/reference strategy societal costs are incurred due to the extra closures needed. The extra closures are needed because of sea level rise and are estimated to cost (commercial) shipping 40 million Euro (Schoemaker, 2016, p.67) over the design life time. No bandwidth is available as this figure has not been analysed in further detail. To calculate extra societal cost of waiting due to an extra lock, this value should be subtracted from the earlier found cost for professional shipping.

6.4.3 Implications

The societal cost of sailing through an extra lock are larger than the cost of extra closures of the current barrier. A further analysis of the lock time would reduce uncertainty of the figures given. Furthermore, it can be seen that societal costs due to delayed commercial shipping are far bigger than those due to recreational shipping. Based on the current estimation, the present value of the extra cost due to shipping delay (proposed strategy) over the lifetime is estimated to vary between 0 and 25 million Euro.

6.5 Safety against flooding

As can be seen in figure 5.1, safety against flooding is dependent on the required safety level and subsidence. Table 6.7 shows the different scenarios that were analysed.

Table 6.7: Sub scenarios for safety against flooding.

Sub scenario	Req. safety level	Subsidence	Reference scenario
1.	Increased	Large	STOOM
2.	Increased	Moderate	(DRUK)
3.	Similar	Large	WARM
4.	Similar	Moderate	RUST

An increase in required safety is mainly expected on the east side if a decision is made to further develop parts of the Krimpenerwaard into residential area. Increased economic value and higher population densities will likely lead to increased economic and societal risk. It is then likely that regulations regarding probability of flooding in this polder will be similar or equal to Central Holland (dike ring 14). Further development is mainly the case in scenario STOOM.

Since a storm surge barrier can close off the river, one could argue that the sea level rise is of no influence on the water table on the Hollandsche IJssel. However this would only be the case if the probability of failure is infinitely small. Schoemaker (2016, p.17-20) shows the effects of sea level rise with a storm surge barrier; sea level rise leads to an increase in closure frequency and increased water levels for the larger return periods. Chapter 7 elaborates further on this.

Next to sea level rise, the increase in river discharge is of influence on the water table as well. The two climate scenarios (WARM/STOOM and DRUK/RUST) predict Rhine discharges for 2050 and 2100. Currently there is a debate on whether or not to cap peak discharges due to flooding in Germany (te Linde, 2012). Schoemaker (2016, p. 219) however showed that this capping is of negligable influence for the water table in the Hollandsche IJssel. The Rhine discharges are tabulated in table 6.8.
The reference situation (2015) and the expected hydraulic load (expected water level + local wind set-up) in

Table 6.8: Rhine discharges [m^3/s].

Sub scenario		2050	2100
STOOM/WARM	'regular'	18,000	20,000
STOOM/WARM	capped	16,000	17,500
DRUK/RUST	'regular'	16,500	18,000
DRUK/RUST	capped	15,000	16,000

2050, 2100 and 2150 are based on an analysis with Hydra-BS and are tabulated in table 6.9. Hydra-BS is a validated software program developed by HKV Lijn in Water for Rijkswaterstaat. The loads have been found by averaging the hydraulic load found at every km using Hydra-BS. For 2125 & 2150, loads were found by linear extrapolation of 2050 and 2100 values using P_f of 1:1,000 for both barriers. As the load is used for a cost calculation, using the average instead of the maximum is considered legitimate.

Table 6.9: Average water levels [m] in the Hollandsche IJssel.

		2015	**2015**	**2050**	**2100**	**2125***	**2150***
P_f MLB		1:100	1:100	1:100	1:1,000	1:1,000	1:1,000
P_f SVK HIJ		1:100	1:200	1:200	1:1,000	1:1,000	1:1,000
STOOM/WARM	1:2,000	2.960	2.837	2.978	3.046	3.181	3.305
	1:10,000	3.128	3.029	3.129	3.171	3.269	3.354
DRUK/RUST	1:2,000	2.960	2.837	2.880	2.810	2.877	2.944
	1:10,000	3.128	3.029	3.056	2.989	3.043	3.087

* Expected water level by linear extrapolation

It can be seen that, by adjusting the reliability of the barrier, the reference strategy is able to keep the expected water levels more or less constant until 2100. A change in design water level (e.g. in DR15 due to urbanisation) will lead to a large increase in hydraulic load.

6.5.1 Buildings

Buildings in the vicinity of dikes result in serious challenges for dike reinforcements concerning both height and stability. Figures 6.7 - 6.13 show the various options conceptually. Figure 6.7 shows the greenfield situation. Subsidence and sea level rise can be countered by earthen measures. These measures are also an option if buildings are located below the dike, however not directly at the toe (figure 6.8). Whenever the toe of the dike reaches a building, conventional raising will lead to removal of the building (figure 6.9). If this is not wished for, a structural measure could be chosen (figure 6.10 and 6.11 (L)). If, due to subsidence and/or sea level rise, the retaining height is insufficient, it could be possible to raise structurally once more. The options are however limited and may be costly. Similar structural measures could be taken when demolition of the building at the top has to be averted. Figure 6.12 shows how this could be done. Again, at a later stage the height could be increased, however this may be costly. Another option would be to raise dikes and remove the building. Buildings along the dikes are however highly valued and removal/demolition may socially and politically not be acceptable on a large scale. The same is valid for option 3 (figure 6.9).

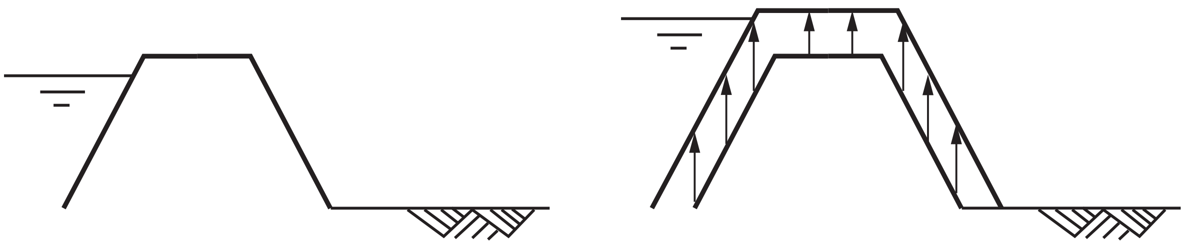

Figure 6.7: Option 1: Conventional dike reinforcement.

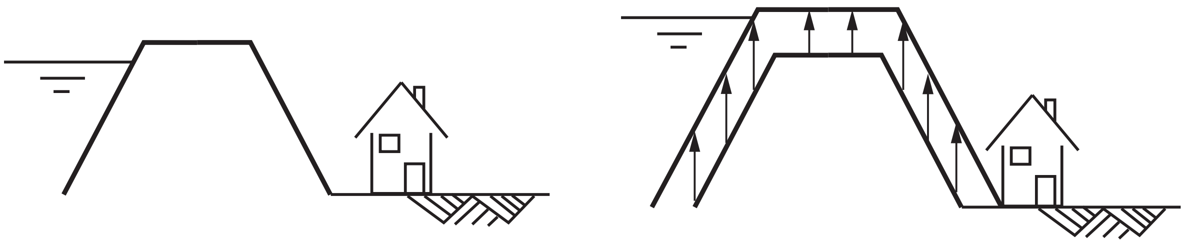

Figure 6.8: Option 2: Conventional dike reinforcement near a building.

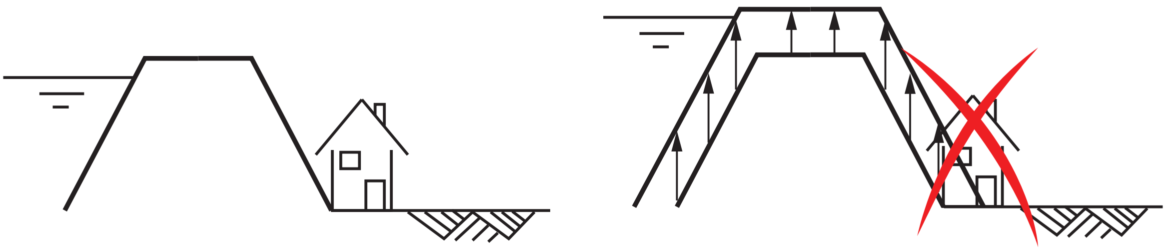

Figure 6.9: Option 3: Removal due to dike reinforcement near a building.

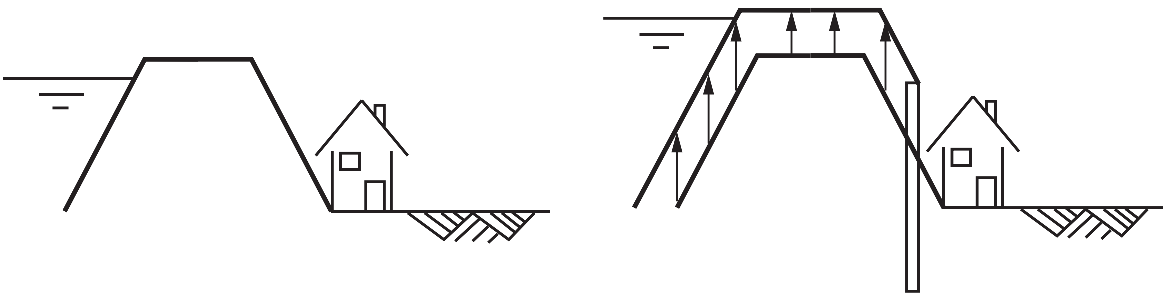

Figure 6.10: Option 4: Sheet piling to prevent removal of a building.

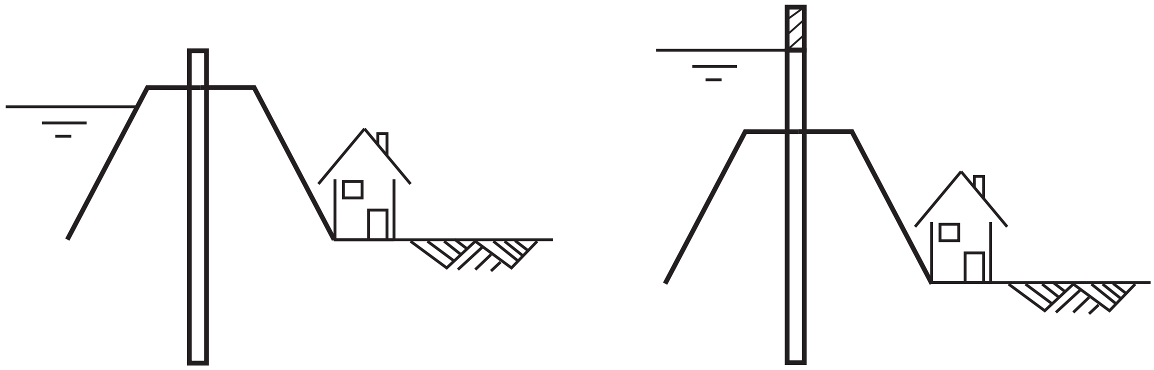

Figure 6.11: Option 5: Structural measure to increase retaining height with a building at toe of dike (L) and further increase due to SLR and subsidence (R).

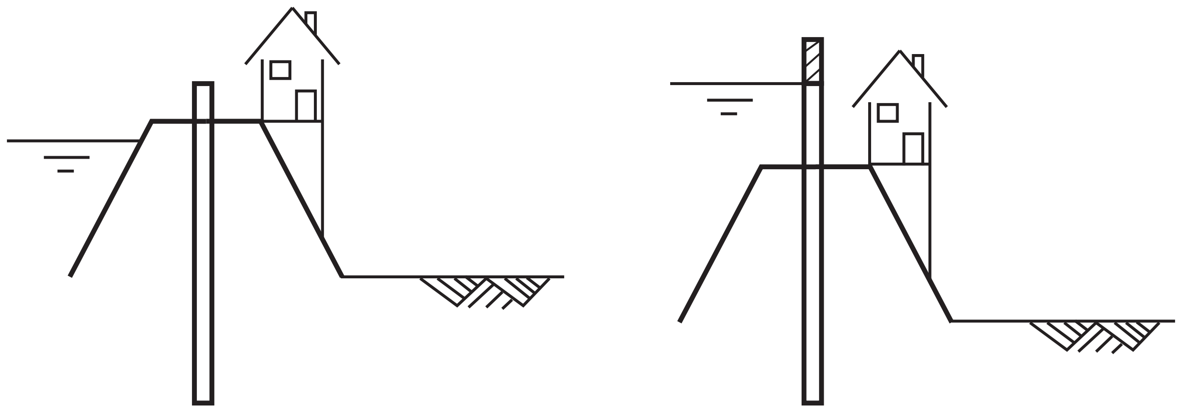

Figure 6.12: Option 6: Structural measure to increase retaining height with a building at top of dike (L) and further increase due to SLR and subsidence (R).

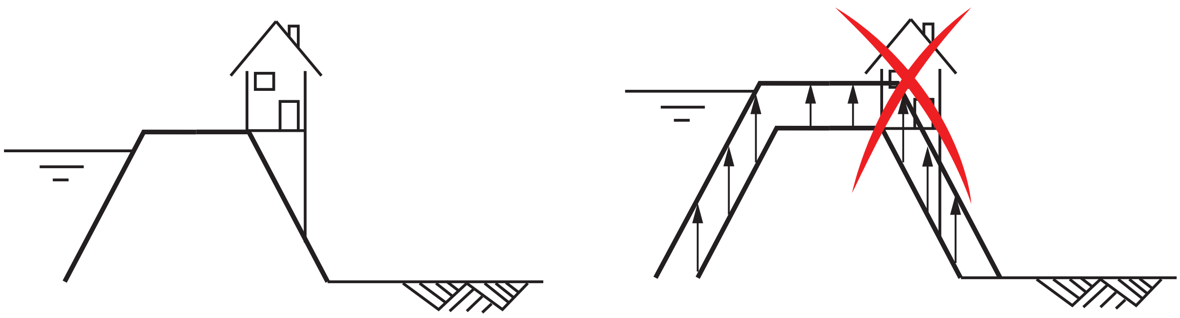

Figure 6.13: Option 7: Removal of a building due to dike reinforcement near a building.

6.5.2 Cost indication continuation current policy

In order to compare the proposed concept with the reference strategy, a cost estimation has been made on the cost of dike improvements. Simplifications have been made, as this cost estimation only serves as means of comparison and is not the core of this thesis. The costs of future dike improvements are based on an initial cost estimation done by DHV (2012) for the current set of dike improvements (KIJK + Capelle a/d IJssel - Moordrecht). From this cost estimation an average price per meter was obtained for slurry walls, soil improvements and sheet pile constructions. For simplicity reasons, neither a distinction was made between stability or height related improvements, nor between different heights. The thorough analysis of the cost estimations led to the following investment cost (2017, incl. VAT) per running meter:

- Earthen improvement: € 11,726 per meter;
- Sheet pile: € 18,229 per meter;
- Slurry wall: € 25,824 per meter.

6.5.3 Assumptions

Various assumptions were made for the cost estimation. The assumptions are listed below.

- Buildings are saved as much as possible. Whenever a building is built at the top of the dike and the dike has to be raised, a slurry wall is constructed. Earthen strengthening is applied up to 3 m from a building at the toe. Whenever a building is present closer to the dike, a sheet pile construction is used up to a length above the surface of 1.5 m.
- The life time for slurry walls, soil improvements and sheet pile constructions are 100 years, 50 years and 50 years respectively.
- For soil improvements a slope of 1:3 is used.
- The amount of sea level rise and subsidence is only taken into account in whether sheet piles can still be constructed: there is e.g. no differentiation in price between a 0.5 m and 0.6 m heightening.
- For simplicity reasons, it is expected that all dikes that have passed *WTI 2011* will be insufficient by 2030 and thus have to be improved.
- Due to the possible policy switch around 2050, earthen and sheet pile dike improvements done up to 2050 are expected to be designed with a lifetime up to this year.

The actual level of safety is dependent on the rate of subsidence. If water levels in the Hollandsche IJssel are not lowered, large subsidence rates will lead to insufficient dike heights reached at an earlier stage . Different scenarios have been considered, based on the Delta scenarios presented in chapter 5. The results are presented in table 6.10. In the STOOM+ and DRUK+ scenario, different land use of the Krimpenerwaard after 2050 has been considered as well. This leads to a 1:10,000 year safety standard after 2050 which in returns requires safer dikes.

In table 6.10, the fourth column shows the present value of cost if dike reinforcements are continued after 2050 (reference strategy). Scenario 1 & 2 and 3 & 4 lead to the same cost. This is because subsidence has a far larger influence on the dike improvements than sea level rise. In the proposed strategy (the most right column of table 6.10) the cost of dike reinforcements are far lower than in the reference strategy (369 - 766 million Euro). Furthermore, it can be seen that the cost for dike improvements are large compared to other costs. This can largely be attributed to the large number of structural improvements in the dikes (sheet piles and slurry walls) needed to save buildings.

Table 6.10: Scenarios and (investment) costs for dike improvements until 2150. Present Value (PV), 2017 in Million Euro [M€].

	Scenario	Subsidence [m/y]	Reference [M€] 2020-2150	Proposed [M€] 2020-2050	Difference [M€]
1	STOOM	0.015	1,336	570	766
2	STOOM+*	0.015	1,336	570	766
3	DRUK	0.009	1,262	570	692
4	DRUK+*	0.009	1,262	570	692
5	WARM	0.015	963	511	452
6	RUST	0.009	880	511	369

** In STOOM+ and DRUK+ different land use will lead to 1:10,000 year safety standard after 2050.*

6.5.4 Implications

Buildings along the dikes of the Hollandsche IJssel are also subjected to the effects of different scenarios. In DRUK and RUST a subsidence of 0.9 m is expected, while for STOOM and WARM, the subsidence per century is even 1.5 m. Assuming that currently the dike height is just sufficient and that subsidence of the dike is equal to that of the hinterland, this would mean a required dike raise of 1 - 1.5 m until 2125 only to counter subsidence. For earthen dike reinforcements slope 1:3, this would mean a 6 to 9 m wider dike, likely leading to removal of buildings. If buildings are saved, this leads to high costs as presented in table 6.10. Continuing dike improvements after 2050 as measure for flood protection leads to present value cost ranging between 369 and 765 million Euro.

6.6 Summary

This chapter has shown the (societal) cost of (1) congestion (2) shipping and (3) dike improvements. Firstly, regarding the societal cost of congestion, a present value (PV) ranging from 20 up to 85 million Euro can be saved by earlier implementation of a new barrier. To improve the situation however, the barrier and road should be realised at a new location (elaborated further in chapter 8). Secondly, *if* ships have to pass through a lock, extra cost (PV) for shipping range between 0 and 25 million Euro (proposed strategy). Thirdly, in case of continuation of current (reference) policy however, dike reinforcements are required between 2050 - 2150 with a PV that ranges between 369 and 766 million Euro. The values show that the cost incurred for dike reinforcements (reference strategy) are enormous compared to extra cost for shipping due to a possible extra lock passage (proposed strategy), or saved traffic cost by realising a better road connection earlier. Furthermore, the societal cost due to congestion between 2016 and 2050 could be up to three times as high as waiting cost for shipping. The road connection is therefore a relevant aspect of the design and should be incorporated when possible. Table 6.11 summarises what the potential costs and savings are when opting for the proposed strategy (section 5.5) instead of the reference strategy (section 5.4). The found cost and savings in this chapter will be used to compare the total investment cost of the reference and proposed strategy in chapter 10.

Table 6.11: Potential cost and savings when choosing the proposed strategy instead of reference strategy.

	[M€]	Type
Congestion	20-85	Saving
Shipping	0-25	Cost
Dike reinforcement	369-766	Saving

7

Hydraulic requirements

7.1 Introduction

In this chapter, the size of the gate(s) in the barrier will be derived. Chapter 6 showed that reinforcing the dikes is complicated and therefore expensive. Starting point for this thesis is to create a safe situation, without dike reinforcements behind the barrier after the new barrier is constructed. *Before* the new barrier is realised, dike reinforcements may be needed to resolve current stability issues (and sometimes insufficient height) (Krol, 2014, p.17).

> **Point of departure**
> A barrier that maintains the tidal range as far as possible, while no dike improvements behind the barrier are needed after implementation.

In order to maintain ecological value, a tidal range should be maintained, see chapter 4. Maintaining a tidal range leads to a *minimum* required cross-section/opening of the gate(s) in normal conditions, see section 7.3. The starting point of no dike reinforcements after implementation of the barrier will lead to subsidence and therefore decreasing strength of the dikes. Sea level rise will lead to increasing hydraulic loads. The combination of subsidence and sea level rise will lead to a moment in time at which the dikes will have insufficient height. In section 7.3, this point in time is calculated. The barrier should function in storm conditions as well. Reduced retaining height of the dikes and increasing water levels will lead to less storage in the river behind the barrier. As section 7.4 will show, small-sized gates are required (a *maximum* allowable opening) to limit the inflow of water in case of failure.

7.2 Assumptions

In order to calculate hydraulic requirements of the barrier, several assumptions were made. This section discusses the strength of the dikes, assumed (design) water levels and the geometry of the river.

7.2.1 Proven strength

This thesis is focused on the design of a barrier at the end of the Hollandsche IJssel. Detailed analysis of the actual strength of dikes along the Hollandsche IJssel is beyond the scope of this thesis. However, for the sake of credibility of the results, a realistic strength is paramount. Therefore, in this thesis the strength of the dikes behind the barrier is defined by the so-called ‘proven strength’ (PS). This is the load that the dikes have proven to be able to resist. It is known that during the floods of 1953 the dikes were able to resist water levels of NAP +3.78 m at Gouda and that the dikes have only been raised, not widened. Therefore, the proven strength related water levels only have to be corrected for subsidence, leading to a current proven strength water level of NAP +3.22 m (Schoemaker, 2016, p.51). For this thesis it is now assumed that until 2050 (the year in which the new barrier is implemented), strength and stability improvements will counter the effects of subsidence leading to proven strength related water levels of NAP +3.22 m in 2050. After 2050, the dikes will start to subside. In this thesis a subsidence of 1.1 m/century is assumed, evenly distributed per year (1.1 cm/year) (HHSK, 2016).

7.2.2 Water levels

The water level in the Hollandsche IJssel is dependent on two aspects, namely (1) discharge from surrounding polders and (2) the water level on the Nieuwe Maas. Discharge from upstream is not present (see chapter 1). In this thesis, an assumption is made on what the increase in water level is due to polder discharge, using HYDRA-BS. An average increase of the water level by 0.56 m was found, which will be used throughout this thesis. Appendix H provides further information on how this value was found. The water level on the Nieuwe Maas is subjected to climate change. Different climate scenarios (see chapter 5) predict different rates of sea level rise. It is assumed that from 2015 until 2050, water in the river system is subjected to linear sea level rise ranging from 0.35 m/century (DRUK/RUST) upto 0.93 m/century (WARM/STOOM) (Bruggeman et al., 2013, p.8) (CBS et al., 2016). From 2050 onwards, water in the river system is subjected to linear sea level rise ranging from 0.4 m/century (DRUK/RUST) upto 1.0 m/century (WARM/STOOM) (Bruggeman et al., 2013, p.8).

7.2.3 Design water levels

In this thesis, the increase in water level caused by discharge from polders is used as an input variable and is not subject to change. This water level increase can occur at any moment in time. In order to obtain a design water level, this water level increase has to be subtracted from the maximum allowed water level (proven strength). For example, in 2050 this leads to a design water level of 3.22 - 0.56 = NAP +2.66 m. The design water levels for 2050 - 2200 are presented in table 7.1. It can be seen that the design water level reduces over time with the subsidence of 1.1 m/century mentioned in section 7.2.1.

Table 7.1: Maximum allowed water levels in the Hollandsche IJssel.

	2050	2100	2150	2200
Max water level [m NAP]	+3.22	+2.67	+2.12	+1.57
Design water level [m NAP]	+2.66	+2.11	+1.56	+1.01

7.2.4 Geometry for first order calculations

For simplicity reasons, a simplification is made of the geometry of the Hollandsche IJssel in first order/hand calculations. The river is assumed to have a length of 17 km with a rectangular cross-section. The river bed lies at NAP -7.0 m and the river has a width of 100 m.

7.2.5 Summary of assumptions

The assumptions have been summarised below.

- The barrier designed is implemented in 2050 and has a proposed lifetime of 100 years;
- In the proposed conceptual design, no further dike reinforcements are carried out after 2050;
- Proven strength of the dikes along the Hollandsche IJssel is assumed to be NAP +3.22 m in 2015 (Schoemaker, 2016), which means that water levels up to this height can be retained;
- Dike strengthening measures upto 2050 will counter subsidence and therefore proven strength is assumed to be NAP +3.22 m in 2050;
- The dikes are subjected to a subsidence of 1.1 m/century (HHSK, 2016);
- The strength of the dikes will decrease with the rate of subsidence;
- A surcharge of 0.56 m is assumed due to water from the polders pumped into the river;
- From 2015 - 2050, water in the river system is subjected to linear sea level rise ranging from 0.35 m/century (DRUK/RUST) upto 0.93 m/century (WARM/STOOM) (Bruggeman et al., 2013, p.8) (CBS et al., 2016)
- From 2050 onwards, water in the river system is subjected to linear sea level rise ranging from 0.4 m/century (DRUK/RUST) upto 1.0 m/century (WARM/STOOM) (Bruggeman et al., 2013, p.8).
- For hand calculations, a river with a length of 17 km is assumed. The river bed lies at NAP -7.0 m and has a constant width of 100 m over the depth and length of the river. The total surface area of the river, A_{river}, is therefore equal to 1,700,000 m^2.

7.3 Normal conditions, opened barrier

Water levels inside the Hollandsche IJssel should not exceed proven strength related water levels at any moment in time. Daily water levels increase over time due to sea level rise, while subsidence reduces the maximum water levels that the dikes are able to retain. From an ecological perspective, tidal action should be conserved, resulting in an open barrier in normal conditions. To reduce hydraulic loads on the dikes, the barrier could dampen the tide. From considerations mentioned above, four situations can be distinguished, under normal conditions:

1. Fast sea level rise (SLR), minimum required amplitude;
2. Fast sea level rise (SLR), current amplitude;
3. Moderate sea level rise (SLR), minimum required amplitude;
4. Moderate sea level rise (SLR), current amplitude.

Figures 7.1 & 7.2 illustrate situation/scenario 3 in 2100 and 2150. The left side of each figure represents the water level in the Nieuwe Maas, the right side the water levels in the Hollandsche IJssel. The design water level is defined as the water level that is allowed to be reached by tidal action, see also table 7.1. On the right side of each image it can be seen that the design water level (DWL) is lowered 1.1 m per century, following the settlement of the dikes. For this situation, the water level is increased by 0.4 m per century due to sea level rise. Furthermore, in this situation the tidal amplitude is dampened by the barrier to 0.5 m. It can be seen that, at least until 2150, there is still a margin between the occurring water levels in the river and the design water level.

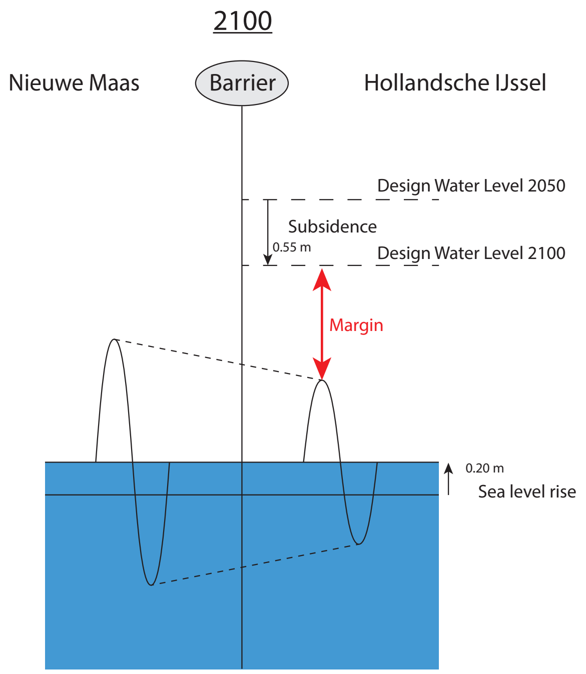

Figure 7.1: Water level and DWL in 2100, scenario 3.

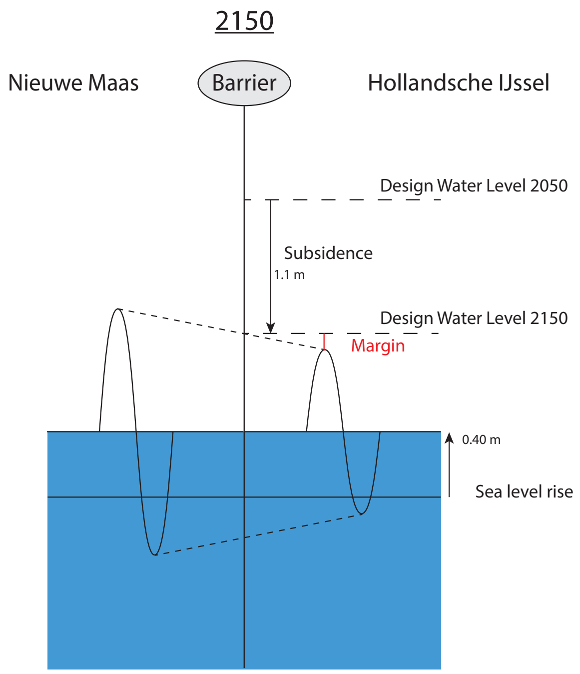

Figure 7.2: Water level and DWL in 2150, scenario 3.

Figures 7.1 & 7.2 showed occurring water levels in 2100 and 2150 respectively. The development of the water level over time for scenario 3 is depicted in figure 7.3. The water level (in blue) increases over time due to climate change. On the other hand, the strength of the dikes and therefore the maximum allowed water levels (in red) and design water level (in black) decrease over time. The intersection of the blue and black line marks the year in which pumping capacity is required to maintain the desired tidal action. For situation 3, this is in 2173. Table 7.2 shows at which point in time pumping capacity is required for all four scenarios.

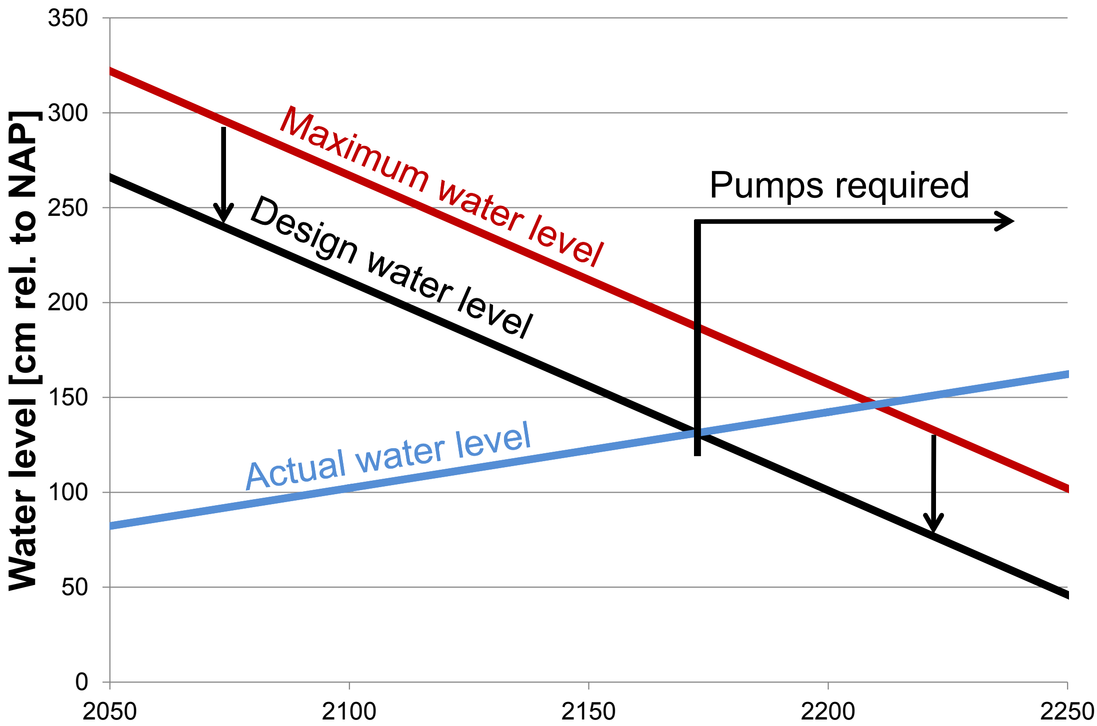

Figure 7.3: Water table development for scenario 3. The occurring water level is depicted in blue, the design water level in black, the maximum water level in red.

Table 7.2: Year in which pumping capacity is needed, different scenarios based on KNMI (2015) and HHSK (2016)

Scenario	Sea level rise [m/100y]	Subsidence [m/100y]	Tidal range [m]	Year pumps needed
1	1.0	1.1	1.0	2128
2	1.0	1.1	1.5	2105
3	0.4	1.1	1.0	2173
4	0.4	1.1	1.5	2140

7.3.1 First approximation pump capacity

Table 7.2 shows that in all scenarios except scenario 3, pumping capacity is needed to ensure a life time of 100 years. In case of fast sea level rise in combination with conservation of current tidal amplitudes, pumps should compensate the last 45 years (2105 - 2150, Scenario 2). In this scenario, the head difference that should be overcome by pumps is 0.95 m, resulting in a required pumping capacity of 114 m^3/s or 1,058 kW. By comparison, the largest pumping station in The Netherlands (IJmuiden) has a capacity of 260 m^3/s (Stam, 2004). The power needed for the proposed pumps is comparable to what can be delivered by an average windmill in The Netherlands (1 MW) (de Windcentrale, 2016). Appendix H elaborates upon how the values have been derived.

7.3.2 Minimum opening

To conserve the current ecosystem, tide should be conserved. The size of the opening has been defined using SOBEK. The required opening increases rapidly whenever a tidal range very close to the original is desired. A hand calculation (see also Appendix J) exemplifies this, see figure 7.4. Figure 7.4 shows that the required cross-section increases rapidly as the tidal amplitude reaches its original value (0.75 m). In this stage of design a preliminary choice has to be made on the amplitude. Due to the asymptotic increase, 90% of the original tidal amplitude (0.675 m) is chosen as an upper value. A 1-Dimensional model of the river in SOBEK shows that a cross-section of approximately 150 m^2 (without losses, μ*A=A) is required to reach 90% of the original tidal amplitude. If losses are included, a cross-section of 226 m^2 is required. A full report is found in Appendix L.

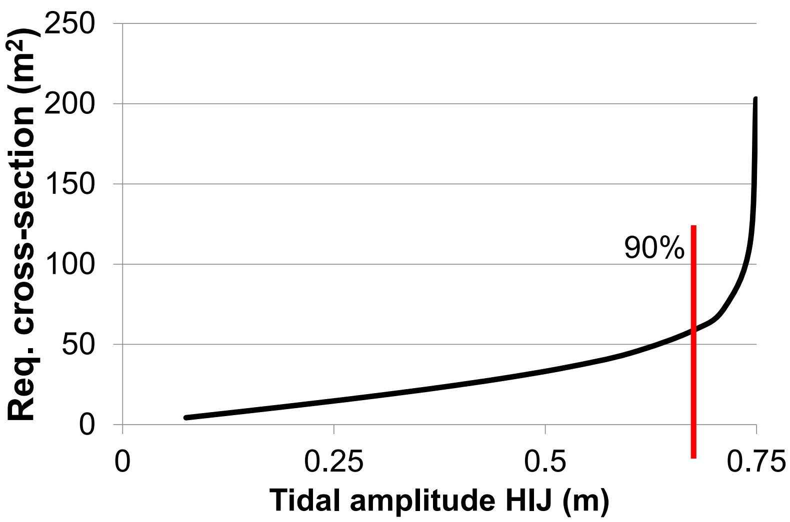

Figure 7.4: Rapid increase required cross-section (no wall friction included and μ*A=A).

7.4 Storm conditions, closed barrier

In the previous section, maximum water levels under normal conditions were discussed. A new barrier should function under storm conditions as well. In this section it is discussed whether a single door solution could suffice or that a multiple gate solution is required. Figures 7.5 & 7.6 illustrate the difference between the two types. With a single gate solution, a gate is meant that closes off a waterway in one go. Failure to close would result in an opening with the size of the entire waterway. A multiple gate solution closes a waterway by using more than one gates. These gates have a smaller cross-section than the gate in the single gate solution. In case of failure of one of the gates, the remaining opening in the barrier would be smaller than if a single gate solution should fail. This in turn leads to a slower water level rise behind the barrier. The current Algera Barrier is a single gate solution (two single gates behind each other).

The effect of single gate and multiple gate solutions on water levels behind the barrier is qualitatively explained in this text box.

Large opening (1x100 m)
Failure will result in water entering the river through a big opening. This results in fast water level rise. Storage capacity is small compared to incoming discharge.

Medium sized openings (e.g. 4x25 m)
Failure of one barrier (25 m) will still lead to fast rise of water levels inside. Storage capacity is still small compared to incoming discharge. As both failure of one door and all doors lead to failure, reliability may even be reduced compared to one large opening. Consider a case where flood gates each have a probability of failure of 1/100, $p_f = 1/100$. Then the probability of failure of the system of four gates is $p_{f,4\ gates}$= 1-(99/100)*4=0.0394 while a system with one flood gate $p_{f,1\ gate}$=0.01).

Small openings
Failure of one opening will lead to small rise of water levels. A small opening can be repaired and thus the inflow of water is limited. Extra safety measures, such as stop locks ('schotbalken'), are small enough to be installed within a few hours. Compared to double safety measures in larger openings, they are relatively easy and cheaper to construct. Furthermore, since the openings are small, pumping capacity could counter failure of a gate.

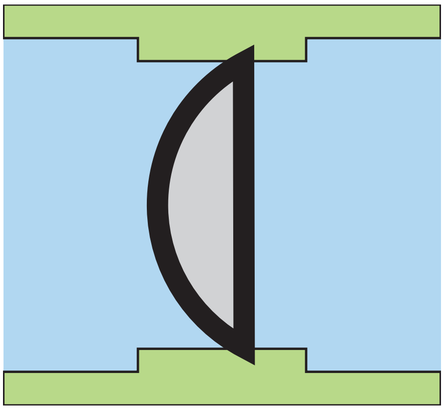

Figure 7.5: Example of a single gate solution.

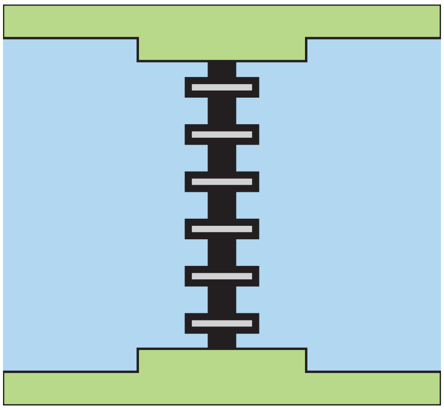

Figure 7.6: Example of a multiple gate solution.

7.4.1 Single gate solution

In this section the applicability of a single gate solution is researched. The first part will provide information on how to assess the probability of a certain water level occuring at the Hollandsche IJssel. Subsequently, factors that have an effect on this probability will be discussed. Lastly, the applicability of a single gate solution is researched using HYDRA-BS and the results are presented.

Probability of exceedance of water levels

For each water level in a water body a certain return period can be determined. The higher the water level, the higher the return period. A higher return period means a lower probability of occurring. This probability can be assessed for the water levels outside the barrier, as well as for inside the barrier. In the design of a barrier, goals regarding the probability of exceedance of a certain water level inside the barrier need to be achieved. To see whether certain measures are sufficient one can assess this probability of exceedance ($P_a(H)$), given the probability of water levels outside the barrier and the failure probability of the barrier. This is approximated by the following relationship:

$$P_a(H) \approx P_{f,SSB} \cdot P_v(H) \quad if \quad P_{f,SSB} \leq 0.01 \quad (Vos,\ 2014) \tag{7.1}$$

In which $P_v(H)$ is the probability of a certain water level in front of the barrier and $P_{f,SSB}$ is the non-closure failure probability of the barrier.

> **Example** (Schoemaker, 2016, p.17):
> Suppose that the exceedance probability of a water level of NAP +3.0 m on the outside of the storm surge barrier ($P_v(H)$ is 1/10 per year and the non-closure failure probability of the storm surge barrier ($P_{f,SSB}$) is 1/100 per event. Then, the exceedance probability of a water level of NAP +3.0 m on the inside of the storm surge barrier ($P_a(H)$) is 1/100*1/10 = 1/1000 per year. The red line in figure 7.7 illustrates the results.

Effect of reliability, sea level rise and closure level on water levels on the return period

Equation 7.1 indicates that by decreasing the failure probability of a barrier, it is possible to decrease the exceedence probability of certain water levels. Schoemaker (2016, p.20) shows how the return periods of water levels are influenced by this closure reliability (figure 7.8), and also by sea level rise (figure 7.9) and the design closure level ((figure 7.10)). HYDRA-BS was used to predict the water levels.

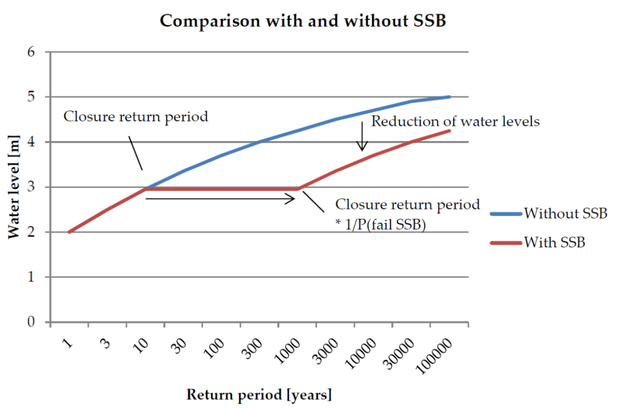

Figure 7.7: Effect of a storm surge barrier (SSB) (Schoemaker, 2016).

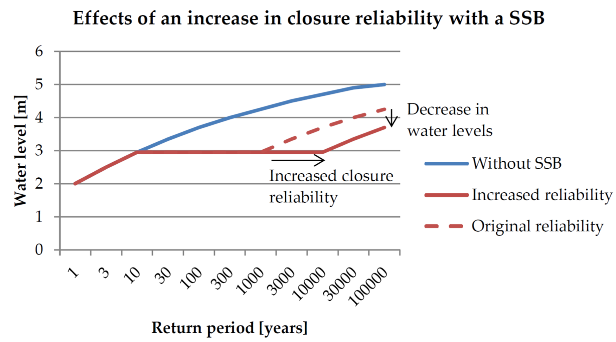

Figure 7.8: Effect of an increase in closure reliability storm surge barrier (SSB) (Schoemaker, 2016).

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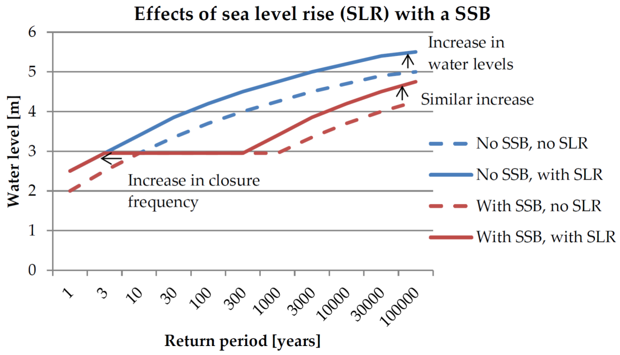

Figure 7.9: Effects of Sea Level Rise (SLR) (Schoemaker, 2016).

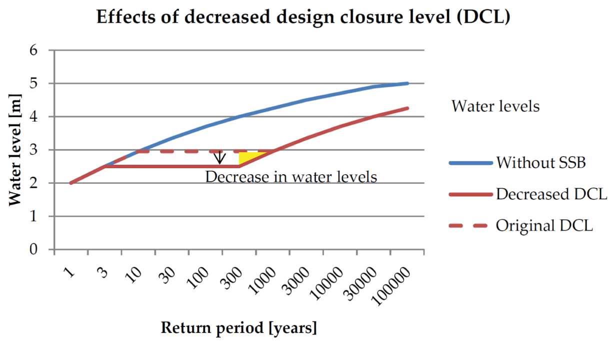

Figure 7.10: Effects of lower design closure level storm surge barrier (SSB) (Schoemaker, 2016).

Conclusions from graphs presented

From figures 7.7 - 7.10 several conclusions can be drawn, listed below (see also Schoemaker (2016, Ch.2)). The reader is recommended to read Schoemaker (2016, Ch.2) for further information.

- **Regarding figure 7.7**: suppose that the exceedance probability of a water level of NAP +3.0 m on the outside of the storm surge barrier ($P_v(H)$ is 1/10 per year and the non-closure failure probability of the storm surge barrier ($P_{f,SSB}$) is 1/100 per event. Then, the exceedance probability of a water level of NAP +3.0 m on the inside of the storm surge barrier ($P_a(H)$) is 1/100*1/10 = 1/1000 per year. The red line in figure 7.7 illustrates the results. Figure 7.7 illustrates that predicted water levels for return periods larger than 1000 years *with* SSB, are the same as the water levels for return periods larger than 10 years *without* SSB.
- **Regarding 7.8**: the exceedance probability of certain water levels is decreased by increasing the closure reliability of the storm surge barrier.
- **Regarding figure 7.9**: an increase in water levels due to sea level rise is transferred one-to-one on water levels in the Hollandsche IJssel.
- **Regarding figure 7.10**: decreasing the design closure level is only of notable influence for the lower water levels with low return periods, where the barrier is now closed instead of open. A small reduction of water levels with higher return periods is achieved (yellow triangle in figure 7.10).

Applicability single gate solution

Using current HYDRA-BS software, the possibilities for a single gate barrier were researched. The results for KM02 (near Gouda) in 2100 and 2150 are summarised in tables 7.3 and 7.4, the full results can be found in Appendix I. In the tables, the 'Norm' is based on the proven strength of the barrier and the assumption that dikes are not reinforced after 2050 (see also section 7.1). Furthermore, an extremely low probability of failure of the storm surge barrier ($P_{f,SSB}$) is used. Two remarks on the analysis should be made:

1. Hydra-BS considers complete failure of the barrier (non-closure). Partial closure is cannot be taken into account. This is a logical assumption in the current situation and the reference strategy. A series of smaller close off structures (e.g. a set of pipes with valves; multiple gate solution) could result in partial failure however;
2. Extra measures, such as extra pumping capacity or extra flooding area, are not taken into account.

Table 7.3: Predicted water levels in case of a single gate solution in 2100 (HYDRA-BS).

Climate scenario	$P_{f,SSB}$	Water level [m NAP]	Norm [m NAP]	Unity Check
DRUK/RUST	1:10,000	2.786	2.67	0.958
WARM/STOOM	1:10,000	3.059	2.67	0.873
DRUK/RUST	1:100,000	2.451	2.67	1.089
WARM/STOOM	1:100,000	2.789	2.67	0.957

Table 7.4: Predicted water levels in case of a single gate solution in 2150 (HYDRA-BS).

Climate scenario	$P_{f,SSB}$	Water level [m NAP]	Norm [m NAP]	Unity Check
DRUK/RUST	1:10,000	2.986	2.12	0.710
WARM/STOOM	1:10,000	3.559	2.12	0.596
DRUK/RUST	1:100,000	2.651	2.12	0.800
WARM/STOOM	1:100,000	3.289	2.12	0.645

Conclusion

From tables 7.3 and 7.4 it can be concluded that a solution consisting of a single gate barrier (even with a very low probability of failure) in combination with no further dike reinforcements after implementation of the barrier, is insufficient. A single gate solution is therefore not applicable, hence a multiple gate solution should be researched (section 7.4.2).

7.4.2 Multiple gate solution

Section 7.4.1 showed that a single gate barrier without dike reinforcements provides insufficient protection. In this section a multiple gate solution is researched. The objective is to define the maximum size of each gate. Firstly, the maximum theoretic life time of a barrier under storm conditions is researched. As the dike strength reduces and the maximum allowed water levels with it, storage capacity in the river decreases over time. Secondly, the effect of different sizes of a breach on water levels in the Hollandsche IJssel is analysed. As multiple gate barriers are not incorporated in HYDRA-BS, a first-order analytic calculation is carried out to analyse the effect of different breach sizes. The effect of a breach on the water level in the Hollandsche IJssel is presented in graphs 7.12 - 7.15, from which a maximum allowed gap in case of failure can be derived.

Theoretical maximum lifetime

Consider a barrier without pumps which is closed before a storm surge. To reach low water levels in the Hollandsche IJssel, the barrier is closed during low water before the storm. In case of (partial) failure of the barrier during the storm surge, the maximum allowable water level rise is defined by the design water level minus the low water level at closure. Figure 7.11 shows the margins over time (grey) for fast (WARM/STOOM) climate change. The maximum low water level at Krimpen a/d IJssel during the 'Sinterklaasstorm' of 2013 (NAP + 107 cm) has been has been extrapolated to 2050 and used as y-intercept (black ascending line). The Design Water Level (DWL) is the black descending line. Theoretically, the barrier could last until 2111 in the WARM/STOOM scenario (figure 7.11) or up to 2148 in case of the DRUK/RUST scenario. At this moment in time the margin between the black lines reduces to zero (intercept), which implies that a barrier is required which has no chance of gate failure: there is no storage volume left in the river. Figure 7.11 shows that the lifetime can be increased by installing pumps. In that case, the water level in the Hollandsche IJssel before the storm can be reduced with the installed pump capacity. The maximum theoretical lifetimes for different scenarios are summarised in table 7.5.

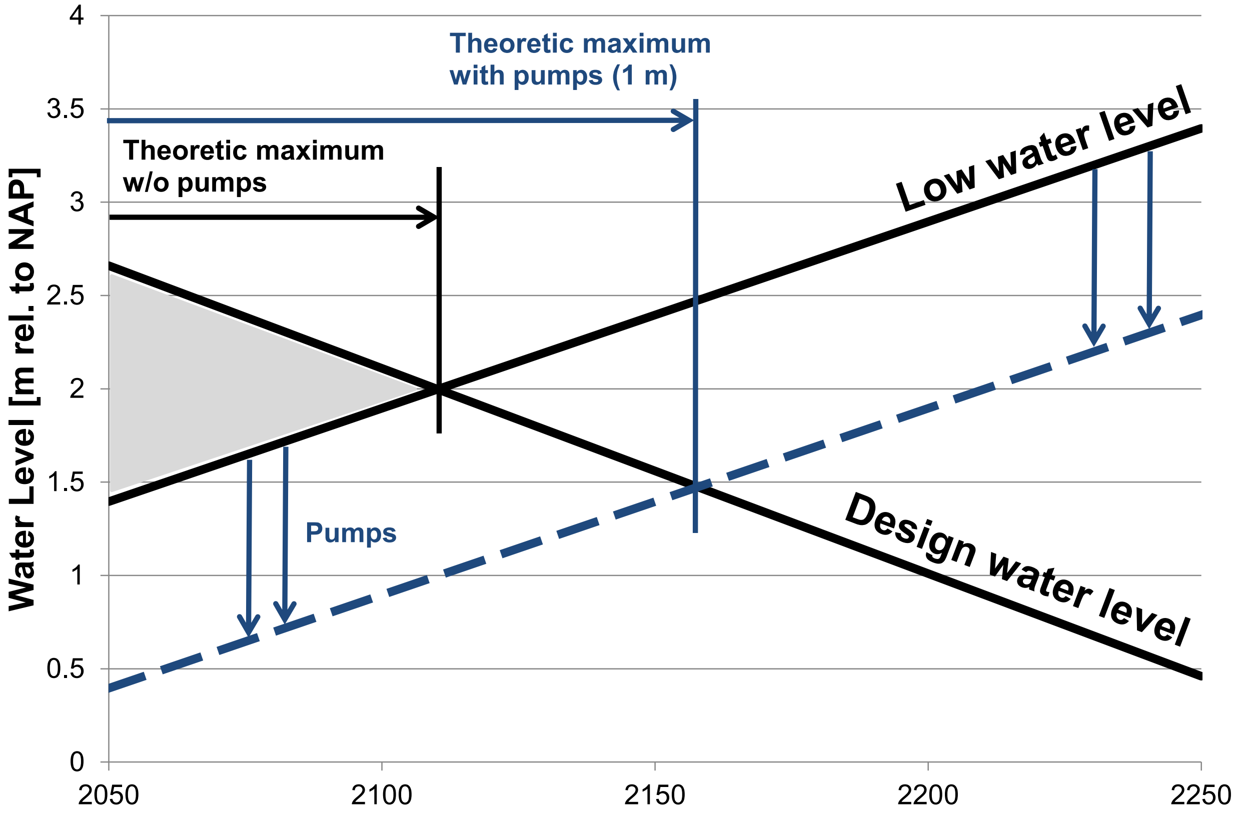

Figure 7.11: Maximum lifetime under storm conditions (WARM/STOOM). Margin (storage) for failing gates in grey.

Table 7.5: Life time of the storm surge barrier (theoretic maximum).

Climate scenario	Pump capacity [m water level reduction]	Theoretic maximum lifetime
DRUK/RUST	0	2148
WARM/STOOM	0	2111
DRUK/RUST	1	2215
WARM/STOOM	1	2158

Example
Consider a 1 in 10,000 year storm with a duration of 36 hours (3 tidal cycles, M2-tide), including two low waters (2 times 6 hours) in 2100 with fast Sea Level Rise (SLR). It is assumed that no water flows out during low tide. The low water level (LWL) before the storm is given by:

$$LWL(t) = LWL_{2015} + SLR_{2015-2050} \cdot 35y + SLR_{2050-2100} \cdot 50y = 1.07 + \frac{0.93}{100} \cdot 35 + \frac{0.50}{100} \cdot 50 = NAP \quad +1.90 \quad m \tag{7.2}$$

The storage volume *S* is calculated by subtraction of *LWL* from Design Water Level at time *t DWL(t)* multiplied by the surface area of the river A_{river}.

$$S(t) = (DWL(t) - LWL(t)) \cdot A_{river} = (2.67 - 1.90) \cdot 1,700,000 = 13.09 \cdot 10^5 \quad m^3 \tag{7.3}$$

This means that on average, the failing gates cannot let in more than

$$\overline{Q} = \frac{S}{1.5T} = \frac{13.09 \cdot 10^5}{18h37.5m} = 19.52 \quad m^3/s \tag{7.4}$$

The volume of water that is allowed enter the Hollandsche IJssel in case of (a) failing gate(s) reduces over time. At a certain moment in time, the design water level (DWL(t)) and low water level (LWL(t)) reach the same values, illustrated in figure 7.11.

Maximum size gates
Unfortunately, partial failure of multiple gate barriers is not incorporated in HYDRA-BS. However, an indication of the maximum size of a gate is required for further design. Therefore, a first-order analytic calculation is carried out, with:

$$Q(t) = \mu A_b \sqrt{2g(h_{NM} - z(t))} \quad (Torricelli) \tag{7.5}$$

$$Q(t) = A_{HIJ} \frac{dz}{dt} \tag{7.6}$$

where A_{HIJ} is the surface area of the the river, A_b is the opening in the barrier and h_{NM} is the water level in the Nieuwe Maas with a return period of 10,000 years. Eliminating Q, this leads to:

$$\frac{dz}{dt} = \frac{\mu A_b \sqrt{2g}}{A_{HIJ}} \sqrt{h_{NM} - z(t)} \tag{7.7}$$

With h_{NM} assumed constant over time and z(0) = z_0, solving this equation leads to:

$$z(t) = z_0 - \frac{1}{4}(kt)^2 + kt\sqrt{h_{NM} - z_0} \quad with \qquad k = \frac{\mu A_b \sqrt{2g}}{A_{HIJ}} \tag{7.8}$$

If a breach occurs, the water flows in during high tide. However, during low tide, the water level flows out. For outflow, similar equations as for inflow are used:

$$Q(t) = \mu A_b \sqrt{2g(z(t) - h_{NM})} \quad (Torricelli) \tag{7.9}$$

$$Q(t) = -A_{HIJ} \frac{dz}{dt} \tag{7.10}$$

Which leads to:

$$z(t) = z_0 - \frac{1}{4}(kt)^2 - kt\sqrt{z_0 - h_{NM}} \qquad with \qquad k = \frac{\mu A_b \sqrt{2g}}{A_{HIJ}} \tag{7.11}$$

The effects on the water level of different entrance openings can now be plotted for different points in time. Figures 7.12 & 7.14 show resulting water levels for 2050 and 2100 for the (WARM/STOOM) scenario including outflow through the failed gate during low tide. In figures 7.13 & 7.15, the cross-section during low tide is increased to 40 m^2 to maximise outflow. A summary of the used values is found in table 7.6.

Table 7.6: Used values for first order calculation (WARM/STOOM).

Description	Symbol	Value	Unit
Contraction coefficient	μ	1.0	-
Gap area	A_b	Variable	m^2
Surface area river	A_{HIJ}	$1.7 \cdot 10^6$	m^2
Water level Nieuwe Maas 2050	h_{NM}	+3.65	m NAP
Water level Nieuwe Maas 2100	h_{NM}	+3.89	m NAP
Low water level 2050	z_0	+1.40	m NAP
Low water level 2100	z_0	+1.90	m NAP
Design water level 2050	DWL	+2.66	m NAP
Design water level 2100	DWL	+2.01	m NAP

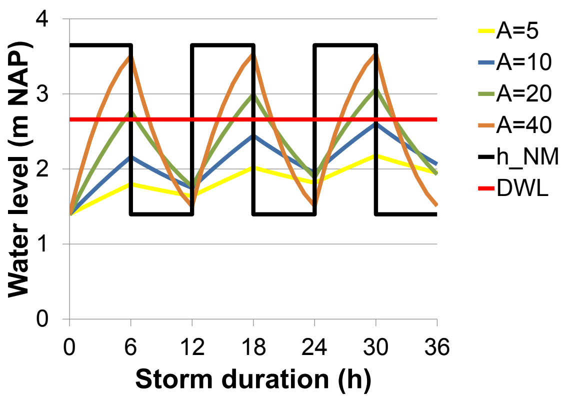

Figure 7.12: Water level in HIJ during storm including outflow in 2050 (WARM/STOOM).

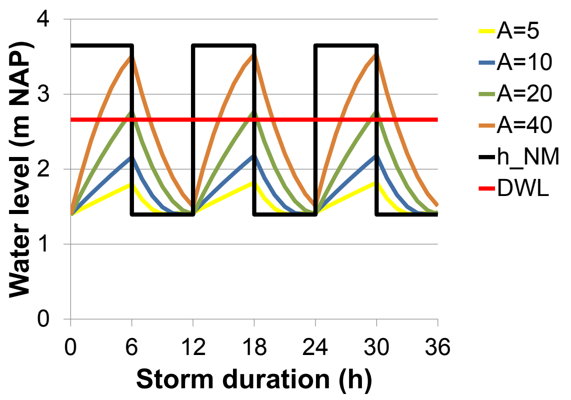

Figure 7.13: Water level in HIJ during storm including increased outflow in 2050 (WARM/STOOM).

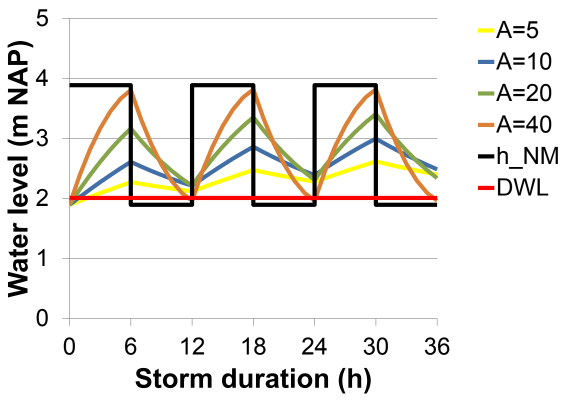

Figure 7.14: Water level in HIJ during storm including outflow in 2100 (WARM/STOOM).

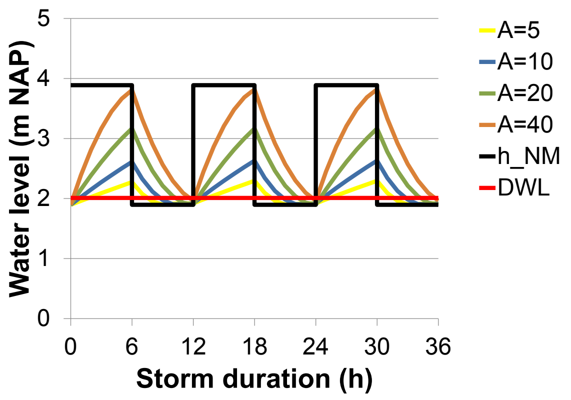

Figure 7.15: Water level in HIJ during storm including increased outflow in 2100 (WARM/STOOM).

2050

Figure 7.12 shows that during the storm in 2050, water levels do not exceed the norm or design water level(DWL, red line in figures 7.12 & 7.13) if the breach is kept under 10 m^2. Increasing the outflow area of water to 40 m^2 (figure 7.13) increases the maximum allowed breach to around 20 m^2.

2100

Reduction of the norm in combination with sea level rise results in failure of the barrier in any circumstance. The norm or design water level (DWL, red line in figures 7.14 & 7.15) is close to the low water level. This results in a very small storage capacity of the river behind the barrier (see also figure 7.11). Increased outflow (figure 7.15) is not sufficient, even if the breach is only 5 m^2.

2150

No calculations were performed for 2150 because the theoretic maximum lifetime of a system without pumps is exceeded in 2111 (see table 7.5). The low water level before the storm is higher than the design water level.

The calculations presented in this section for the WARM/STOOM scenario were also carried out for the DRUK/RUST scenario. The results of all calculations are tabulated in table 7.7. Again, no dike reinforcements are carried out after implementation of the new barrier. In order to prevent flooding of the dikes, table 7.7 shows that the size of a failing cross-section should be smaller than 5 m^2.

Table 7.7: Maximum allowable size inflow for different scenarios in storm conditions

Description	Year	Outflow ($\mu\cdot$ A) [m^2]	Maximum allowed inflow ($\mu\cdot$A) [m^2]
DRUK/RUST	2050	≈15	≈15
DRUK/RUST	2050	40	≈20
DRUK/RUST	2100	<5	<5
DRUK/RUST	2100	40	<5
DRUK/RUST	2150	<5	<5
DRUK/RUST	2150	40	<5
WARM/STOOM	2050	≈10	≈10
WARM/STOOM	2050	40	≈20
WARM/STOOM	2100	«5	«5
WARM/STOOM	2100	40	«5

Limitations

The first order calculations presented in this section, have several limitations. First of all, the water level in the Nieuwe Maas, h_{NM}, alters between low water and high water, which is obviously a schematisation of the actual situation. Secondly, with the use of first order calculations, it is assumed that no fluctuations of water levels over the length of the river are present. This is obviously a simplification of reality. Thirdly, only the influence of failure during the entire storm surge was analysed. One can imagine that a small gap can be closed or at least narrowed during a storm surge. This would have a positive effect on limiting the water levels in the Hollandsche IJssel. Fourthly, the influence of pumping capacity on the ‘allowed’ gap in the barrier was not looked into. Installing pumping capacity could reduce water levels in the Hollandsche IJssel before the surge occurs (see figure 7.11). Additionally, in case of failure, water flowing in can be pumped out by the installed pumps.

Conclusion

Table 7.7 showed that, to prevent failure of the dikes along the river, the size of failing cross-section should be smaller than 5 m^2. However, as mentioned under limitations, fast closure of a failing gate and the effect of pumps have not been taken into account. In the remainder of this thesis, a maximum value of μ·A = 5 m^2 will be used for further calculations.

7.5 Shipping through barrier

If ships have to go through a lock at all times, extra societal costs are incurred for shipping (chapters 5 & 6). An opening maintaining the tidal action in the river in combination with a shipping channel could nullify this societal cost. In the previous section it was found that a failing cross-section should be smaller than 5 m^2. From this finding, it is concluded that shipping cannot pass the new barrier without going through a lock.

7.6 Summary of findings

In this chapter, a number of results have been presented regarding the lifetime of the structure and the size of the opening for tidal action. The findings have been listed below and will be used as input for chapters 8 and 9.

- A single gate solution in combination with no further dike improvements after 2050 (implementation new barrier) leads to insufficient safety against flooding.
- If the current tidal range is maintained, the barrier can function under normal conditions without pumps upto 2105 in the WARM/STOOM scenario or 2140 in the DRUK/RUST scenario.
- Under storm conditions, a situation without pumps can theoretically be realised until 2111 (WARM/STOOM) - 2158 (DRUK/RUST). This is however a theoretic maximum. The actual lifetime of the barrier without pumps a topic for further research.
- In order to maintain 90% of the original tide, the minimum required cross-section is ≈150 m^2 (excluding friction and entry and exit losses). The actual size is dependent on the length and configuration (and therefore friction) of the barrier and is approximately 225 m^2, see appendix K.
- To prevent failure of the dikes along the river during a storm, the size of failing cross-section in the barrier should be smaller than 5 m^2. The maximum allowed size of each gate is assumed to be μ·A=5 m^2. This value will be used for further design purposes (chapters 8 and 9).

8

Conceptual design

8.1 Introduction

In this chapter, several decisions are made that rougly define the dimensions of the barrier. First of all, the location of the new barrier is determined. Various aspects will be taken into consideration as described in section 8.2. After that, the type of barrier is chosen in section 8.3. A multitude of concepts is taken into consideration. This section is followed by a short reminder of the aspects that should be taken into consideration to enable fish migration (section 8.4). The required pumping capacity found in chapter 7 has to be incorporated in the design. This is discussed in section 8.5, followed by the proposal to reduce the width of the river (section 8.6). All findings are summarised in section 8.7.

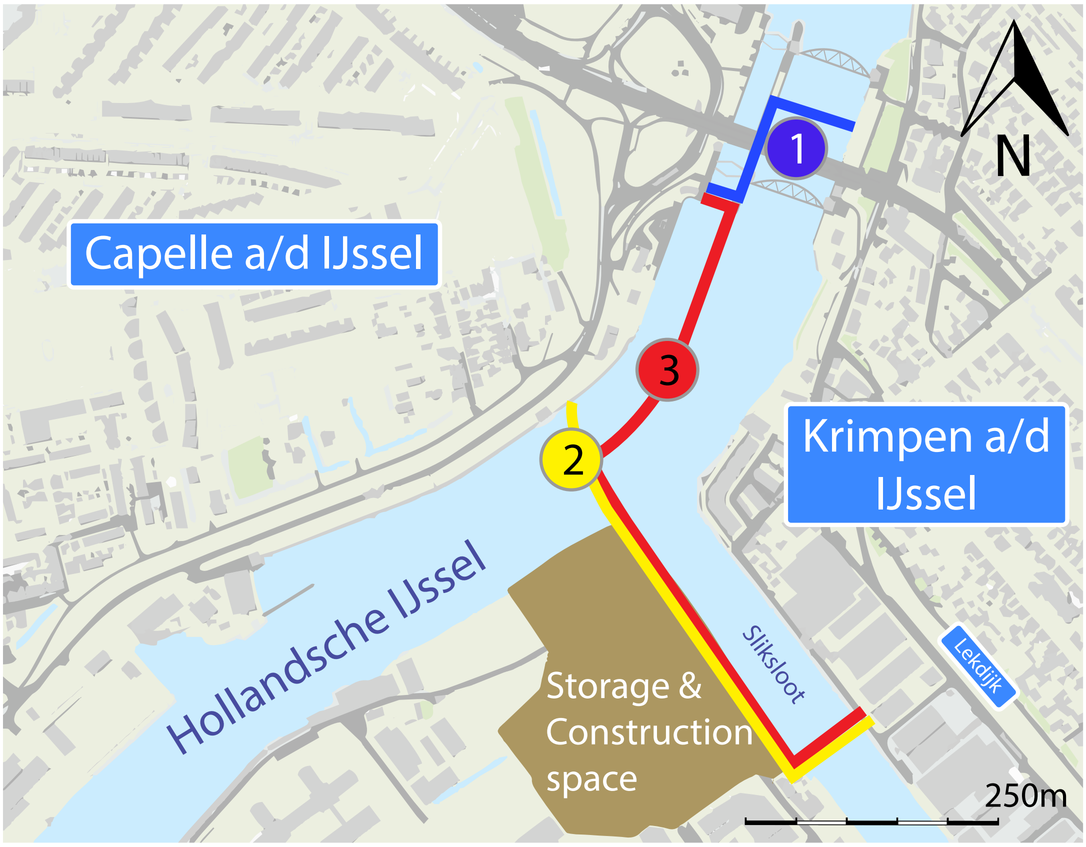

Figure 8.1: Possible locations of new barrier complex. Source: World Topographic Map.

8.2 Location

In choosing a location, three options have been considered (see figure 8.1). All locations considered keep the current barrier intact. The barrier has reached its technical design lifetime in 2050 and loses its function as storm surge barrier. The flood doors could however be conserved as a landmark/heritage. The next paragraphs will discuss the various aspects looked at and how each of the option scores on them. The three options considered are:

1. **Keep current location**. The current situation is conserved as much as possible. The new barrier is constructed between the current flood doors of the Algera barrier, ships pass the barrier through the current Algera lock and road traffic use the road connection that is already present (Algera bridge).
2. **New start**. A completely new barrier is realised just downstream of the current barrier. More space is available, so that more locks can be constructed when needed. The current Algera lock becomes obsolete. Furthermore, due to the new location, the new barrier can be combined with a improved road connection, which is not possible at location 1 (elaborated in chapter 6).
3. **Middle solution**. This location is a combination of *'Keep current location*' and *'New start'*. Ships can pass the barrier through the current Algera lock, therefore a new lock does not need to be constructed. Additionally, it is possible to combine the new barrier with the realisation of a new road connection.

8.2.1 Improvement road connection

Initial calculations (chapter 6) on congestion length show that, when closed, the bridge connection has sufficient capacity for current traffic intensities. It is the junction in Krimpen a/d IJssel that has insufficient capacity. Since space here is very limited, improving the capacity of the junction is highly problematic. To improve the situation, the road connection Capelle - Krimpen should have a different trajectory. It seems most logical to combine the barrier with a road connection. Therefore, from a mobility standpoint, it is advised to construct the barrier at a new location (location 2 or 3). At these locations, the barrier and road connection can be combined. By constructing a barrier complex at location 2 or 3, the problematic junction in Krimpen a/d IJssel could be by-passed and a better road connection could be established. Appendix O provides an analysis on the possible types of crossings. An example of a trajectory is illustrated in Appendix F, figure F.17.

8.2.2 Use of current lock

Currently, the Algera lock is only used for tall ships and when the barrier is closed (Gemeente Krimpen aan den IJssel, nd). When the new barrier is implemented, *all* ships need to go through a lock. The capacity of the Juliana lock complex has recently doubled (van de Sandt, 2014). It is therefore logical to investigate whether the capacity of the existing Algera lock is sufficient if all ships need to go through a lock, now and in the future. The scenarios on the development of shipping (chapter 6) were used to check whether the capacity of the current lock is sufficient. It was found that in three out the four scenarios tested, the capacity of the lock under current operation times is sufficient until 2150. In one scenario the capacity was tested insufficient. Construction of an extra lock is an option to increase the capacity. However, various mitigating measures, such as a different road connection that does not cross the current lock, could also increase the capacity of the current Algera lock. As mentioned in the previous paragraph, a new road connection is also needed from a mobility standpoint. It is therefore concluded that an extra lock is not necessary. Appendix L provides an elaborate analysis on the lock capacity. At location 1 and 3 the existing lock can be used, while in case of location 2 a new larger lock is constructed. Therefore, locations 1 and 3 score higher than location 2 on this aspect.

8.2.3 Barrier length

The length of the barrier is an important cost driver. As can be seen in figure 8.1, location 3 has a far longer length than location 1 and 2.

8.2.4 Connection to the dike rings

All structures should be connected to both dike rings. As location 1 is similar to the current Algera barrier, no difficulties are foreseen to connect it to the dike rings. If location 2 or 3 is chosen, the connection to DR15 is more difficult to achieve. The Sliksloot, see figure 8.1, has an open connection with the Nieuwe Maas and the Hollandsche IJssel. Figure 8.2 shows two options to connect to DR15, option A and B. If location A is chosen, the connection of the Sliksloot to the Hollandsche IJssel is lost. This would seriously damage nautical activities in the Sliksloot. If location B is chosen, nautical activities are saved. However, the new barrier becomes very long and therefore costly. It is therefore concluded that location 1 scores better on the connection to the dike rings than location 2 and 3.

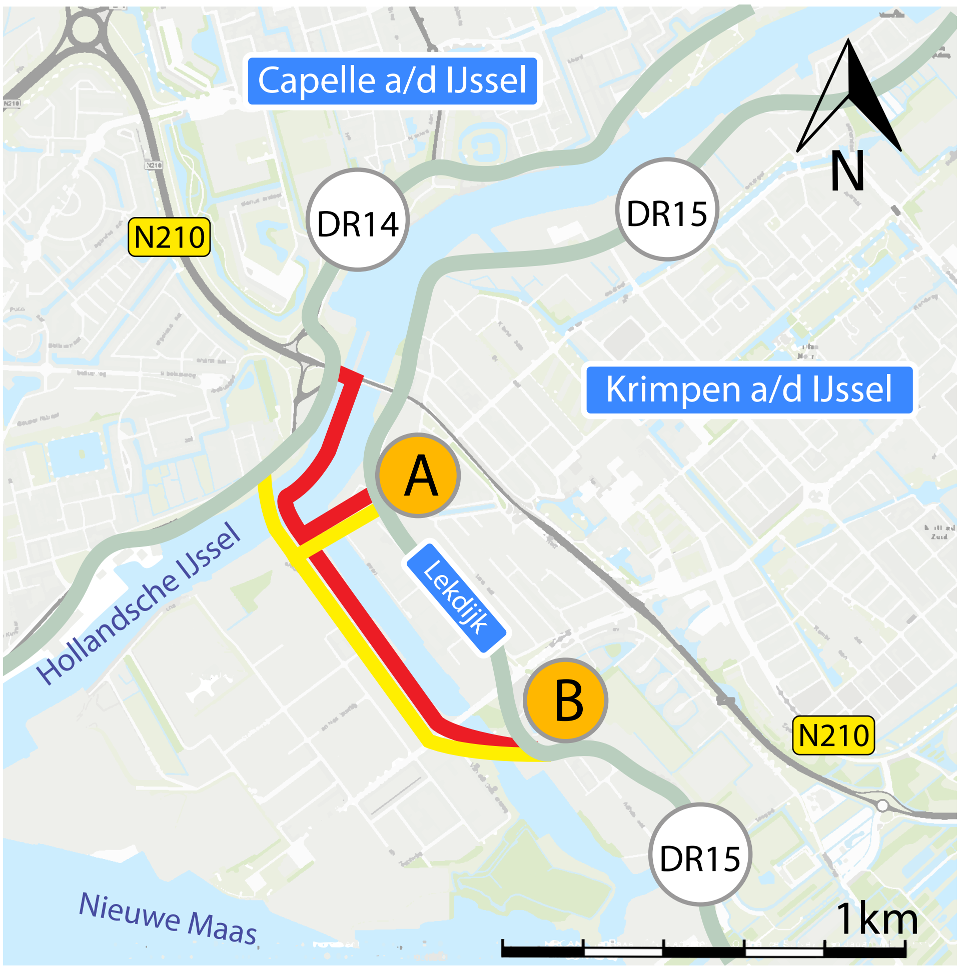

Figure 8.2: Possible connections of location 3 (red) and 2 (yellow) to DR15. Source: World Topographic Map.

8.2.5 Temporary situation

The temporary situation has three aspects, namely (1) flood protection, (2) shipping and (3) traffic. At all 3 locations the current Algera barrier is kept intact. Flood protection in the temporary situation is therefore not an issue. Furthermore, shipping will be hampered in the temporary situation, independent of the location chosen. Close to location 2 and 3 space is available (old EMK terrain, indicated in brown in figure 8.1) to construct a structure or to store building materials (see figure 8.1). This can reduce hindrance to shipping and traffic. The old EMK terrain however contains contaminated soil. The cost of removing the soil should therefore be weighed against hindrance for traffic and shipping. It is concluded that location 2 and 3 score better on the temporary situation than location 1.

8.2.6 Summary of findings and choice of location

The results of the location analysis are tabulated below (table 8.1). When adding all scores (assuming equal importance), location 1 has the most positive score and is therefore chosen. It should be noted that this comparison is purely qualitative. The results show that location 1 scores positive on the *length*, *use of the existing lock* and *connection to the dike rings*. The barrier length is an important cost driver for storm surge barriers (Mooyaart and Jonkman, 2017). At this location the road connection cannot be improved and is not taken into account any further. As the barrier and the new connection are now decoupled, no further attention is paid to dimensioning and designing this connection. However, based on the estimated societal cost for congestion (see chapter 6), it is advised that a new road connection in combination with a by-pass around Krimpen a/d IJssel is researched.

Table 8.1: Aspects different locations.

Location	Name	Road conn.	Use current lock	Length	Conn. dike rings	Temp. situation
1	Keep current	-	+	+	+	-
2	New start	+	-	-	-	+
3	Middle	+	+	- -	-	+

8.3 Type of barrier

The barrier to be constructed has several functions namely: (1) a flood defence function (2) a tidal/ecological function and (3) a shipping function. It consists of two parts, namely:

1. Navigation lock;
2. Closable barrier section.

Both parts of the barrier are used for flood defence. The navigational lock has a shipping function and the closable barrier section assures tidal action. Now that location 1 is chosen (see section 8.2) a navigational lock does not need to be designed. Focus is placed on the closable part of the barrier. A complete overview of barrier options is provided by van der Toorn and de Gijt (2013) and de Vries (2014). The overview is summarised and taken into consideration in Appendix N. As this thesis focuses on incorporation of ecological value into design considerations, considering a natural barrier at this stage of design is self-evident.

At first instance, conceptual designs are based on hydraulic boundary conditions over a life time of 100 years. Here, three situations can be distinguished, namely: the current situation (1), the surge situation (2) and the future situation (3). The barrier should:

1. Allow for tidal action behind the barrier;
2. Be able to protect the hinterland from storm surges;
3. Allow control of the water table behind the barrier and alteration of the water level over the design lifetime.

One could say that criterion 2 is a special case of criterion 3. It should be noted that dimensions, water levels and tidal amplitudes of the figures presented below are not to scale.

8.3.1 Natural barrier

A natural solution could be a form of a reduction barrier, reducing the inflow of water into the Hollandsche IJssel. A barrier of reed or a wide floodplain are options to consider. Direct after construction, water could flow over the barrier to and fro the Hollandsche IJssel, creating a new habitat of intertidal areas and tidal channels.

A natural barrier thus satisfies the first requirement mentioned above. The two other requirements can however not be met as is shown in figure 8.3. To allow for daily tidal action, the barrier should be designed at mean water level. Storm surges, criterion 2, can therefore not be prevented (figure 8.3 (Middle)). Furthermore, a lowered water table in the Hollandsche IJssel (criterion 3) limits the outflow of water over the barrier (figure 8.3, right). This problem is similar to the structural solutions, however in this case the tidal opening cannot be altered quickly. To do so, a structural measure should be added.

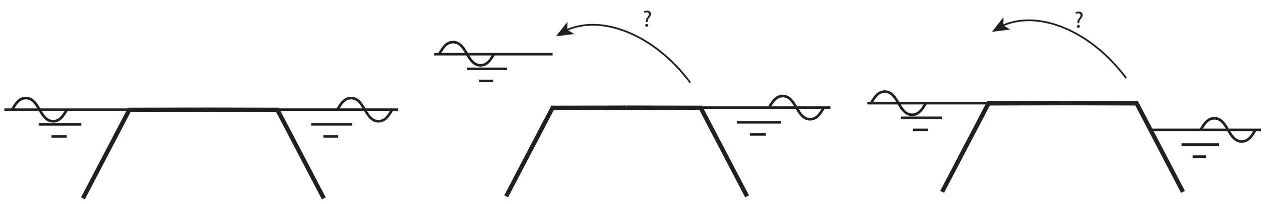

Figure 8.3: Natural solution. Current (Left), Surge (Middle), Future (Right) situation.

In conclusion, a natural barrier does not comply with all requirements and cannot be used as a hydraulic conceptual design. To fulfil all hydraulic requirements, structural measures are needed.

8.3.2 Structural measures

In the paragraphs below, different structural measures have been described that could be used to create the hybrid barrier. All meet the hydraulic requirements described in Appendix N. The barrier should e.g. be able to resist negative head and it should be possible to alter the tidal opening.

Siphon

A siphon could be used to transfer water from one side to the other. In case of a storm surge, air is let into the tubes on the top of the dike, making entering of the water into the Hollandsche IJssel impossible. Siphons can be used to overcome head differences upto 7 m (van Rossum, 2016).

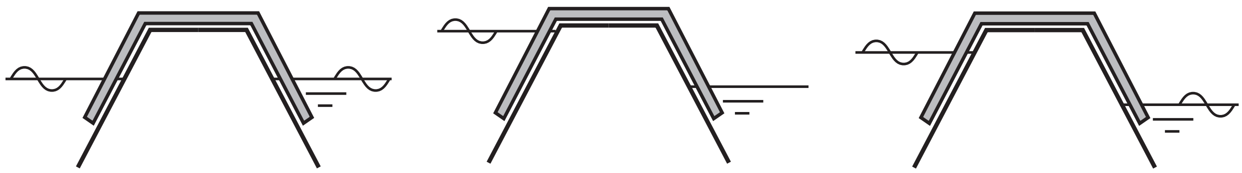

Figure 8.4: Siphon solution. Current (Left), Surge (Middle), Future (Right) situation.

Check valve

A system of culverts with (check) valves could be implemented as well. Closing off or opening the culverts allows alteration the tidal opening. In storm conditions, all valves are closed.

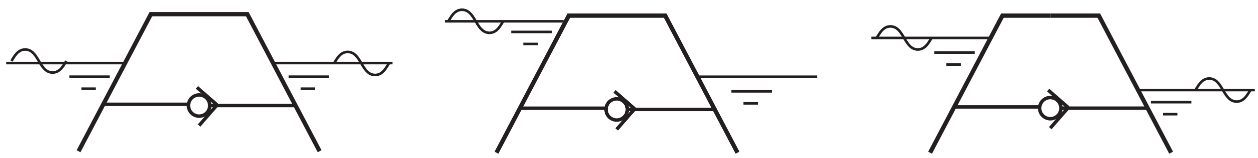

Figure 8.5: Check valve system. Current (Left), Surge (Middle), Future (Right) situation.

Horizontal rotation

A barrier with horizontal rotating doors has been constructed in the Thames. The rotation allows for different positions, enabling maintainance to be carried out in the dry. As hinges may be problematic (which was the case in the Maeslantbarrier (Cobouw, 2004)), this may not be the most durable solution.

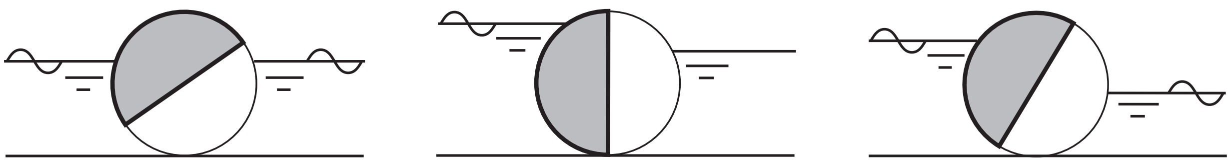

Figure 8.6: Horizontal rotating doors. Current (Left), Surge (Middle), Future (Right) situation.

Vertical lift

Vertical lifting doors have been applied in many barriers. The current Algera barrier, and the Eastern Scheldt barrier are famous examples. In storm conditions, the barrier is closed. In the future, the tidal opening can be altered easily.

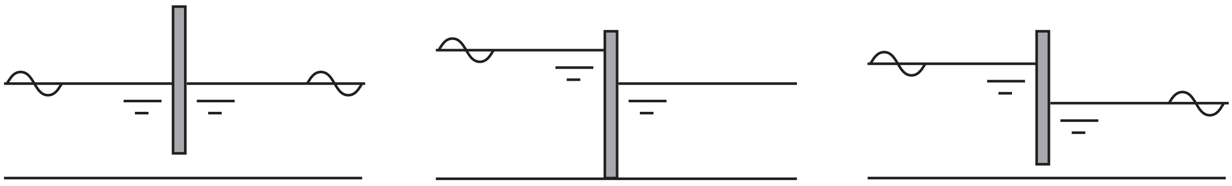

Figure 8.7: Vertical translating doors. Current (Left), Surge (Middle), Future (Right) situation.

Inflatable rubber barrier

Rubber barriers acting as storm surge barrier are relatively new. The concept has been applied in The Netherlands once as a surge barrier, in Ramspol. The barrier can be closed in storm conditions. In the future, sections of the barrier may be closed to cap inflow of water. Partial inflation might also be an option.

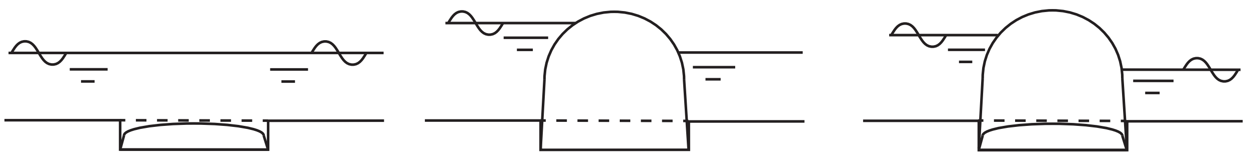

Figure 8.8: Inflatable rubber barrier. Current (Left), Surge (Middle), Future (Right) situation.

8.3.3 Choice of barrier type

Section 7.4 showed that for all climate scenarios, the barrier needs pumping capacity to counter a breach in storm conditions. In order to limit inflow in case of a single gate failure, the openings in the barrier should be kept small. Under normal conditions, the WARM/STOOM scenario also shows that pumping capacity is needed before 2150 (section 7.3). So, pump capacity is needed during the intended technical lifetime (100 years) of the barrier. If pump capacity can be incorporated relatively easy, adaptation cost (e.g. demolishing the super structure before pumps can be installed) can be saved. Siphons, horizontal rotating, vertical lifting and inflatable barriers would require significant adaptations. A culvert system with valves could be adaptive and is therefore preferred. The in- and outlet structure can be designed in such way that pumps can be installed easily.

8.4 Fish migration

In order to enable fish to migrate through the barrier, the culverts should have minimum dimensions. From section 4.8 it is obtained that a minimum width of 1.0 m and a water depth of 0.5 m is required. Culverts (see section 8.3) should therefore have minimum dimensions of 1.0 x 0.5 m.

8.5 Pump capacity

Pumping capacity is required during the lifetime of the barrier (see chapter 7). In order to make the design adaptive to required pumping capacity, several culverts have to be designed as such that pumps can be installed at a later stage. At first glance this may seem an expensive solution. However, pumping capacity can be used in both regular and storm conditions:

- In regular conditions the pumps can be used to maintain desired tidal ranges and thus conserve ecological value;
- In storm surge conditions, the pumps can be used to lower the water level before the storm makes landfall, thus increasing storage capacity;
- In storm surge conditions, the pumps can be used to pump out water in case of door failure;
- The future head differences can be used to generate part of the power needed to pump out water.

8.6 Width of the river

The current cross-section of the river is approximately 600 m^2 (6x100 m). To illustrate this, figure 8.9 shows a cross-section of the river about half-way the river (km 8.4). The cross-section was very likely to be historically needed to convey discharge. Nowadays however, the river has almost no discharge (see chapter 1). This cross-section is not necessary from different points of view:

- **Discharge**: the river is not connected upstream, therefore maximum discharge is determined by the pumping capacity, which is about 100 m^3/s (see Appendix G). Assuming a flow velocity of 1 m/s, a cross-section of 100 m^2 is needed.
- **Tide/ecology**: for an average tidal difference of 1.45 m, a cross-sectional area of ≈225 m^2 at the mouth is required, (see Appendix K).
- **Shipping**: The desired width of the shipping canal is equal to 4x width + side wind addition (Bezuyen et al., 2011, p.193) . The desired depth = 4.9 m for Class Va ships (Rijkswaterstaat, 2011, p.41). Side wind addition is 5% of length for land region. This means that for CEMT class Va ships (110 x 11.4 m x 3.30 m) a width of 45.6 + 5.5 = 51.1 m is needed.

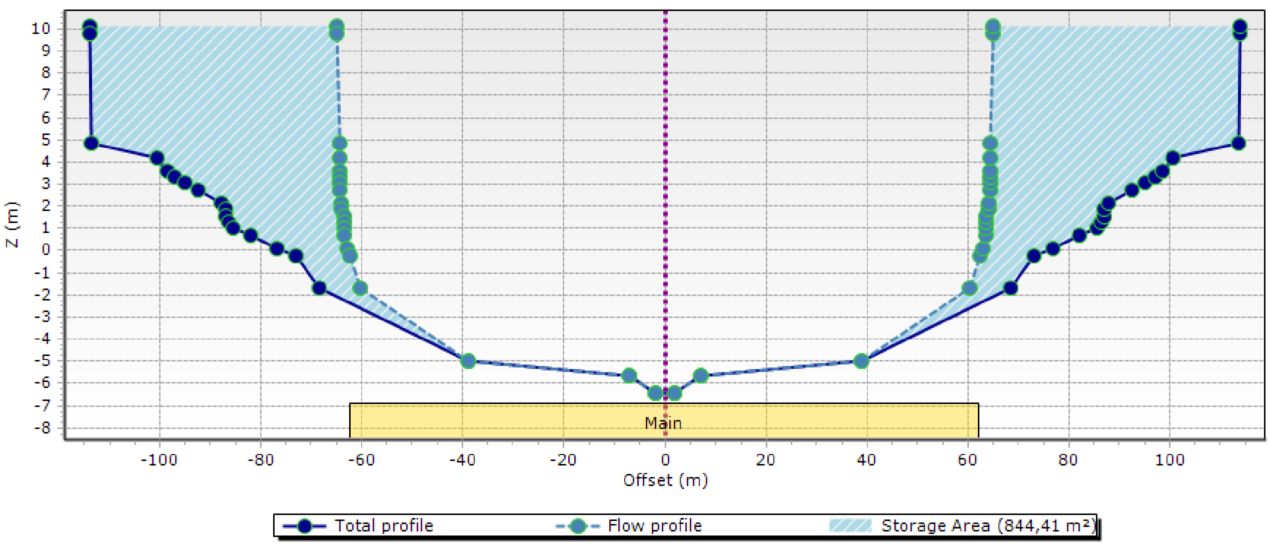

Figure 8.9: Schematised cross-section of the Hollandsche IJssel, km 8.4. Source: SOBEK, RMM model.

8.6.1 Narrowing the river

Although the barrier is designed to conserve as much as the tidal range as possible, a reduction of ≈10% is expected. Reason for this is the rapid increase in required cross-section if a tidal range larger than 90% of the original value is required (see figure 7.4). One could say that the realisation of extra intertidal area could compensate the loss in tidal range. Currently, the river has a navigable width of about 100 m, while the required width for CEMT Va ships is about 50 m (see previous paragraph). This means that there is space available for the creation of intertidal nature. The navigable profile is reduced to minimal dimensions, providing space for nature development. Figures 8.10 and 8.11 provide a visualisation. Depending on the space available, the river can be narrowed from one side or two sides. Besides nature creation, the created fore-lands may increase stability and reduce run-up.

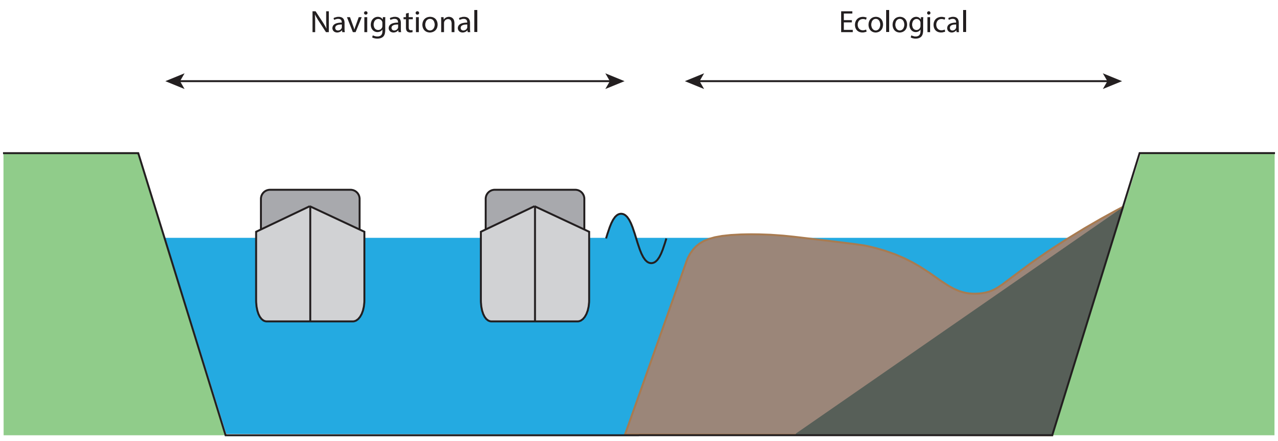

Figure 8.10: Nature maximisation at one side of the river.

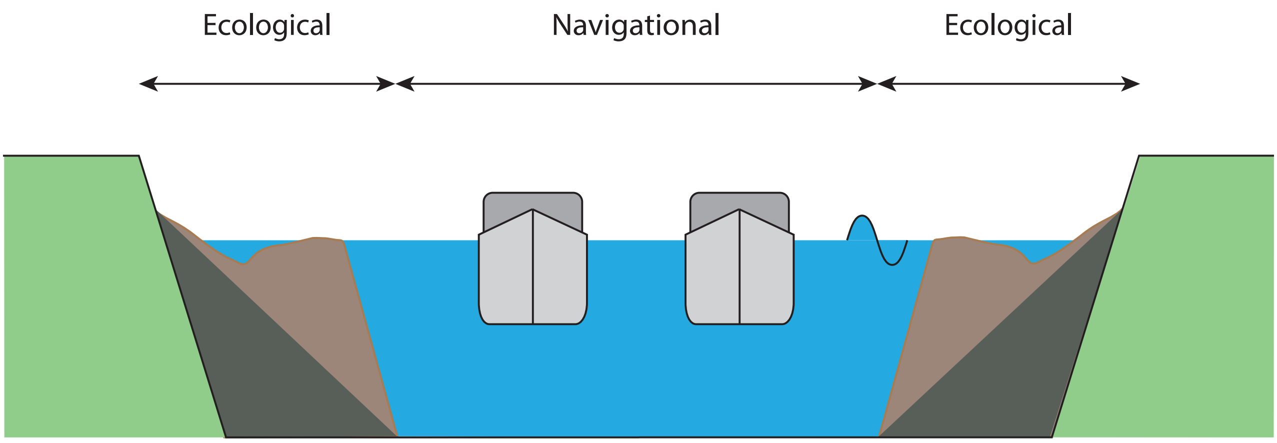

Figure 8.11: Nature maximisation at two sides of the river.

Reducing the width of the river has several advantages and disadvantages, which have been summed up below.

Advantages

- **New nature**: extra shallow water and intertidal zones are created. Currently the majority of the water in river is deep water.
- **Separation of functions**: the nature function and navigational function of the river are separated.
- **Reduction hydraulic load**: wave run-up is reduced due to the shallow area in front of the dikes. Furthermore, fetch is reduced.
- **Increased stability**: a part of the filled up river can be used to increase the stability of the dikes: the outer slopes can be made more gentle (dark brown in figures 8.10 & 8.11).
- **Reduced volume**: the water volume in the basin is reduced. Less pumping capacity is needed to lower water levels. Less water needs to be refreshed, less effort is needed to maintain water quality.

Disadvantages

- **Cost**: cost are incurred for the creation of nature.
- **Maintenance**: the river bed may not settle as fast as the surrounding dikes. In order to maintain similar flood rates of these areas, soil may need to be removed over time from the newly created tidal flats. Furthermore, a solid maintenance and risk management plan is required in advance that is specific about what actions are needed at which moment in time and who is responsible for it.
- **Possible siltation and succession**: Due to the new type of barrier, the sediment balance may be influenced. Possibly, sediment flows in but cannot flow/be pumped out, thus creating a sediment trap. In that case the extra sediment available could lead to siltation and succession. Siltation could also lead to an insufficient navigational depth. Further research is required to find out whether, and if so, to what extent the Hollandsche IJssel would become a sediment trap.
- **Reduction storage**: The storage capacity is slightly reduced. Only the storage capacity around MWL is reduced. In storm conditions, the low water level lies above MWL (at Krimpen, MWL= ±NAP +0.46, LWL= ±NAP +1.07). Therefore, only in case of severely reduced water levels (that can only be achieved by means of pumping) the storage capacity is altered.

Available space
Narrowing the river is not possible along the entire river. The concept is considered to be less applicable near villages and close to existing mooring facilities. Figures P.1 and P.2 in Appendix P show the locations that are suggested to narrow the river. In total, approximately 650,000 m^2 of intertidal area could be realised, compared to the current 250,000 m^2.

8.7 Summary and conclusions

In this chapter the barrier has been described conceptually. A barrier located in between current flood doors of the Algera barrier was found to be most suitable. The capacity of the existing lock is sufficient. In one of the scenarios analysed, extra capacity is required. However, mitigating measures are possible that could increase the capacity of the current lock. An extra lock is therefore unnecessary and not advised. Although the improved road connection could not be integrated at the location chosen, it is strongly advised to look into a new connection (e.g. a tunnel). Appendix O gives the reader a qualitative comparison of the different options applicable for this case. However, as the barrier and the new connection are now decoupled, no further attention is paid to dimensioning and designing this connection. Different barrier types were analysed and a set of closable culverts was chosen. Furthermore, as discussed in chapter 7, pumping capacity is required during the lifetime of the barrier. A number of culverts should be adaptable as to install of pumps at a later stage. The barrier might reduce the tidal range by ≈10%. Narrowing the river has several benefits, of which the realisation of extra intertidal area is one. By narrowing the river as proposed, an extra 400,000 m^2 of intertidal nature can be realised. The disadvantages (e.g. possible siltation) should however be looked at closely as well.

The conceptual design consists of the following elements:

- The current **Algera lock**;
- A **barrier with** a set of **culverts**, in some of which pumps can be installed. All culverts should be closable and are of limited size;
- **Pumps** (future): pumps are required in the future to conserve the tide in normal conditions and reduce water levels in storm conditions.
- The **river** will be altered by **narrowing** it. This can compensate small losses in tidal range and increase the available intertidal areas.

9

Preliminary design

9.1 Introduction

In this chapter, the conceptual design presented in the previous chapter is worked out in further detail. In section 9.2, the extreme load situations are presented, followed by information about the subsoil conditions and wind wave loading. After this, a design will be presented in section 9.3. First the required height of the structure is determined based on the load situations presented in section 9.2. After that, the bottom level of the structure is determined, based on the required bottom level of the culvert and pump structure. This section ends with a description of a possible way to close off the culverts. In section 9.4, a number of checks is carried out to check whether the proposed structure is stable and can resist piping. Section 9.5 shows a method to construct the pump and culvert structure, followed by a conclusion (section 9.6).

9.2 Input

9.2.1 Load situations

It is not always the most extreme surge condition that is normative for all failure mechanisms. Therefore, in designing the structures, two situations will be considered:

1. Maximum positive head;
2. Maximum negative head.

In situation 1, a 1 in 10,000 year water level on the outside of the barrier is present at the outside of the barrier (Nieuwe Maas side). Sea level rise results in that maximum water levels occur at the end of the design life time, 2150. Hydra-BS calculations were extrapolated to calculate the expected water levels in 2150. For the WARM/STOOM scenario, a water level of NAP +4.25 m with a return period of 10,000 years is expected by 2150. At the inside of the barrier (Hollandsche IJssel side), the water level is limited by the Design Water Level. The design water level in 2150 is NAP +1.56 m (see table 7.1). However, pumps could create extra storage capacity by lowering the water table in the Hollandsche IJssel. It is assumed that the pumps could reduce the water level by 1 m, resulting in a water level at the inside of the barrier of NAP +0.56 m.

Maximum negative head, situation 2, occurs right after construction (2050). At this moment in time, the dikes along the river can resist the highest water levels. A situation can occur in which water levels on the Hollandsche IJssel have reached maximum values. However, the barrier could fail to open and the water table is not lowered during low tide (on the Nieuwe Maas). This leads to the situation of maximum negative head. Water levels inside the barrier (Hollandsche IJssel) could reach values upto the maximum water level (NAP +3.22 m, see table 7.1), while water levels outside the barrier are based on low tide in the Nieuwe Maas (NAP +1.40 m (Rijkswaterstaat, ndd, extrapolated)). Table 9.1 provides an overview of the situations considered.

Table 9.1: Considered situations.

Situation	Description	Year of occurrence	h_{NM} [m NAP]	h_{HIJ} [m NAP]
1	Maximum positive head	2150	+4.25	+0.56
2	Maximum negative head	2050	+1.40	+3.22

9.2.2 Subsoil conditions

Bathimetry

Figure 9.1 shows the bathimetry at the location where the new barrier will be realised. The lock on the left and the current navigational channel on the right can clearly be distinguished. On average, the river bed in the navigational channel lies at NAP -8.5 m. In front of the barrier, the river bed is at NAP -7.0 m on average, see Appendix Q.

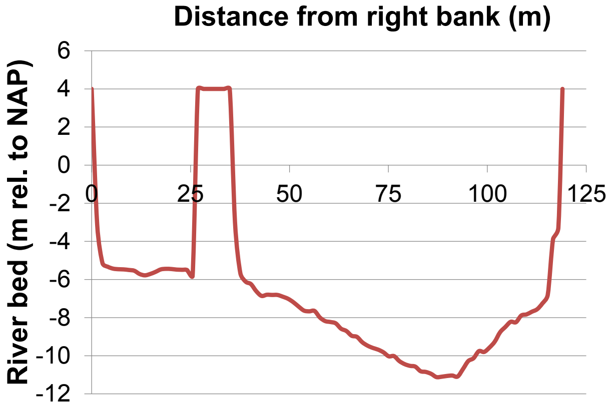

Figure 9.1: Bathimetry of the river at the Algera barrier. Source: Rijkswaterstaat.

Soil classification

Soil classification is performed from the level of the river bed downwards. Six cone penetration tests (CPTs), obtained from DINO loket (TNO Geologische Dienst Nederland, nd), were analysed to determine subsoil conditions. The interpreted CPTs can be found in Appendix Q. Six different soil types were used to classify, tabulated in table 9.2.

Table 9.2: Soil classification and parameters (NEN, 2012, table 2.b).

Type	Name	Density $[kN/m^3]$	Cohesion, c $[kN/m^2]$	Friction angle, ϕ [°]
A	"Clay"	15/15	3	17.5
B	"Clay silty"	16/16	4	20
C	"Clay, peaty"	15/15	2	15
D	"Peat"	10.5/10.5	2	15
E	"Sand"	18/20	0	32.5
F	"Sand clayey"	18/20	0	30

It was found that a (pleistocene) sand layer is present between NAP -6 m and NAP -12 m. In most CPTs, a clay layer is present at the riverbed level. In case of a shallow foundation, this layer should be removed. From analysis of the CPTs, two schematised soil profiles have been drafted, see table 9.3.

Table 9.3: Soil profile A and B.

Profile	Top level [m NAP]	Botom level [m NAP]	Soil type
A	-6	-20	Sand
B	-6	-8.7	Clay
	-8.7	-9.0	Clay peaty
	-9.0	-10.2	Clay
	-10.2	-11.0	Clay peaty
	-11.0	-20.0	Sand

9.2.3 Wind wave loading

The wind wave height is calculated using the formulas of Bretschneider, which are considered to give reasonable results (Ministerie VWS and ENW, 2007, p.98). The Bretschneider equations are (Ministerie VWS and ENW, 2007, p.137):

$$\tilde{H} = 0.283 \cdot tanh(0.53\tilde{d}^{0.75})tanh\left(\frac{0.0125\tilde{F}^{0.42}}{tanh(0.53\tilde{d}^{0.75}}\right) \tag{9.1}$$

$$\tilde{T} = 2.4\pi \cdot tanh(0.833\tilde{d}^{0.375})tanh\left(\frac{0.077\tilde{F}^{0.25}}{tanh(0.833\tilde{d}^{0.375}}\right) \tag{9.2}$$

in which:

$$\tilde{H} = \frac{gH_s}{U^2} \qquad \tilde{T} = \frac{gT_p}{U} \qquad \tilde{F} = \frac{gF}{U^2} \qquad \tilde{d} = \frac{gd}{U^2}$$

Here, F is the fetch (m), U is the wind velocity (m/s) at an altitude of 10 m , d is the water depth (m) and T_p is the peak wave period (s). For the newly constructed barrier, the following parameters are used:

- The fetch perpendicular to the structure is used;
- The fetch for the inlet structure is 2140 m (along the river axis in SW direction);
- The river bed is estimated to be at NAP -7 m on average (see figure 9.1). With water levels in 2150 at NAP +4.25 m (see section 9.2.1), the water depth is 11.25 m;
- In extreme conditions, the wind speed is expected to be 42 m/s (Ministerie VWS and ENW, 2007, p.120).

Using the values above, leads to a significant wave height (H_s) of 1.65 m and a peak period (T_p) of 4.61 s for the inlet structure.

9.3 Design

9.3.1 Height structure

A water level of NAP +4.25 m can be expected by 2150 (see section 9.2.1. In case no waves are expected a freeboard of 0.5 m (Ministerie VWS, 2007) should be added. This leads to a required height of NAP +4.75 m.

Overtopping

However, incoming waves do hit the inlet structure. The height of the structure is therefore dependent on the allowable overtopping. It is assumed that in storm conditions, people could be present at the top of the structure. Therefore, the maximum allowable overtopping discharge is assumed to be 1 l/s/m (Pullen et al., 2007, p.31). Only results are presented in this paragraph; Appendix R shows the full analysis on how crest levels were derived. This analysis shows that the required overtopping discharge (1 l/s/m) is reached if the crest height is at NAP +9.4 m, compared to the required NAP +4.75 m for the surge water level alone. Therefore, two mitigating measures are considered:

1. **Safety zone**: close to the edge (5 m) of the structure no people are allowed.
2. **Bull nose**: a bull nose could deflect back up rushing water (see figures 9.2 and 9.3).

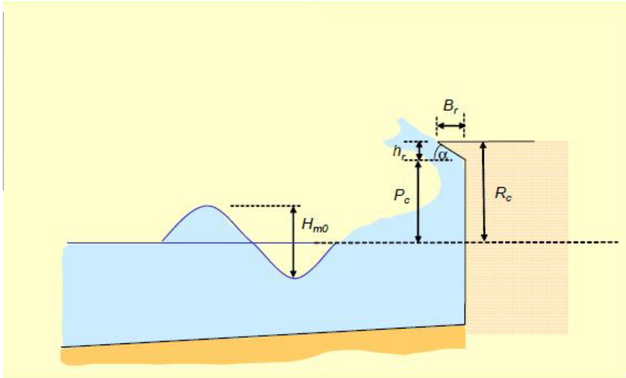

Figure 9.2: Schematisation of a bull nose. Source: Pullen et al. (2007)

Figure 9.3: Bull nose at the Donaghadee sea wall (UK). Source: www.moore-concrete.com

Summary of findings

Introducing a safe zone leads to a required crest height at NAP +7.9 m, a reduction of 1.5 m compared to the initial calculation. A bull nose leads to an enormous reduction of overtopping discharge for crest heights above NAP +6.4 m. Therefore, if a nose is used in a more detailed design, this value should be treated with caution. However, given the conceptual nature of this thesis, the value of NAP +6.4 m is used. Table 9.4 summarises the required crest heights for both structures. Use of a bull nose results in a relatively low crest height and is advised.

Table 9.4: Summary of findings for required crest level of inlet structure.

Option	Crest level [m NAP]
No measures	+9.4
Safety zone	+7.9
Bull nose	+6.4

The inner side of the barrier is subjected to lower water levels. Without going into further detail, the retaining height is assumed equal to the proven strength of the dikes along the Hollandsche IJssel in 2050; NAP +3.22 m (see table 7.1).

Crest levels structure:

- Nieuwe Maas side: NAP +6.4 m;
- Hollandsche IJssel side: NAP +3.22 m.

9.3.2 Bottom level structure

The bottom level of the structure is determined by the level of the culverts. Two types of culverts are distinguished, namely (1) culverts in which pumps can be installed at a certain point in time, hereafter called 'pumping structure,' and (2) culverts that operate only under free flow, the 'culvert structure'.

Pumping structure

At a certain moment in time, pumps are installed to ensure acceptable water levels. Large pumps are required to pump out the volumes necessary to maintain the tidal range. Large pumps require large diameters and a certain head above the pumps to function correctly. The capacity of the pumps under *normal* conditions was roughly defined (114 m^3/s) in chapter 7. Pumps will pump out water half of the time. The water level rises again by letting in water through the culvert structure the remainder of the time. The pump structure is only used to pump out water. The found value is used as a first estimation of both the required diameter and the depth. Consultation of an expert within RHDHV led to 6 pumps, each with an inlet with sides of 3.7 m. To ensure sufficient head, the bottom of the inlets should be at NAP -8.9 m (van Beveren, 2017). Including a margin of 0.5 m between each pump, the pump structure takes up roughly 26 m of the total 80 m available. The other 54 m can be used for the culverts structure.

It should be mentioned that pumps are also needed to ensure safety in *surge* conditions. To ensure sufficient storage capacity in case a gate fails, water levels may have to be lowered beforehand (e.g. by 1 m, see figure 7.11). The installed pumping capacity needed to ensure tidal action (*normal conditions*) can however be used in *surge conditions*. Before a storm, water is not let in anymore and the pumps will pump out water continuously (instead of ≈50% of the time in normal conditions). An in-depth failure analysis is required to determine whether extra capacity (on top of the capacity installed for normal conditions) is required for surge conditions or not and at which moment in time the pumps should be installed. This is beyond the scope of this thesis and it is assumed that the installed pumping capacity in normal conditions is sufficient in storm conditions as well.

Culvert structure

To allow optimal inflow, the culverts should be beneath the water level at the Nieuwe Maas at all times under normal conditions. The yearly minimum low water level directly after implementation is governing (water levels are expected to rise over time). Minimum Low Water Levels lie around NAP -0.70 m (2015) (Rijkswaterstaat, ndd). When including sea level rise (DRUK/RUST), this leads to NAP -0.58 m in 2050.

9.3.3 Size and configuration culverts

The length of the barrier is limited to the length of the current barrier (80 m). Furthermore, the size of each opening is limited to $\mu \cdot A$= 5 m^2, based on the assumption made in section 7.4. Rectangular culverts are chosen because, compared to round culverts, they make optimal use of the available length. In this thesis, square culverts are assumed. For small culverts, a μ of 0.7 is expected (see Appendix K). This leads to culverts with sides of 2.7 m.

Appendix J showed that 31 squared culverts of 2.7 x 2.7 m are required to ensure ≈90% of the original tidal range (table K.10, configuration 7). This means that about 100 m of length is required (assuming 0.5 m of concrete between each culvert), while only 54 m is available. To increase the amount of culverts, the culverts will be stacked as can be seen in figure 9.4. This type of stacking allows to still close the lower culverts from the top as will be discussed in the next paragraph (see e.g. figure 9.7). The bottom side of the lowest row of culverts is at NAP -6.48 m.

Pump structure:
Square culverts available for pumping have sides of 3.7 m. The bottom of each culvert lies at NAP - 9.0 m. The pump structure is only used to pump out water and is not used as inlet.

Culvert structure:
The 'normal' square culverts are placed in two layers. The culverts are relatively small (sides of 2.7 m) to limit inflow in case of failure (chapter 7). The bottom of the upper row culverts lies at NAP -3.28 m, the bottom of the lower row culverts lies at NAP -6.48 m.

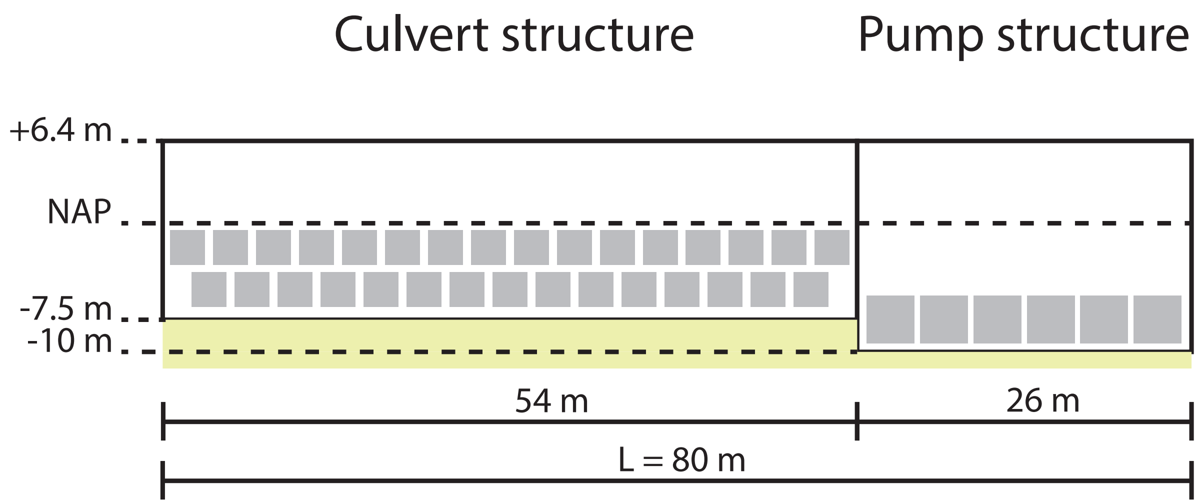

Figure 9.4: Front view of the structure

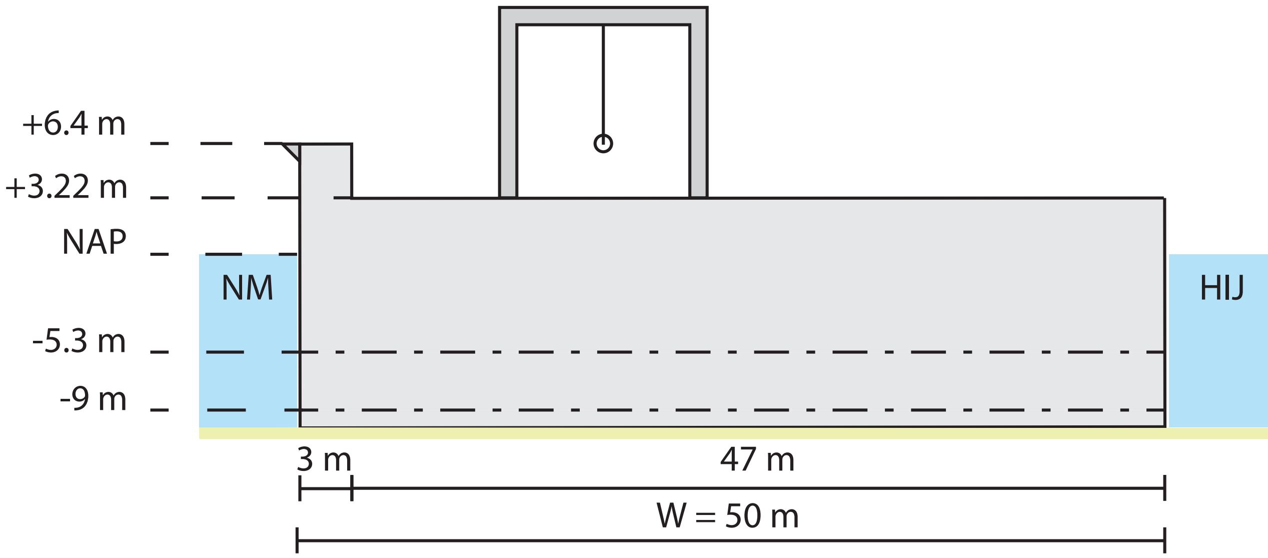

Figure 9.5: Side view of the structure (Pump structure side).

9.3.4 Closure mechanisms

Closing-off culverts to limit or stop inflow of water is paramount. Multiple closure mechanisms are therefore advised and are briefly discussed in this section.

Pivoted valves

Pivoted valves can be used to close off the culverts in regular use. They close by turning around the midpoint of the culvert/valve. Figure 9.6 shows what this valve looks like. An advantage is that no direct access to the culvert is required. Figure 9.7 shows a part of the culvert structure with the pivoted valves installed in the culvert section. The choice to stack the culverts in this way, allows for an hydraulic closing system without bends. The same type of valves can be installed in the pump section.

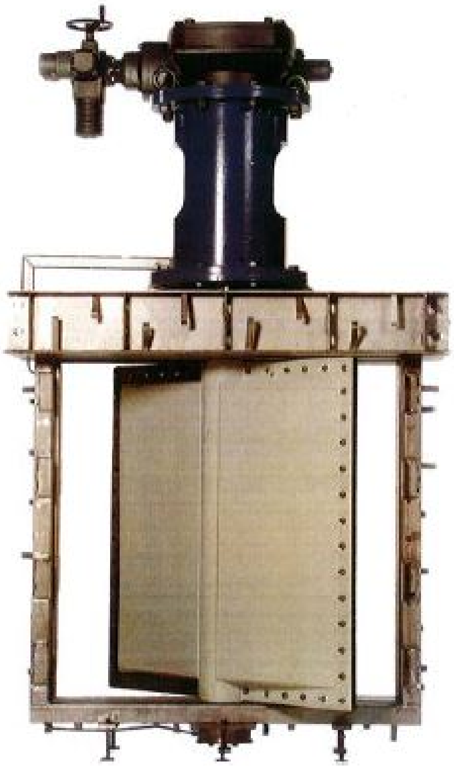

Figure 9.6: Pivoted valve Source: RHDHV

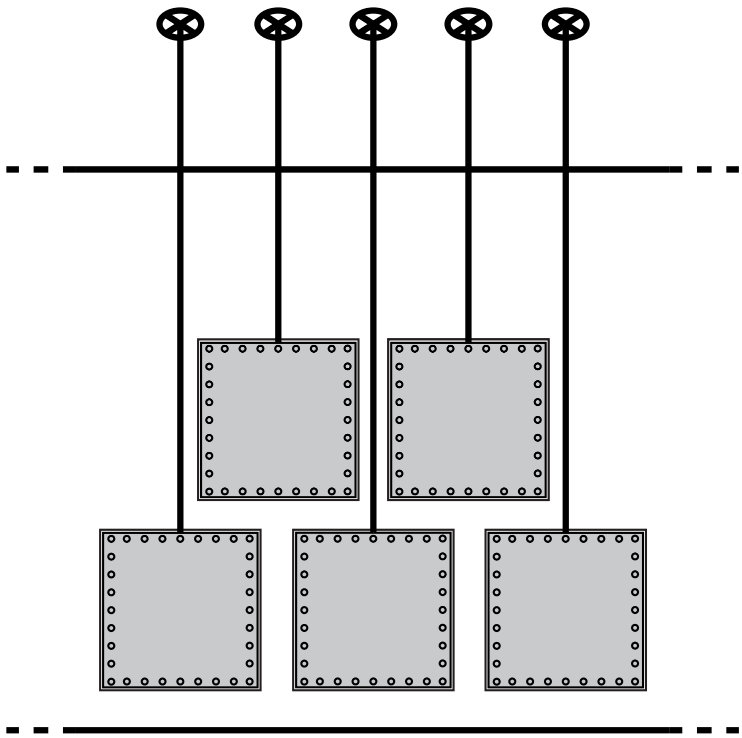

Figure 9.7: Pivoted valves with stacked culverts.

Stop locks & and hydraulic valves

In the culvert structure, a double set of stop locks (*schotbalken*) is placed at each side of the upper culverts. They can be used as final measure to close the culvert in case of failure and for maintenance purposes (e.g. removing debris from the culvert). Figure 9.8 shows the placement of stop locks. Due to the design choice not to place the culverts directly above each other, stop locks cannot be used for the lower row of culverts. The slots guiding the stop locks would be blocking inflow of water in normal conditions and therefore reduce the tidal range. Therefore, a double set of hydraulic valves is placed at each side of the lower culverts. Figure 9.9 shows a large version of the required hydraulic valves in Hamburg. The implemented stop locks and hydraulic valves at each end of the culvert are depicted in figure 9.10. In the pump structure, hydraulic valves or stop locks are required as emergency closure mechanism as well.

Figure 9.8: Placement of stop locks. Source: hanviskie.blogspot.nl

Figure 9.9: Large hydraulic valves in Hamburg. Source: http://www.bildarchiv-hamburg.de

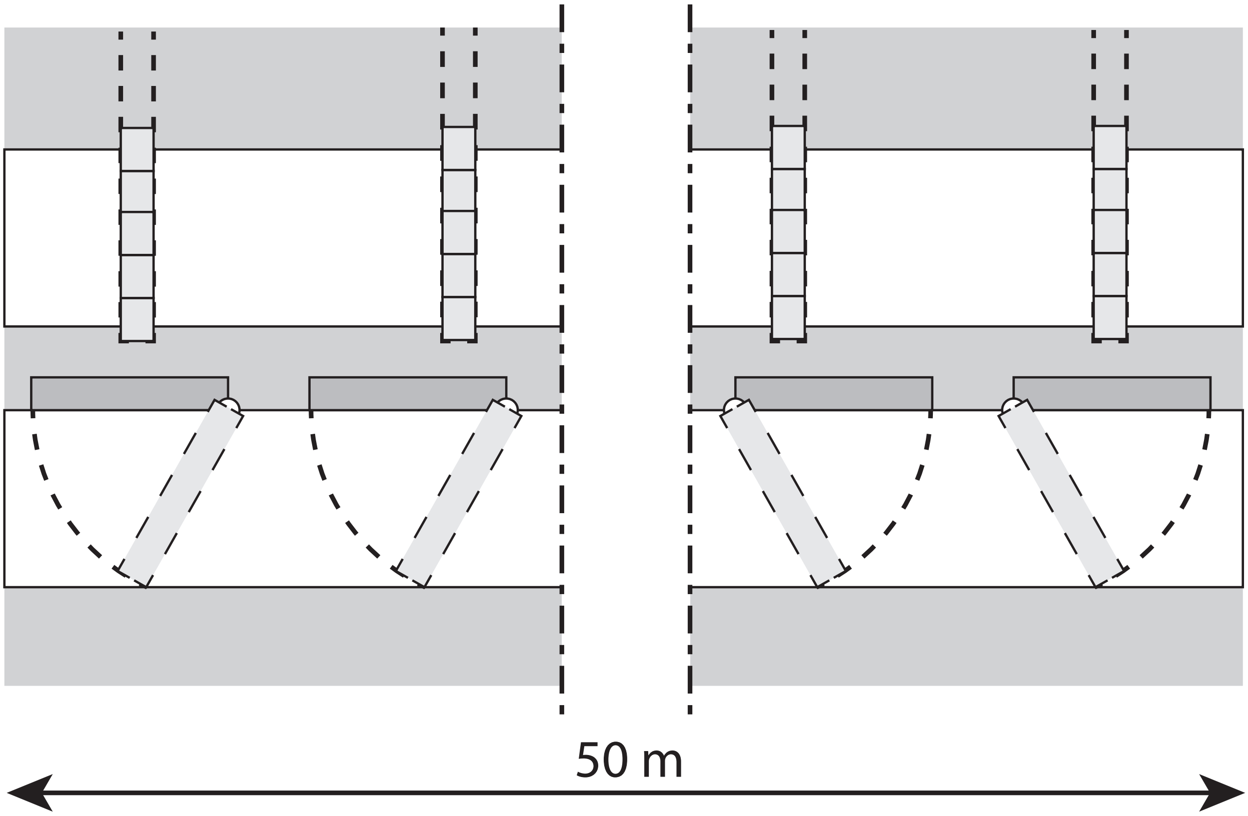

Figure 9.10: Emergency and maintenance closure mechanism culvert structure.

9.3.5 Other aspects

A crane is required to replace a pump or for maintenance purposes. The crane track can be seen in figure 9.5. A smaller crane is also required to insert and lift stop locks.

9.4 Stability and Piping

In the previous sections, the crest level at both sides of the barrier and the bottom level of the structure were derived. In this section the structure will be tested on stability and piping. A few more assumptions will be made to do so.

9.4.1 Further assumptions

To check the structure on stability and piping, a number of assumptions were made:

- The width of the structure is 50 m, see figure 9.5;
- The crest at the Nieuwe Maas side has a width of 3 m;
- The exact dimensions of the construction are unknown at this stage of design. Therefore, the structure is modelled as a hollow concrete box with a wall thickness of 2 m.

9.4.2 Stability analysis

For the inlet structure, a horizontal (1), a rotational (2), and a vertical (3) stability analysis (Vrijling et al., 2011, Ch.38) were carried out, taking into account both static (hydrostatic and dead weight) and dynamic (wave) loadings. To guarantee horizontal stability, the shallow foundation should be able to resist the horizontal force by a friction force on the structure. To guarantee rotational stability, the total of acting moments divided by the vertical forces should not be outside the core. Vertical stability is guaranteed when stress in the subsoil does not exceed maximum values. A safety factor of 1.5 was used. The results of the calculations are presented in table 9.5. It was found that the inlet structure is stable in all situations considered. Here it can be seen that the unity checks do not differ significantly for the different load situations. This is a because the structure itself is heavy compared to hydraulic loads that act on it.

Table 9.5: Results stability analyses (Unity Checks).

Structure	Situation	Horizontal stability	Rotational stability	Vertical stability
Culvert	1	30.6	72.5	2.3
	2	30.6	72.5	2.3
Pump	1	14.8	31.8	1.8
	2	14.8	31.8	1.8

9.4.3 Piping analysis

The parameters by Bligh and Lane were both used to check for piping. A safe seepage distance (Vrijling et al., 2011, p.222) is calculated by:

$$L \geq \gamma \cdot C \cdot \Delta H \quad with \quad \gamma = 1.5 \tag{9.3}$$

Table 9.6: Used values for piping analysis (Vrijling et al., 2011, p. 222).

Method	Symbol	Value	Unit
Bligh	C_B	12	[-]
	i_{max}	8.3	[%]
Lane	C_L	5	[-]
	i_{max}	20	[%]

Table 9.7: Results piping analysis.

Situation	L [m]	L_{req} [m]	Unity Check [-]
1	50	66	0.75
2	50	32	1.53

Table 9.7 shows that the length of the structure is considered insufficient as a safe piping distance (situation 1). Therefore, based on current assumptions, the seepage length has to be increased. Various options (Vrijling et al., 2011, p. 222) can be considered to make a pipe safe design:

- A sheet pile or slurry wall underneath the construction;
- Grout columns;
- Protective textile;
- Filter structure;
- Increase width structure.

So, various mitigating options are possible. Furthermore, the actual soil conditions are uncertain. Therefore, the design will not be tailored further towards a pipe safe design. A suggestion is made however: the river bed in front and behind the barrier probably needs scour protection. Therefore, a combination of a filter structure and a bed protection is a good way to tackle both piping and scour.

The structure can withstand the load situations mentioned in section 9.2.1. The structure scores sufficient on horizontal, vertical and rotational stability. The length of the barrier is insufficient for a safe piping design. Several mitigating measures are mentioned.

9.5 Construction method

In this section, a possible construction method is qualitatively described. Further research could result in the finding that another method is cheaper. A full comparison of construction methods is beyond the scope of this thesis. A loam layer is assumed to be present at NAP -23 m. This loam layer was used to create a watertight layer in the building pits for the current Algera barrier (Rijkswaterstaat, 1959, p.37). This layer will be used as watertight layer for the construction pit. An uplift calculation (Appendix S) showed that the downward pressure of soil and underwater concrete is sufficient to resist upward water pressure. The construction process is divided into the following steps:

1. **Dredge river bed**
   The clay layer (approximately until NAP -9 m, Appendix Q) is entirely removed. At the location where the culvert structure is realised, the bed is lowered to NAP - 9 m. The pump location is dredged until NAP -10 m, see figure 9.12.
2. **Install sheetpiles culvert structure**
   The building pit is bordered by sheetpiles (figure 9.13). To allow tidal action during construction, the culvert and pump structure are built separately.

3. **Underwater concrete**
   Underwater concrete is poured into the building pit, see figure 9.14. The concrete also serves as foundation slab.

4. **Dewater building pit**
   The building pit is dewatered (figure 9.14) and construction of the culvert structure can commence.

5. **Construct culvert structure**
   The culvert structure is constructed (figure 9.15). Valves are installed.

6. **Remove sheet piles**
   The sheetpiles are removed (figure 9.16). The valves are still closed.

7. **Place scour protection**
   Scour protection (assumed length of protection at each side: 50 m) is placed in front of the culvert structure, see figure 9.16. After this, the valves can be opened. The culvert structure is operational.

8. **Repeat step 2 - 6 for the pump structure**
   The sheetpiles used for the first building pit can be reused for construction of the second building pit. The construction steps for the pump structure are depicted in figures 9.17 through 9.20. The pump structure remains closed until pumps are needed. Pumps can be installed whenever pumping capacity is required.

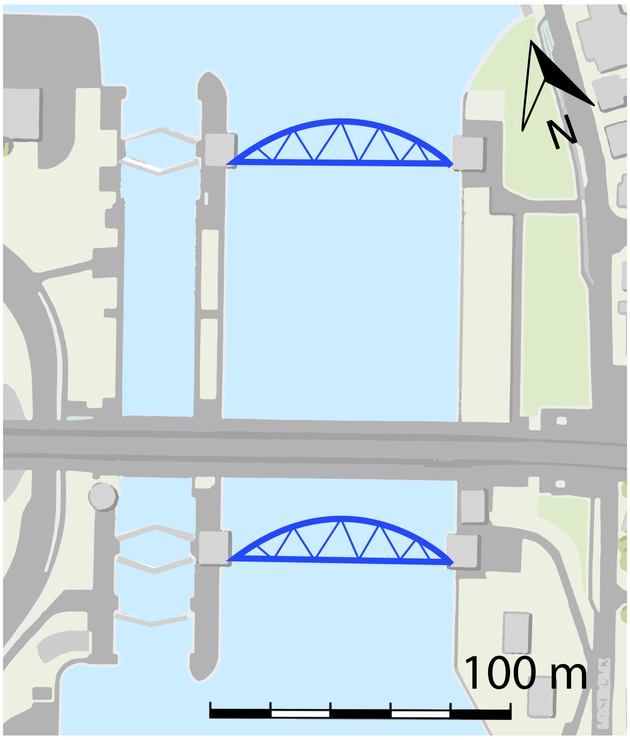

Figure 9.11: Initial situation.

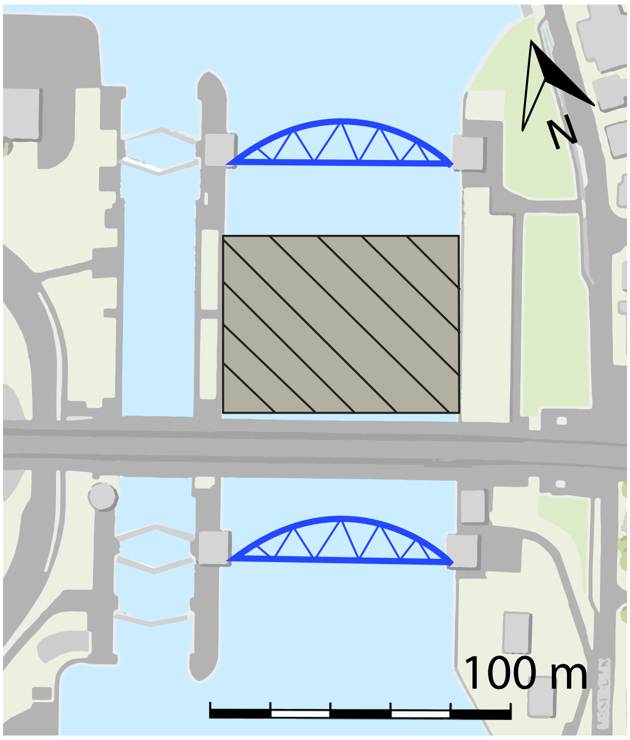

Figure 9.12: Step 1: Dredging river bed.

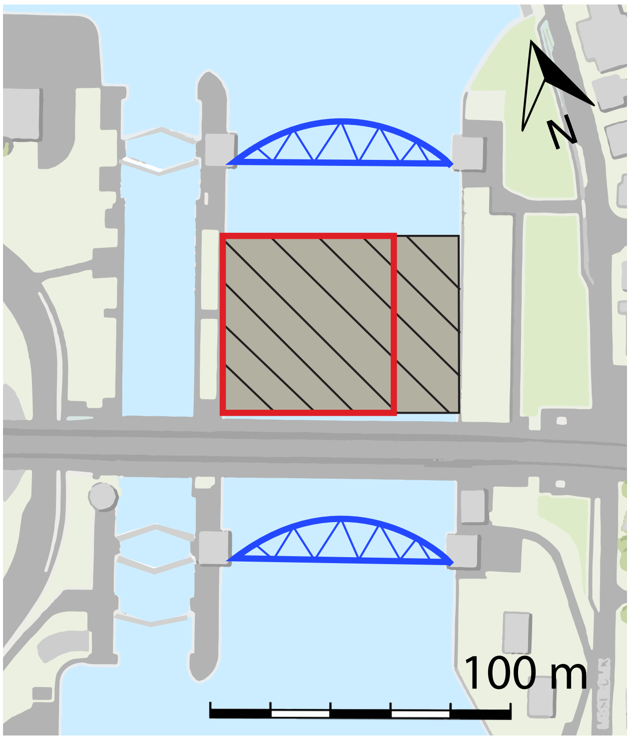

Figure 9.13: Step 2: Install sheetpiles culvert structure. Sheetpiling is depicted in red.

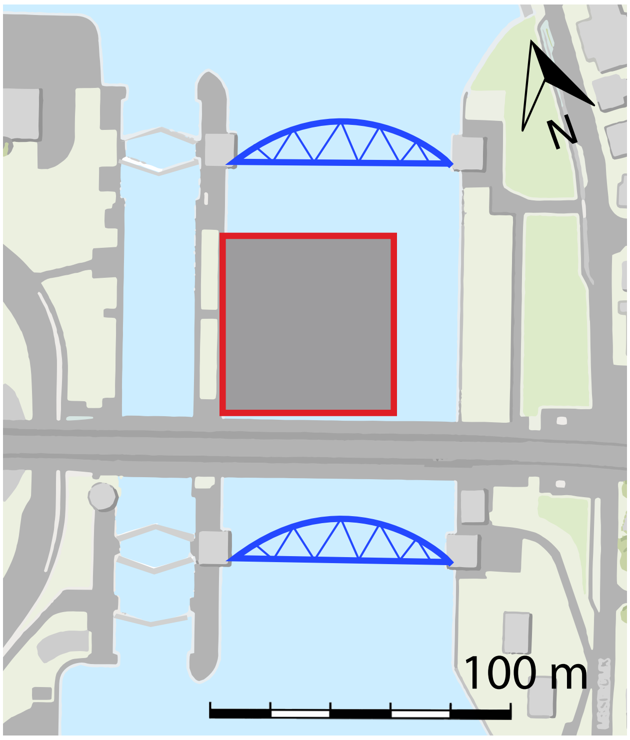

Figure 9.14: Step 3 & 4: Pour underwater concrete (dark grey) and dewater building pit.

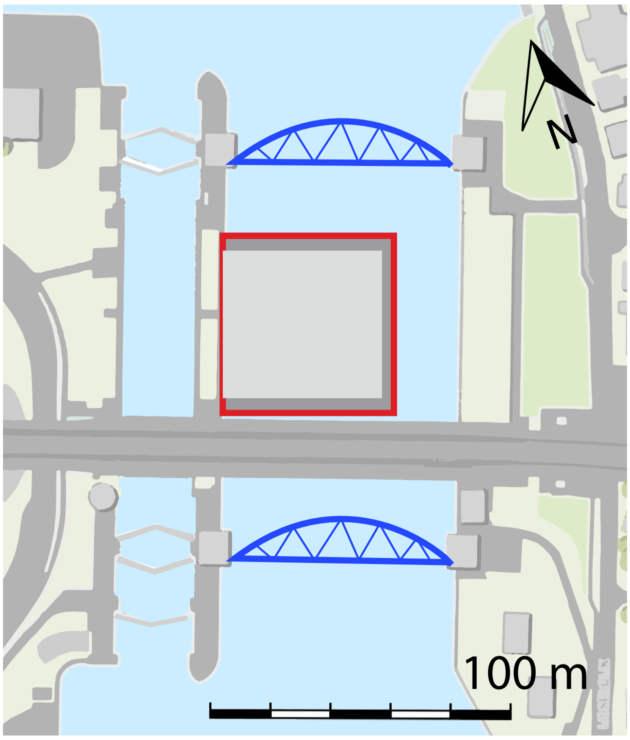

Figure 9.15: Step 5: Construct culvert structure (light grey).

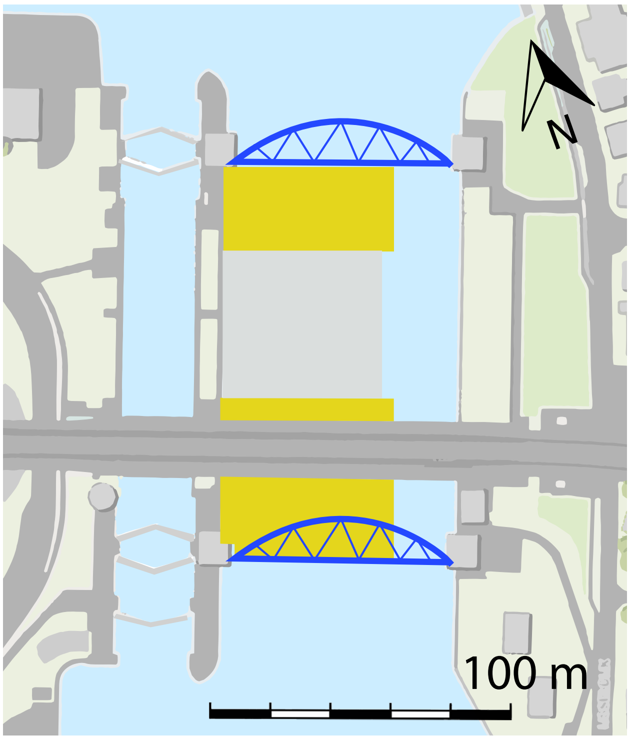

Figure 9.16: Step 6 & 7: Remove sheetpiles and place scour protection (yellow).

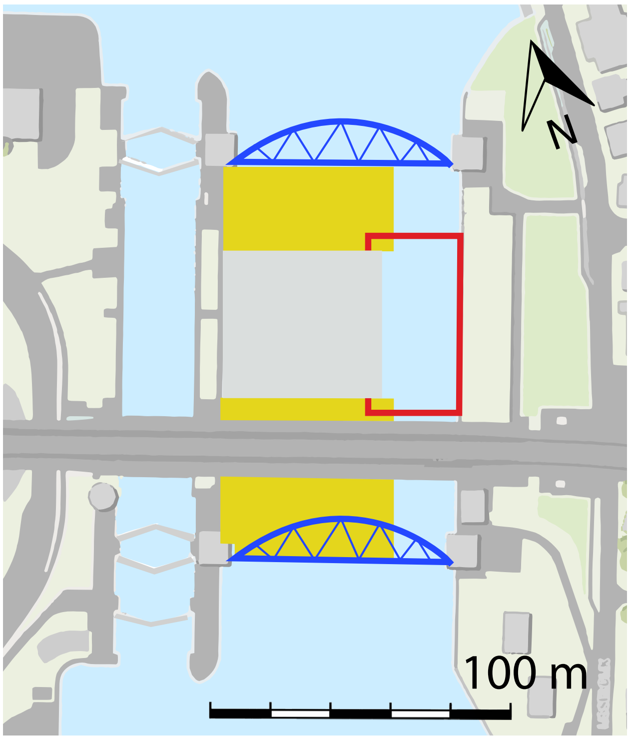

Figure 9.17: Step 8: Repeat process for pump structure (2.1).

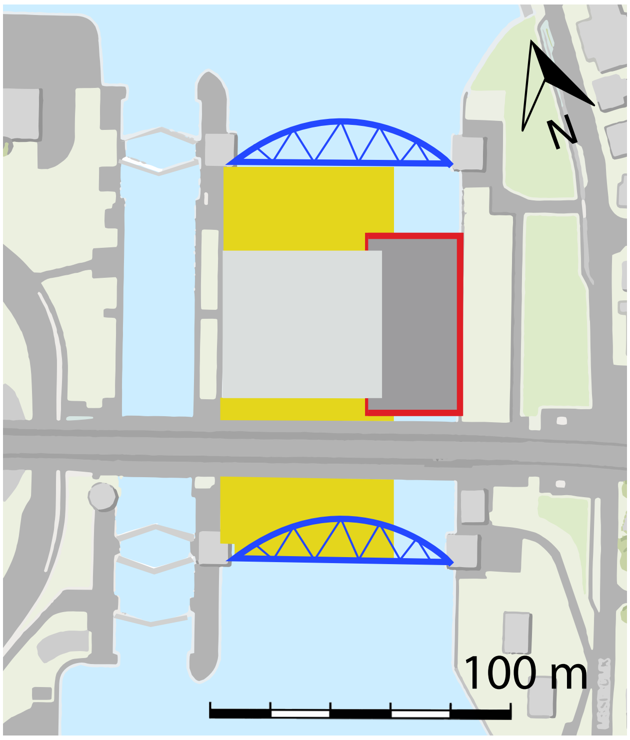

Figure 9.18: Step 3.1 & 4.1: Pour underwater concrete (dark grey) and dewater building pit.

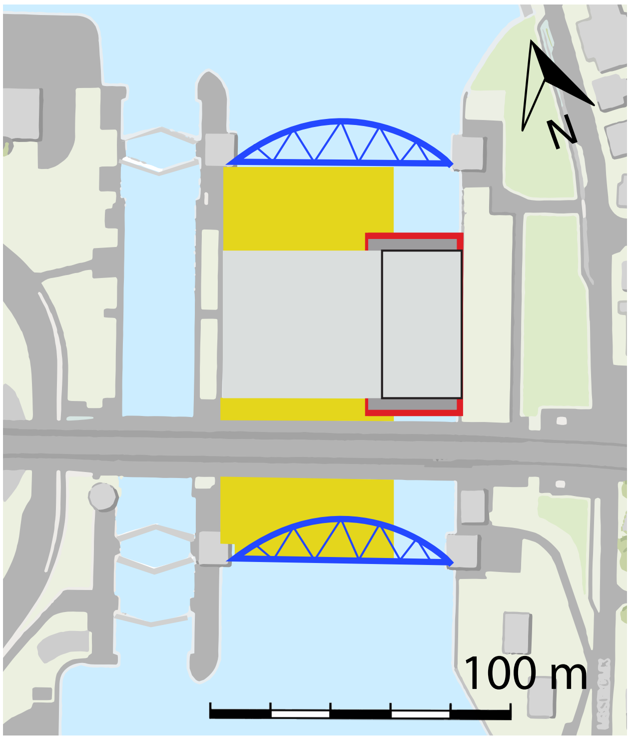

Figure 9.19: Step 5.1: Construct culvert structure (light grey, with black stroking).

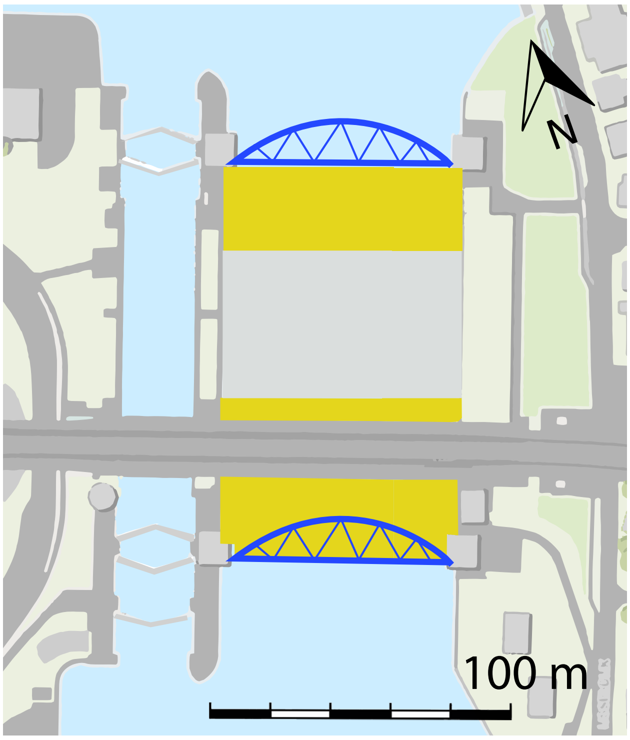

Figure 9.20: Step 6.1 & 7.1: Remove sheetpiles and place scour protection (yellow).

9.6 Summary

In this chapter, the main dimensions of the structure were defined. The structure is horizontally, vertically and rotationally stable. The length of the structure is insufficient to prevent piping, however various mitigating measures are possible. Furthermore, examples were provided how to close the culverts with valves and how the barrier can be constructed. A visualisation of the barrier is depicted in figures 9.21 & 9.22.

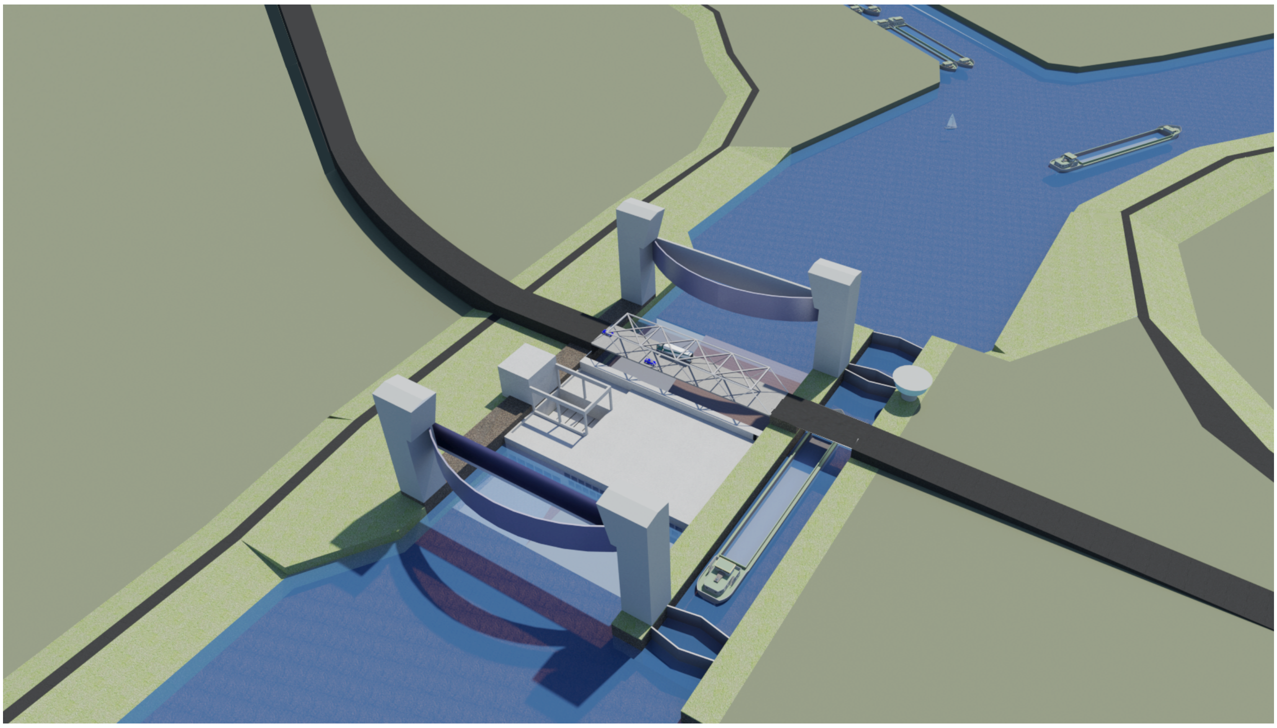

Figure 9.21: Visualisation of the new barrier complex.

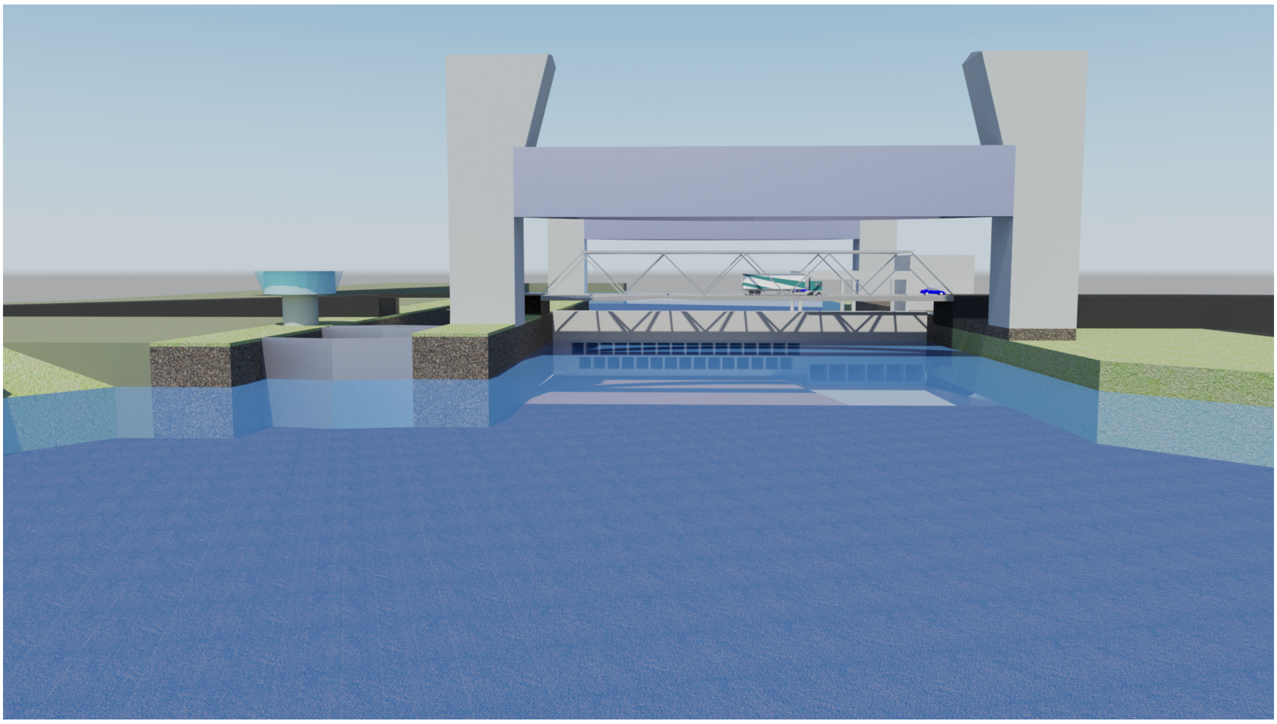

Figure 9.22: Visualisation of the new barrier complex.

10

Strategy overview

10.1 Introduction

In the previous chapter, the design of an eco-friendly barrier was elaborated upon. This chapter will give an indication of the cost of construction itself (section 10.2) and value of the proposed strategy to nature (section 10.3). Furthermore, a comparison is made between the proposed strategy (the eco-friendly barrier) and reference strategy as introduced in chapter 5. A distinction is made between three points in time in which the barrier could be implemented: 2030, 2050 and 2070. In section 10.4, the investment cost are compared.

Summary reference and proposed strategy
As a reminder, the strategies introduced in chapter 5 are summarised in this text box.

Proposed strategy

- Realisation of an ‘eco-friendly’ barrier (multiple gate solution);
- Until 2050: Improve dikes when necessary;
- 2017 - 2030: Improve current barrier to failure probability 1:200;
- 2050: Realisation new barrier;
- 2050 - 2150: No further dike improvements along Hollandsche IJssel;
- Ships have to pass the barrier through a lock;
- Water level in the Hollandsche IJssel can be lowered.

Reference strategy

- Continuous dike improvements;
- 2015 - 2030: Improve current barrier to failure probability 1:200;
- 2058: Replace current barrier with a 1:1,000 barrier (single gate solution);
- Ships can pass the barrier ‘lock-free’;
- Water levels on Hollandsche IJssel are not lowered.

10.2 Cost of construction

In this section the investment cost (including VAT) of the proposed design/strategy are presented. A full analysis can be found in Appendix U. The cost are split into five aspects, namely:

1. Barrier;
2. Building pit;
3. Valves;
4. Pumps;
5. Intertidal areas.

Pumps have to be installed at a certain point in time. In chapter 7, the moment in time when pumps are required to allow tidal action was derived (*normal conditions*). For *storm conditions*, a theoretical end of lifetime without pumps was derived. However, in order to increase safety against flooding and reliability of the barrier system, it is very well possible that pumps are needed before theoretical end of lifetime. As a reliability analysis of the barrier is not part of this thesis, this point in time was not defined. It is therefore not certain at which point in time pumps need to be installed. A safe/conservative assumption for a cost calculation is to assume that the pumps are installed when the barrier is built (2050). Later installation of pumps would reduce the present value cost. Furthermore, by later installation, maintenance cost on pumps could be saved. Therefore, with regard to the pumps, one could either (1) place the pumps directly or (2) reserve space and place the pumps when they are required to maintain the tidal range. Further research on whether the pumps are required in storm conditions and a more thorough cost analysis (including maintenance) could give more insight on which option is preferred.

Table 10.1 provides an overview of the non-discounted investment cost. The cost of realisation of intertidal areas, as introduced in section 8.6, have been incorporated as well. From table 10.1 it follows that the creation of intertidal areas is by far the largest cost (roughly three quarters). However, the intertidal areas may be necessary to limit the impact of the reduction on tidal range on the ecological value (section 10.3).

Besides the cost of the intertidal areas, the valves and pumps make up the largest part of construction cost of the barrier itself (see also Appendix U). The choice of configuration of the culverts (depicted in figure 9.7) is one of the reasons for the high cost of the valves. A different configuration could lead to lower cost, but has not been researched.

Table 10.1: Overview non-discounted investment cost, including VAT (price level 2017) [M€].

	Cost [M€]
Barrier	10.0
Building pit	21.0
Valves	25.3
Pumps	32.5
Subtotal (barrier)	*88.8*
Intertidal areas	229.7
Total*	**318.5**

**Excluding maintenance cost*

10.2.1 Note on the cost and realisation of intertidal areas

The cost to create the intertidal areas form a large part of the total construction cost and are relatively high compared to other ecological restoration projects in The Netherlands, such as in the Hedwigepolder (80 M€ (ANP, 2016)), the Marker Wadden (75 M€ (Natuurmonumenten, nd)) and the Sand Engine (70 M€ (Rijkswaterstaat and Provincie Zuid Holland, nd)). To reduce cost, the river could be allowed to deposit sediment in the projected areas naturally instead of the assumed realisation in one go. Because an analysis on sediment deposits is not part of this thesis, this potential cost reduction is however not quantified. Furthermore, in this thesis, a complete implementation of the proposed intertidal areas (Appendix P) is assumed. A more incremental approach, where a number of new intertidal areas are realised as a pilot to see the effect ('wait and see') could also be an option. This could save cost as (1) the implementation is spread over multiple years and (2) not all proposed intertidal areas are necessarily implemented.

The realisation of intertidal areas that has been proposed, has only been elaborated upon on a conceptual level (section 8.6). If it is decided to realise new intertidal areas it is advised to work out a plan in higher detail. Questions that one can think of that should be answered before the areas are realised include:

- Are the intertidal areas created in one go or incrementally?
- Are the intertidal areas realised by men or could one shape the conditions in such way that nature delivers sediment/sand to the projected areas?
- Are breakwaters required to keep the sediment in place or can the areas be created with a natural slope?
- Are intertidal channels required? What should be the dimensions of the channels?
- Are all the proposed areas suitable, or is shipping hampered too much at some locations?

10.3 Natural value

The reduction in tidal range caused by the new structure and possible fish migration were used to assess ecological impact (further elaborated upon in Appendix T).

Tidal range

A one-dimensional model was used to assess the impact of the structure on the tidal range. Appendix K shows how this was done. The structure reduces the tidal range to approximately 90% of the original value. This could however be compensated by creation of extra intertidal areas. An overview of the possible locations is provided in Appendix P. A possible way to score and compare the original situation with the new situation, is to multiply the available intertidal area with the Habitat Suitability Index (HSI) for the Scirpus triqueter at every location, see equation 10.1 and Appendix T. The results are presented in table 10.2. It should be noted however that the HSI is only an index to indicate the suitability to a certain species. Whether an indicator species (in this case the Scirpus triqueter) will actually be present in an area, is dependent on more factors than a high HSI (in this case a wide tidal range). Other factors that could indicate a healthy freshwater tidal river system are water depth, natural dynamics, available sediment, contaminants etc. (elaborated in chapter 4).

$$Score = A_{total} \cdot HSI \tag{10.1}$$

Table 10.2: Summary of current and future intertidal flats.

Situation	Total area	Avg. Tidal range	Habitat Suitability Index	Score
Current	249,974	1.54	0.90	224,741
New	691,091	1.35	0.58	381,998

Fish migration

For fish migration, culverts should have a width above 1.0 m (safe side, see chapter 4). Water depths should be greater than 0.5 m. The projected culverts have dimensions well above the minimum values: the square culverts have sides of 2.7 m. It is therefore concluded that the cross-sections are large enough for fish to migrate. Average velocities in the culverts were obtained through the SOBEK schematisation (Appendix K). The results for begin September 2016 are shown in figure 10.1. Fish can migrate with flow velocities smaller than 0.8 m/s, see chapter 4. It was found that values are below 0.8 m/s about 67% of the time. Fish are therefore able to migrate, although not continuously. The effects of this ‘migration window’ could be a subject for further research.

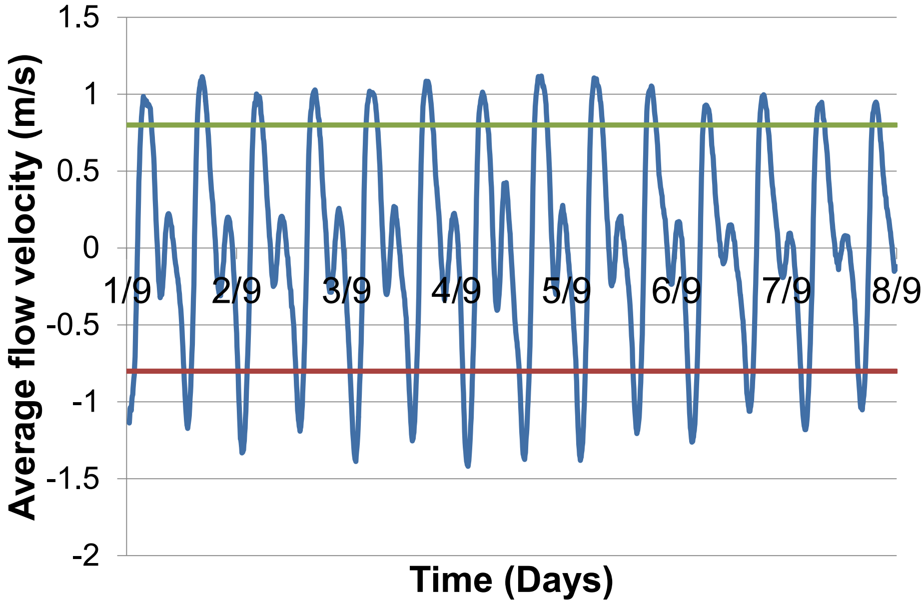

Figure 10.1: Average flow velocity through the barrier (blue) and advised flow velocities for fish migration (red and green).

Summary

A small reduction in tidal range has a large impact on the Habitat Suitability Index of the Scirpus triqueter (see also figure 4.6). Although the tidal range is reduced, the score has improved due to the increased amount of intertidal area. Furthermore, fish will be able to migrate, but not continuously (67% of the time). It is concluded that the impact of the barrier on ecological value is acceptable.

10.4 Sensitivity to year of implementation

Up to this point, it was assumed that the barrier would be realised in 2050. In this section the present value of implementation at different moments in time will be compared. The predictions made in chapter 6 for shipping and dike reinforcement will be used to compare implementation in 2030, 2050 and 2070 and the reference strategy (see chapter 5). Only aspects at which the reference strategy and proposed strategy differ are considered here.

10.4.1 Construction cost

This section describes the cost of construction (barrier + intertidal areas) for each of the variants. As future investments are discounted, later implementation of the new structure will lead to lower present value cost. The cost of the construction mentioned in section 10.2 are used. It should be noted that maintenance cost have not been taken into consideration. The results are presented in table 10.3.

Table 10.3: Present value (investment cost, incl. VAT) of constructing a new barrier, including intertidal areas [M€].

	Reference	2030	2050	2070
RUST/WARM	n.a.	184.8	75.6	31.6
DRUK/STOOM	n.a.	218.9	123.0	69.1

Improvement new structure

If the construction is built in 2030, it will reach the end of its designed life time in 2130. At that moment in time, improvements will be required to let the structure function correctly until 2150. The improvements are estimated to cost 25% of the initial investment cost (Present value, table 10.4). The cost are discounted as a future cash flow.

Table 10.4: Present value (investment cost, incl. VAT) of extending the lifetime of the new barrier [M€].

	Reference	2030	2050	2070
RUST/WARM	n.a.	1.4	n.a.	n.a.
DRUK/STOOM	n.a.	8.8	n.a.	n.a.

New structure reference situation

In the reference strategy, a new barrier is proposed in 2070 with a probability of failure of 1:1,000. A solution that keeps the HIJ open for shipping is expected and assumed. The expected cost of such a solution are based on the construction cost of the current Algera barrier (40 M*f*). In 2016, the same barrier would cost 136 M€ (Mooyaart and Jonkman, 2017). The cost of the barrier are extrapolated to 2070 and discounted as future cash flow. The present value in the different scenarios is presented in table 10.5.

Table 10.5: Present value of investment cost new construction reference scenario (incl. VAT) [M€].

	Reference	2030	2050	2070
RUST/WARM	12.9	n.a.	n.a.	n.a.
STOOM/DRUK	28.6	n.a.	n.a.	n.a.

Summary present value of construction

The Present value of the investment cost presented in this section are summarised in table 10.6.

Table 10.6: Summary present value investment cost barrier and intertidal areas (incl. VAT) [M€].

	Reference	2030	2050	2070
RUST/WARM	12.9	186.2	75.6	31.6
STOOM/DRUK	28.6	217.7	123.0	69.1

10.4.2 Dike reinforcements cost

Dike reinforcements are an important cost driver. Later implementation of the barrier results in more dike reinforcements that need to be carried out (a certain level of safety against flooding should be maintained). Table 10.7 shows the present value (2017) in case of each of the scenarios. The investment cost for 2050 are presented in chapter 6 (table 6.10). The factors used to make this translation are presented in Appendix U. Similar calculations were carried out for the 2030 and 2070 scenario.

Table 10.7: Present value investment cost (incl. VAT) [M€].

	Reference	2030	2050	2070
DRUK	1,261.7	366.2	569.9	1,129.4
STOOM	1,335.6	366.2	569.9	1,305.7
RUST	879.8	345.2	511.0	849.9
WARM	963.4	345.2	511.0	956.7

10.4.3 Waiting cost shipping

Later implementation will lead to less harm to shipping. The present value (2017) of the societal cost are tabulated in table 10.8. The present value for implementation in 2050 are based on the cost presented in chapter 6 (figures 6.4 and 6.6). Similar calculations were carried out for 2030 and 2070.

Table 10.8: Present value shipping cost [M€].

	Reference	2030	2050	2070
DRUK	40.0	105.3	64.6	38.0
STOOM	40.0	89.2	50.5	26.9
RUST/WARM	40.0	31.7 - 32.6	10.8 - 11.7	3.2 - 3.9

10.4.4 Summary

Table 10.9 summarises the net present values of the investment cost (incl. VAT) for the reference strategy and implementation of the eco-friendly barrier at different points in time. The presented bandwidth is a result of the different Delta scenarios used. The table shows that the proposed strategy is cheaper than the reference strategy when implemented in 2030 or 2050 or just as expensive when implemented in 2070. The main reason for this is that expensive dike reinforcements can be averted. An earlier implementation of the barrier leads to lower cost. Therefore, early implementation is advised.

Table 10.9: Investment cost comparison (Net Present value) [M€].

	Reference	2030	2050	2070
Barrier*	13-29	55 - 61	21 -34	8 - 19
Intertidal areas*	-	130 - 158	55 - 89	22 - 50
Dike reinforcements	880 - 1,336	345 - 366	511 - 570	850 - 1,307
Shipping	40	32 - 105	11 - 65	3 - 38
Total	933 - 1,405	563 - 689	598 - 758	885 - 1,413

** Maintenance cost were not taken into consideration*

10.5 Conclusion

In this chapter, the cost of the proposed barrier system have been discussed. Construction of the barrier itself costs approximately 89 million Euro, the intertidal areas cost approximately 230 million Euro (price level 2017). Depending on the Delta scenario, the cost of implementation of the barrier and intertidal areas in 2050 has a present value 76-123 million Euro. The barrier reduces the tidal range by approximately 10%, however the available intertidal area has almost tripled. Fish will still be able to migrate the majority of the time. The design is considered acceptable from an ecological point of view. Furthermore, a brief sensitivity analysis has been carried out. It was found that the proposed barrier including the creation of new intertidal areas is cheaper than the reference strategy in two cases considered (implementation in either 2030 or 2050) or just as expensive as the reference strategy (implementation in 2070). Dike reinforcements are a large cost driver. Early realisation leads to aversion of dike reinforcement cost. Figure 10.2 shows the present value of the cost for both the reference and implementation of the proposed scenario in 2030, 2050 and 2070.

The creation of new intertidal areas make up a large part (±75%) of the cost. The areas are however not required from a flood risk perspective. Furthermore, only the cost of the intertidal areas were presented. Besides the compensation to *nature*, various benefits to *mankind* cling to extra nature, such as possible increased value of neighbouring property, a landscape that can be enjoyed etc. The benefits (e.g. by ecosystem services approach) were not quantified as part of this thesis. It is up to policy-makers to decide whether the benefits of realisation of intertidal areas weigh up to the costs.

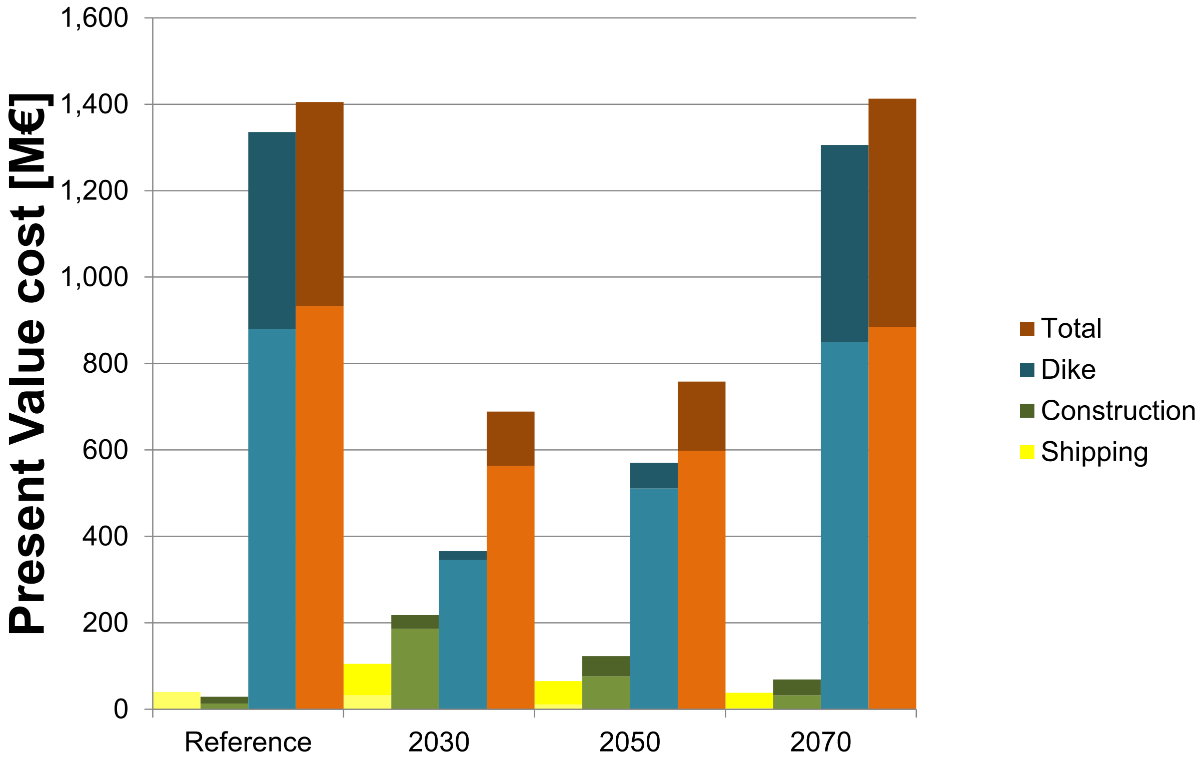

Figure 10.2: Present value of the cost in the reference scenario, and implementation of the proposed strategy in 2030, 2050 and 2070.

11 Conclusions

This chapter wraps up the conclusions that can be drawn from the results presented in this thesis. Besides the conclusions presented in this chapter, the reader is advised to read the more elaborate summaries and conclusions presented at the end of each individual chapters. By means of the case study on the Hollandsche IJssel, an effort was made to answer the question:

What is an appropriate conceptual design of a storm surge barrier for the Hollandsche IJssel that takes into account ecological aspects?

One of the objectives of this thesis was to translate ecological value into boundary conditions for the design of the proposed barrier. Various boundary conditions were distilled based on the uniqueness of the fresh water system present in this case study. Focus was placed on preservation of the tidal range and the ability for fish to migrate. An ecological framework was introduced and used to conceptually design the new barrier. Furthermore, ecological aspects were introduced that are deemed to be important for every engineer who wants to fully take into account natural value. The review of the current state of 'eco-software', legislation and societal forces presented in chapter 4 could serve as a building block to be used by other engineers who want to incorporate natural value into their design considerations.

It was found that a single gate barrier in combination with no dike reinforcements after realisation of this barrier does not lead to sufficient safety levels. The dikes are subject to subsidence and cannot withstand the expected water levels behind the barrier. A multiple gate barrier is therefore required. Initial calculations suggested individual openings of approximately 5 m^2. This finding results in a barrier in which ships cannot pass freely; the openings are too small for ships to pass. The current Algera lock, located next to the Algera barrier, has however sufficient capacity to accommodate the expected growth in ships passing the barrier complex.

An appropriate conceptual design of a surge barrier in the Hollandsche IJssel from an ecological point of view (main research question) was presented in chapter 9. The main reason for calling the design presented 'appropriate,' is the preservation of roughly 90% of the tidal range in the Hollandsche IJssel. This is achieved by realisation of 31 culverts with a total area of 226 m^2. Furthermore, fish are able to migrate through the barrier the majority of the time. To ensure sufficient tidal action, pumps are required between 2105 and 2173 (depending on the severity of climate change). Pumps are possibly needed earlier to create a safe situation in storm conditions. However the point in time in which pumps are required for storm conditions was only briefly researched and is a topic for further research (chapter 12). A barrier was chosen in which pumps can be incorporated easily.

From the predictions made in chapter 6, it is concluded that the societal cost of shipping delay are lower than the delay caused by the insufficient road connection. Improving the road connection could not be integrated in the proposed strategy: it is not the bridge connection that is most problematic, but the insufficient capacity of the junction in Krimpen a/d IJssel. However, compared to the cost of dike reinforcements, the societal cost of shipping delay and congestion are small.

It was found that the proposed barrier including the creation of new intertidal areas is cheaper than the reference strategy in two cases considered (implementation in either 2030 or 2050) or just as expensive as the reference strategy (implementation in 2070). Earlier implementation leads to lower present value cost, mainly because expensive dike reinforcements are averted. The dike reinforcements are expensive due to buildings built close to, or directly on the dike. Saving this property requires expensive dike reinforcements including sheet piles and slurry walls. While the reference strategy has investment costs roughly ranging between 900 M€ and 1,400 M€, the investment cost of implementation of the proposed strategy in 2030 roughly ranges between 500 M€ and 700 M€ (Net Present Value, including VAT).

12 Further research

The results presented in this thesis are subject to a certain scope. This chapter provides directions for further research.

12.1 Design of the barrier and probability of failure

The design of the barrier was focused on ensuring a tidal range behind the barrier. Safety and reliability of the barrier were taken into account qualitatively and with relatively simple analyses. In order to consider an eco-friendly closure of the river system as a legitimate alternative, a thorough analysis of the reliability of the barrier is however paramount. It is therefore suggested to further research the probability of failure of the proposed design and analyse the results if a single culvert or several culverts do not close. Such an analysis could also shed light on whether the assumed maximum size of each of the culverts can be considered legitimate. In current software (HYDRA-BS), only failure of a single gate solution in the Hollandsche IJssel is incorporated. The effect of a multiple gate solution on the water levels on the Hollandsche IJssel is yet unknown, but expected to be positive.

Besides the reliability of barrier, other topics that can be researched in further detail are:

- The new barrier could trap sediment in the river, leading to siltation. The effect of the barrier on erosion/siltation should be researched with a more detailed (numerical) model.
- The barrier has insufficient length to prevent piping. Several mitigating measures were suggested in this research. A more detailed design should be safe against failure due to piping.
- The barrier was designed on a conceptual level. Assumptions were made on e.g. the percentage of reinforcement steel and the construction method. A more thorough analysis could shed new light on the assumptions made. It is however not expected that the general design is changed significantly.
- The water level rise due to polders discharging into the Hollandsche IJssel was assumed constant and non-negotiable. The loads on the dikes could be decreased (and thus their lifetime extended) by reducing the polder discharge. Research into reducing polder discharge e.g. in storm conditions is therefore an interesting topic to look at.

12.1.1 Pumps

To ensure sufficient tidal action in the river, pumps are required. The required pumping capacity to ensure this tidal fluctuation was calculated in this study. The calculations were based on a simplified bathimetry and therefore volume of the river. Taking into account the actual volume including the reduced cross-section due to the creation of extra intertidal areas, could alter the required pumping capacity. In this research, the installed pumping capacity was assumed to be sufficient in storm conditions as well. It is suggested to further research the required pumping capacity in storm conditions.

At the moment in time in which pumps are required to ensure sufficient tidal action, the culverts will be used to let the water into the Hollandsche IJssel. Water is pumped out by the pumps. Energy could be generated by installation of turbines in the culverts which in turn can be used to (partly) power the pumps (see appendix H). Further research could elaborate on whether installation of turbines is economically viable and what their influence is on fish migration.

12.2 Ecology

In this research, ecological value was translated into boundary conditions based on the unique aspects of a freshwater tidal river system. A wider scope, including e.g. less unique species, could provide an even more complete view of the impact the proposed strategy has on ecological value.

Furthermore, it was assumed that by maintaining tidal action, nutrient supply and the sediment balance will remain intact. Detailed modelling of sediment and nutrients was beyond the scope of this thesis. The effect of the barrier and the river narrowing on erosion/siltation and nutrient supply could be researched with a more detailed (numerical) model.

Next to researching nutrient and sediment supply, the presented findings on the required tidal opening (Appendix K) could be researched in further detail as well. First of all, the used loss coefficients could be researched further. Secondly, the intertidal areas have been introduced in a basic rectangular shape. A different shape of the intertidal areas and possibly different locations of the areas could (positively) influence the tidal range in the river. Thirdly, a basic assumption on the maximum gap area was made. Further research regarding the probability of failure of the barrier (section 12.1) could shed light on whether this assumption was right or that the gap area should be made smaller/larger. Fourthly, the reason behind the under prediction of the tidal range in the SOBEK model could be researched. An unbiased model will produce more accurate results.

Besides the subjects mentioned, several other topics could be looked into:

- Design of a bed protection in front of and behind the barrier to protect the barrier itself and washing out of contaminants.
- Further elaboration on the proposed intertidal areas. This includes, among other aspects (section 10.2.1), a decision on whether, and if so, which intertidal areas should be realised and if the areas should be realised incrementally or in one go. One could research what the best design (combination of tidal channels and intertidal flats) is for different types of species.
- Fish are not able to migrate the whole time. The effects of this ‘migration window’ could be a subject for further research.

12.3 Societal cost and benefits

The development of the value of property over time along the river has not been fully taken into account. One can imagine that raising dikes does not have a beneficial effect on the value of property. Furthermore, the societal cost of hindrance because of dike strengthening has not been taken into account either. Besides direct hindrance, dike reinforcement leads to a situation in which traffic is temporarily not allowed. Including the societal cost of dike reinforcement is a relevant topic for further research.

Only the cost of the intertidal areas were presented, while, besides the compensation to *nature*, various benefits to *mankind* cling to extra nature. Examples are the possible increased value of neighbouring property, a landscape that can be enjoyed, more recreation along the river etc. The benefits (e.g. by eco system services approach) were not quantified as part of this thesis. Further research could quantify the instrumental value of the creation of the proposed intertidal areas.

In this study assumptions were made on the waiting time for a lock (1 hour) and the capacity of the Algera lock (Appendix L). Furthermore, the available data on ship passages was relatively general. It is suggested to perform a more detailed analysis of ship passages, the capacity of the Algera lock and the expected delay.

12.4 Implementation

It was found that implementation of the proposed strategy as soon as possible is cheapest because dike reinforcements can be averted. If the proposed strategy is chosen, it is advised to research whether and to what extend already projected reinforcements are required. Sooner implementation of the proposed barrier could lead to adjustment or even cancellation of (a part of) the intended dike reinforcement programs: the dikes need to withstand the design water levels for a shorter period of time. If it is decided to realise (part of) the proposed intertidal areas, a solid maintenance plan is required in advance that is specific about what actions are needed at which moment in time and who is responsible for it. Furthermore, a risk management plan is required which indicates the possible risks of the new intertidal areas and what actions should be taken. It is advised to consult and involve the relevant stakeholders described in chapter 3 when writing further implementation plans.

12.5 Reflection and applicability in other cases

In this research, implementation of an eco-friendly storm surge barrier at the Hollandsche IJssel was researched. Besides implementation at the Hollandsche IJssel, case studies could be performed at other locations. In Appendices A and B a number of promising locations are provided. In the Hamburg region for example, various river branches were closed off and canalised in the past. Reopening such a river stretch and including a barrier similar to the one proposed in the Hollandsche IJssel case, could introduce an intertidal climate, while safety against flooding is maintained at similar levels.

It is more difficult however to implement the proposed design in cases where river discharge is present. The barrier reduces the discharge capacity of the river, which could lead to problematic situations due to increased water levels upstream. Furthermore, a waterway with discharge is likely to be longer than the tidal part of the Hollandsche IJssel (17 km). Due to their length, water levels cannot be lowered easily over the entire river.

The concept presented in this thesis is therefore considered not suitable for cases in which river systems have significant discharges.

The combination of a further close off of an estuary *and* maintenance ecological value is also difficult to achieve. In an estuary, the size of the mouth is defined by the size of the estuary; they are in equilibrium. Reducing the size of the mouth or inlet by e.g. realisation of the barrier proposed in this research, disrupts this equilibrium. This could result in e.g. reduction of the size of the estuary, 'sand hunger' and a reduction of the tidal amplitude (Bosboom and Stive, 2012, Ch.9), negatively influencing natural value. The Eastern Scheldt in The Netherlands is an example of an estuary where sand hunger and reduction of tidal amplitude has occurred after a storm surge barrier was realised. Implementation of the proposed concept would thus lead a reduced natural value, and therefore not be eco-friendly.

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Appendices

A

Quick scan locations application of hybrid closure

As climate predictions lean towards rising sea levels and the increasing occurrence and magnitude of storm surges, flood defences need improvements to maintain current safety standards. To protect hinterland, flood defences are often located on the border between water and land. On the other hand, the border between land and water, such as river banks, is an attractive place to live and is in many situations filled with high value property. Due to the value of this property, it might not always be possible to protect the hinterland by highering the river banks. Full canalisation might seem to be the most logical option if the value of the waterway is only of limited importance as shipping route. However, by closing off the river completely, tidal amplitude, and therefore precious ecological value accompanied with it, is lost. Economical value is then maintained or improved at cost of the ecological value, an issue much encountered in interventions. Therefore, a concept preserving the tide and maintaining or even improving the level of safety is wished for, preserving both economical and ecological value as much as possible, is much wanted.

In order to indicate the relevance of the problem, a quick scan has been performed. This scan is meant to indicate a number of possible cases that might be suitable for a concept that reduces the flood risk by near-canalisation and maintains a tide. In first instance, cases are selected where:

- a meso or macro tidal amplitudes is present;
- a high level of wealth is present in combination with a high population density;
- the delta is situated on a trailing edge in a stormy climate.

Subsequent paragraphs will further explain why these criteria have been chosen. It may very well be possible that a some cases have been omitted, or that the concept can be applied in regions with slightly different conditions. The mere object of this scan is to show the relevance of the problem at hand.

A.1 Tidal amplitude

To be of relevance, a relevant tidal amplitude has to be present. Davies and Clayton (1980) make a distinction between micro, meso and macro-tidal regimes, see figure A.1. Based on this figure, areas subjected to mean spring tidal ranges smaller than 2 m (micro-tidal regimes) are excluded.

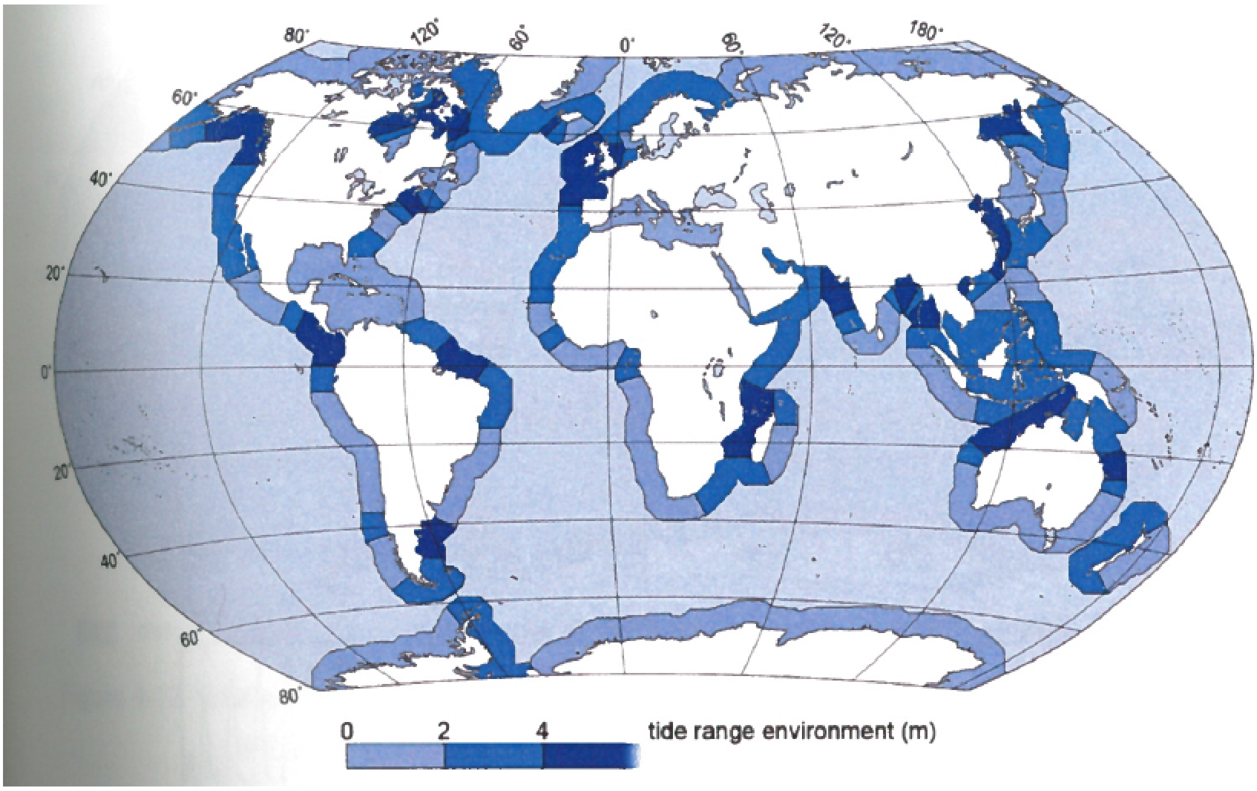

Figure A.1: Spring tidal range, Source: Bosboom and Stive (2012).

A.2 Wealth and population density

The possible regions are further narrowed down by further selecting on wealth by Gross Domestic Product (GDP) per capita. Figure A.2 shows GDP per capita on a subnational or regional rather than national basis. It is assumed that areas with high levels of wealth are more likely to have high value property along river banks than areas with smaller wealth accumulation. For this scan, regions with average GDP per capita larger than $10,800 have been taken into account. This data is now combined with a rough estimate of population density, based on a light map seen in figure A.3. Reason for a further selection on density is the assumption that in areas with high density it will be less likely that measures along the river banks are possible or wanted. Regions that meet the above-mentioned criteria have been tabulated in table A.1.

Table A.1: Densely populated regions with meso or macro spring tidal range and high GDP per capita.

Area	Country	Region
Europe	Norway; Denmark; United Kingdom; Ireland; Germany; The Netherlands; Belgium; France Spain; Portugal	
Asia	South Korea	West coast
	Japan	South
	China	East
	Malaysia; Singapore; Brunei	
South America	Panama	South
	Mexico	West (Baja California)
North America	USA	West
		East
	Canada	Vancouver area
		South East

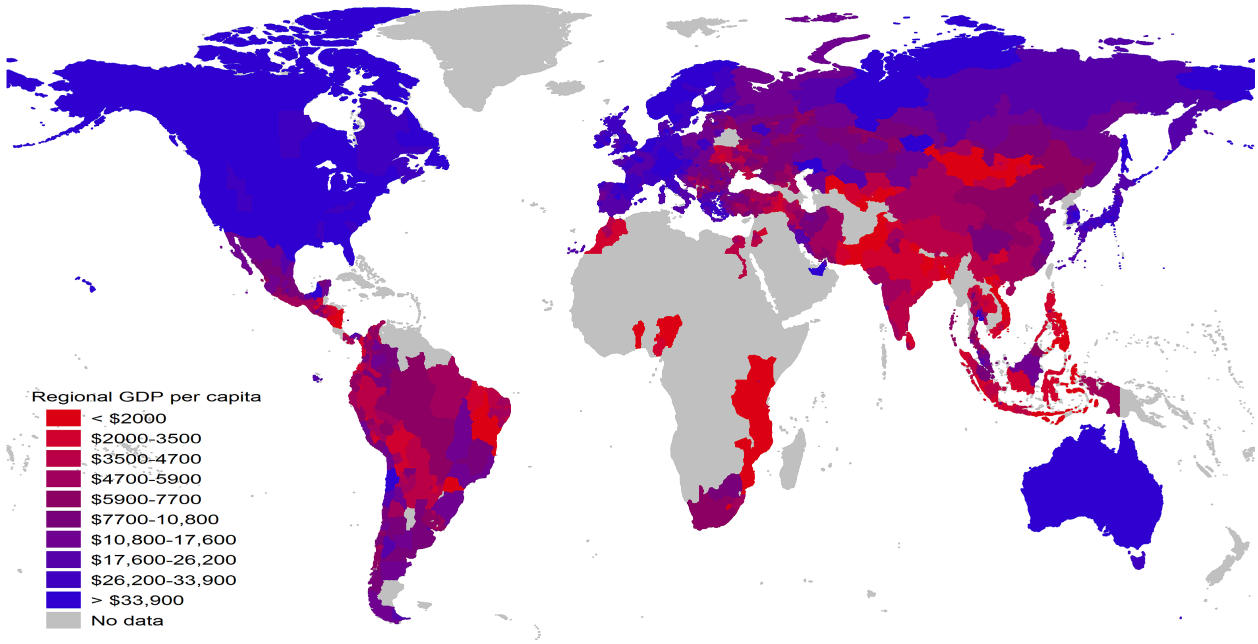

Figure A.2: Regional Gross Domestic Product (GDP), (Gennaioli et al., 2011).

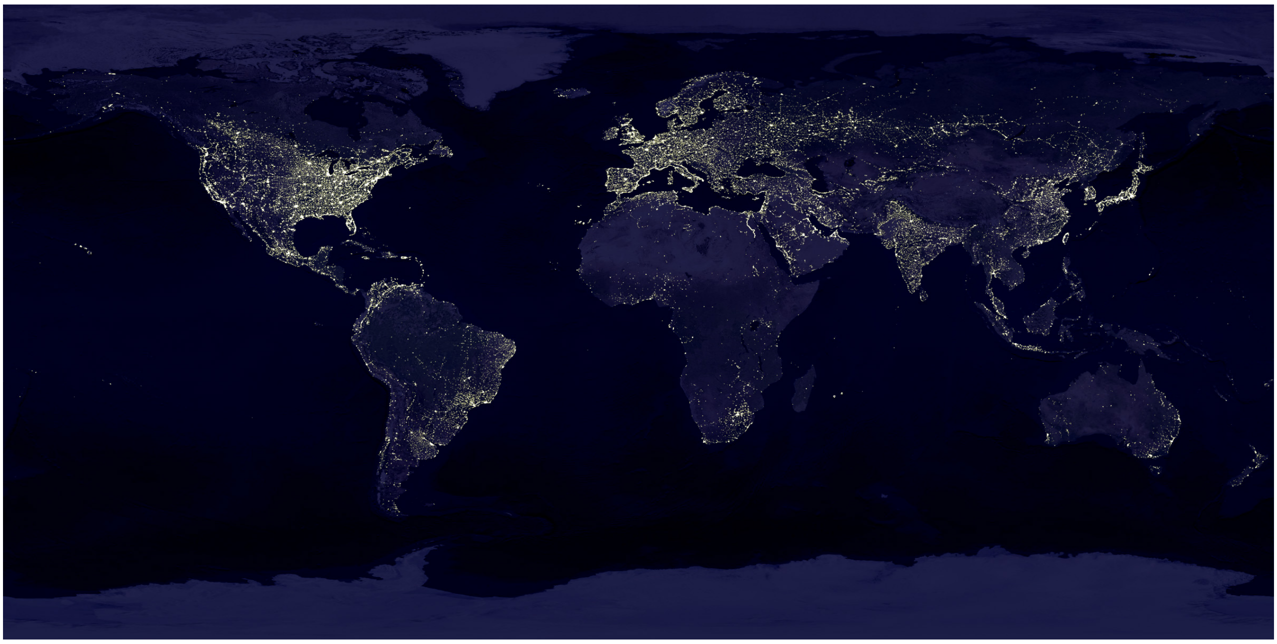

Figure A.3: Satellite image of the world at night. Source: (NASA, 1995).

A.3 Edge type and storm climate

To reduce the number of regions to areas where it is likely to find related and relevant problems, the type of coast is taken into account. Problems with relocation are likely to be found in areas where river deltas are vast and flat. A rise in water level due to sea level rise, subsidence or increased set up due to climate change is more likely to cause problems to its surroundings in flat river mouths than small deltas with steep slopes close to the river bank. Widespread flat deltas can be found along the trailing edges of continents or bordering marginal seas. Neo-trailing edges are very similar to leading edges however, meaning that their deltas are small and bordered by steep slopes as well Bosboom and Stive (2012, Chapter 2). Due to set up caused by storms, the water level varies more in areas subjected to storms and cyclones than in areas subjected to swell. In general, this leads to higher protections compared to the water level in storm or cyclone areas, something that might be unwanted along these banks covered with high value property as mentioned before. A further selection of cases based on the type of edge and wave climate, based on Bosboom and Stive (2012) is summarised in table A.2.

Table A.2: Type of coast and wave environment, Bosboom and Stive (2012).

Area	Country	Region	Edge type	Type of waves
Europe	Norway		Trailing (Amero)	Storm waves
	Denmark	West coast	“ ”	“ ”
	United Kingdom		“ ”	“ ”
	Ireland		“ ”	“ ”
	Germany		“ ”	“ ”
	The Netherlands		“ ”	“ ”
	Belgium		“ ”	“ ”
	France		“ ”	“ ”
	Spain	North and West coast	“ ”	“ ”
	Portugal		“ ”	Swell
Asia	South Korea	West coast	Marginal Sea	Protected Sea
	Japan	South	Leading	Protected Sea/Cyclones
	China	East	Marginal Sea	Cyclones
	Malesia		“ ”	Protected Sea
	Singapore		“ ”	Protected Sea
	Brunei		Leading	Swell
South America	Panama	South	Leading	Swell
	Mexico	Baja California	Trailing (Neo)	Swell/Cyclones
North America	USA	West	Leading	Storm waves
		East	Trailing (Amero)	Swell/Storm waves
				Cyclones
	Canada	Vancouver area	Leading	Storm waves
		South East	Trailing (Amero)	Storm waves

Based on table A.2, the population density has been further assessed. Only the regions with an amero-trailing edge and a cyclone or storm wave climate have been further investigated. Municipalities with more than 500,000 inhabitants in these areas are further looked into. Table A.3 shows the municipalities per country with more than 500,000 inhabitants and situated relatively close to the coast. The rivers in the neighbourhood of these agglomerations are mentioned as well. Agglomerations with more than 1 million inhabitants are marked with a asterisk United Nations Statistics Division (2015).

Table A.3: Selected areas and rivers, United Nations Statistics Division (2015).

Continent	Country; State/Province	City	River
Europe	United Kingdom	London*	Thames; Darent; Mar Dyke; Medway; River Crouch
		Manchester*	Mersey
		Glasgow*	Clyde; Black/White Cart Water
		Newcastle	Tyne
		Bristol	Lower Avon
	Ireland	Dublin*	Liffey; Dodler; Tolka river
	Germany	Hamburg*	Elbe + tributaries; Wezer; Hunte; Ochtum
	The Netherlands	Amsterdam*	-
		Rotterdam*	Hollandsche IJssel; Spui; Oude Maas; Dordtse Kil; Noord; Beneden Merwede; Wantij; Hollandsch Diep
	France	Paris*	Seine
		Nantes	Loire; Erdre; Sèvre
		Bordeaux	Garonne; Bourde
Asia	China; Zhejiang	Hangzhou*	Qiantang Jiang
		Ningbo*	Yongjiang; Yuyao; Fenghua
		Zhoushan	Inland canals
		Taizhou*	Jiaojiang; Yongning
		Wenzhou*	Oujiang; Nanxi
	Chinga; Fujian	Fuding	unknown river
		Fu'an	unknown river
		Fuzhou*	Minjiang
		Putian*	unknown river
		Quanzhou*	Jinjiang
		Xiamen*	Downtown lake; Several small rivers
		Zhangzhou	Tributary
North America	USA; Florida	Jacksonville	St. Johns
	USA; New York	New York*	Raritan; Rahway; Freshkills; Passaic; Hackensack Overpeck; Arthur Kill; Harlem River; East River; Hudson; Westchester Creek ; Bronx River
	USA; Massachusetts	Boston	Charles River; Mystic River; Neponset River; Chelsea Creek

B

Case specification

In this appendix, four different cases are selected from the quick scan with possible implementation locations (appendix A) and further looked into. The cases are spread over different continents and exemplify the relevance of the subject at hand. The cases are selected based on the main function the river or river branch has. While a half open barrier may delay shipping and thus reduce economic value: river (branches) mainly used for transportation are left out of the scope. River (branches) with a residential or an office function along the banks haven been chosen for further elaboration. It may however very well be the case that an ecological function of the river is already combined with an urban function. Within the four cases, two of the cases currently have an open connection to the river system. The other two cases are locked off by a sluice. The cases that have been selected are:

1. Hamburg region (sluice), section B.1;
2. Taizhou region (sluice), section B.2;
3. Rotterdam region (open), section B.3;
4. New York region (open), section B.4.

B.1 Hamburg region

Being an agglomeration of 1.7 million inhabitants (United Nations Statistics Division, 2015) and situated along the tidal part of the river Elbe, tributaries of this river would be a good case to look at. The spring tidal range is around 4 m (Bundesamt für Seeschifffahrt und Hydrographie (BSH), 2016). The Elbe's tributaries around Hamburg are either in open connection when used for shipping, or closed off by a sluice when this is not the case. As it is not the intention of this further elaboration to be in conflict with main economical/shipping function, a further look will be taken at closed off river arms, for which (1) the Dove Elbe and Gose Elbe and (2) the Bille river have been selected.

B.1.1 Dove Elbe and Gose Elbe

The Dove Elbe is a 18 km river stretch that is closed off by the Tatenberger Sluice (Norddeutscher Rundfunk, 2015). The Gose Elbe is connected to the Dove Elbe approximately 3 km east of the Tatenberger Sluice. Another sluice, the Reitschleuse, separates both waters (Heinrich, 2016). Aerial imaging (Google Inc., 2016) shows that in the past, both branches were connected upstream to the Elbe as well (near former concentration camp Neuengamme). Currently the upstream connection is blocked by the Elbe river dike. In the two river branches there is currently no tidal action (Norddeutscher Rundfunk, 2015). Both the Dove and Gose Elbe have are bordered by dikes that currently have little hydraulic function. With no industry present, the rivers have a recreational, residential and nature function.

B.1.2 Bille River

The Bille river has a length of 65 km, has no significant discharge and connects to the Elbe river in Hamburg (Albrecht, 2000). The downstream area close to the Elbe river primarily has an industrial function. The latest development plans do however not consider this area to be entirely harbour area (Hamburg Port Authority, 2012). Further upstream, east of the quarters Rothenburgsort and Billbrook, housing, agriculture and natural preserves border the river. With port extension to Altenwerder along the southern Elbe and redevelopment (such as Hafencity) around the northern parts of the harbour (HafenCity Hamburg GmbH, 2016), it might very well be possible that the utmost southern stretch of the Bille river may lose its main industrial/harbour function. Reintroduction of (a part of) the tide could be taken into account when optimising this area for new functions such as water storage, residential area and recreation. Further upstream nature may benefit from the reintroduction of the tide.

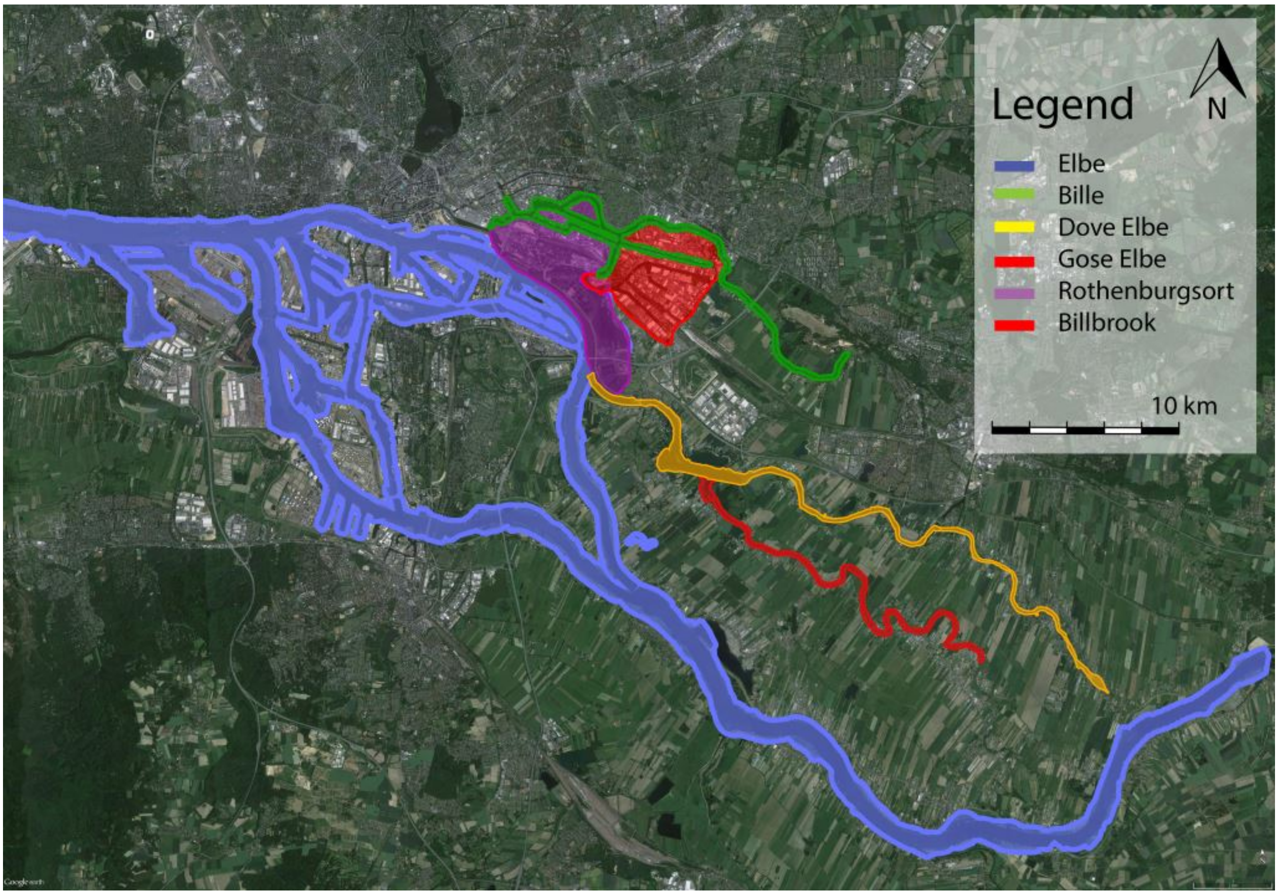

Figure B.1: Map of Hamburg. Source: Google Inc. (2016).

Figure B.2: The Dove Elbe, showing its different functions. Source: ASVHH (nd).

Figure B.3: The Gose Elbe. Source: Bellin (nd).

B.2 Taizhou region

Taizhou is a city in the province of Zhejiang, China. The city itself has 1.9 million inhabitants and is located along the Jiaojiang river. The agglomeration has more than 5.9 million inhabitants of which Huangyan houses 570,000 inhabitants (Brinkhoff, 2016). The city is located along the Yongning river which flows into Jiaojiang river.

B.2.1 Yongning river

Both rivers seem to be separated by a lock and a dam, figure B.4, preventing tidal intrusion into the Yongning river. Maximum tidal amplitude near Shanghai is approximately 3.3 m (Mobile Geographics, 2016) and is expected to be similar in the Jiaojiang river mouth. Discharge of the Jiaojiang river remains unknown. The recent typhoons in the area such as Haikui (2012) (The Telegraph, 2012) and Chan-hom (2015) (The Wall Street Journal, 2015) stress the necessity of proper defence against wind set up due to typhoons and tropical storms, e.g. by the already constructed dam and locks. On the other hand, the initiative to realise a river park shows the community's interest in improving the natural value and living standards along the river. Figure B.6 shows an overview of the constructed river park. Figure B.5 shows the old situation with concrete river banks. A concept that prevents set up to reach Huangyan and enhances ecological value by reintroducing the tide could therefore be a viable option.

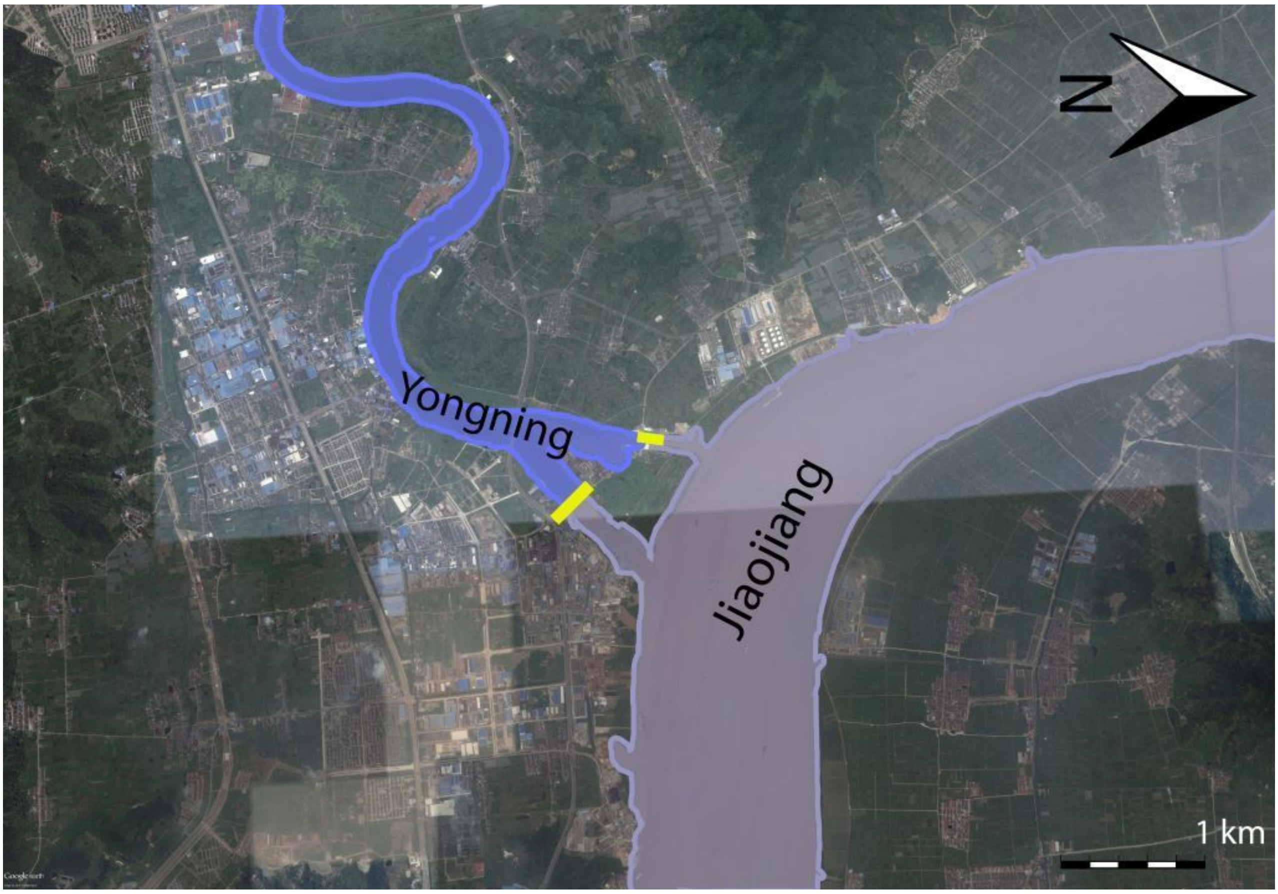

Figure B.4: Yongning Jiaojiang connection, the lock and dam are marked yellow. Source: Google Inc. (2016).

Figure B.5: Old situation, Huangyan. Source: Turenscape (2004).

Figure B.6: Overview River Park, Huangyan. Source: Turenscape (2004).

B.3 Rotterdam region

The Rotterdam (combined with The Hague) agglomeration consists of about 3 million inhabitants (Brinkhoff, 2016) and houses Europe's largest port (World Shipping Council, 2016). Its open connection to the sea is seen as a major benefit for shipping (Port of Rotterdam, 2016) but also allows the tide to reach far upstream (Rijkswaterstaat, nda). From the various tributaries in the Rhine delta with tidal influence, the Hollandsche IJssel is chosen due to the insufficient strength of the current dikes around the IJssel, subsidence (Hoogheemraadschap van Schieland en de Krimpenerwaard, ndb) and a previous study on river stretch by Schoemaker (2016).

B.3.1 Hollandsche IJssel

The Hollandsche IJssel is one of the side branches of the Dutch delta. It connects to the New Meuse near Cappelle a/d IJssel a suburb of Rotterdam. It has a length of 46 km of which the last 19 km are influenced by the tide. Near Gouda, the 'Waaiersluis' and Juliana locks block any further tidal influence. The upstream connection to the Lek river was closed off in 1285, as described by Rijkswaterstaat (ndc). The main natural phenomenon that is able to stir the water, is the tide, with a maximum tidal amplitude of approximately 1.8 m (Rijkswaterstaat, nda). The river only has a minor discharge, that is mainly due to pumps discharging into the Hollandsche IJssel (van Balen et al., 2010).

There are around 40,000 shipping passes per year, of which 23,000 are commercial (Bückmann et al., 2012). Shipping may encounter negative effects, as locks may be needed to move vessels across the half-open construction. Closing off the river would be beneficial in terms of flood protection as the water level can be controlled and lowered in accordance with subsidence of the surrounding area.

B.4 New York region

New York alone already has more than 8 million inhabitants (United Nations Statistics Division, 2015), while 22 milion inhabitants are part of the New York agglomeration (Brinkhoff, 2016). The recent damage and loss of lives caused by hurricane Sandy (2012) show that this metropole is prone to extreme weather with accompanying storm surges. The flooded area can be seen in figure B.9. Multiple rivers and creeks end in the Atlantic Ocean near New York, of which the Hudson and the East River are probably best known to the public. The rivers mentioned are however used for shipping and therefore not considered in this specification. The Passaic river and Hackensack River in New Jersey will be considered.

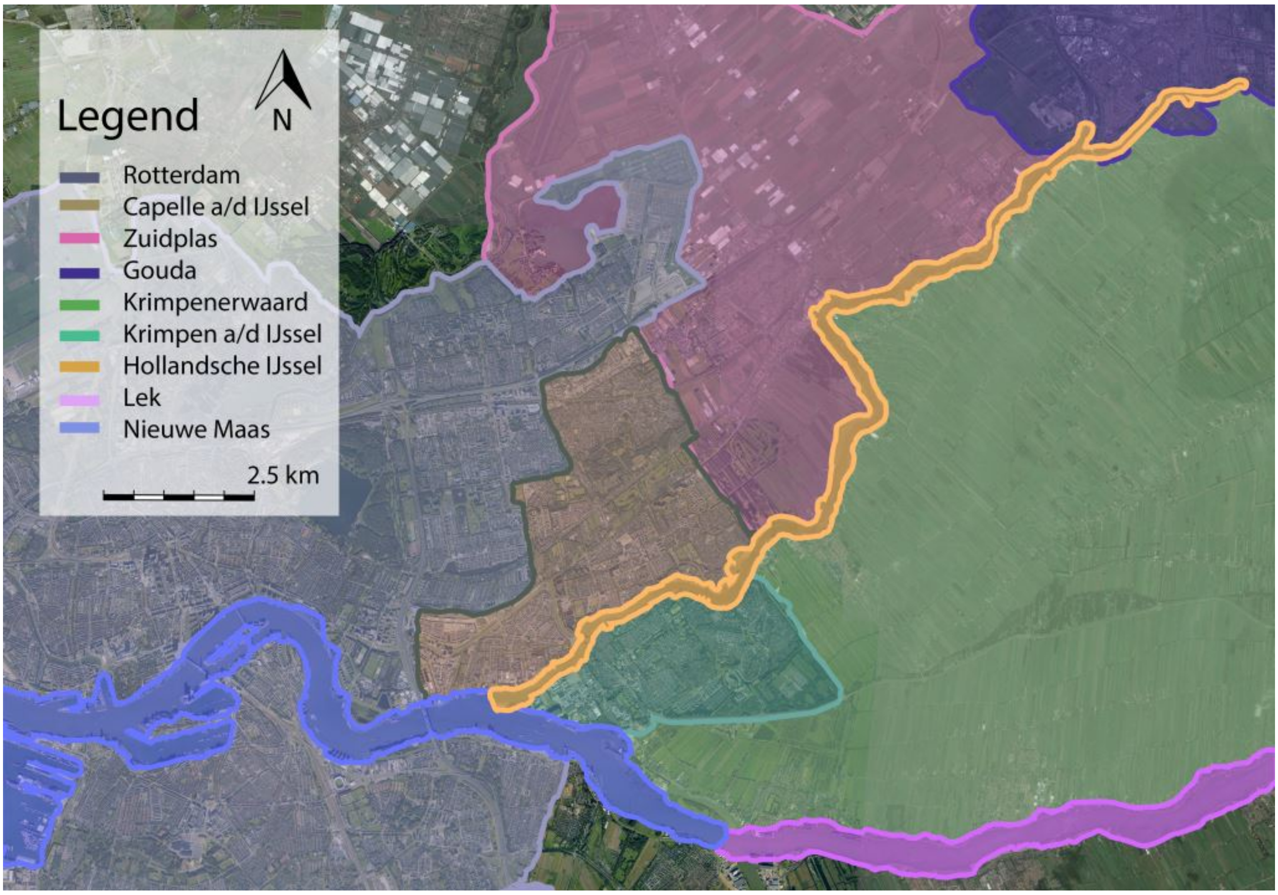

Figure B.7: The Hollandsche IJssel. Source: Google Inc. (2016).

B.4.1 Passaic River

The 145 km Passaic River is situated in the state of New Jersey (Passaic River Institute, nd). Average discharges are in the order of 75 m$^3/s$ with extremes of 1200 m$^3/s$ (USGS, 2016). At the river mouth, maximum tidal amplitudes are approximately 2.7 m (USGS, 2016). Near Passaic, around 25 km upstream (Google Inc., 2016), the maximum tidal amplitude remains approximately 2.7 m (Mobile Geographics, 2016). Tidal fluctuations will probably continue further upstream into the Saddle River and the Passaic river. The Dundee dam near Garfield, 30 km upstream of the river mouth (Google Inc., 2016), prevents further propagation of the tide into the Passaic river.

The banks of the last approximately 15 km used to be heavily in use by industry. The river is considered to be one of the most polluted in the United States and recently a $1.4 billion plan was presented by the U.S. Environmental Protection Agency to clean up this part of the river (Herzog, 2016). The banks of the final 15 km of the river are however facing declining industrial activity. With only 48 round trips in 2004, hardly any commercial shipping is present (US Army Corps of Engineers, 2010). Restoration of the river and protection from storm surges mentioned before could be combined which making this a valid case.

B.4.2 Hackensack River

Another case that could be looked at is the Hackensack river. The discharge in the river is small with averages of approximately 5 m^3/s and peaks of 25 m^3/s near New Milford, NJ (USGS, 2016). Its banks however house major industrial sites, that may be harmed by a half open barrier. The (former) industry along the Hackensack also polluted the river, to levels comparable in the Passaic River (o'Neill, 2015). Considering the decreasing industrial activity along the neighbouring Passaic river and the ongoing debate on nature preservation and restoration in the area could however make this case a viable one in the future.

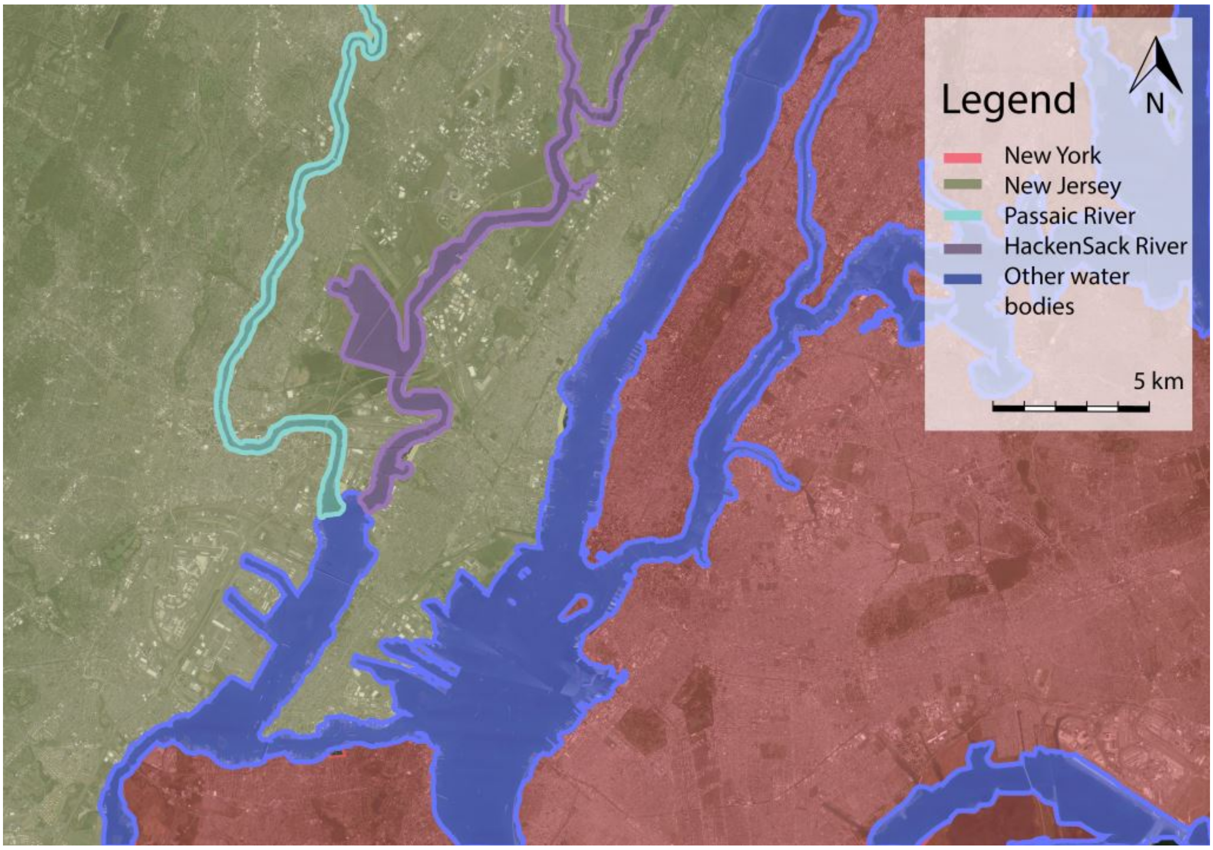

Figure B.8: Location of the analysed rivers. Source: Google Inc. (2016).

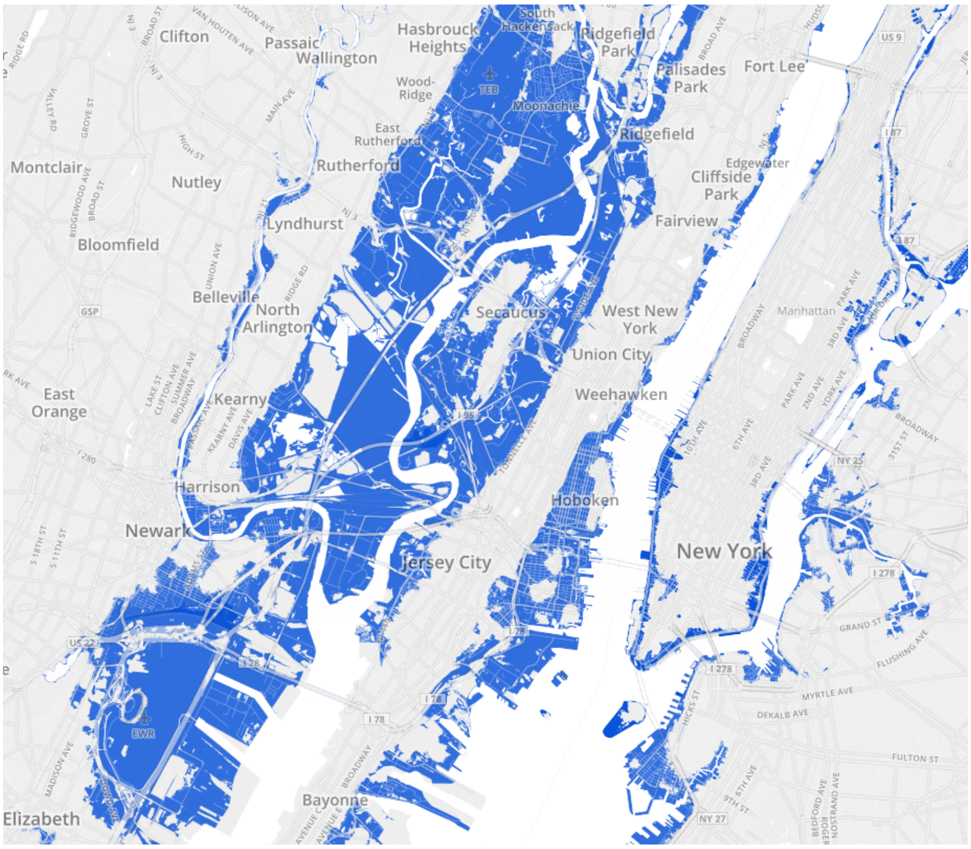

Figure B.9: Floodings caused by hurricane Sandy. Source: Keefe et al. (2016).

C

Closure of tidal areas in The Netherlands

This chapter provides information on other estuaries/deltas that have been closed in The Netherlands in the last century. Information on why they were closed and why a specific closure was chosen could give insights on the direction to look at for the Hollandsche IJssel. The following sections provide information on the different factors that have led to this decision.

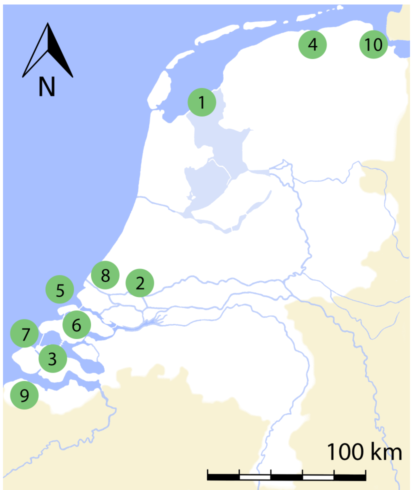

Figure C.1: (Closed off) estuaries and rivers in The Netherlands. Image source: lesidee.nl.

C.1 Zuiderzee (1932)

The Zuiderzee was a marginal sea that was connected to the North Sea via the Wadden Sea. Factors that led to closure and construction of the Afsluitdijk (number 1 in figure C.1) were:

- Increase agricultural area: the first world war showed the country's fragile food supply;
- Flooding in 1825, 1916 showed the necessity due to caused damage and loss of lives. Shortening the dike length from approximately 400 km to 32 km meant that available funds for flood protection could be spend on a shorter dike;

Increase in fresh water supply and decreasing influence of salt water on agricultural activities were additional benefits. The towns along the sea were home to salt water fishers whose fishing grounds would disappear. Protests could however not prevent the construction of a dam; the government initiated a compensation fund to support the fishermen (Lak, 2007).

C.2 Hollandsche IJssel (1958)

The Hollandsche IJssel was the first water body (number 2 in figure C.1) in which a storm surge barrier was introduced after the flooding of 1953. A storm surge barrier was according to Krol (2014) chosen because:

- In daily circumstances there is no shipping delay (for vessels that fit underneath the barrier);
- A storm surge barrier was faster to build than a dam;
- There is no disturbance of the regional water system.

C.3 Veerse Gat (1961)

The Veerse Gat (number 3 in figure C.1) was a water body that separated two islands (Walcheren and Noord-Beveland) from each other in the south western delta. Connecting both island with a dam meant:

- A reduction of the coast line;
- An opportunity to apply the lessons learnt on larger closures.

C.4 Lauwerszee (1969)

The Lauwerszee (number 4 in figure C.1) was an estuary that connected to the Wadden Sea. Initially the government planned to reinforce current dikes, which was a cheaper solution. Local inhabitants were however in favour of a closure and persuaded the government to do so, reducing the coast line from 32 km to 13 km. Local shrimp fishermen and nature organisations were however against closure. Besides the 13 km dam, the closure encompassed construction of outlets and a shipping lock (Nationaal Park Lauwersmeer, nd).

C.5 Haringvliet (1970)

The Haringvliet (number 5 in figure C.1) has been closed off by an outlet construction. A ‘simple’ dam was not possible because the Haringvliet is used to discharge (peak) discharges of the Rhine and Meuse into the North Sea. The construction of the dam and outlets led to disappearance of tidal action and salt water intrusion. The fresh water body provided opportunities for agricultural activities and the supply of drinking water. Recently, the Dutch government has decide to open the outlet more permanently. This ‘kierbesluit’ will lead to a situation in which the outlets will be opened slightly to allow fish to pass the structure (Rijksoverheid, 2013).

C.6 Grevelingen (1971)

The Grevelingen lake (number 6 in figure C.1) is the largest salt water lake of Europe (Ministerie van Economische Zaken, nd). Initially, the lake was meant to become fresh and function as supply for agricultural activities on the neighbouring islands. In the mid ’70s it was decided that it should remain salt and the Brouwerssluis was constructed. However, tidal amplitude has disappeared (Rijkswaterstaat, ndb). Reasons for the current construction were:

- Natural value of salt water lake;
- Reduction of coastline.

In the past years, the water quality of the Grevelingen lake is a point of debate. Wortelboer and Dirkx (2011) mention that the disappearance of tidal action has led to:

1. Bad water quality due to of accumulation nutrient-rich sediment in the former tidal channels;
2. Vegetation develops on the intertidal area, leading to a decrease of area favourable for sea birds;
3. Erosion of the tidal flats due to ‘sand hunger,’ a process in which sand is transported from the flats into the tidal channels.

Recently, different stakeholders have joined forces (Zicht op Grevelingen) to improve the value of the area. A tidal amplitude of 0.5 m is mentioned to be ideal to flush away high accumulations of nutrient-rich sediment and keep current nature functions. Wortelboer and Dirkx (2011) however mention that this insufficient to restore nature fully.

C.7 Eastern Scheldt (1986)

Originally, it was proposed that the Eastern Scheldt (number 7 in figure C.1) should be closed off completely. However, protests from local fishermen and in a later stage nature conservation organisations prevented a total closure. Construction had already commenced when this decision was taken (van Houten, 2013). Finally the iconic half open barrier has been constructed.

C.8 Nieuwe Waterweg and Hartel Kanaal(1997)

The Maeslant and Hartel barriers are the last barriers that have been constructed as part of the Delta Works. Initially, no barriers were scheduled due to the importance of the Nieuwe Waterweg and the Hartel Kanaal (number 8 in figure C.1) as a transport routes to the port of Rotterdam. Later studies showed that raising the dikes in the Rijnmond area was costly and conflicted with the current urban function (Stichting Deltawerken Online, 2004). The importance of shipping to and from the Rotterdam harbour and the higher price of raising dikes led to the decision to construct the two arms in the Nieuwe Waterweg. The large span however comes with a price: the barrier has a relatively high probability of failure: 1:100 (Ministerie van Verkeer en Waterstaat, 2006).

- Raising dikes in Rijnmond area in conflict with urban function;
- Open connection to Port of Rotterdam is of high importance.

C.9 Western Scheldt (open)

The Western Scheldt (number 9 in figure C.1) is the last open estuary of the south western delta. Initially, the harbour of Antwerp and its connection through this estuary to open sea was an important reason not to construct a barrier (Ministerie van Verkeer en Waterstaat, 2006). Nowadays it is flagged as Natura 2000 area (Ministerie van Economische Zaken, nd). Recent raising of the dikes around the Western Scheldt (PZC, 2012) show that maintenance of its current ecological and economical function are preferred over e.g. shortening of the coastline.

C.10 Dollard (open)

The Dollard (number 10 in figure C.1) is an open estuary in utmost north east of The Netherlands. As a result of the 1953 flooding, the dikes around both the German as the Dutch part were raised (Essink, 2013). No information is available on plans for an entire closure of the Dollard. Possible historic reasons are the large size of the estuary, the low economic value and low population density. Nowadays the ecological value of the area is acknowledged by its status as natural preserve.

C.11 Summary

The closure of estuaries shows a clear trend (except for the Hollandsche IJsselkering, which seems to be a frontrunner). Where in the past estuaries were closed off and only the benefits were looked at, recent closures show a trend towards mitigation of loss of other functions. The Eastern Scheldt barrier and the recent decision to open the Haringvliet dam are clear examples of the current value that is accredited to nature, while the construction of the Maeslant barrier and Western Scheldt remaining open show the importance of the shipping function. More than ever, stakeholders' interests should be and are taken into account. Table C.1 summarises the properties for the different nautical 'users' of the water bodies in normal conditions.

Table C.1: Closures and their influence their 'users'

Water body	Nautic	Biotic
Zuiderzee	Closed	Closed
Hollandsche IJssel	Open	Open
Veersche Gat	Closed	Closed
Lauwerszee	Closed	Closed
Haringvliet	Closed	Closed*
Grevelingen	Closed	Closed**
Eastern Scheldt	Closed	Open
Nieuwe Waterweg	Open	Open
Hartel Kanaal	Open	Open
Western Scheldt	Open	Open

* Will be opened in the future

** Plans are made to open up the barrier

D

Mitigating measures for fish migration

Various mitigating measures can be thought of in case passage of fish through the barrier is difficult. This appendix will describe various measures found. The report *Nederland leeft met Vismigratie* was used as first guideline (Kroes et al., 2015). At the end of the appendix, a qualtitative comparison compares the different options.

D.1 Fish friendly pumps

Different pumps that are fish friendly are available. However, a widely accepted definition of fish friendlyness does not exist in The Netherlands. Work is in progress to define regulations on how to test pumps on fish friendliness (NEN, 2016).

Several manufacturers claim to have fish friendly pumps on the market. The difference between a conventional pump and a fish friendly pump is the shape of the blade. In a fish friendly pump the blade is created in such way that they do not harm the fish. Figures D.1 and D.2 show the difference between both blades. Bosman watermanagement presents the vision pump with a capacity up to 5 m^3/s. According to the firm, its capacity could be scaled up (bosman watermanagement, nd). Hidrostal also produces fish friendly pumps with capacities upto 12 m^3/s (Hidrostal, nd). A report by Visadvies (Spierts and Vis, 2012) showed that at least a smaller pump (capacity 2.3 m^3/s) is fish friendly.

Figure D.1: Conventional pump (IJmuiden). Source: www.gemalen.nl

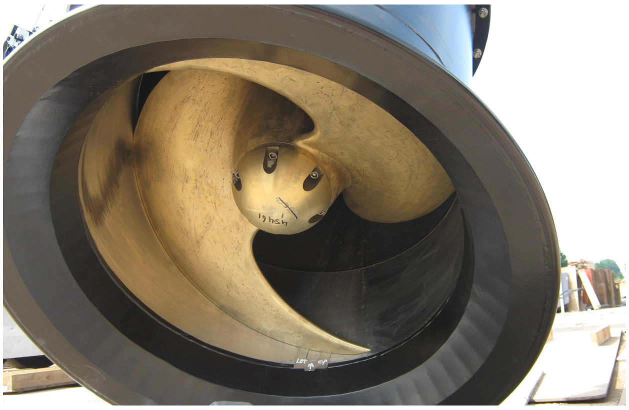

Figure D.2: Fish friendly pump (Ouwenaar/Haarrijn). Source: www.roelofsgroep.nl

D.1.1 What if capacity is too small?

Although manufacturers claim that the pumping capacity can be scaled up, the capacity might still be insufficient. If that is the case, one could install at least one fish friendly pump in combination with conventional pumps. Fish should then be deterred from the conventional pumps, e.g. by means of a stroboscope (FishFlow innovations, nd), see figure D.3.

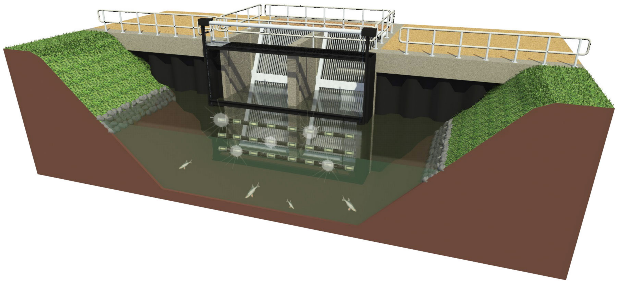

Figure D.3: Stroboscope to deter fish from a conventional pumping station. Source: fishflowinnovations.nl

D.2 Fish ladder

Fish ladders or stairs are an open connection around a construction. Various configurations are possible, such as a pool-and-weir fish ladder or a baffle fish ladder, see figures D.4 and D.5. The fish ladder is a proven concept to transfer fish across a structure. However, a permanent discharge will flow into the lower lying water, which might be problematic. If a fish ladder in combination with lower lying water is considered, a calculation should be made whether this continuous discharge is not problematic. Furthermore, in order to be implemented, the conventional fish ladder requires quite some space.

Figure D.4: Pool-and-weir fish ladder. Source: www.wrij.nl

Figure D.5: Baffle fish ladder. Source: www.youtube.com

D.3 De Wit lock fish passage

Compared to the conventional fish ladder, the lock fish passage is a cheap way to overcome height differences larger than 1 m (de bruijn, nd). The lock fish passage was designed in 2010 by Wim de Wit. Figure D.6 shows a schematisation of the lock fish passage. Always, one of the doors is opened, while the other is nearly closed. The nearly closed door generates a little discharge to attract the fish. Attracted by the flow, fish swim into the culvert. At a certain moment in time, the opened door closes, and the closed door opens, allowing the fish to swim out at the other end.

D.3.1 Carousel

In order to prevent salt intrusion through a fish passage Hoek and Leunge (2014) wrote an article about a fish carousel through the Afsluitdijk in The Netherlands. The concept is similar to the lock passage, but also prevents salt intrusion. Figure D.7 shows a schematic overview.

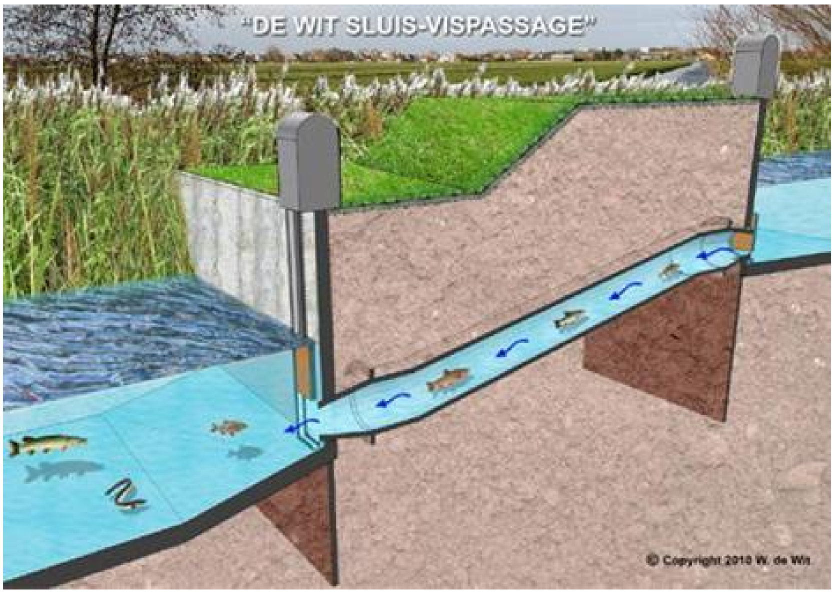

Figure D.6: De Wit lock fish passage. Source: www.hdsr.nl

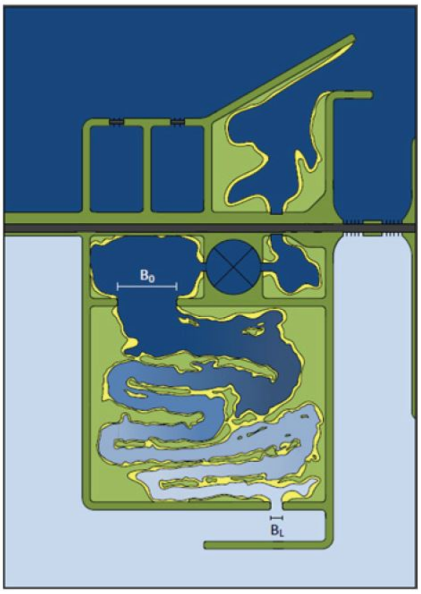

Figure D.7: Carousel fish passage (Hoek and Leunge, 2014).

D.4 Fish lift

In comparison with the fish lock, fish lifts can overcome even larger head differences. Basically it is a bucket of water to which fish are attracted, e.g. by a little flow which ascends and descends at an interval. Figure D.8 shows a fish lift at the Paradise Dam in Australia.

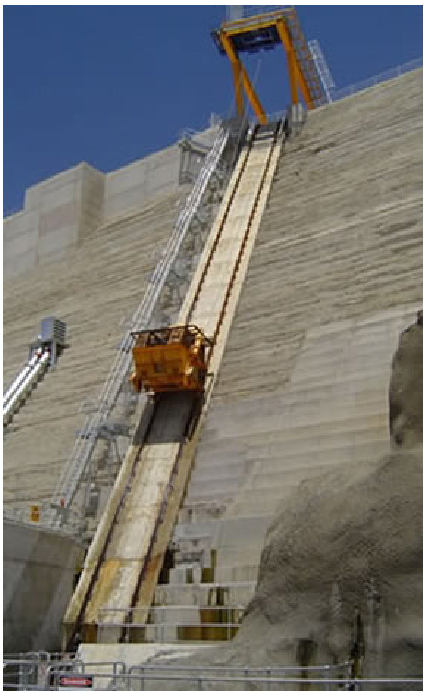

Figure D.8: Fish lift at the Paradise Dam, Australia. Source: tlsc.co

D.5 Summary of findings

Table D.1 shows a qualitative comparison of different mitigation measures for fish migration. It can be seen that a traditional ladder scores lower than the other options on the criteria considered. However, no cost estimate is made which could of course influence the decision for a measure if one is needed. Independent of which measure is chosen, it can be concluded that enough measures are available to mitigate issues with fish passability.

Table D.1: Qualitative comparison of different mitigation measures for fish migration.

	Leakage	Space required	Head difference
Pumps	+	+	±
Ladder	-	-	-
Lock	+	+	+
Lift	+	+	++

E

Dose effect relationships

This appendix shows a number of relevant dose effect relationships for a freshwater tidal river system. Naturally, since the Scirpus triqueter or *Driekantige Bies* is the only unique species for a freshwater tidal river system, the relationships for this plant are shown. Other species that are aimed for, or at least the ecological boundary conditions could be created for, are various mussels (such as Unionidae (stroommosselen) *and Anodontinae (zwanenmossels))* for shallow tidal water and the Osmerus eperlanus (smelt/spiering) and the Dreissena polymorpha (driehoeksmossel) for deeper water. These species are not unique for tidal river systems, but species such as the Thick shelled river mussel (Bataafse stroommossel) and the Smelt (Spiering) have become a rare in The Netherlands. Shaping the right conditions for these species therefore leads to value creation based on their rareness.

E.1 Scirpus triqueter/Driekantige bies

- A waterdepth between 25 cm and 200 cm is required, while a waterdepth between 100 and 130 cm is optimal (figure E.1) (Deltares, nd).
- A minimal tidal amplitude of 50 cm is required, while a tidal amplitude of more than 80 cm is optimal (figure E.2) (Deltares, nd).

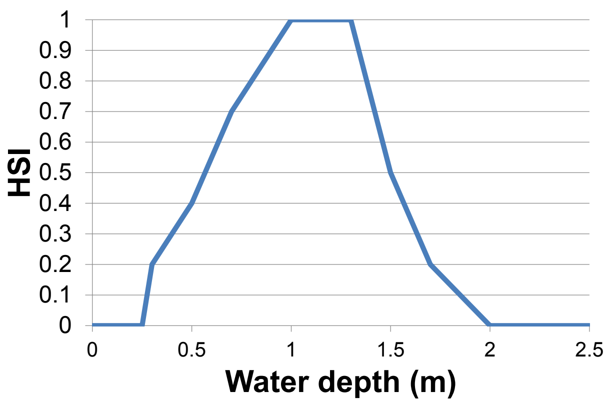

Figure E.1: HSI for water depth Driekantige Bies (Deltares, nd).

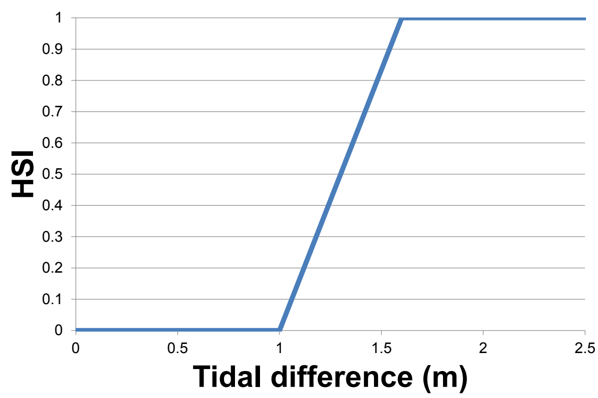

Figure E.2: HSI for tidal range Driekantige Bies (Deltares, nd).

E.2 Molluscs

- **Unioniae (stroommossels) & Anodontinae (zwanenmossels)** are indicative species for shallow tidal waters. No detailed info on habitat suitability is known. It is assumed that all shallow tidal water is suitable. For flow velocities, a similar HSI as for the Driehoeksmossel is assumed, see figure E.4.
- **Dreissena polymorpha/Driehoeksmossel** is an indicative species for deeper tidal water. Figures E.3 and E.4 show suitability indices for this type.

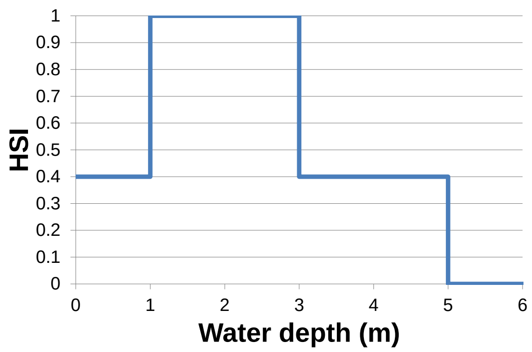

Figure E.3: HSI for flow velocity Driehoeksmossel (Deltares, nd).

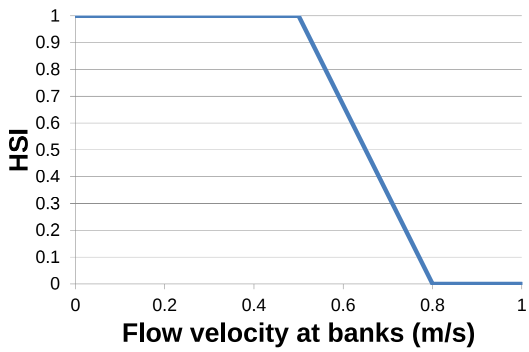

Figure E.4: HSI for flow velocity Driehoeksmossel (Deltares, nd).

E.3 Osmerus esperlanus/Spiering/Smelt

Currently, the KRW scores regarding fish are sufficient (table 4.1). However, as a barrier could negatively influence accessibility, attention should be paid to fish. Therefore, the following boundary conditions are of importance:

- The dam should be passable by migratory fish species (Bal et al., 2001);
- It is assumed that any deep water is just as suitable for fish species: no distinction is made;
- The length of the 'shoreline': the amount of deep water bordering shallow tidal and intertidal waters (Nijboer et al., 2000) is an indication for habitat suitability for Smelt. The more shoreline, the more space available for the fish to dispose their eggs.

F

Road capacity calculations

The road connecting Krimpen and Cappelle aan den IJssel (N210) is notorious for its traffic jams. Currently the plan is to replace the bridge connection together with the barrier around 2058. If an alternative solution would tackle this problem at an earlier stage, this save societal costs in terms of reduced travel time. In order to see where the problem lies, first order capacity calculations have been performed, see section F.4.

F.1 Situation

The regional road N210 runs from the highway A16 near Capelle aan den IJssel over the Algera bridge to Krimpen aan den IJssel. The road then continues further into the Krimpenerwaard. West of the Algera bridge, the two lanes for each direction (50 km/h) are separated by two bus lanes. On the bridge and on the east side there are two lanes (50 km/h) present plus a lane which can be used in both directions, depending on which direction the rush hour is directed. Measurements of intensity just east of Krimpen aan den IJssel (NDW, nd) and analysis of data provided by Google Traffic (Google Inc., nd) show that congestion occurs from Krimpen to Cappelle in the morning, while in the afternoon congestion is the reversed way. From north to south, six potential bottle necks see figure F.1 near the Algera bridge can be distinguished:

1. Connection N210/N219;
2. Junction N210 with Ketensedijk and Nijverheidsstraat;
3. Algera bridge;
4. Access and exit near Koningin Wilhelminaplein;
5. Junction N210 with Industrieweg and Nieuwe Tiendweg;
6. Roundabout N210 connection Tiendweg.

F.2 Bottle necks north west of Algera bridge

The first two potential bottle necks mentioned can be dealt with regardless the future strategy (current or proposed). There is enough space to increase capacity on the current trajectory. A first analysis with Google Traffic (Google Inc., nd) showed that the processing capacity of the junction N210/N219 (1) is insufficient. This causes congestion in the morning upto the junction with the Ketensedijk (2). A suggestion would be to let the roads cross at different levels, which has been done at the N44/N14 crossing in Wassenaar. Just as the junction N210 with Ketensedijk/Nijverheidsstraat, a further look into the capacity of N210/N219 crossing is beyond the scope of this research.

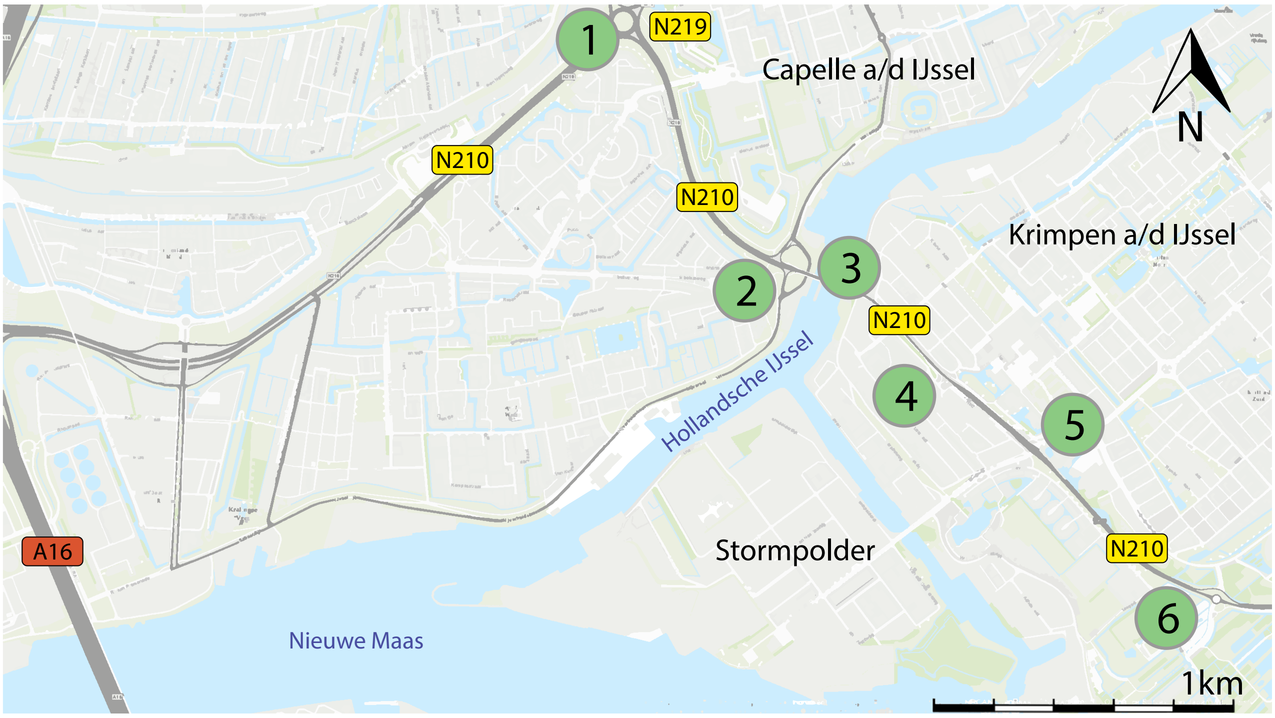

Figure F.1: Overview potential bottle necks. Source: World Topographic Map.

F.3 Bottle necks south east of Algera bridge

The potential bottle necks south east of the Algera bridge are potentially a bigger problem. Currently the N210 is running straight through Krimpen aan den IJssel. As buildings are build in close vicinity of the road, space is limited to increase its capacity. Furthermore the viaduct connecting the bridge has a negative impact on the quality of life.

1. **Algera bridge**: Congestion showed up in some cases just before the bridge and not on the bridge (an indication of a bottle neck). However in most cases the congestion seems to come from either the connection N210/N219 (morning) or the junction Industrieweg/Nieuwe Tiendweg (afternoon);
2. **Access and exit near Koningin Wilhelminaplein**: Potential congestion caused could not be distinguished from the congestion caused by the junction downstream (Industrieweg/Nieuwe Tiendweg);
3. **Junction N210 with Industrieweg and Nieuwe Tiendweg**: Google Traffic showed congestion around the junction, predominantly in the direction of Capelle in the morning and towards Krimpen in the afternoon. This direction stays busy all day;
4. **Roundabout N210 connection Tiendweg**: No congestion could be identified.

F.4 Capacity calculation

Based on the findings using Google Traffic presented in the previous section capacity calculations have been performed. For both the bridge and the junction this has been done using the fundamental diagram (van Nes et al., 2010, C.4). This diagram is based on equation F.1,

$$Q = k \cdot u \tag{F.1}$$

where Q represents the capacity, k the vehicle density and u the velocity. As this is a first order calculation, a bilinear diagram has been used, see figure F.2.

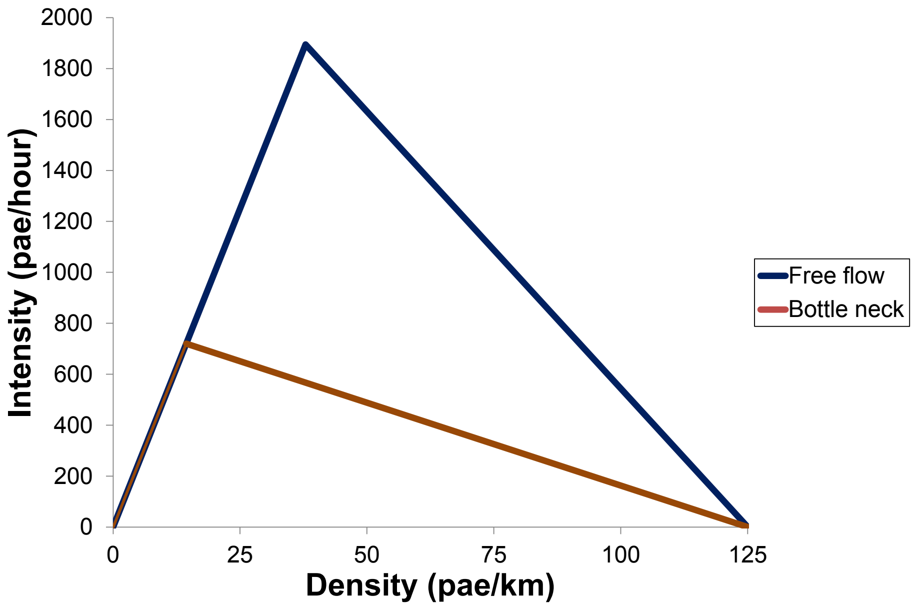

Figure F.2: Fundamental diagram.

When the critical vehicle density is unknown, the capacity can be calculated using the lead time h, defined as the time between two fronts of a car passing a certain point. The lead time is said to vary between 1.6 and 2.0 seconds. With

$$Q = 1/h \tag{F.2}$$

and a maximum speed of 50 km/h the capacity varies between 1800 and a theoretical maximum of 2250 vehicles per lane per hour. For 50 km/h roads within the urban areas CROW gives similar numbers: 1800 pae/h per lane (CROW, nd).

Three scenarios have been modelled, the differences have been tabulated in table F.1. The growth percentages represent the annual economic growth upto 2050 and 2125 respectively. The option to change use of land in combination with slow economic growth is not considered to be realistic.

Table F.1: Sub scenarios for number of crossings

Sub scenario	Land use	Economic growth	2050	2125	Reference scenario
1.	Similar	Strong	0.5%	0.5%	DRUK
2.	Similar	Slow	0.25%	0.5%	RUST/WARM
3.	Different	Strong	0.5%	1%	STOOM

To give an indication, the current intensities in both directions are shown in figure F.4. A clear peak is seen in Capelle direction in the morning and another peak towards Krimpen in the after noon. In figure F.3 the development peak intensities for the three scenarios are presented.

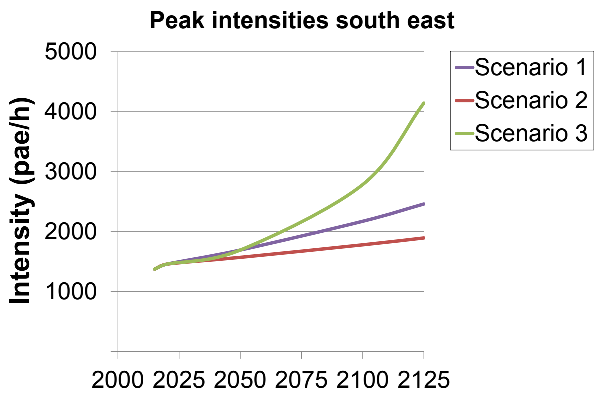

Figure F.3: Development of peak intensities for different scenarios on the N210.

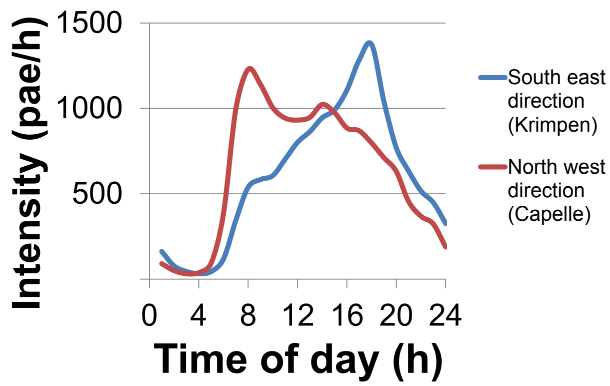

Figure F.4: Intensities on the N210, measured east of Krimpen a/d IJssel in 2015 Source: (NDW, nd).

F.4.1 Assumptions and simplifications

For simplicity reasons the following assumptions and simplifications have been made:

- The intensities measured just east of Krimpen aan den IJssel (NDW, nd) are used and transformed to intensities based on daily counts provided by Rotterdam Vooruit (2010, p.39);
- The intensity in both directions is considered to be the same;
- The division in the different directions is considered the same during the morning and the afternoon;
- Potential congestion occurring on lane 1 and 3 have no influence on the congestion upstream;
- For this first assumption it is assumed that the commuter lane is open towards Krimpen the entire day;
- In order to fit the model to the current situation, a lead time *h* of 1.9 seconds has been used;
- Congestion does not occur during the weekends.

F.4.2 Algera bridge

The capacity of a road section can vary between 1800 and 2250 pae/h/lane, see equation F.2. During rush hour, the closed bridge can support 3600 - 4500 pae/h one way. For this analysis a lead time of 1.9 s has been used, resulting in a peak capacity of 3789 pae/h. Presently, the capacity of the bridge itself seems sufficient during the day. Except for scenario 3 the bridge seems to have sufficient capacity in the future as well. In this scenario the capacity becomes insufficient around 2100.

F.4.3 Junction

Now, the junction is examined. Often it is the capacity of the crossings that form the limiting factor. As can be seen in figure F.5, the bus lanes and the jam lane make the junction quite complex. The rush hour lane can only be used for traffic coming from and travelling to Krimpen. Daily averaged data is available for direction 1, 2 and 3 together and 4 separately. The division between the directions is however based on insights gained during a field visit. For the main directions, 2 and 4, it is assumed that intensity is proportional to the amount of time the traffic light is green. Directions 1 and 3 are assumed to process 5% of the incoming traffic.

When the amount of vehicles exceeds the capacity, a traffic jam will begin to form. The upstream travel speed or shockwave of a jam from t_1 to t_2 is ω_{1-2} and calculated by:

$$\omega_{1-2} = \frac{q_1 - q_2}{k_1 - k_2} \tag{F.3}$$

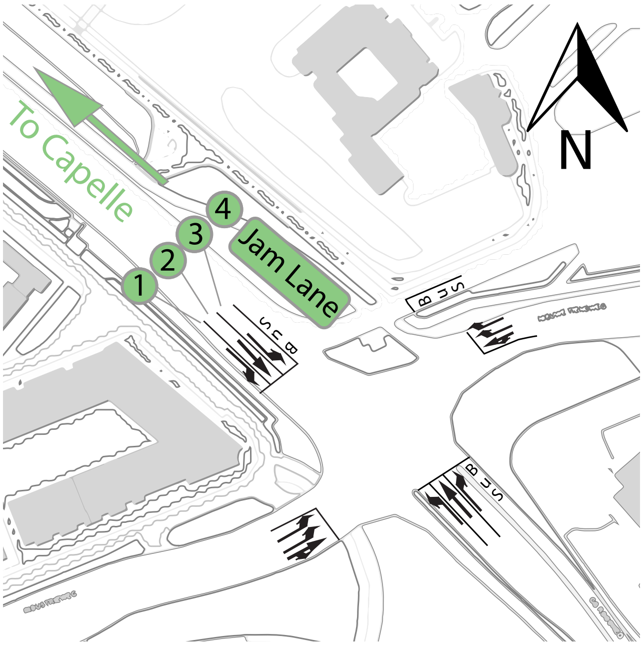

Figure F.5: Junction N210 with Industrieweg/Nieuwe Tiendweg, Krimpen a/d IJssel. Source: World Topographic Map.

Table F.2: Capacity junction eastward.

Crossing	'Green time' [%]	Capacity [pae/h]	Traffic division [%]
1	55	982	5
2	38	681	53
3	21	377	5
4	26	476	37

$$x(t) = \omega(t) \cdot t \tag{F.4}$$

The average jam length for a vehicle entering during t_1 to t_2 is given by dividing the integral of x (trapezium rule) by t:

$$L = \int_{t_1}^{t_2} x(t)dt = (t_2 - t_1)\frac{x(t_1) + x(t_2)}{2(t_2 - t_1)} \tag{F.5}$$

Next, the average extra travel time can be calculated using:

$$T = \frac{L}{u_2} - \frac{L}{u_1} \tag{F.6}$$

where u_1 is the free flow velocity (50 km/h) and u_2 the average velocity in congestion. The total cost of congestion is then calculated by multiplying T by the incoming intensity $q(t)$ and the value of time (VoT).

$$C = T \cdot q(t) \cdot VoT \tag{F.7}$$

Findings

It was found that, with current configurations, direction 1 and 3 did not cause congestion and are therefore not analysed further. For every scenario presented in table F.1 the congestion length has been calculated. As an indication, the congestion lengths for lane 2 and 4 have been presented for scenario 1 for different years (Figures F.6 - F.13).

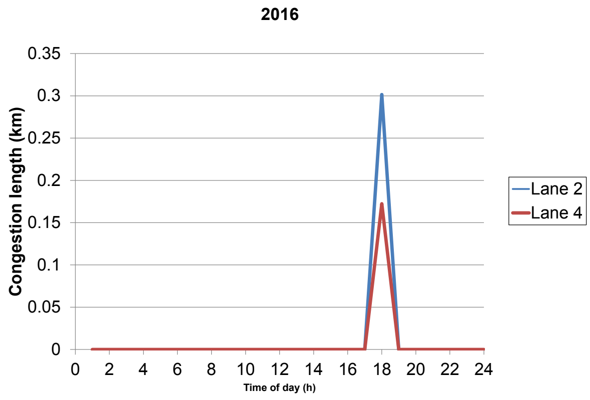

Figure F.6: Congestion length in 2016.

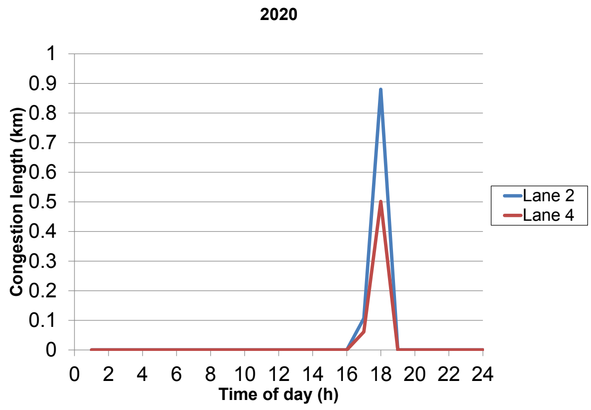

Figure F.7: Congestion length in 2020.

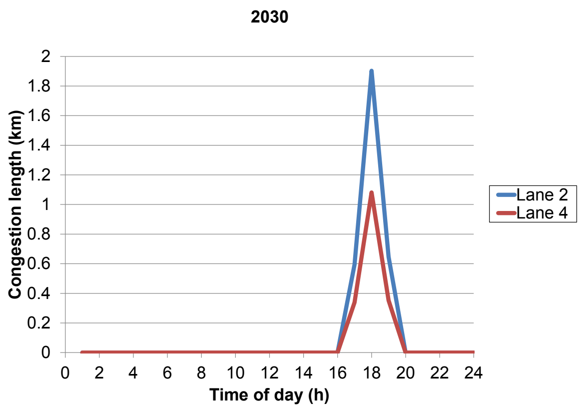

Figure F.8: Congestion length in 2030.

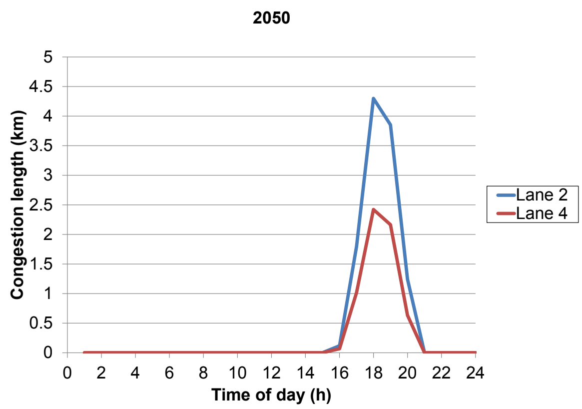

Figure F.9: Congestion length in 2050.

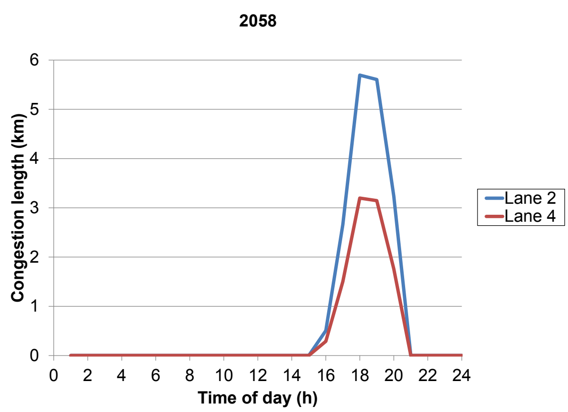

Figure F.10: Congestion length in 2058.

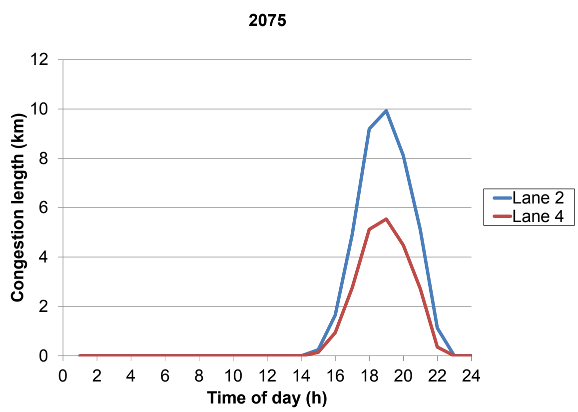

Figure F.11: Congestion length in 2075.

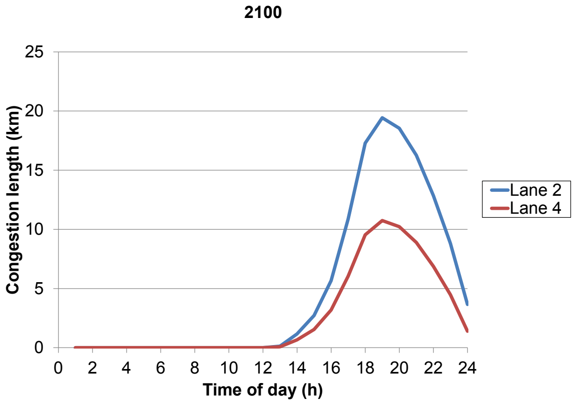

Figure F.12: Congestion length in 2100.

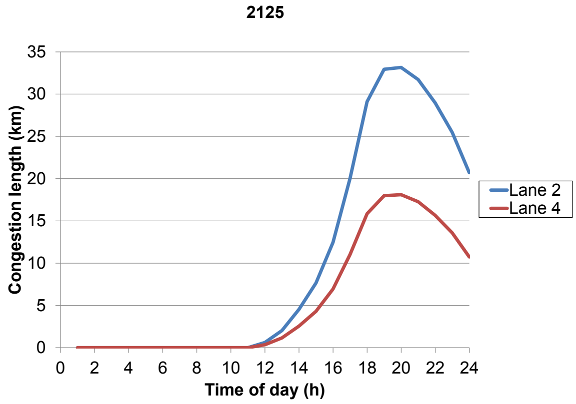

Figure F.13: Congestion length in 2125.

Next, using the equations presented above, the cost of congestion have been calculated. The Value of Time is considered to be 9 €/h (Kennisinstituut voor Mobiliteitsbeleid, 2013). The present value of societal congestion costs in each year are presented in figure F.14. The accumulated cost are presented in figure F.15. As decision makers will be most interested in the possible saved cost until 2058, the accumulated cost until this year have been presented again in figure F.16.

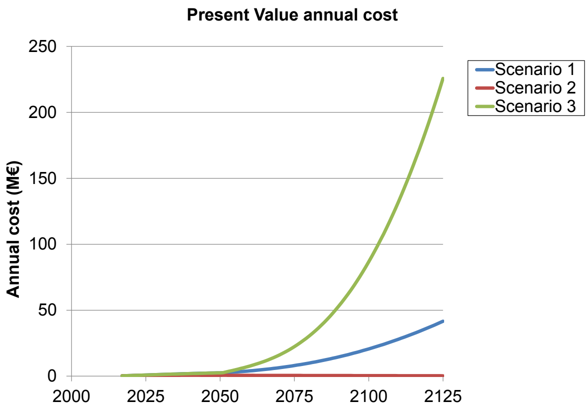

Figure F.14: Present value of societal cost (cash flows).

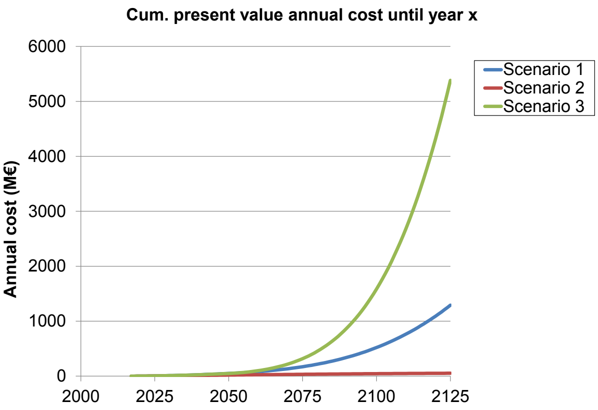

Figure F.15: Present value of societal cost (accumulated).

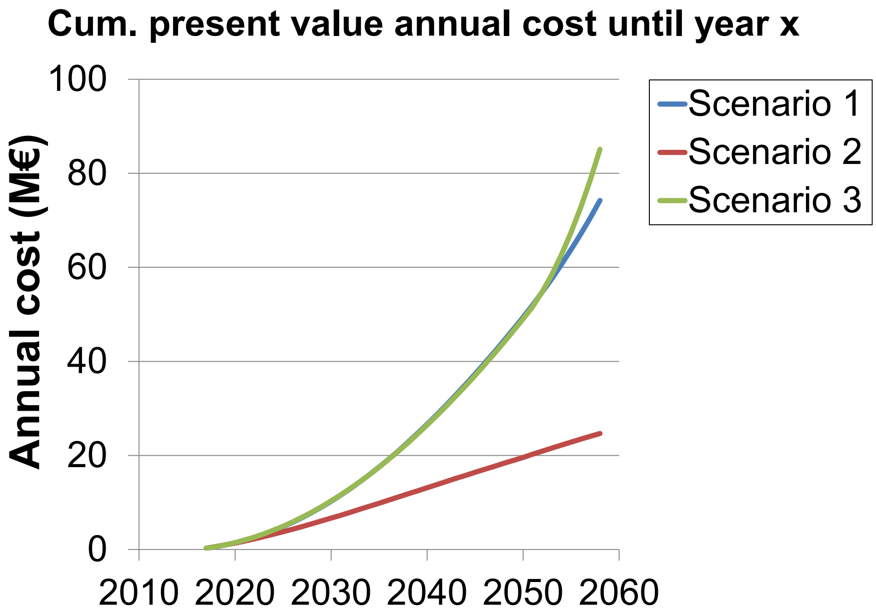

Figure F.16: Present value of societal cost until 2058 (accumulated).

Table F.3 shows the societal costs of the lack of capacity and the resulting congestion. By improving the capacity of the road connection in 2025 instead of in 2058, societal cost ranging from 20.9 (Scenario 2) upto 80.1 million euro (Scenario 3) could be averted. It should be noted that congestion occuring in the other directions, e.g. from the industrial area Stormpolder, towards the junction have not been taken into account and would lead to a further increase in societal cost.

In the second half of this century, the predictions start to become more uncertain and deviate significantly. The annual increase in traffic by 1% in scenario 3 due to urbanisation has a substantial influence. The length of the jam also creates gridlocks by blocking other junctions. E.g. in scenario 1 the jam has a length of 2 km in 2030, meaning that it reaches the connection of the N210 with the N219. Congestion lengths upto 10 km in 2075 in scenario 1 show the seriousness of the problem.

Table F.3: Societal costs congestion junction in million Euro.

Scenario	2020 [M€]	2025 [M€]	2050 [M€]	2058 [M€]	2100 [M€]	2125 [M€]
1	2.0	4.9	49.4	74.2	520.4	1,291.1
2	1.8	3.8	19.6	24.7	45.3	53.5
3	2.0	5.0	49.0	85.1	1,583.7	5,381.5

F.5 Possible new road trajectory

The previous sections have shown that the current road connection is insufficient. The junction in Krimpen a/d IJssel is most problematic. Unfortunately, there is insufficient space available to expand the road capacity. Therefore, a by-pass is suggested, which is illustrated in figure F.17.

Figure F.17: Proposed new trajectory N210.

G

Water in- and outlets

Figures based on Seip (2012), De Nederlandsche Gemalen Stichting (nd), Hoogheemraadschap van Schieland en de Krimpenerwaard (nda), Rijkswaterstaat (2005) and Revet (nd).

Table G.1: Outlet capacity of different pumping stations.

Pumping station	Out [m^3/s]	Out [m^3/min]	Water board
De Waaier	7.5	450	HHdSR
Waaiersluis and Spuisluis	14	840	HHdSR
Willens Goejanverwelledijk	0.9	54	HHR
Hanepraai	1.2	70	HHR
Mallegat	1.3	80	HHR
mr. Pijnacker Hordijk	35	2100	HHR
Stolwijkersluis	0.8	50	HHSK
Verdoold, M.C. zn	7.5	450	HHSK
Middelblok	0.5	30	HHSK
Westeinde	0.2	10	HHSK
Abraham Kroes	14.2	850	HHSK
Boezemland	0.1	6	HHSK
de Nesse	0.7	40	HHSK
Hitland	1.3	75	HHSK
Johan Veurink	5	300	HHSK
Oostgaarde	0.8	50	HHSK
Torenhof	0.0	0.5	HHSK
Middelwatering	1.5	90	HHSK
Waste water treatment	0.7	40	HHSK/HHR
Total	**93.1**	**5585.5**	
Stormpolder gemaal A*	0	1.7	
Stormpolder gemaal B*	0.1	3.3	

*West of Algera barrier

Table G.2: Inlet apacity of different pumping stations.

Pumping station	In [m^3/s]	In [m^3/min]	Water board
Waaiersluis	5	300*	HHdSR
mr. Pijnacker Hordijk	35	2100**	HHR
Johan Veurink	1.3	80	HHSK
Snelle Sluis	2.5	150	HHSK
Total	**43.8**	**2630**	

*Daily averaged, maximum intake 1800 m^3/min

** Normal conditions. Maximum intake 2700 m^3/min (short period)

H

The barrier under normal conditions

The water table at both sides of the barrier develops over time, due to sea level rise and subsidence. The basic assumption for the water level behind the barrier is that the water level should follow the rate of subsidence of the dikes. Additionally, a tidal amplitude has to be maintained from an ecological perspective. This amplitude ranges from 50 cm (minimum) to 75 cm (current). From this, four situations are introduced:

1. Fast SLR, minimum amplitude;
2. Fast SLR, maximum amplitude;
3. Moderate SLR, minimum amplitude;
4. Moderate SLR, maximum amplitude.

Figure H.1 illustrates situation/scenario 3. The design water level is defined as the water level that is allowed to be reached by tidal action. On the right side of each image it can be seen that design water level is lowered 110 cm per century, following the settlement of the dikes. Furthermore, the tidal amplitude is reduced to 50 cm. It can be seen that, at least until 2150, there is still a margin between the tide and the design water level. In other scenarios this is not the case, see the following paragraph.

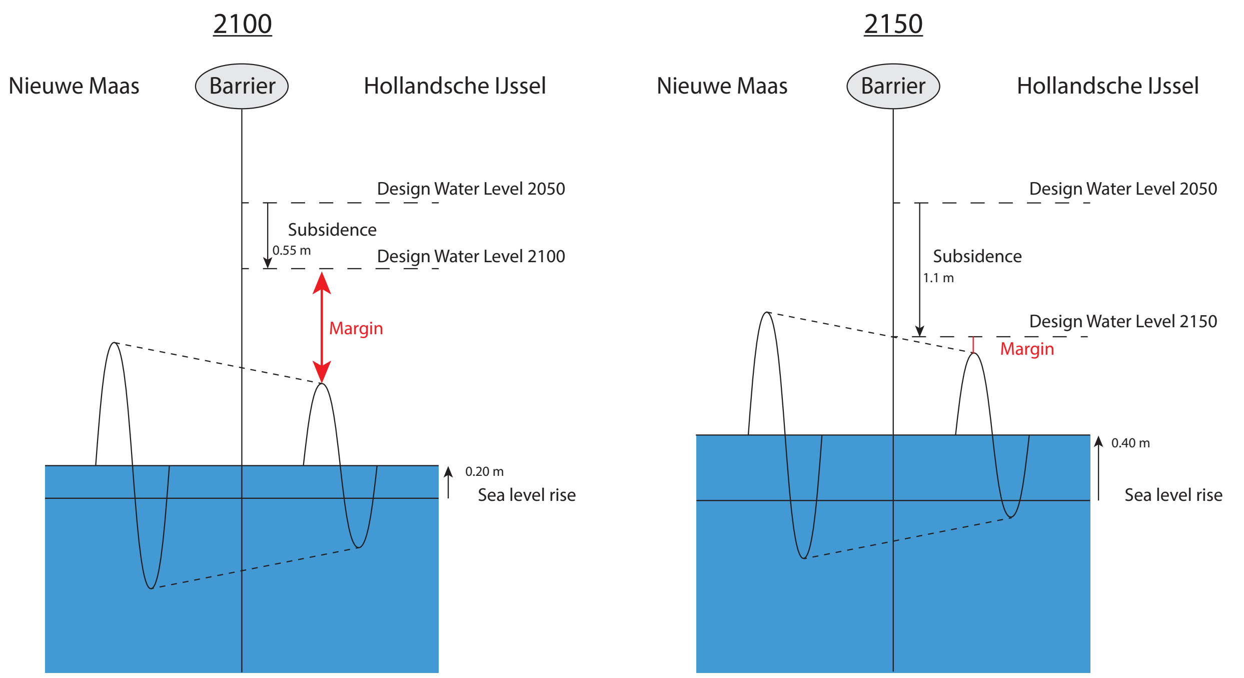

Figure H.1: Water table development for scenario 3

H.1 When is pumping capacity in normal conditions needed?

The design water level is determined by the water level related to the proven strength at time *t* minus the water level increase caused by influx of water from the polders (table H.1). This water level increase is calculated in HYDRA-BS by substraction of Design Closure Level from the occurring water level if the barrier has no probality of failure (P_f=0). On average, the water level increases an extra 0.56 m. The decrease in design water level DWL is dependent on the rate of subsidence S:

$$DWL(t) = DWL_{2050} - S \cdot t \tag{H.1}$$

Table H.1: Maximum allowed water levels in the Hollandsche IJssel

	2050	2100	2150	2200
Max water level [m NAP]	3.22	2.67	2.12	1.57
Design water level [m NAP]	2.66	2.01	2.56	1.01

Without pumps, the low water level in the HIJ can never reach lower values than low water in the Nieuwe Maas. The Mean High Water in the Hollandsche IJssel (MHW_{HIJ}) can therefore be calculated by adding the desired tidal range to Mean Low Water of the Nieuwe Maas (MLW_{NM}) at year t:

$$MHW_{HIJ} = MLW_{NM}(t) + 2 \cdot amplitude \tag{H.2}$$

where MLW_{NM} at t is given by adding the expected sea level rise (SLR) to the MLW in 2050.

$$MLW_{NM}(t) = MLW_{NM,2050} + SLR \cdot t \tag{H.3}$$

Equating equations H.1 and H.2 gives the point in time when pumping capacity is needed to maintain tide. Figures H.2 and H.3 show the development of SLR and subsidence for the different scenarios.

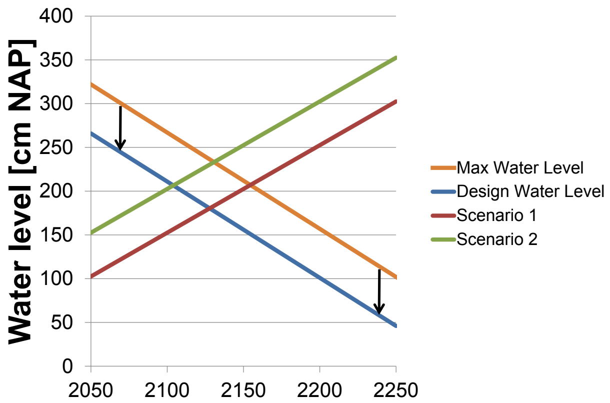

Figure H.2: Water table development for scenario 1 and 2

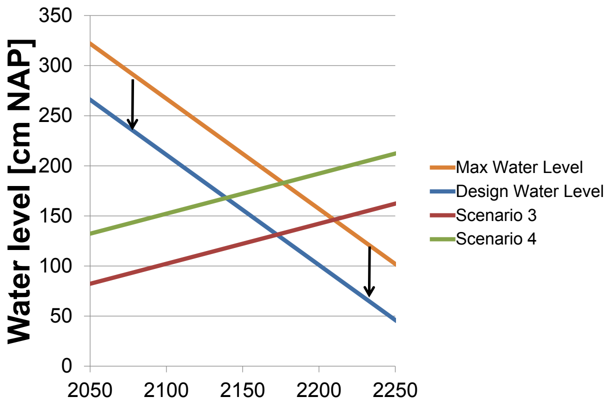

Figure H.3: Water table development for scenario 3 and 4

Table H.2: Pumping capacity needed, different scenarios based on KNMI (2015) and HHSK (2016)

Scenario	SLR [m/100y]	Subsidence [m/100y]	Tidal range [m]	Year pumps needed
1	1.0	1.1	1.0	2128
2	1.0	1.1	1.5	2105
3	0.4	1.1	1.0	2173
4	0.4	1.1	1.5	2140

H.2 First approximation pump capacity

A first approximation of the required capacity is calculated using a rectangular crossection of the Hollandsche IJssel.

$$Q = \frac{WhL}{0.5T} \tag{H.4}$$

with W=100 m, h=1.5 m, L=17km and T=12h 25m (M2 tide), this leads to a required pumping capacity of approximately 114 m^3/s. By comparison, the largest pumping station in the Netherlands (IJmuiden) has a capacity of 260 m^3/s (Stam, 2004).

The required pumping capacity is calculated by:

$$P = \rho g Q \Delta H \tag{H.5}$$

To ensure a life time of 100 years in the scenario 2 (worst scenario), pumps are needed the last 45 years (see table H.2). The difference in water level in 2150 is given by:

$$\Delta H = (S + SLR) \cdot t = (0.011 + 0.01) \cdot 45 = 0.945 \quad m \tag{H.6}$$

Here, S is the subsidence per year and SLR the sea level rise. In this case, the required power is 1,058 kW, similar to the power produced by a modern windmill in optimal conditions. It can therefore be concluded from this first approximation that maintaining tidal amplitude with a pumping station is technically feasible and economically attainable. Furthermore, the influx of water could be used to produce a part of the power needed. Design alterations should be made to accommodate power generation. These alterations are out of the scope of this thesis and the notion of power generation serves as a recommendation for future research.

Results HYDRA-BS Calculations

This appendix shows predicted water levels for a barrier that has a span similar to the current Algera barrier. A barrier with a failure probability $P_{f,SSB}$ of 1:10,000 and 1:100,000 has been simulated for different design closure levels (DCLs). HYDRA-BS only takes into account a non-closure failure into account: partial failure is not accounted for. The results presented in this appendix show that a barrier that spans the entire river without any dike improvements after 2050 is not sufficient to protect its hinterland. The results presented in sections I.1 & I.2 are the outer data points in HYDRA-BS. Figure I.1 shows that KM02 and KM16 also represent the extreme values.

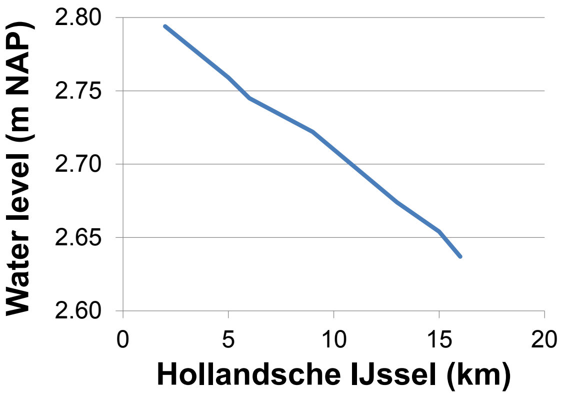

Figure I.1: Water levels in 2100. Return period 10,000 years, DCL 1.65 m.

I.1 KM02

Table I.1: Water levels at KM02 in the Hollandsche IJssel in 2100. P_f 1:10,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:10,000	2.25	2.786	2.67	0.958
DRUK/RUST	1:10,000	2.00	2.786	2.67	0.958
DRUK/RUST	1:10,000	1.65	2.786	2.67	0.958
WARM/STOOM	1:10,000	2.25	3.059	2.67	0.873
WARM/STOOM	1:10,000	2.00	3.061	2.67	0.872
WARM/STOOM	1:10,000	1.65	3.060	2.67	0.873

Table I.2: Water levels at KM02 in the Hollandsche IJssel in 2100. P_f 1:100,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:100,000	2.25	2.772	2.67	0.963
DRUK/RUST	1:100,000	2.00	2.562	2.67	1.042
DRUK/RUST	1:100,000	1.65	2.451	2.67	1.089
WARM/STOOM	1:100,000	2.25	2.789	2.67	0.957
WARM/STOOM	1:100,000	2.00	2.79	2.67	0.957
WARM/STOOM	1:100,000	1.65	2.79	2.67	0.957

Table I.3: Water levels at KM02 in the Hollandsche IJssel in 2150. P_f 1:10,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:10,000	2.25	2.986	2.12	0.710
DRUK/RUST	1:10,000	2.00	2.986	2.12	0.710
DRUK/RUST	1:10,000	1.65	2.986	2.12	0.710
WARM/STOOM	1:10,000	2.25	3.559	2.12	0.596
WARM/STOOM	1:10,000	2.00	3.561	2.12	0.595
WARM/STOOM	1:10,000	1.65	3.56	2.12	0.596

Table I.4: Water levels at KM02 in the Hollandsche IJssel in 2150. P_f 1:100,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:100,000	2.25	2.972	2.12	0.713
DRUK/RUST	1:100,000	2.00	2.762	2.12	0.768
DRUK/RUST	1:100,000	1.65	2.651	2.12	0.800
WARM/STOOM	1:100,000	2.25	3.289	2.12	0.645
WARM/STOOM	1:100,000	2.00	3.29	2.12	0.644
WARM/STOOM	1:100,000	1.65	3.29	2.12	0.644

I.2 KM16

Table I.5: Water levels at KM16 in the Hollandsche IJssel in 2100. P_f 1:10,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:10,000	2.25	-	2.67	-
DRUK/RUST	1:10,000	2.00	2.633	2.67	1.014
DRUK/RUST	1:10,000	1.65	2.633	2.67	1.014
WARM/STOOM	1:10,000	2.25	2.905	2.67	0.919
WARM/STOOM	1:10,000	2.00	2.905	2.67	0.919
WARM/STOOM	1:10,000	1.65	2.907	2.67	0.918

Table I.6: Water levels at KM16 in the Hollandsche IJssel in 2100. P_f 1:100,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:100,000	2.25	-	2.67	-
DRUK/RUST	1:100,000	2.00	2.378	2.67	1.123
DRUK/RUST	1:100,000	1.65	2.188	2.67	1.220
WARM/STOOM	1:100,000	2.25	2.637	2.67	1.013
WARM/STOOM	1:100,000	2.00	2.638	2.67	1.012
WARM/STOOM	1:100,000	1.65	2.637	2.67	1.013

Table I.7: Water levels at KM16 in the Hollandsche IJssel in 2150. P_f 1:10,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:10,000	2.25	-	2.12	-
DRUK/RUST	1:10,000	2.00	2.833	2.12	0.748
DRUK/RUST	1:10,000	1.65	2.833	2.12	0.748
WARM/STOOM	1:10,000	2.25	3.405	2.12	0.623
WARM/STOOM	1:10,000	2.00	3.407	2.12	0.622
WARM/STOOM	1:10,000	1.65	3.407	2.12	0.622

Table I.8: Water levels at KM16 in the Hollandsche IJssel in 2150. P_f 1:100,000

Climate Scenario	$P_{f,SSB}$	DCL	Water level (m NAP)	Norm (m NAP)	Unity Check
DRUK/RUST	1:100,000	2.25	-	2.12	-
DRUK/RUST	1:100,000	2.00	2.578	2.12	0.822
DRUK/RUST	1:100,000	1.65	2.388	2.12	0.888
WARM/STOOM	1:100,000	2.25	3.137	2.12	0.676
WARM/STOOM	1:100,000	2.00	3.138	2.12	0.676
WARM/STOOM	1:100,000	1.65	3.137	2.12	0.676

J

First order analytical calculation required cross-section

Dynamics of flow due to tidal action are described by the shallow water equations: the continuity and momentum balance equations result in a set of partial differential equations (Battjes and Labeur, 2014). The set of equations can be simplified by discrete modeling if certain conditions are met and simplifications are made.

- Storage and transport of water can be separated. The tidal opening transports and the river serves as storage. Inertia in the channel is neglected.
- The length of the storage basin is relatively short in comparison with the tidal wave.
- The tidal wave is sinusoidal.

Although this is not entirely true for the Hollandsche IJssel, e.g. water levels vary from place to place, the simplification can be used as a first estimate. A full analysis, using the full set of equations, requires a numerical modelling with a program such as SOBEK. The discrete system simplified is described by the following equations:

$$\Gamma = \frac{8}{3\pi}\chi\left(\frac{A_b}{A_c}\omega\right)^2\frac{\hat{\xi}_{NM}}{g} \tag{J.1}$$

$$A_c = \frac{A_b\omega}{\sqrt{\Gamma\frac{3\pi g}{8\chi\hat{\xi}_{NM}}}} \tag{J.2}$$

$$r = \frac{\hat{\xi}_{HIJ}}{\hat{\xi}_{NM}} = \frac{1}{\sqrt{2}\Gamma}\sqrt{-1+\sqrt{1+4\Gamma^2}} \tag{J.3}$$

Where:

$$\chi = \frac{1}{2} + c_f\frac{l_c}{R}$$

This leads to:

$$4(r\Gamma)^4 = \left(-1+\sqrt{1+4\Gamma^2}\right)^2 \tag{J.4}$$

$$\Gamma = \frac{\sqrt{1-r^2}}{r^2} \tag{J.5}$$

In first instance, friction of the tidal opening is not taken into account (c_f=0). The results are presented in table J.1.

Table J.1: First approximation tidal opening needed

$\hat{\xi}_{NM}$ [m]	$\hat{\xi}_{HIJ}$ [m]	r [-]	HSI [-]	Γ	A_c [m^2]
0.75	0.740	.987	0.80	0.167	106.0
0.75	.675	.900	0.58	0.538	58.7
0.75	0.600	.800	0.33	0.796	44.5
0.75	0.500	0.667	0	2.350	33.3

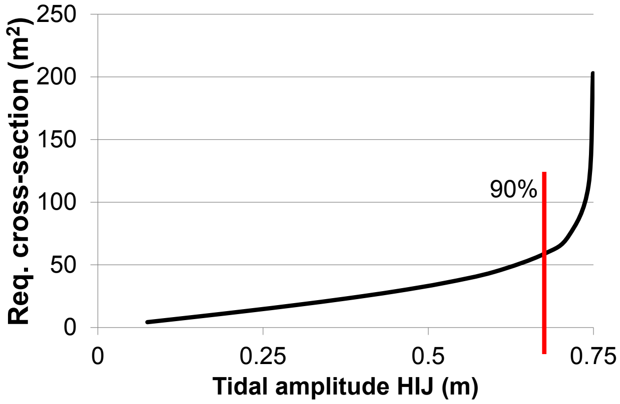

Figure J.1: Rapid increase required cross-section (no friction).

Figure J.1 shows that the required cross-section increases rapidly as the tidal amplitude in the reaches its original value. In this stage of design a preliminary choice has to made on the amplitude. Due to the asymptotic increase, 90% of the original tidal amplitude (0.675 m) is chosen as upper value.
When including friction into the calculation, calculation of A_c through equation J.2 becomes an iterative process. Table J.2 shows the result for different lengths of channels (barrier openings). The approximated dimensions of the channels have been incorporated as well.

Table J.2: First approximation tidal opening needed, prismatic with c_f=0.004 (Battjes and Labeur, 2014, p.19)

l_c [m]	b_c*h_c $[m^2]$	$\hat{\xi}_{NM}$ [m]	$\hat{\xi}_{HIJ}$ [m]	r [-]	Γ	A_c $[m^2]$
10	≈ 12.0*5	0.75	0.675	0.90	0.538	60.0
25	≈ 12.4*5	0.75	0.675	0.90	0.538	61.9
50	≈ 13.0*5	0.75	0.675	0.90	0.538	64.9
100	≈ 14.1*5	0.75	0.675	0.90	0.538	70.3
10	≈ 6.8*5	0.75	0.50	0.67	1.677	34.1
25	≈ 7.1*5	0.75	0.50	0.67	1.677	35.4
50	≈ 7.5*5	0.75	0.50	0.67	1.677	37.4
100	≈ 8.2*5	0.75	0.50	0.67	1.677	40.9

Analytical calculation for SOBEK
The first order analysis led to a required cross-section of 70 m^2, see table J.2. In this calculation an M2-tide was used (T=12h25m). In order to compare the analytical results with SOBEK results, the analytical calculation was performed again with a tidal period of 12h. For a 100 m long rectangular culvert, a cross-section of 72.6 m^2 is required to maintain 90% of the tide (r=0.90). Dimensions of the culvert (WxHxD) are 14.5x5*50 m.

K

SOBEK calculation required cross-section

K.1 Introduction

In this Appendix a closer look will be taken at the realised tide in the Hollandsche IJssel, when different measures are implemented. In order to do so, SOBEK 3 will be used for schematisation. SOBEK 3 is a relatively new version. The latest schematisation of the Dutch delta (Rijn-Maasmodel (RMM)) and it is therefore a logical step to use this latest version of both SOBEK and RMM in this thesis. The impact of two measures will be considered, namely: (1) narrowing of the river, (2) an inlet structure. First the impact of both measures will be assessed using simplified models (sections K.3 - K.5). After that, the structure and narrowing will be implemented in a more realistic representation of the river (section K.6).

K.2 Resonance from the length of the basin

Resonance in a tidal basin occurs when:

$$k_0 \cdot l = \frac{\pi}{2} + 2\pi \cdot i \quad with \quad (i = 0, 1, 2 \ldots n) \tag{K.1}$$

$$k_0 = \frac{\omega}{c_0} \tag{K.2}$$

$$\omega = \frac{2\pi}{T} = 1.45 \cdot 10^{-4} \tag{K.3}$$

$$c_0 = \sqrt{\frac{gA_c}{B}} = \sqrt{9.81 * 7} \quad = 8.29 \quad [m/s] \tag{K.4}$$

$$k_0 l = \frac{1.4 \cdot 10^{-4}}{8.29} \cdot 18000 = 0.316 << 1 \tag{K.5}$$

Battjes and Labeur (2014, p.158) shows that the found 0.316 corresponds to amplification of the tide at the closed end of the basin of approximately 10%.

K.3 The simple model

To get a grasp of the effects of both narrowing the river and placement of a structure, a simple 1D-model with a prismatic channel is set up. The properties of the model are presented in table K.1. The tide is triangular of form.

Table K.1: Settings for SOBEK calculations

Situation	Value	Unit
Width	100	m
Depth	7	m
Chézy coëfficient	45	$m^{1/2}/s$
Tidal period	12	h
High water 'mouth'	+0.75	m NAP
Low water 'mouth'	-0.75	m NAP
Calculation interval	5	min
Grid size	500	m

A dry run of the model showed resonance at the closed end of the basin of 9.8%, see table K.2. This is in line with the prediction made in section K.2.

Table K.2: Resonance in the Hollandsche IJssel

Distance from open end [m]	ξ_{HIJ} [m]	Increase [%]
0	0.75	n.a.
9,000	0.78	4.3
18,000	0.83	9.8

K.4 Simple model: the effect of narrowing a river

The narrowing of estuaries land inward could either lead to damping or excitation of tide inside the estuary. On one hand convergence of the banks leads to excitation, on the other hand (bed) friction results in dampening of the tidal wave (Savenije, 2012). In tidal river systems the same processes take place. Here, a reduced width of the river can lead to dampening and excitation as well. Savenije (2012) presents extensive calculative procedures to analyse tidal wave propagation into estuaries. In order to perform this analysis, the full estuary, including all other river branches, should be taken into account. This is considered to be beyond the scope of this thesis. To show the effect of narrowing of a river, an analysis has been carried out in SOBEK. Here, the Hollandsche IJssel was schematised by a 100 m wide prismatic channel with a length of 18 km and a water depth of 7 m. Eight situations of narrowing the river were compared to a reference situation. The situations have been presented in table K.3. A distinction has been made between instantaneous and gradual narrowing. With an instantaneous narrowing, a reduced profile over almost the entire river is meant (except close to the mouth). In case of gradual narrowing, the cross-section gradually funnels towards the reduced profile at the closed end of the river.

Table K.3: Different tested situations of narrowing.

Situation	Channel width [m]	Floodplain width [m]	Type of narrowing [-]
0	100	0	n.a.
1	50	50	Instant
2	50	50	Gradual
3	25	75	Instant
4	25	75	Gradual
5	50	0	Instant
6	50	0	Gradual
7	25	0	Instant
8	25	0	Gradual

Table K.4: Results tested situations.

Situation	18,000 [m]	9,000 [m]	Increase [%]	0 [m]	Increase [%]
0	1.5	1.56	n.a.	1.65	n.a.
1	1.5	1.55	-0.7	1.65	0.1
2	1.5	1.55	-0.9	1.65	0.3
3	1.5	1.38	-11.6	1.45	-12.2
4	1.5	1.58	0.7	1.65	0.1
5	1.5	1.56	-0.1	1.64	-0.4
6	1.5	1.54	-0.1	1.61	-2.4
7	1.5	1.56	-0.3	1.64	-0.4
8	1.5	1.57	0.4	1.62	-1.4

It can be seen that only in situation 3 (direct narrowing with large flood plains and a small channel) significant damping occurs compared to the reference or 0-situation. This damping can be explained by the low water levels over the large floodplains which are subjected by the bottom friction of this flood plain.

K.4.1 Implication for the case

Table K.4 showed that in only one of the eight analysed cases (case 3) the tide was seriously dampened due to narrowing of the river. In this single case the channel was reduced by 75% and included a large floodplain. In the Hollandsche IJssel, the navigable width should be at least 51 m (CEMT V class, see section x). Since the average width of the river is 100 m, on average the channel can be reduced upto 50%. Narrowing the Hollandsche IJssel would therefore be more alike case 1 or 2. In case 1 and 2 the influence of the narrowing is very limited (±1%, see table K.4). It is therefore likely that narrowing the river will not significantly influence the river.

K.5 Simple model: the effect of an inlet structure

In this section the effect of an inlet structure is assessed. Goal is to create a structure with such dimensions that 90% of the original tide is maintained. The required cross-section of the inlets is calculated with the simple model presented in section K.3.

K.5.1 Modelling losses in SOBEK

When using programs as SOBEK, one should be aware of how to fill in the parameters. In SOBEK, the discharge coefficient μ for a culvert is calculated by the following formula (Deltares, 2015, p.29):

$$\mu = \frac{1}{\sqrt{\xi_i + \xi_o + \xi_f + \xi_v}} \tag{K.6}$$

Here, ξ_i is the entrance loss coefficient, ξ_o is the exit loss coefficient, ξ_f is the friction loss coefficient and ξ_v is the valve loss coefficient. A valve loss will not be modelled and will therefore not be discussed into further detail. Although not mentioned in the manual (Deltares, 2015) a bend loss coefficient ξ_b is also a parameter that can be adjusted. In this subsection, four parameters will be discussed, namely:

1. Entrance loss coefficient;
2. Exit loss coefficient;
3. Friction loss coefficient;
4. Bend loss coefficient.

To give an estimation of each factor, a rectangular culvert with dimensions WxHxL = 10x4x50 m is used.

Entrance loss coefficient (ξ_i)
The entrance loss can only be filled in as a constant value (Deltares, 2015, p.29). Its default value is 0.1. However, Nortier and de Koning (1998) mention values between 0.6 and 0.8. Therefore, a value of 0.7 is used.

Exit loss coefficient (ξ_o)
For submerged flow, the exit loss coefficient is calculated by:

$$\xi_o = k\left(1 - \frac{A_{fc}}{A_{fr}}\right)^2 \tag{K.7}$$

Here, k is a user defined exit loss coefficient, A_{fr2} is the flow area downstream of the culvert and A_{fc} is the flow area in the culvert. In this thesis, the exit loss will be fully taken into account. Therefore, a k-value of 1 is used. The downstream flow area is large compared to the area of the culvert. Using k=1, for the culvert, with dimensions WxHxL =10*4*50 m, this leads to a ξ_0 of 0.89. For smaller culverts, values of ξ_o around 1 are to be expected.

Friction loss coefficient (ξ_f)
The friction loss is dependent on the roughness C of the culvert, its length L and the hydraulic radius R. The coefficient is calculated by:

$$\xi_f = \frac{2gL}{C^2 R} \tag{K.8}$$

For a rectangular culvert, with dimensions WxHxL = 10x4x50 m and C=80 (n=0.013), this will lead to a ξ_f of 0.28. Compared to the inlet and outlet loss coefficients, the friction loss coefficient is small.

Bend loss coefficient (ξ_b)
Although not mentioned in the manual (Deltares, 2015), the bend loss coefficient ξ_b is a parameter that can be filled in. By default, it is set to 99.9, leading to large losses. This is an error that was confirmed by Deltares support (de Wit, 2016) and should be set to 0.

Estimation μ-value
Using the formulas and assumed values presented in this section, μ can be estimated. For a rectangular culvert, with dimensions WxHxL =10*4*50 m,

$$\mu = \frac{1}{\sqrt{0.7 + 0.89 + 0.21}} = 0.77 \tag{K.9}$$

For smaller culverts, a μ-value of approximately 0.7 is expected.

Summary
The inlet and outlet loss coefficients are comparable in size. The friction loss is small compared to the inlet and outlet losses. The bend loss should be set to 0. The example culvert resulted in a μ of 0.77. For smaller culverts, μ-values around 0.7 are to be expected.

K.5.2 Adaptations made in SOBEK

The analytic calculation (Appendix J) was a simplification of reality. The SOBEK calculation has been made more realistic by altering the following:

1. Alter the friction to realistic values;
2. Introduce entrance and exit losses;
3. Introduce pipes instead of a rectangular cross-section.

Friction values
A rough estimate of the friction was made in the analytic analysis. In the first order calculation, a c_f of 0.004 has been used.

$$Q = A \cdot C\sqrt{R \cdot i} \qquad (Ch\acute{e}zy) \tag{K.10}$$

For steady, uniform flow the discharge is given by (Battjes and Labeur, 2014, p. 16):

$$Q = A\sqrt{\frac{gRi}{c_f}} \tag{K.11}$$

Elimination of Q leads to:

$$C = \sqrt{\frac{g}{c_f}} \qquad and \qquad n = \frac{R^{\frac{1}{6}}}{C} \tag{K.12}$$

Using c_f=0.004, L=100 m and T=12h in Appendix J led to a required cross-section of 72.6 m^2 (5*14.52 m). This value corresponds with the Chézy coefficient C of 49.52 or manning's n of 0.022, which will be used to compare the analytic results with the result from the simple model (table J.2).

For steel closed conduits a manning's n of 0.012 is normal. For concrete unfinished steel form 0.013 is normal. As this is a preliminary design, calculations with 0.013 are made only (Te Chow, 1959). Compared to the first order calculations, there will be less wall friction.

Entrance and exit losses
The entrance loss varies between 0.6 and 0.8 (Nortier and de Koning, 1998). For this modelling an entrance loss coefficient of 0.7 will be used. The exit loss is less clear. The SOBEK manual mentions an exit loss coefficient of 1, taking exit losses fully into account. As this is a preliminary design, this value will be used.

It is advised to further investigate the entrance and exit losses when a further detailing the dimensions of the inlet structure (further research).

Pipes
Several openings instead of one were chosen to limit the effects of single door failure. A pipe solution was chosen because pumps could be introduced without mayor design changes. Compared to the first order calculation, there will be more friction as the cross-section is split up.

K.5.3 Results simple model

In total, more than 30 alterations were carried out to get a grasp of the influence of the various factors. The most relevant results are presented in table K.5. Table K.5 presents the realised tide behind the barrier. Two points are taken into account: (1) half-way the basin (9,000 m) and (2) at the closed end of the basin (0 m). Model 0 represents the 0-situation in which no inlet structure is modelled.

Table K.5: Results tested situations

Model	L [m]	Type	A [m^2]	No. [-]	A_{total}	n	ξ_i	k	9,000 [m]	r [-]	0 [m]	r [-]
0	n.a.	n.a.	n.a.	n.a.	n.a.	0.022	n.a.	n.a.	1.56	n.a.	1.65	n.a.
1	100	Rect.	72.6	1	72.6	0.022	0	0	1.36	0.87	1.42	0.86
2	100	Rect.	85.0	1	85.0	0.022	0	0	1.41	0.90	1.48	0.90
3	50	Rect.	85.0	1	85.0	0.022	0	0	1.41	0.90	1.48	0.90
4	50	Pipe	5	17	85.0	0.022	0	0	1.41	0.90	1.48	0.90
5	50	Pipe	5	17	85.0	0.022	0.7	1	1.41	0.90	1.48	0.90
6	50	Pipe	5	20	100	0.022	0.7	1	1.41	0.90	1.48	0.90
7	50	Pipe	5	21	105	0.022	0.7	1	1.42	0.91	1.50	0.91
8	50	Pipe	5	21	105	0.013	0.7	1	1.46	0.93	1.53	0.93
9	50	Pipe	5	20	100	0.013	0.7	1	1.45	0.93	1.52	0.92
10	50	Pipe	5	19	95	0.013	0.7	1	1.43	0.92	1.51	0.91
11	50	Pipe	5	18	90	0.013	0.7	1	1.42	0.91	1.49	0.90
12	50	Pipe	5	17	85	0.013	0.7	1	1.40	0.90	1.47	0.89

Models 1-4
Acccording to the analytic calculation (Appendix J), a cross-section of 72.6 m^2 is required to maintain 90% of the tide (r=0.9). Models 1-4 are used to compare the analytic results. Model 1 models the same cross-section in SOBEK. Here an r-factor of 0.87 is obtained, which is is close to the found 0.9 in the analytic first order approximation. In models 2 & 3 the cross-section is increased to reach an r-value of 0.9. It can be seen that reducing the length of the culvert is of very little influence to the tide. Also, the introduction of pipes is of little influence to the realised tide behind the structure

Models 5-7
In models 5 through 7 the inlet and outlet losses are introduced. Here, again the losses are of little influence. Models 6 and 7 show that increasing the tidal range above 90% is difficult to achieve, in concurrence with figure J.1.

Models 8 - 12
In models 8 through 12, the friction is adjusted to 0.013, according in to section K.5.2. As expected, the realised tidal range behind the structure increases. From the table it can be seen that model 11 leads to r-values above 0.90. The cross-section in this model, 90 m^2 will be used as starting point for further analysis (section K.6).

K.6 Implementation in realistic situation

Now that the influence of both the extra intertidal areas and the inlet structure have been shown, structure and narrowing can be inserted in the Rijn-Maasmonding (RMM) model. This section will first describe the model used (section K.6.1), followed by a short analysis on the accuracy of the model (section K.6.2). After that, the results of implementation of the inlet structure into the RMM will be presented (section K.6.3), followed by a combination of the structure and the extra intertidal areas (section K.6.4).

K.6.1 Description of the model used

The model used is based on the Rhine Meuse Model or RMM. This model is provided by Rijkswaterstaat and is a detailed schematisation of the Dutch delta. To reduce calculation times, the Hollandsche IJssel branch of the model was distilled. The model consists of 40 cross-sections, distributed almost evenly over the entire length (18 km) of the river analysed. At the mouth, the tide measured at Krimpen a/d IJssel was used as a boundary condition. Figure K.1 shows the depth profile of the Hollandsche IJssel. It can be seen that the river becomes shallower near Gouda.

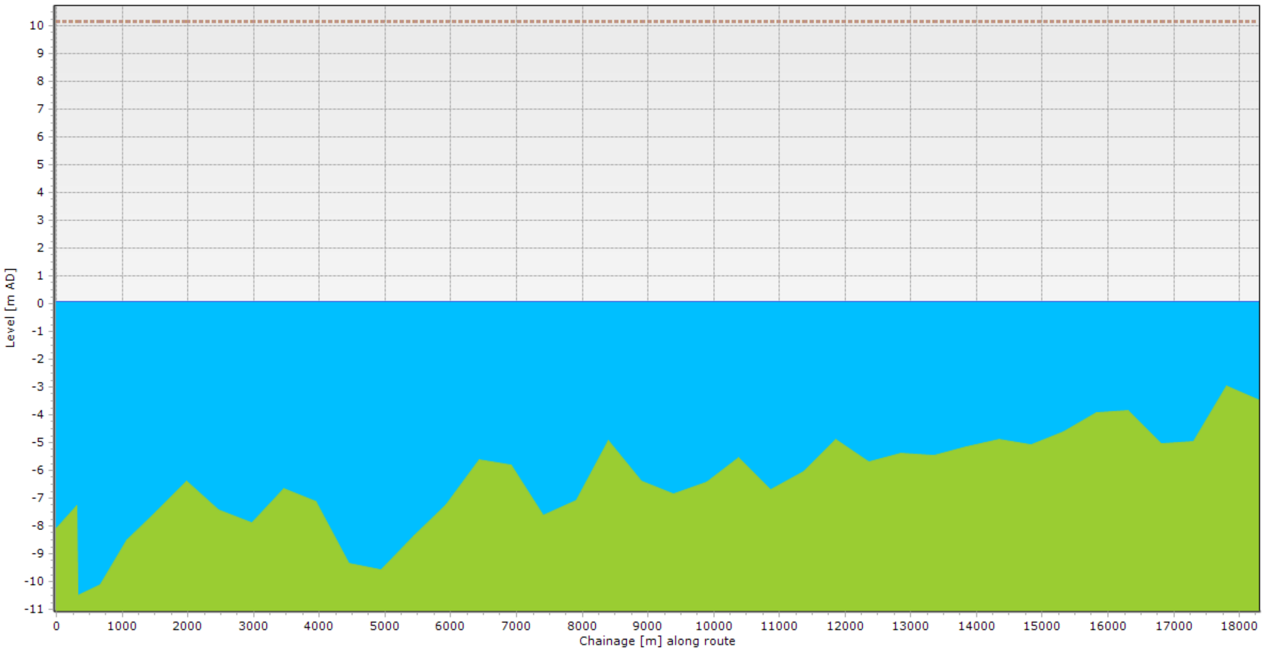

Figure K.1: Length profile of the Hollandsche IJssel in SOBEK. The left side represents Krimpen/Capelle, the right side Gouda.

K.6.2 Accuracy of the model

To check the accuracy of the model, a dry run was performed with the water levels recorded at Krimpen a/d IJssel between 1 and 8 september 2016 (Rijkswaterstaat, ndd). The resulting water levels at Gouda were compared with the water levels measured at "Gouda Brug" (Rijkswaterstaat, ndd).

Results

It was found that the results show a phase lag of 2 hours and 40 minutes: in reality high water reaches Gouda two hours earlier than in the model. As the focus of this thesis is not on when tide occurs, but to what extent. No further attention is given to this aspect.

After correction for the phase lag, it was found that the average absolute error in water level is 0.0356 m. Furthermore, the high water and low water peaks were analysed. In general, high water peaks were predicted too low and low water troughs too high. The results can be found in table K.6. Adding the errors of high and low water leads to an under prediction of the tidal range for this month of 0.042 m. This may seem little, but leads to an underestimation of the tide by 3%! Since it is the aim to reduce the tide by a maximum of 10%, including this error in the results limits the 'playfield' by almost a third. To verify the reported error, an analysis over a longer period of time should be made. This is a tedious job and considered outside the scope of this thesis. Another (partial) explanation of the error may lie within the measuring accuracy of the water levels: the water level is measured in centimetres.

Table K.6: Errors in high and low water, 1 - 8 September 2016.

$HW_{rec.}$ [m]	$HW_{pred.}$ [m]	Abs. Error [m]	$LW_{rec.}$	$LW_{pred.}$ [m]	Abs. Error [m]
1.58	1.577	-0.003	-0.10	-0.100	0.000
1.51	1.456	-0.054	-0.47	-0.436	0.034
1.53	1.492	-0.039	-0.25	-0.234	0.016
1.49	1.431	-0.059	-0.30	-0.294	0.007
1.69	1.635	-0.055	-0.16	-0.153	0.007
1.54	1.510	-0.030	-0.41	-0.408	0.002
1.64	1.594	-0.046	-0.12	-0.101	0.019
1.74	1.734	-0.006	-0.11	-0.082	0.028
1.67	1.679	0.009	-0.23	-0.203	0.027
1.48	1.432	-0.048	-0.53	-0.518	0.012
1.39	1.390	0.000	-0.22	-0.213	0.007
1.38	1.330	-0.050	-0.45	-0.422	0.028
1.40	1.408	0.008	-0.26	-0.279	-0.019
1.30	1.264	-0.037			
1.52	1.49	-0.029	-0.28	-0.26	0.013

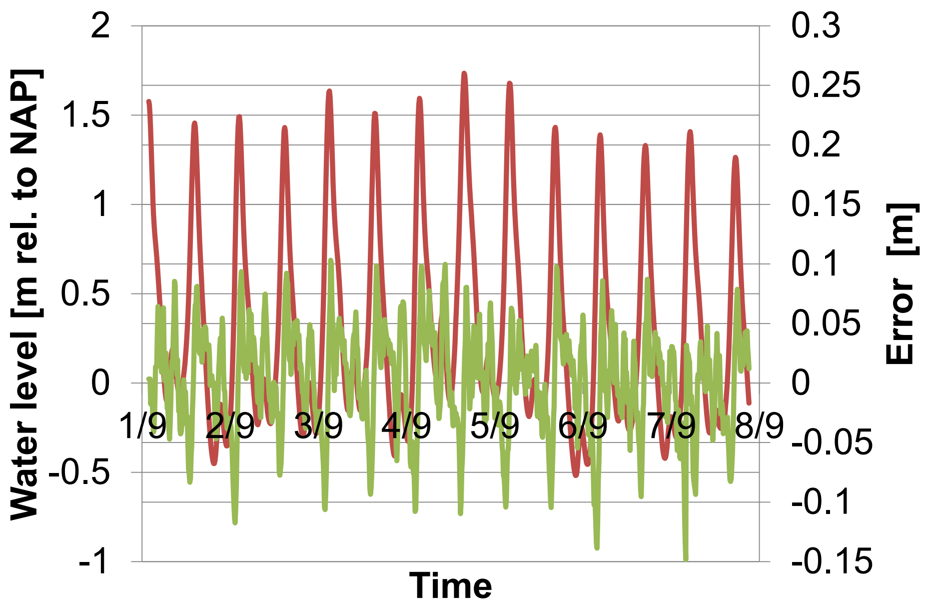

Figure K.2: Water levels (red) and errors (green) from 1-8 September 2016.

K.6.3 Inlet structure in RMM

In this subsection the inlet structure is modelled in the RMM. The cross-section required to maintain 90% of the tide in the simple model (model 11 in table K.5), is used as a starting point. The settings used are presented in table K.7.

Table K.7: User defined settings for SOBEK calculations.

Situation	Value	Unit
Length	50	m
manning's n	0.013	$s/m^{1/3}$
Calculation interval	5	min
Grid size	Every cross-section (±500)	m
ξ_i	0.7	-
ξ_b	0	-
k	1	-

Maximum size inlet
Section 7.4 showed that in case of door failure in storm conditions, a gap of 5 m^2 is already problematic. Therefore, for this preliminary design μ*A=5 m^2 is chosen as a maximum opening. For this size of openings, μ is approximately 0.7, see section K.5.1. Therefore, the maximum area of an inlet can have maximum values of 7.14 m^2. This results in pipes with a maximum diameter of 3.0 m or square culverts with dimensions 2.7x2.7 m.

Results
Table K.8 shows five configurations of the inlet structure. The first structure was sufficient in the simple model, see table K.5. It can be seen that the realised tidal range behind this structure is now well below 90%. The reduction in water depth along the river is probably the reason for the smaller tidal action in the basin. In configurations 2 through 5 the opening is increased. In the first two situations (2 & 3) only pipes are used. In structure 2 space is reserved for an extra lock.

Not all pipes are needed needed for pumps. Therefore, in situation 4 and 5, all except 5 pipes are replaced by rectangular culverts to increase the total area. Again, in structure 4 space is reserved for an extra lock, situation 5 uses all space available to maximise tidal action.

Table K.8: Results tested situations in RMM.

	No. Pipes	Ø [m]	No. Rect.	B [m]	A_{total} [m^2]	0 [m]	r [%]	9,392 [m]	r [%]	17,618 [m]	r [%]
0	n.a.	n.a.	n.a.	n.a.	n.a.	1.67	100	1.57	100	1.40	100
1	18	2.5	0	n.a.	90.0	1.26	75.3	1.20	76.9	1.12	80.0
2	19	3.0	0	n.a.	134.3	1.44	86.5	1.37	87.5	1.23	88.0
3	22	3.0	0	n.a.	155.5	1.49	89.0	1.41	90.2	1.28	91.7
4	5	3.0	15	2.7	144.7	1.47	88.1	1.39	89.0	1.26	90.4
5	5	3.0	19	2.7	173.9	1.52	90.8	1.44	91.8	1.29	92.7

K.6.4 Inlet structure and intertidal flats in RMM

To fully model the total design, the intertidal areas are introduced in the model. The flats will be introduced as basic shapes; no iteration or optimization is carried out here. Figures K.3 and K.4 give an idea of the inserted tidal flats in SOBEK and how they have been adjusted. Configurations 4 & 5 of table K.8 will be modelled.

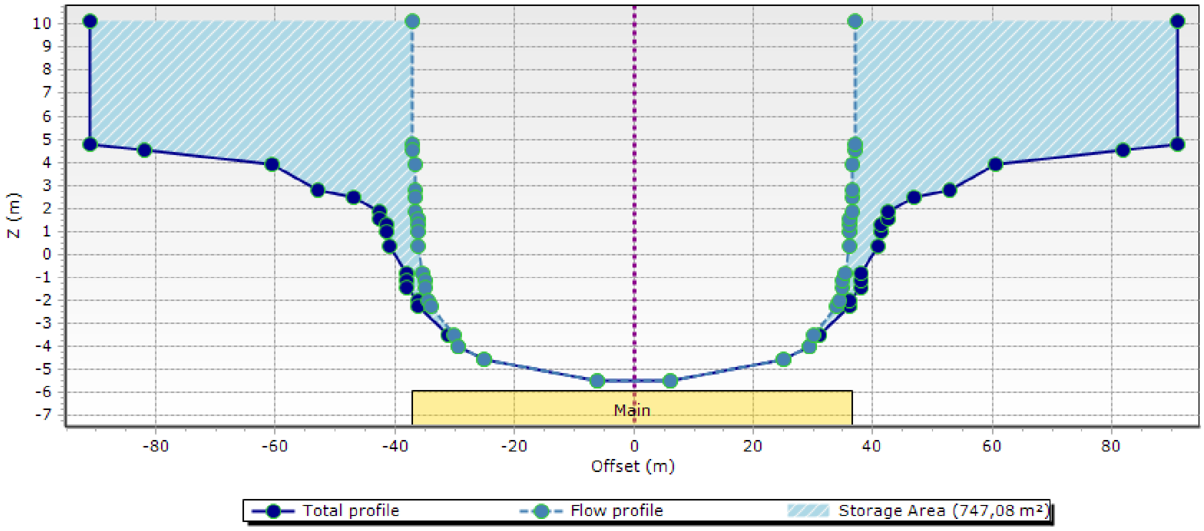

Figure K.3: Example of a cross-section of the Hollandsche IJssel in SOBEK.

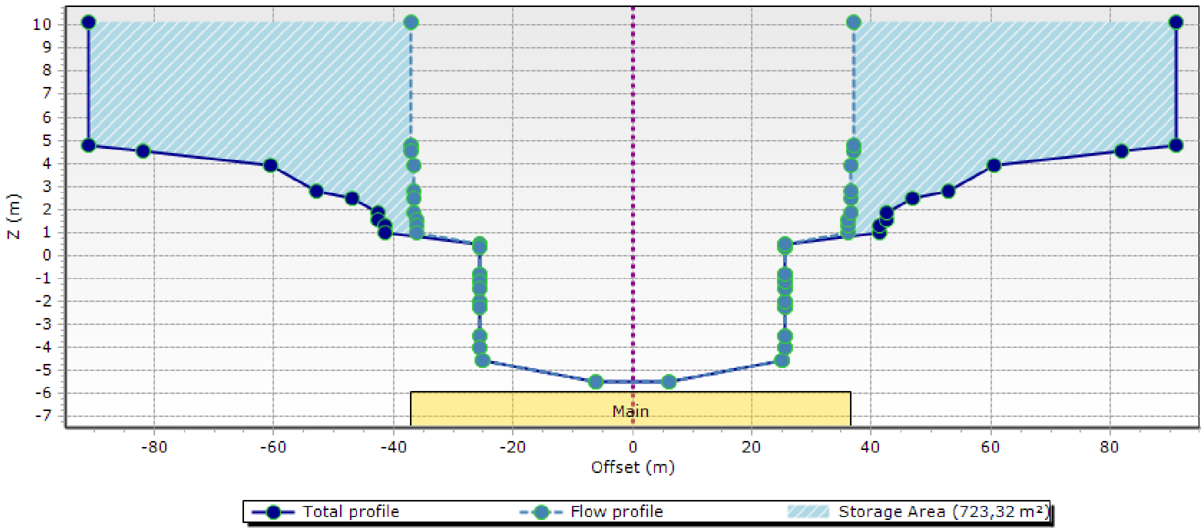

Figure K.4: Adjusted cross-section of the Hollandsche IJssel in SOBEK.

Properties intertidal flats

The intertidal flats will be implemented at 11 locations, as described in Appendix P. The intertidal flats will be modelled at the average slack water level, NAP +0.50 m. Table K.9 shows average HWL and LWL at Gouda and Krimpen. From these values the average slack water level is derived.

Table K.9: Average high water level (HWL) and low water level (LWL) in the Hollandsche IJssel.

	HWL [m NAP]	LWL [m NAP]	Slack [m NAP]
Krimpen	+1.20	-0.27	+0.47
Gouda	+1.40	-0.34	+0.53
Average	+1.3	-0.31	+0.50

Results
Table K.10 shows the tidal range behind the inlet structure when the intertidal flats are included. Water that flows in the river is hampered by the introduced intertidal flats resulting in a reduction of the tidal range half-way the river (m 9,392) and at the end (m 0). Just behind the barrier (at m 17,618), the tidal action has increased. This could be explained by the fact that the friction does not have a large influence at the beginning and that the opening of the inlet is large enough. The tidal ranges have become a little smaller than 90% of the original values. However, when taking into account the 3% under prediction of the tide mentioned in section K.6.2, the values are close to the desired 90%.

Later consultation of an expert within RHDHV resulted in that the diameter of the pumps should be 4.2 m instead of 3.0 m. Furthermore, not 5, but 6 pumps are required. The increased required cross-section leaves less space for the other culverts. This led to the decision to install a double row of culverts. Furthermore, the cross-section of each 'pump section' increased to μ·A=0.7 x 13.9 =9.7 m^2 almost double the size of the assumed maximum of 5 m^2 (see section 7.4). This led to the decision not to allow water through the 'pump section' before the pumps are installed. This was modelled calculation no. 6 & 7 in table K.10. It shows that the tidal amplitude has slightly increased. Configuration number 7 is schematised in figure K.5 and is used throughout this thesis.

Table K.10: Results tested situations with intertidal flats.

	No. Pipes	Ø [m]	No. Rect.	B [m]	A_{total} [m^2]	0 [m]	r [%]	9,392 [m]	r [%]	17,618 [m]	r [%]
0	n.a.	n.a.	n.a.	n.a.	n.a.	1.67	100	1.57	100	1.40	100
4	5	3.0	15	2.7	144.7	1.43	85.7	1.35	86.4	1.31	94.0
5	5	3.0	19	2.7	173.9	1.44	86.5	1.36	87.1	1.32	94.8
6	6	4.2	27	2.7	196.8	1.45	86.7	1.37	87.2	1.33	95.5
7	6	4.2	31	2.7	226.0	1.46	87.2	1.37	87.8	1.35	96.5

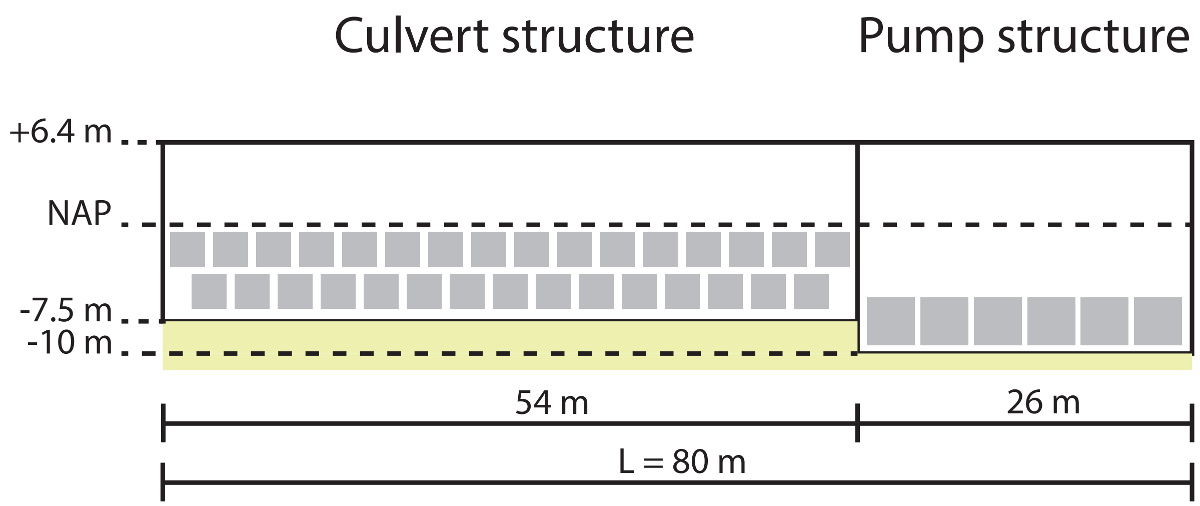

Figure K.5: Front view of the structure

K.7 Further research

The model used is of course far from flawless. This section provides some takeaways on where to commence further research as to optimise the model and its results. First of all, the loss coefficients could be researched further. When the design becomes more detailed, more will be known about the shape of the inlet and therefore inlet losses. Some other sources mention ξ_i-values of 0.5 and 0.6 for respectively rectangular and round cross-sections (Cultuurtechnische Vereniging, 1988). Secondly, the intertidal areas have been introduced in a basic rectangular shape. A different shape of the intertidal areas and possibly different locations of the areas could (positively) influence the tidal range in the river. Thirdly, a basic assumption on the maximum gap area was made. Further research regarding the probability of failure of the barrier could shed light on whether this assumption was right or that the gap area should be made smaller/larger. Fourthly, the reason behind the under prediction of the tidal range could be researched. An unbiased model will produce more accurate results.

L

Required lock capacity

L.1 Introduction

In this appendix, the required lock capacity is calculated. The required capacity defines whether the current Algera lock is sufficient, or that extra capacity is required. This extra capacity can be realised either by an extra lock or an entirely new lock complex. Since a new lock (complex) requires significant amounts of both space and budget, this is an important aspect of the barrier design. As mentioned in chapter 2, there are only few berthing facilities and harbours along the tidal part of the Hollandsche IJssel. The majority of the ships will therefore continue via the Juliana locks to the Gouwe canal. With the completion of a second lock in 2014, the capacity of the Juliana lock complex in Gouda was doubled (van de Sandt, 2014). This raises the question whether the capacity of the Algera lock is sufficient in case all vessels need to use the Algera lock. First the capacity is estimated by comparing the Juliana locks and the Algera lock (section L.2). After that, a more thorough analysis is carried out. The data used and methodology is shortly explained in section L.3, followed by a number of assumptions made to carry out the analysis (section L.4). In section L.5 the capacity of the lock is defined. Using the assumptions made, the available data (Appendix M) is transformed to data that can be used for analysis and extrapolated from 2008 to 2010. The results of this extrapolation for different scenarios introduced in chapter 6 is presented in section L.7 followed by a conclusion and implications in sections L.8 and L.9.

L.2 Quick assessment capacity

The lock chamber of the Algera lock measures 24 x 110 m (Van Hove, 2016). This means that it can accommodate 2 CEMT IV ships in each lock cycle. The area of the two Juliana locks chambers combined is similar to the Algera lock: the old lock measures 115 x 12 m, the new lock 115 x 14 (VolkerWessels, 2013). Therefore, one could conclude that the two lock complexes have a similar capacity. It should be noted that besides the size of the lock chamber, the amount of locks also play a role in the capacity. The availability of two lock chambers at the Juliana locks could provide a higher reliability: in case of a defect or during a maintenance period, the other lock can be used. If one blindly trusts the calculations made for the second Juliana locks, the Algera lock will have sufficient capacity now and in the future, at least until the end of the design life time of the Juliana locks. In order to back up this preliminary conclusion, the future predictions for shipping presented in chapter 6, combined with extra data on the current amount of lock cycles (Appendix M) will be used to assess whether the current lock is indeed sufficient.

L.3 Data used and methodology

Data gathered by the Provincie Zuid-Holland Provincie Zuid-Holland (2011) and IVS data (Rijksoverheid, 2016) were used. The data has been added as Appendix M. The data of Provincie Zuid-Holland (2011) shows the average number of lock cycles per hour for each month in 2008 of the old Juliana lock. This data is combined with the number of commercial and recreational vessels that passed the lock each month. This then leads to the monthly averaged required amount of lock cycles required for either commercial or recreational vessels at each hour of the day for 2008, which can then be extrapolated for 2050, 2100 and 2150 for each of the scenarios described in chapter 6.

L.4 Assumptions

The dataset of Rijksoverheid (2016) (Table M.1, Appendix M) provides the number of passages each month for commercial and recreational vessels. Unfortunately, no distinction was made between the times of day the vessels passed. In order to define the number of lock cycles at each hour for either commercial or recreational vessels, the split should be known. Therefore, the assumption is made that the split at each hour of day is equal to the split of the month. However, recreational vessels are much smaller than commercial vessels. A CEMT IV ship has a length of 85 m, a CEMT III ship has a length of 67-80 m. If recreational vessels are assumed to be of 10 m in length on average, six to eight recreational ships fit in the length of one commercial ship. It is therefore assumed that commercial vessels take up seven times more space than recreational ships.

Example
In May, out of a total of 2,722 ships, 909 (33%) were commercial (table M.1). It is assumed that at each time of the day, 33% of the ships are commercial ships. Since commercial vessels take up seven times more space than a recreational ship, the weighed share is taken:

$$\frac{n_{commercial} \cdot weight}{n_{commercial} \cdot weight + n_{recreational}} = \frac{909 \cdot 7}{909 \cdot 7 + 1,813} = 0.778 \tag{L.1}$$

So, although the commercial ships only form 33% of the total amount of vessels, they take up 77.8% of the space and therefore 77.8% of the lock cycles in May.

Summary of assumptions
The assumptions mentioned have been summarised below:

- All vessels that pass the Juliana locks will pass the Algera lock(s);
- The relative occurrence of commercial and recreational vessels per hour is assumed to be equal to the relative occurrence in that month;
- Commercial vessels take up seven times more space than recreational vessels.

L.5 Capacity current lock

The average lock cycle of the Algera lock is 10 minutes (Van Hove, 2016). If the lock could operate 24 hours per day, this would mean that 144 lock cycles could be performed. However, operating the lock is bounded to several restrictions (Van Hove, 2016):

- The maximum number of lock cycles is limited to 4 per hour;
- The lock is closed during rush hours, between 6:45-9:00 and 15:30-18:30.

The limitations mentioned lead to a maximum capacity of 75 cycles per day and a maximum of 4 cycles per hour. Under the current regime, no shipping is possible between 7:00h and 9:00h and between 16:00 and 18:00h. Between 15:00h and 16:00h and between 18:00h and 19:00h, three lock cycles are possible. Furthermore, since the area of the lock chamber of the Algera lock is twice the size of the old Juliana lock, it is assumed that the Algera lock needs to perform half the amount of lock cycles of the old Juliana lock.

L.6 Transformation and extrapolation

The amount of cycles per hour presented in table M.2 is transformed to the required amount of cycles per day for the Algera lock. This is done by dividing the number of cycles by two (the Algera lock has twice the size of the old Juliana lock) and dividing by the number of days of each month. Since the predictions made in chapter 6 use 2010 as starting point (see tables L.2 and L.3), the data has to be transformed to 2010. Table L.1 shows the extrapolation factors. The results for the required number of lock cycles per hour are presented in tables L.4, L.5 and L.6.

Table L.1: Development shipping 2008 - 2010 (Rijksoverheid, 2016).

	2008	2010	Conversion factor
Recreational	16,017	14,990	0.935
Commercial	8,254	7,840	0.950
Total	24,271	22,830	0.941

Table L.2: Scenarios for professional shipping (chapter 6).

	2010	2050	2100	2150
1	1	1.6	2.2	2.6
2	1	1.5	1.5	1.5
3	1	0.9	0.5	0.5
4	1	0.9	0.7	0.7

Table L.3: Scenarios for recreational shipping (chapter 6).

	2010	2050	2100	2150
1&2	1	1.5	1.5	1.5
3&4	1	1	0.5	0.5

Table L.4: Required average amount of lock cycles Algera lock per hour for commercial ships in 2010.

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0	0	0	0
6	0	1	0	1	1	0	0	0	1	0	0	0
7	1	1	1	1	1	0	0	0	0	0	0	0
8	0	1	0	1	1	1	1	0	1	1	0	0
9	0	1	1	1	1	1	1	1	1	1	1	0
10	1	1	1	1	1	1	1	1	1	1	1	1
11	1	1	1	2	2	1	1	1	1	1	1	1
12	1	1	1	2	2	1	1	1	1	1	1	1
13	1	1	1	1	1	1	1	1	1	1	1	1
14	1	1	1	1	2	1	1	1	2	1	1	1
15	1	1	1	1	1	1	1	1	1	1	1	1
16	1	1	1	2	2	1	1	1	2	1	1	1
17	1	1	1	1	1	1	1	1	1	1	1	1
18	1	1	1	1	1	1	1	0	1	1	1	1
19	1	1	1	1	1	1	1	0	1	1	1	1
20	0	1	0	1	1	1	1	0	1	1	1	0
21	0	1	0	1	1	1	1	0	1	0	0	0
22	0	0	0	1	0	0	0	0	0	0	0	0
23	0	0	0	1	0	0	0	0	0	0	0	0
Total	12	17	13	22	21	19	12	8	18	18	14	11

Table L.5: Required average amount of lock cycles Algera lock per hour for recreational ships in 2010.

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0	0	0	0
6	0	0	0	0	0	0	0	0	0	0	0	0
7	0	0	0	0	0	0	0	1	0	0	0	0
8	0	0	0	0	0	0	1	1	0	0	0	0
9	0	0	0	0	0	0	1	1	0	0	0	0
10	0	0	0	0	0	0	1	1	0	0	0	0
11	0	0	0	0	0	1	1	1	0	0	0	0
12	0	0	0	0	0	1	1	1	0	0	0	0
13	0	0	0	0	0	1	1	1	0	0	0	0
14	0	0	0	0	0	1	1	1	0	0	0	0
15	0	0	0	0	0	1	1	1	0	0	0	0
16	0	0	0	0	0	0	1	1	0	0	0	0
17	0	0	0	0	0	0	1	1	0	0	0	0
18	0	0	0	0	0	0	1	1	0	0	0	0
19	0	0	0	0	0	0	1	0	0	0	0	0
20	0	0	0	0	0	0	1	0	0	0	0	0
21	0	0	0	0	0	0	1	0	0	0	0	0
22	0	0	0	0	0	0	0	0	0	0	0	0
23	0	0	0	0	0	0	0	0	0	0	0	0
Total	0	0	0	3	6	7	12	15	4	2	0	0

Table L.6: Total required average amount of lock cycles Algera lock per hour in 2010.

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0	0	0	0
6	0	1	0	1	1	1	1	1	1	1	0	0
7	1	1	1	1	1	1	1	1	1	1	0	0
8	0	1	0	1	1	1	1	1	1	1	1	0
9	0	1	1	1	2	1	1	2	1	1	1	0
10	1	1	1	1	2	2	2	2	2	1	1	1
11	1	1	1	2	2	2	1	2	2	1	1	1
12	1	1	1	2	2	2	2	2	2	1	1	1
13	1	1	1	2	2	2	2	2	2	2	1	1
14	1	1	1	2	2	2	2	2	2	1	1	1
15	1	1	1	2	2	2	2	2	2	2	1	1
16	1	1	1	2	2	2	2	2	2	1	1	1
17	1	1	1	1	2	2	2	2	1	1	1	1
18	1	1	1	1	1	1	2	1	1	1	1	1
19	1	1	1	1	1	1	1	1	1	1	1	1
20	0	1	0	1	1	1	1	1	1	1	1	0
21	0	1	0	1	1	1	1	1	1	1	0	0
22	0	0	0	1	1	0	0	0	1	0	0	0
23	0	0	0	1	0	0	0	0	0	0	0	0
Total	12	17	13	24	27	25	24	22	23	19	14	11

In table L.6 it can be seen that in 2010, the lock capacity of the Algera lock would be sufficient if only the lock could be used and no vessels could sail under the current Algera barrier. Of course no ships can pass during rush hours (see section L.5). The rush hours are marked in red in table L.6. However there is sufficient capacity before and after rush hours. Furthermore, when comparing table L.4 and L.5, it can be seen that commercial ships have a far larger impact on the amount of lock cycles than recreational ships.

L.7 Results

The expected required amount of lock cycles has been calculated in 2050, 2100 and 2150 for each of the scenarios mentioned in chapter 6 (see also tables L.2 and L.3). It was found that for scenarios 2, 3 and 4, besides the periods in which the lock cannot operate (rush hours, see section L.5), there are no capacity problems. Scenario 1 is however problematic. First scenarios 2, 3 and 4 will be discussed. After that, scenario 1 is discussed followed by some mitigating measures.

L.7.1 Scenario 2, 3 and 4

Table L.7 shows the required amount of lock cycles in 2150 for scenario 2. During rush hours no shipping is possible. In case the available capacity is reached the cells are coloured red. There is sufficient capacity in the hours before and after rush hours to accommodate the ships. To illustrate this, the redistribution of ships for scenario 2 in 2150 has been tabulated in table L.8. Here, whenever the capacity is reached the cell is made blue.

Table L.7: Scenario 2: total required average amount of lock cycles Algera lock per hour in 2150, before redistribution.

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	0.3	0.4	0.2	0.4	0.4	0.3	0.3	0.1	0.2	0.3	0.3	0.3
1	0.0	0.2	0.3	0.4	0.2	0.2	0.4	0.0	0.3	0.1	0.3	0.2
2	0.0	0.2	0.0	0.1	0.1	0.2	0.2	0.0	0.2	0.2	0.3	0.2
3	0.0	0.0	0.2	0.0	0.1	0.1	0.1	0.0	0.1	0.0	0.3	0.1
4	0.0	0.2	0.1	0.1	0.1	0.1	0.1	0.2	0.0	0.1	0.1	0.0
5	0.3	0.4	0.3	0.6	0.5	0.7	0.4	0.4	0.4	0.3	0.5	0.1
6	0.7	1.1	0.4	1.1	1.1	1.0	1.1	0.9	1.0	0.8	0.7	0.6
7	0.8	0.9	0.9	1.0	1.3	1.0	1.3	1.2	0.8	0.8	0.7	0.5
8	0.7	1.3	0.7	1.5	1.7	1.5	1.6	2.0	1.0	1.3	0.8	0.6
9	0.7	1.1	0.8	1.5	2.3	2.0	2.2	2.4	1.8	1.4	1.0	0.7
10	1.0	1.5	1.2	2.1	2.8	2.5	2.4	2.6	2.3	2.1	1.6	1.1
11	1.1	1.8	1.5	2.7	3.2	2.8	2.2	2.6	2.7	1.9	1.3	1.0
12	1.7	2.3	1.7	3.3	3.1	2.9	2.4	2.6	2.6	2.2	2.0	1.2
13	1.5	2.2	1.4	2.4	2.8	3.0	2.4	2.8	2.7	2.4	1.8	1.8
14	1.6	1.9	1.4	2.5	3.1	2.9	2.6	2.6	2.9	2.2	1.3	1.3
15	1.3	1.7	1.5	2.4	2.7	2.9	2.6	2.6	2.6	2.3	1.1	1.2
16	1.1	1.9	1.2	2.8	3.1	2.6	2.8	2.5	2.8	2.0	1.4	1.1
17	0.9	1.2	0.9	2.2	2.5	2.7	2.5	2.5	2.0	2.1	1.5	0.9
18	1.0	1.7	1.0	2.0	2.1	2.1	2.4	1.8	2.0	1.5	1.2	1.4
19	0.9	1.4	1.0	1.9	1.7	2.2	2.0	1.1	1.6	1.6	1.0	1.0
20	0.7	1.2	0.7	1.6	2.0	1.6	1.9	1.1	1.3	1.4	0.8	0.7
21	0.6	0.8	0.6	1.5	1.9	1.7	1.6	0.9	1.2	0.8	0.7	0.5
22	0.7	0.3	0.4	1.0	0.8	0.6	0.7	0.2	0.9	0.7	0.5	0.4
23	0.5	0.6	0.6	0.9	0.4	0.4	0.3	0.2	0.5	0.5	0.5	0.5
Total	18.2	26.3	19.4	36.3	40.1	38.1	36.4	33.6	34.1	29.1	21.7	17.2

Table L.8: Scenario 2: total required average amount of lock cycles Algera lock per hour in 2150, after redistribution. Cells with maximum capacity are marked blue.

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	0.3	0.4	0.2	0.4	0.4	0.3	0.3	0.1	0.2	0.3	0.3	0.3
1	0.0	0.2	0.3	0.4	0.2	0.2	0.4	0.0	0.3	0.1	0.3	0.2
2	0.0	0.2	0.0	0.1	0.1	0.2	0.2	0.0	0.2	0.2	0.3	0.2
3	0.0	0.0	0.2	0.0	0.1	0.1	0.1	0.0	0.1	0.0	0.3	0.1
4	0.0	0.2	0.1	0.1	0.1	0.1	0.1	0.2	0.0	0.1	0.1	0.0
5	0.3	0.4	0.3	0.6	0.5	0.7	0.4	0.4	0.4	1.1	1.2	0.7
6	1.5	2.1	1.3	2.1	2.4	2.0	2.4	2.1	1.8	0.0	0.0	0.0
7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	1.5	1.1
9	1.4	2.4	1.5	3.0	4.0	3.5	3.8	4.0	2.8	1.4	1.0	0.7
10	1.0	1.5	1.2	2.1	2.8	2.5	2.4	3.0	2.3	2.1	1.6	1.1
11	1.1	1.8	1.5	2.7	3.2	2.8	2.2	2.6	2.7	1.9	1.3	1.0
12	1.7	2.3	1.7	3.3	3.8	3.3	2.4	2.6	2.6	2.2	2.0	1.2
13	1.5	2.2	1.4	3.1	4.0	4.0	3.4	3.5	4.0	2.4	1.8	1.8
14	1.6	2.5	1.4	4.0	4.0	4.0	4.0	4.0	4.0	3.5	1.3	1.6
15	2.4	3.0	2.7	3.0	3.0	3.0	3.0	3.0	3.0	3.0	2.5	3.0
16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	1.9	2.9	1.9	3.0	3.0	3.0	3.0	3.0	3.0	3.0	2.7	2.3
19	0.9	1.4	1.0	3.1	3.3	4.0	3.9	2.4	2.6	2.1	1.0	1.0
20	0.7	1.2	0.7	1.6	2.0	1.6	1.9	1.1	1.3	1.4	0.8	0.7
21	0.6	0.8	0.6	1.5	1.9	1.7	1.6	0.9	1.2	0.8	0.7	0.5
22	0.7	0.3	0.4	1.0	0.8	0.6	0.7	0.2	0.9	0.7	0.5	0.4
23	0.5	0.6	0.6	0.9	0.4	0.4	0.3	0.2	0.5	0.5	0.5	0.5
Total	18.2	26.3	19.4	36.3	40.1	38.1	36.4	33.6	34.1	29.1	21.7	17.2

L.7.2 Scenario 1

Scenario 1 is problematic however. By 2100, redistribution becomes difficult because also outside rush hours the capacity is reached. This leads to long waiting times. Table L.9 shows the expected demand for 2100 in case of scenario 1. An example redistribution is shown in table L.8. Again, whenever too much capacity is desired the cell was made red. Blue cells mark that the lock works at its maximum capacity that hour.

Table L.9: Scenario 1: total required average amount of lock cycles Algera lock per hour in 2100, before redistribution.

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	0.4	0.6	0.3	0.6	0.5	0.4	0.4	0.1	0.3	0.5	0.4	0.5
1	0.1	0.2	0.4	0.5	0.3	0.3	0.5	0.1	0.5	0.2	0.5	0.2
2	0.0	0.3	0.1	0.2	0.2	0.3	0.2	0.0	0.3	0.2	0.4	0.3
3	0.0	0.0	0.2	0.1	0.1	0.1	0.1	0.0	0.2	0.1	0.4	0.1
4	0.1	0.3	0.1	0.2	0.1	0.1	0.2	0.2	0.0	0.2	0.1	0.0
5	0.4	0.5	0.4	0.8	0.6	1.0	0.5	0.5	0.5	0.4	0.7	0.1
6	1.0	1.6	0.6	1.6	1.5	1.3	1.4	1.1	1.4	1.1	1.1	0.8
7	1.1	1.4	1.4	1.4	1.8	1.3	1.6	1.4	1.1	1.1	1.0	0.7
8	1.1	1.9	1.1	2.1	2.3	2.1	1.9	2.3	1.3	1.8	1.1	0.9
9	1.0	1.6	1.2	2.2	3.2	2.6	2.7	2.8	2.5	2.0	1.4	1.0
10	1.5	2.2	1.7	3.0	3.8	3.4	3.0	3.0	3.2	3.0	2.4	1.7
11	1.6	2.6	2.2	3.8	4.4	3.8	2.7	3.0	3.7	2.8	1.9	1.4
12	2.5	3.3	2.5	4.7	4.3	3.9	3.0	3.0	3.5	3.2	2.9	1.8
13	2.2	3.3	2.0	3.5	3.8	4.0	3.0	3.3	3.8	3.4	2.6	2.7
14	2.3	2.7	2.1	3.5	4.3	3.9	3.2	3.0	4.0	3.1	1.9	1.8
15	1.9	2.5	2.2	3.4	3.7	3.9	3.2	3.1	3.5	3.2	1.7	1.7
16	1.7	2.7	1.8	3.9	4.2	3.5	3.4	2.9	3.9	2.8	2.0	1.6
17	1.3	1.8	1.3	3.1	3.4	3.7	3.1	3.0	2.8	2.9	2.1	1.2
18	1.4	2.5	1.5	2.9	2.8	2.8	2.9	2.1	2.7	2.1	1.8	2.0
19	1.2	2.1	1.5	2.7	2.4	3.0	2.5	1.3	2.2	2.3	1.5	1.5
20	1.0	1.7	1.1	2.3	2.8	2.2	2.4	1.3	1.8	2.0	1.2	1.0
21	0.8	1.1	0.9	2.1	2.6	2.3	1.9	1.1	1.6	1.1	1.0	0.8
22	1.0	0.5	0.6	1.5	1.1	0.9	0.8	0.2	1.2	1.0	0.7	0.5
23	0.8	0.9	0.9	1.3	0.6	0.6	0.4	0.2	0.7	0.7	0.8	0.8
Total	27	38	28	51	55	51	45	39	47	42	32	25

Table L.10: Scenario 1: total required average amount of lock cycles Algera lock per hour in 2100, after redistribution. Cells with maximum capacity are marked blue.

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	0.4	0.6	0.3	0.6	0.5	0.4	0.4	0.1	0.3	0.5	0.4	0.5
1	0.1	0.2	0.4	0.5	0.3	0.3	0.5	0.1	0.5	0.2	0.5	0.2
2	0.0	0.3	0.1	0.2	0.2	0.3	0.2	0.0	0.3	0.2	0.4	0.3
3	0.0	0.0	0.2	0.1	0.1	0.1	0.1	0.0	0.2	0.1	0.4	0.1
4	0.1	0.3	0.1	0.2	0.1	0.1	0.2	0.2	0.0	0.2	0.1	0.0
5	0.4	0.5	0.4	2.9	2.1	1.0	1.4	2.8	0.5	0.4	0.7	0.1
6	2.2	3.0	2.0	3.0	4.0	3.3	4.0	2.5	3.8	2.2	2.1	1.5
7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	2.2	3.5	2.3	2.2	3.7	4.0	2.7	2.8	2.5	3.8	2.5	1.9
10	1.5	2.2	1.7	3.5	4.0	3.9	3.0	3.0	3.2	3.0	2.4	1.7
11	1.6	2.6	2.2	4.0	4.0	4.0	2.7	3.0	4.0	2.8	1.9	1.4
12	2.5	3.5	2.5	4.0	4.0	4.0	4.0	3.0	4.0	3.7	2.9	1.8
13	2.2	4.0	2.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.6	2.7
14	2.9	4.0	3.1	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.6	2.5
15	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	3.0	3.0	2.8	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
19	2.2	3.4	1.5	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	1.7
20	1.0	1.7	1.1	4.0	4.0	4.0	4.0	2.4	4.0	3.3	1.2	1.0
21	0.8	1.1	0.9	4.0	4.0	4.0	2.6	1.1	4.0	1.1	1.0	0.8
22	1.0	0.5	0.6	2.9	4.0	3.4	0.8	0.2	1.7	1.0	0.7	0.5
23	0.8	0.9	0.9	1.3	1.7	0.6	0.4	0.2	0.7	0.7	0.8	0.8
Total	27	38	28	51	55	51	45	39	47	42	32	25

L.7.3 Mitigating measures

Redistribution of ships occurs automatically because ships simply cannot pass the lock complex at the desired time. Taking mitigating measures could increase the capacity and reduce waiting times. Several mitigating measures have been summed up below.

- **Extra lock**: An extra lock with dimensions of the new Juliana lock increases the capacity to 6 cycles per hour. This will lead to sufficient capacity in all scenarios until 2150;
- **Other road connection**: A road connection at another location would take away the capacity limitations mentioned in section L.5. This can either be a bridge high enough to sail underneath or a tunnel/aqueduct. A lock bridge, see Appendix O could be an option as well. In this case the lock capacity is 6 cycles per hour, also during rush hours. This will lead to sufficient capacity in all scenarios until 2150;
- **Residual capacity**: It is likely that the lock cycles counted in the 2008 dataset (Provincie Zuid-Holland, 2011) were not entirely filled. There is probably extra capacity that can be used. It remains unknown what this residual capacity is;
- **Service time for yachts**: Install service times for the recreational shipping. Since the societal costs are much smaller for recreational shipping than for commercial shipping (€8.25 versus €338/ship/hour (Kennisinstituut voor Mobiliteitsbeleid, 2013)), the societal costs could be limited. However, due to their small size, commercial ships have limited impact on the required amount of lock cycles. In scenario 1 for example, this could be a solution for 2100. The rapid increase of vessels in this scenario and the limited impact make that this measure is only short-term solving a problem.

L.8 Conclusion

As shown in section L.7, three out of the four scenarios analysed show that the current Algera lock has sufficient capacity until 2150. In scenario 1, under the current limitations on the capacity the lock has an insufficient capacity by 2100. However, mitigating measures could result in sufficient capacity for the Algera lock. Relocation of the road connection in combination with an increase in capacity (based on the findings in chapter 6), results in sufficient capacity of the current Algera lock. Reservation of space for an extra lock creates flexibility in the design and would leave all options open for future decision-makers if a drastic increase in shipping numbers takes place. However, the majority of the predictions point towards a single lock. Therefore,reservation of space is not paramount.

L.9 Implications

The conclusion that the current lock is sufficient to accommodate future shipping intensities has implications on the design. A new lock complex is not necessary and therefore locations 2 and 3 mentioned in chapter 8 have become less attractive.

M

Data for lock capacity calculation

Table M.1: Shipping through Juliana lock in 2008 (Rijksoverheid, 2016).

Month	Commercial	Recreational	Total	Com. [%]	Rec. [%]	$Weighed_{com.}$ [%]	$Weighed_{rec.}$ [%]
January	583	52	635	91.8	8.2	98.7	1.3
February	776	74	850	91.3	8.7	98.7	1.3
March	551	125	676	81.5	18.5	96.9	3.1
April	942	806	1748	53.9	46.1	89.1	10.9
May	909	1813	2722	33.4	66.6	77.8	22.2
June	804	2078	2882	27.9	72.1	73.0	27.0
July	697	4756	5453	12.8	87.2	50.6	49.4
August	334	4400	4734	7.1	92.9	34.7	65.3
September	737	1248	1985	37.1	62.9	80.5	19.5
October	791	512	1303	60.7	39.3	91.5	8.5
November	611	102	713	85.7	14.3	97.7	2.3
December	519	51	570	91.1	8.9	98.6	1.4
Total	8,254	16,017	24,271				

Table M.2: Number of cycles per hour for Juliana lock (2008), accumulated per month 2008 (Provincie Zuid-Holland, 2011).

Hour	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0	11	15	10	17	16	14	14	3	10	15	12	14
1	2	6	12	15	10	9	16	2	14	5	14	7
2	1	9	2	5	5	10	8	1	8	7	13	8
3	0	1	7	2	4	3	4	1	6	2	11	3
4	2	8	4	6	3	3	6	9	0	6	4	0
5	13	14	13	24	20	30	18	19	16	13	20	3
6	29	44	19	47	48	42	49	40	43	35	31	25
7	34	37	41	43	56	42	58	54	35	35	30	20
8	32	51	32	64	73	65	68	87	41	56	32	28
9	31	44	37	65	102	83	95	106	78	60	41	31
10	45	58	52	90	123	107	105	114	97	93	69	50
11	49	70	66	114	141	119	95	113	114	84	55	42
12	75	89	76	140	136	122	106	115	108	98	83	53
13	66	88	59	103	123	125	107	124	116	104	74	80
14	68	74	63	104	136	122	113	114	123	94	56	55
15	57	67	65	101	119	122	114	116	108	99	48	51
16	50	73	53	116	135	111	121	108	120	86	57	48
17	39	49	40	93	108	115	109	112	86	90	62	37
18	43	68	44	86	90	89	104	81	83	65	51	59
19	37	57	45	79	76	95	88	50	69	71	42	44
20	30	47	32	69	89	68	84	49	56	62	35	30
21	24	30	27	62	83	71	69	41	50	35	28	23
22	29	13	18	44	36	27	29	8	37	32	21	16
23	23	23	27	40	18	18	14	8	22	20	23	23
Total	790	1,035	844	1,529	1,750	1,612	1,594	1,475	1,440	1,267	912	750

N

Types of barriers

When closing off a waterway, different possibilities are possible. Factors limiting the design space are e.g. depending on the time a barrier needs to be open and (closure) reliability of the concept. An extensive overview of the possibilities for storm surge barriers is provided by van der Toorn and de Gijt (2013) and de Vries (2014, Appendix B). The following types of barriers can be distinguished:

- Mitre gates;
- Radial gates;
- Vertical lifting gates;
- Flap gates;
- Sector gates;
- Visor gates;
- Horizontal rotating gates;
- Inflatable rubber dam;
- Parachute barrier;
- Barge gate;
- Reduction barrier;
- Mailbox gate;
- Caisson structure (concrete vertical lifting gate).

Due to the more permanent character of the closure, different options are added. Here, ideas of Knook (2012) have been used as a source of inspiration.

- Check valve;
- Siphon.

When deciding which concepts to take into further consideration, the two different functions should be taken into account: the shipping function and the tidal/water management function.

N.1 Tidal function

The tidal section of the barrier is a structure that has to ensure tidal influx and outflux for ecological purposes. When a lower water table and/or a reduced tidal regime is wished for in the Hollandsche IJssel, the required tidal opening is different for in and outflow. As a semi-diurnal cycle is wished for, flow reverses four times per day.

- Tidal opening can be altered relatively quickly;
- Tidal opening can be entirely closed in case of extreme high water;
- Barrier is able to resist hydraulic head from both sides;
- Fish is able to pass;
- Water level can be regulated.

The description of specifications lead to the following assessment criteria:

1. **Negative head** Due to flow reversal and different water levels at each side of the dam, the structure needs to be able to resist heads from both sides.

2. **Permanent close off** The combination of high design water levels with discharge from the surrounding polders and limiting storage capability in an economically valuable area make the possibility to close the barrier paramount.
3. **Closing speed** The flow reversal also leads to the demand that a closure or alteration of tidal opening should take place in short time span.
4. **Current** The tidal flow causes a current in which the doors need to be able to close.
5. **Fish passability** The major reason to maintain the tidal amplitude is the uniqueness of the ecotopes related to tidal action. To prevent isolation and exploit the potential of the ecosystem to a maximum fish migration is paramount.
6. **Maintenance** Two aspects are considered concerning maintenance, being (1) presence of hinges and (2) the ability to repair in the dry. Hinges often provide challenges for maintenance. The hinge in the Maeslant barrier has led to problems as cushions wore down very fast (Cobouw, 2004). The ability to repair movable parts in the dry eases inspection runs and reduces maintenance costs.

Table N.1: Scores of the concepts for tidal function

	Mitre	Radial	Vertical lift	Flap	Sector	Visor	Horizontal rotation	Rubber	Parachute	Barge	Reduction	Mailbox	Check valve	Siphon
1. Negative head	±	-	+	±	-	-	+	+	+	-	+	±	+	+
2. Close off	+	+	+	+	+	+	+	+	+	+	-	±	+	+
3. Closing speed	±	±	±	-	-	±	±	±	-	±	n.a.	±	+	+
4. Current	-	+	+	+	+	+	+	+	+	±	+	+	+	+
5. Fish passability	+	+	+	+	+	+	+	+	+	+	+	+	+	±
6. Maintenance														
- 6.1. No Hinges	-	-	+	-	-	-	-	+	+	-	+	+	-	+
- 6.2. Repair in the dry	-	+	+	-	+	+	+	n.a.	-	-	-	+	-	+

N.1.1 Killing conditions

Concepts unable to comply with the first five criteria are considered to be insufficient. The abovementioned concepts and scores, five concepts are taken into further consideration.

- Check valve;
- Siphon;
- Vertical lifting gates;
- Horizontal rotating gates;
- Rubber barrier.

N.2 Shipping function

The navigational section of the barrier has to ensure passage of shipping. As this waterway is part of the 'staande mast route' (see section 3.5), height limitations are unwanted for the navigational section.
Low probability of failure, therefore a proven concept is wished for. This leaves two types of lock doors, being:

- Mitre gates;
- Rolling door.

O

Types of river crossings

There are various ways to cross a river. This Appendix discusses various options and their (dis)advantages. The following crossings will be discussed:

- Ferry;
- Bridge;
- Tunnel;
- Aqueduct.

Main points of comparison are cost, capacity for crossing traffic and hindrance to shipping. A qualitative comparison is made in section O.5.

O.1 Ferry

A ferry can be an effective way as means of transport over a waterway. Apart from minor constructions needed at the river banks, no structures are needed. A ferry connection can therefore be implemented relatively quickly and requires low initial investments. However, several disadvantages cling to a ferry connection. Compared to the current situation (fixed connection), travel time is likely to increase. Time is needed to (dis)embark and generally ferries need more time to pass the river than vehicles over a bridge. Also, Vehicles need a place to queue to prevent clogging of the surrounding road network. Lastly, The crossing of ferries conflicts with the direction of vessels sailing up or down the river. This can potentially lead to dangerous situations and a reduced capacity of the river. To summarise, the (dis)advantages have been listed below:

- Little construction needed;
- Low initial investments;
- Increased travel time;
- Buffering capacity needed;
- Ferry crossings conflict with shipping.

O.2 Bridge

Currently both sides of the river are connected by a bridge. The bridge connection currently has a low capacity and considered to be a bottle neck (Deltaprogramma, 2014, p.88) (Bureau Alle Hosper landschapsarchitectuur en stedebouw, 1998, p.15). Most ships are able to sail underneath the closed part of the bridge. Genererally speaking, bridge connections are cheaper than tunnel connections (source). Bridges are especially favourable compared to tunnels when the banks have a ctain elevation so that no 'aanbruggen' are needed. These connections require some space as can be seen in Krimpen a/d IJssel, which may not be the most beautiful solution. In the new situation all vessels have to cross a sluice. If the bridge is constructed too low, the capacity of both the waterway and the road connection may be reduced significantly. A final remark is made on the impact a bridge has on the landscape. Its visibility could serve as an eye catcher, but could also be seen as ugly. To summarise, the (dis)advantages have been listed below:

- Cheaper than a tunnel;
- Space needed for road connection;
- Could limit the passable height;
- Capacity shipping;
- Impacts landscape.

O.2.1 'Lock bridge'

A special type of bridge is a bridge over a lock. Here, the lock is used for road navigation as well. A road connection at either side of the lock ensures continuous passage. In the Netherlands towns such as IJmuiden and Terneuzen have such a connection. A positive aspect is the multiple use of the lock. Furthermore, the waterway can be crossed with a lower height. The capacity may however be limited. The additional (dis)advantages have been listed below:

- Multiple use lock;
- Less high bridge needed;
- Limited capacity.

O.3 Tunnel

A tunnel has the advantage that no height limitations will be present for shipping. Another plus is that traffic will not be visible for the surrounding area and nuisance will be reduced. Tunnels are generally more expensive than bridges (source). However, the closed nature of tunnels has resulted in high safety standards that may include a separate lane for emergencies and two tubes to evacuate users in case of fire. Furthermore, the road needs to go over the dikes at the banks to prevent the surrounding area from flooding in case of a leak. The tunnel is then situated in a separate polder. Similar to a bridge connection, serious space is needed to cover for the elevation differences. Once completed, expansion of the road connection may proof to be difficult. To summarise, the (dis)advantages have been listed below:

- No height limitations shipping;
- Minimal impact on landscape;
- More expensive than a bridge;
- Space needed for road connection;
- High safety standards;
- Separate polder;
- Difficult to expand.

O.4 Aqueduct

An aqueduct can be seen as the short version of a tunnel. Similar benefits cling to the aqueduct. The reduced length results in less limiting safety regulations. A viaduct will generally be cheaper than a tunnel. The additional (dis)advantages have been listed below:

- Less expensive than a tunnel;
- Less limiting safety standards;
- Only for narrow waterways.

O.4.1 Naviduct

A naviduct is a special kind of aqueduct. It is an aqueduct combined with a lock. To date, this concept has been applied once near Enkhuizen (Krabbersgat). Here, the frequent opening of the former bridge over the water led to serious congestion. With the construction of the naviduct, water and road bound traffic has been separated. A few years ago, the possibility to construct a naviduct in the Afsluitdijk has been researched.

O.5 Summary of applicability

Previous sections have shown the different (dis)advantages of different types of river crossings. Subsequently, a qualitative comparison has been made between the different options. Reason to opt for qualitative comparison is to early narrow down the available options. Furthermore no extensive data is readily available, making a quantitative comparison a tedious effort. Exact scales could give the impression that the differences are exactly known, which is not the case. Therefore, the different alternatives have been ranked with simple pluses and minuses. Besides the costs and the impact a crossing type has on road traffic and shipping, the impact on the tidal function has been assessed as well. The crossings that go underneath the waterway need a polder that reduces the available size to realise the tidal opening and a pumping station. The results have been tabulated in table O.1. Due to the need of a navigational lock in the future situation, a naviduct can be considered superior over an aqueduct. Therefore, only the traits of a naviduct have been compared.

Table O.1: Score of river crossing types

	Ferry	Bridge	Lock bridge	Tunnel	Naviduct
Cost	-	±	±	++	+
Hindrance to shipping	±	±	±	-	-
Hindrance to traffic	+	±	±	-	-
Hindrance tidal function	-	-	-	±	+

The qualitative analysis summarised in table O.1 shows that a ferry connection provides significant hindrance to road traffic. Based on the wish to improve current situation, the option of a ferry connection is considered not viable. Although the costs of crossing for a lock bridge could be lower than a conventional bridge, the costs incurred due to the double connection may nullify the initial savings. The required polder for the naviduct and tunnel reduce the area to realise a tidal opening and a pumping station. Calculations from the required tidal opening and pumping capacity give insight whether a tunnel or a naviduct is feasible. However, the combination of a further close-off of the river make that an entire underwater road connection does not seem logical.

P

Locations for intertidal areas

In this Appendix a closer look will be taken at the possible locations for the realisation of intertidal areas. First the current intertidal areas are indicated (section P.1), followed by a suggestion for creation of new intertidal areas (section P.2).

P.1 Current intertidal areas

The intertidal areas currently present were identified using Bing maps (Microsoft, 2016) and Google Earth (Google Inc., 2016). The areas are bordered by riprap. Because it is not known whether every square meter lies within the intertidal range, strictly speaking the areas may not be purely intertidal. However, for the sake of this thesis, the found areas are considered to be purely intertidal. Table P.1 shows the current intertidal areas. The column 'Location' shows the approximate distance along the river from the Waaier lock (Gouda). The areas are also depicted in figures P.1 and P.2.

Table P.1: Current intertidal areas, bordered by riprap.

Location	Side	Description	Location [m]	Area [m^2]
1	R	Near graveyard	494	6,330
2	L	Upstream sewage treatment plant	1,483	7,429
3	R	Near pumping station	2,966	2,907
4	R	Upstream of bungalow park	3,460	5,937
5	R	Downstream of bungalow park	3,954	27,340
6	L	Second area upstream Gouderak	3,954	1,272
7	L	First area upstream Gouderak	3,954	9,864
8	R	First area downstream Moordrecht	4,449	1,607
9	R	Second area downstream Moordrecht	5,932	5,204
10	L	First area downstream Gouderak	5,932	7,551
11	R	Third area downstream Moordrecht	6,426	6,024
12	R	Fourth area downstream Moordrecht	6,920	3,085
13	L	Second area downstream Gouderak	6,920	9,741
14	R	Fifth area downstream Moordrecht	7,415	13,604
15	L	Third area downstream Gouderak	7,415	18,307
16	L	Fourth area downstream Gouderak	7,909	8,188
17	R	First area upstream Nieuwerkerk	8,403	6,835
18	R	First area downstream Nieuwerkerk	8,897	1,662
19	R	Second area downstream Nieuwerkerk	9,886	19,547
20	L	Fifth area downstream Gouderak	10,380	2,347
21	R	First area downstream Klein Hitland	11,863	53,358
22	L	First area upstream Ouderkerk	11,863	4,981
23	R	Capelle, upstream marina	14,335	21,623
24	L	Krimpen	16,312	5,231
Total				249,974

P.2 Locations for new intertidal areas

This section shows the locations where intertidal areas can be realised. Where possible, the river was narrowed as far as possible from a navigational point of view. Current harbours or mooring places form an exception: here no narrowing (at least at the mooring side) was applied. Table P.2 describes the possible locations and their area. Again, the column 'Location' shows the approximate distance along the river from the Waaier lock (Gouda). The locations have been depicted in green in figures P.1 and P.2.

Table P.2: 'New' intertidal areas

Location	Description	Location [m]	L [m]	Avg. width [m]	Area [m^2]
25	Juliana lock - Zellingwijk Gouderak	3,954	1,440	45	64,800
26	Zellingwijk Gouderak	4,449	250	25	6,250
27	Close to ferry Moordrecht - Gouderak	4,943	110	25	2,750
28	Downstream Moordrecht - Zelling Kortenoord	6,426	1,800	50	90,000
29	Zelling Kortenoord - Klein Hitland	9,392	1,940	85	164,900
30	Klein Hitland	10,380	390	25	9,750
31	Klein Hitland - River bend	10,380	540	50	27,000
32	River bend - Groot Hitland	11,863	1,575	65	102,375
33	River bend Ouderkerk	12,852	560	50	28,000
34	Harbour Groenendijk - Marina Capelle	14,335	1,140	50	57,000
35	Marina Capelle - Berth location Krimpen	15,818	1,850	50	92,500
Total					645,325

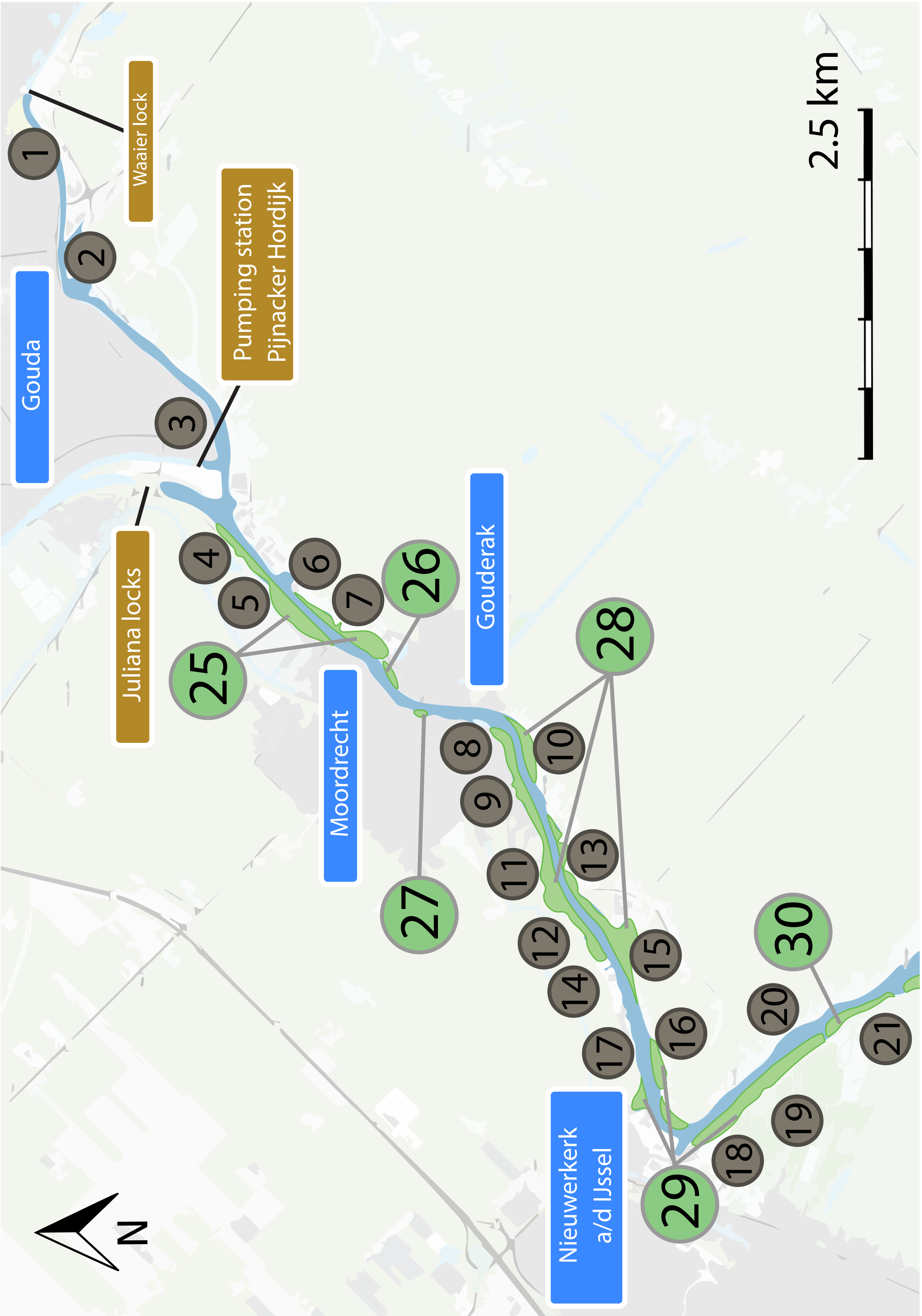

Figure P.1: Locations of intertidal areas. Current areas are marked grey, new areas are marked green.

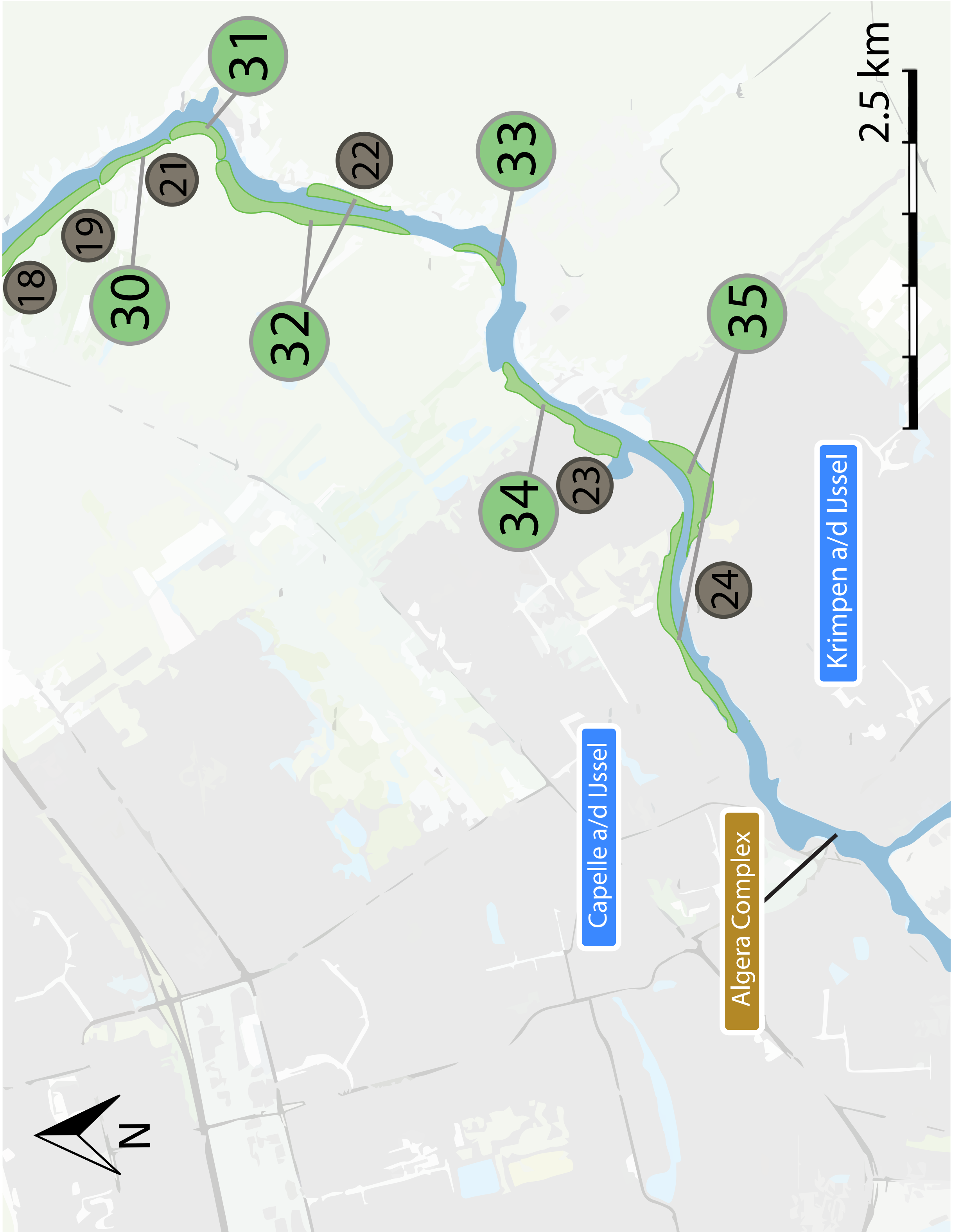

Figure P.2: Locations newly created intertidal areas. Current areas are marked grey, new areas are marked green.

Q

Subsoil conditions

This appendix shows the subsoil conditions. Section Q.1 shows the bathimetry, section Q.2 the CPTs and interpretation.

Q.1 Bathimetry

Figures Q.1 shows the bathimetry of the river in front of the current barrier. Figure Q.2 shows the bathimetry between the current doors of the Algera barrier. At the left side of this graph the lock can be distinguished.

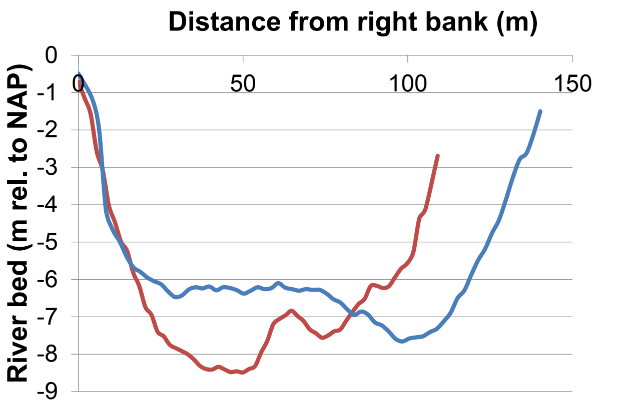

Figure Q.1: Bathimetry of the river 125 m (blue) and 350 m (red) south of current storm surge barrier. Source: Rijkswaterstaat.

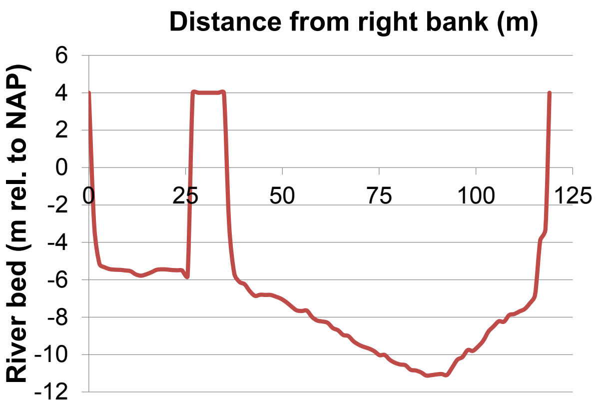

Figure Q.2: Bathimetry of the river in between current flood doors Algera barrier. Source: Rijkswaterstaat.

Q.2 Soil interpretation

This appendix shows the CPTs used for design. The soil types have been abbreviated by capital A-F, see table Q.1.

Table Q.1: Soil classification and parameters (NEN, 2012, table 2.b)

Type	Name	Density $[kN/m^3]$	Cohesion, c $[kN/m^2]$	Friction angle, ϕ [°]
A	“Clay”	15/15	3	17.5
B	“Clay silty”	16/16	4	20
C	“Clay, peaty”	15/15	2	15
D	“Peat”	10.5/10.5	2	15
E	“Sand”	18/20	0	32.5
F	“Sand clayey”	18/20	0	30

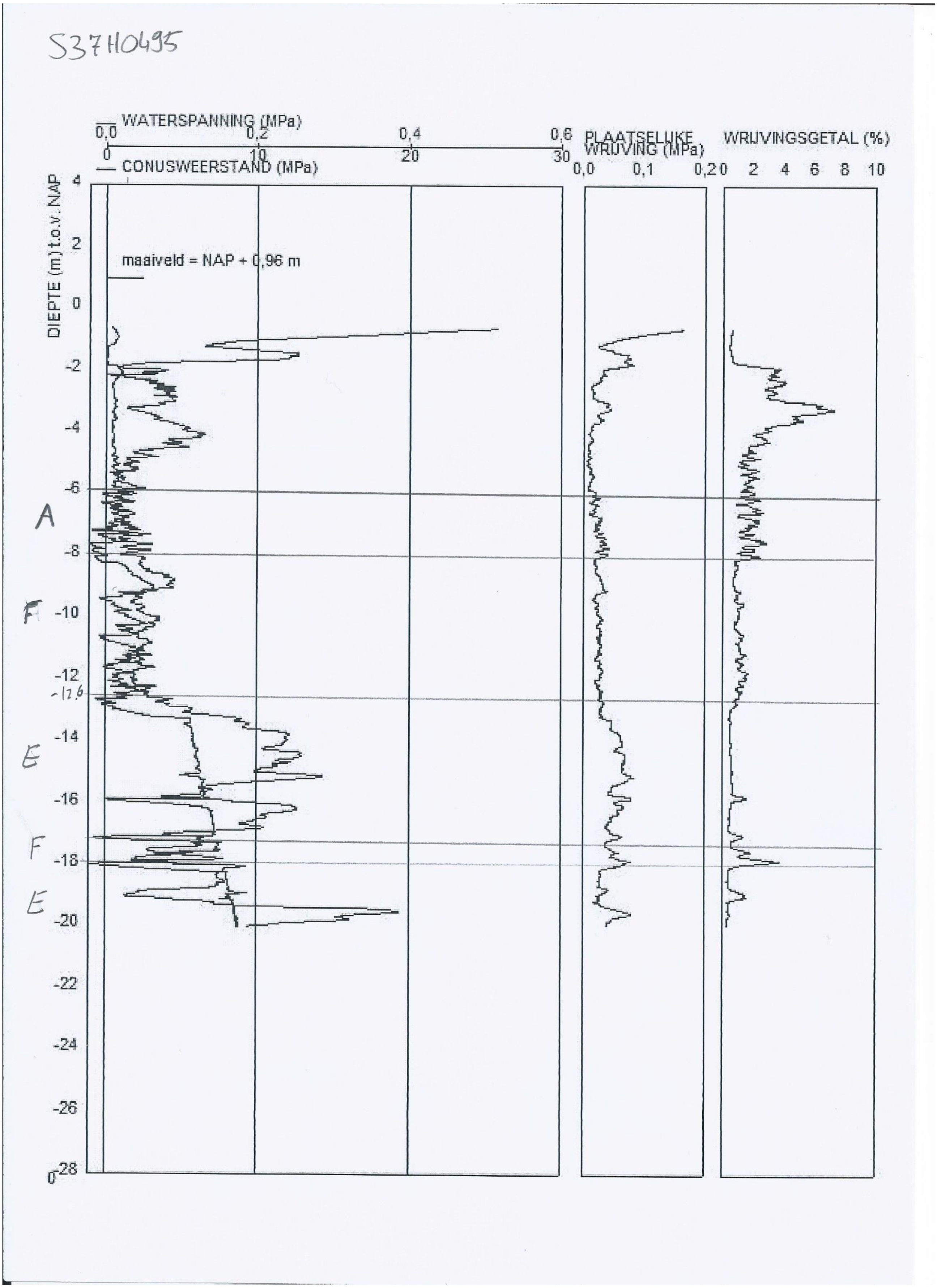

Figure Q.3: CPT S37H0495 (TNO Geologische Dienst Nederland, nd)

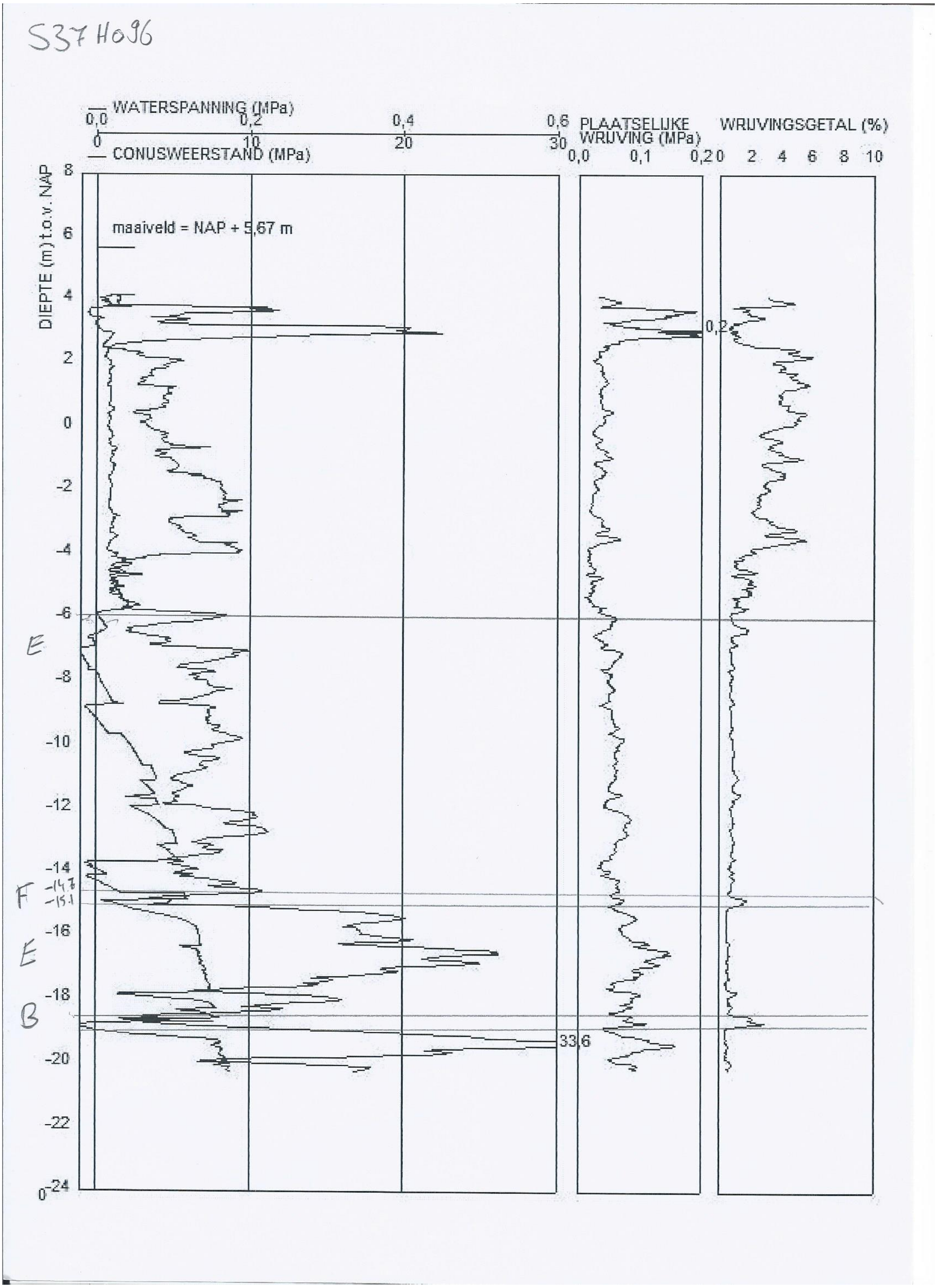

Figure Q.4: CPT S37H096 (TNO Geologische Dienst Nederland, nd)

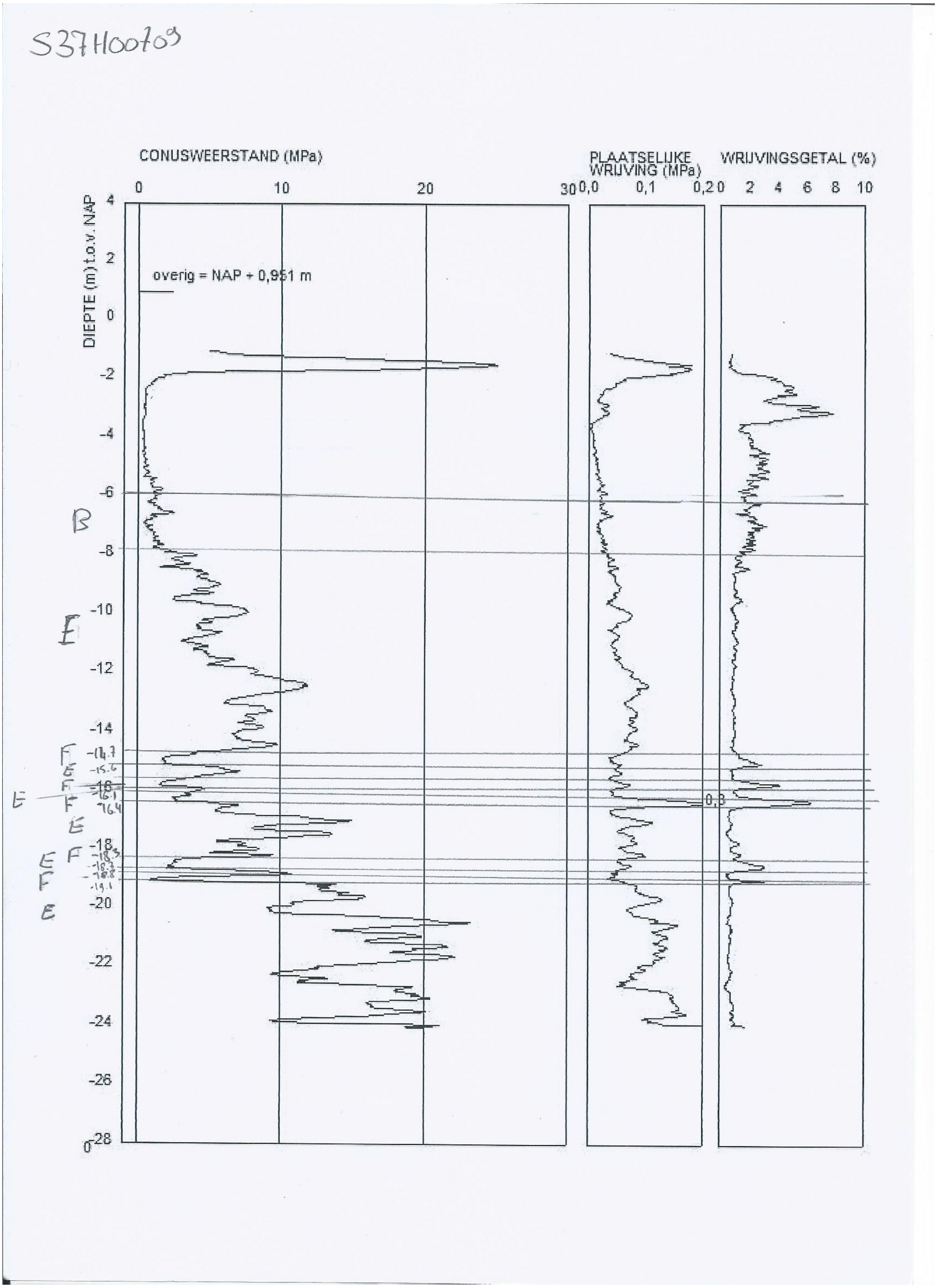

Figure Q.5: CPT S37H00709 (TNO Geologische Dienst Nederland, nd)

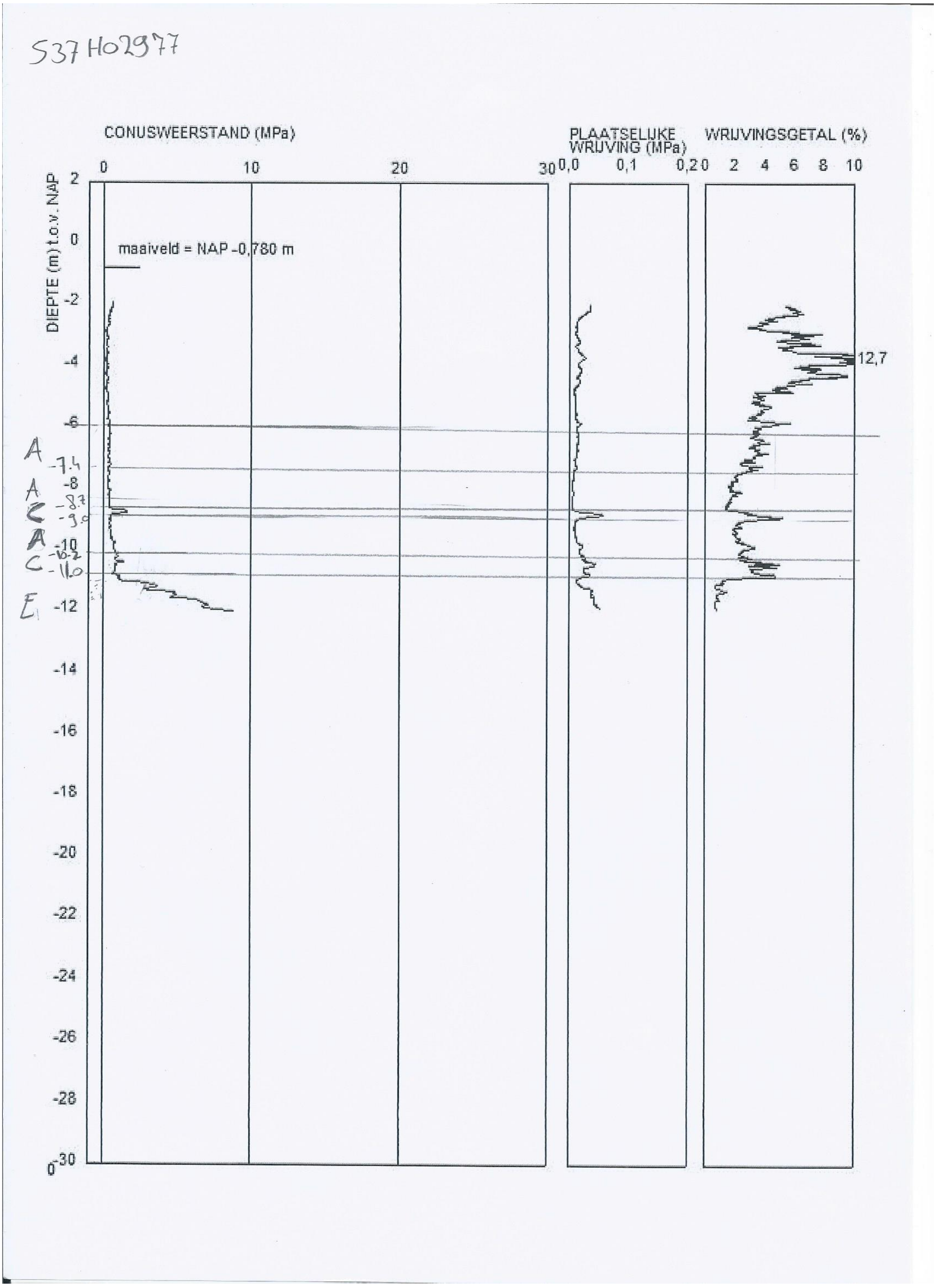

Figure Q.6: CPT S37H02977 (TNO Geologische Dienst Nederland, nd)

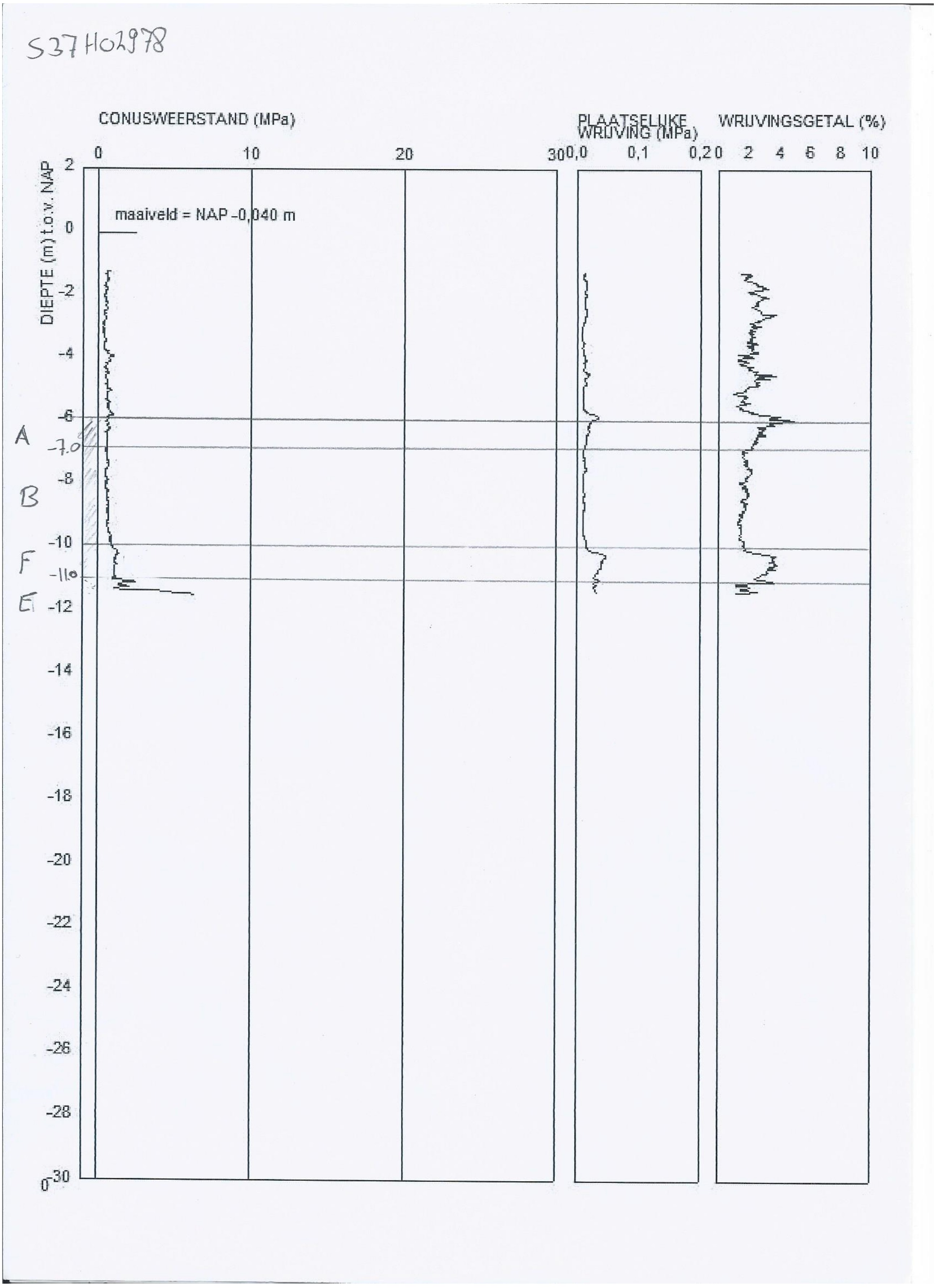

Figure Q.7: CPT S37H02978 (TNO Geologische Dienst Nederland, nd)

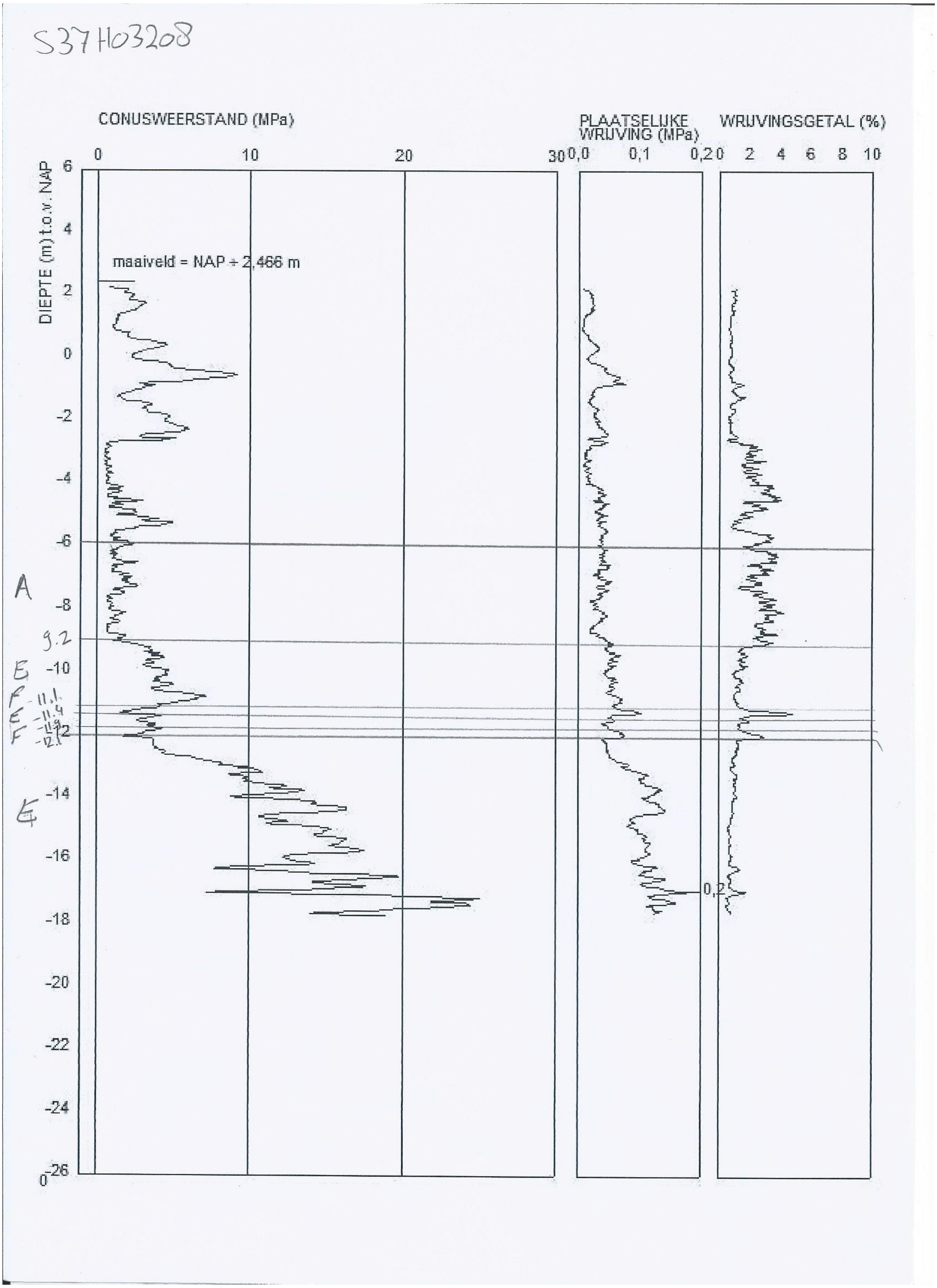

Figure Q.8: CPT S37H0496 (TNO Geologische Dienst Nederland, nd)

R
Overtopping analysis

This Appendix provides the reader background information on how the required crest height presented in chapter 9 were derived. Incoming waves hit a vertical wall at the inlet structure. The height of the structure is dependent on the allowable overtopping. It is assumed that in storm conditions, people need to be able to be present the top of the structure. Therefore the maximum allowable overtopping discharge is assumed to be 1 l/s/m (Pullen et al., 2007, p.31). The used values for this analysis are presented in table R.1.

Table R.1: Used values for wind wave loading

Symbol	Description	Input value	Unit
d	Water depth	11.25	[m]
u	Wind velocity at 10 m	42.0	[m/s]
F_{inlet}	Fetch	2140	[m]
$T_{p,inlet}$	Peak period	4.61	[s]
$H_{s,inlet}$	Significant wave height	1.65	[m]
$F_{connection}$	Fetch	60	[m]
$T_{p,connection}$	Peak period	2.00	[s]
$H_{s,connection}$	Significant wave height	0.40	[m]

To define whether impulsive or non-impulsive conditions are present, the “impulsiveness” parameter d_* needs to be calculated. If this parameter is larger than 0.3, non-impulsive (pulsating) conditions govern (Pullen et al., 2007, p.131).

$$d_* = 1.35 \frac{d \cdot 2\pi \cdot h_s}{H_s g T_{m-1,0}^2} \tag{R.1}$$

where according to TAW (2002),

$$T_{m-1,0} = \frac{T_p}{1.1} \tag{R.2}$$

Using the wave and geometrical values for the situation presented above results in d_*=3.51: a non-impulsive environment. Equation R.3 provides the overtopping discharge for a vertical wall in this environment (Pullen et al., 2007, p.132).

$$q = 0.04 e^{\left(-1.8 \frac{R_C}{H_S}\right)} \sqrt{g H_S^3} \tag{R.3}$$

Here, R_c represents the freeboard. The required overtopping discharge is reached if the crest height is at NAP +9.4 m, compared to the required NAP +4.75 m for the surge water level. Therefore, two mitigating measures are considered:

1. **Safety zone**: close to the edge (5 m) of the structure no people are allowed.
2. **Bull nose**: a bull nose could deflect back uprushing water.

Safety zone
The overtopping discharge at a distance x can roughly be estimated by (Pullen et al., 2007, p.31):

$$q_{effective} = \frac{q}{x} \tag{R.4}$$

This leads to required crest height at NAP +7.9 m, a reduction of 1.5 m compared to the initial calculation.

Bull nose
A bull nose deflects back uprushing water (figure R.1). The reduction factor is k for a nose towards the water, $\alpha < 90°$, is calculated using a decision chart, see figure R.2. A nose with a height and depth of 0.5 m ($\alpha = 45°$) is used.

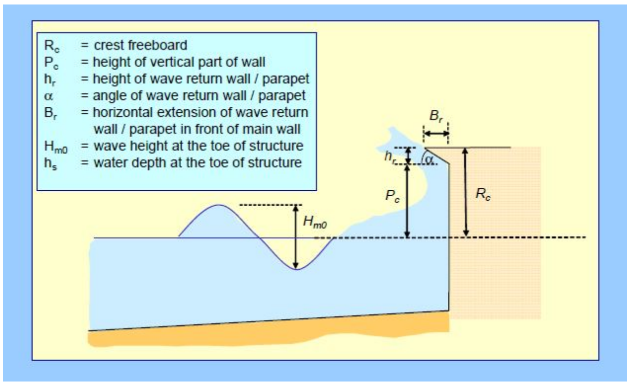

Figure R.1: Schematisation of a bull nose. Source: (Pullen et al., 2007)

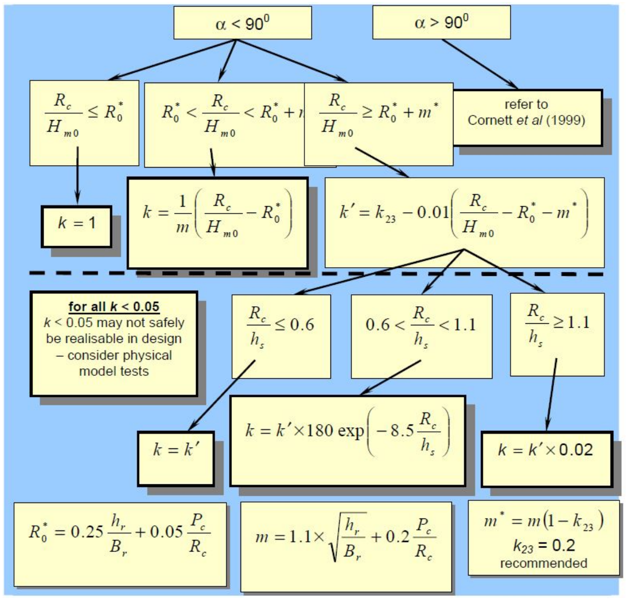

Figure R.2: Decision chart k-value. Source: (Pullen et al., 2007)

Iteration shows an enormous reduction of overtopping discharge for crest heights above NAP +6.4 m. Therefore, this value should be treated with caution if a nose is used in further design. However, given the conceptual nature of this thesis, the value of NAP +6.4 m is used.

Table R.2: Summary of findings for crest level of inlet structure

Structure	Option	Crest level [m NAP]
Inlet	No measures	+9.4
	Safety zone	+7.9
	Bull nose	+6.4

S
Uplift building pit

In this appendix the uplift of the building pit is discussed. When the current Algera barrier was constructed, a loam layer at NAP -23 m was used to make the building pit watertight (Rijkswaterstaat, 1959, p.37). This layer will be used to seal off the building pit. Sheet piles will be installed until NAP -23 m.

S.1 Pump structure

Assumptions have been made to make the uplift calculation. Furthermore, the bottom level of the building pit is based on chapter 9. An overview of the situation is provided in figure S.1.

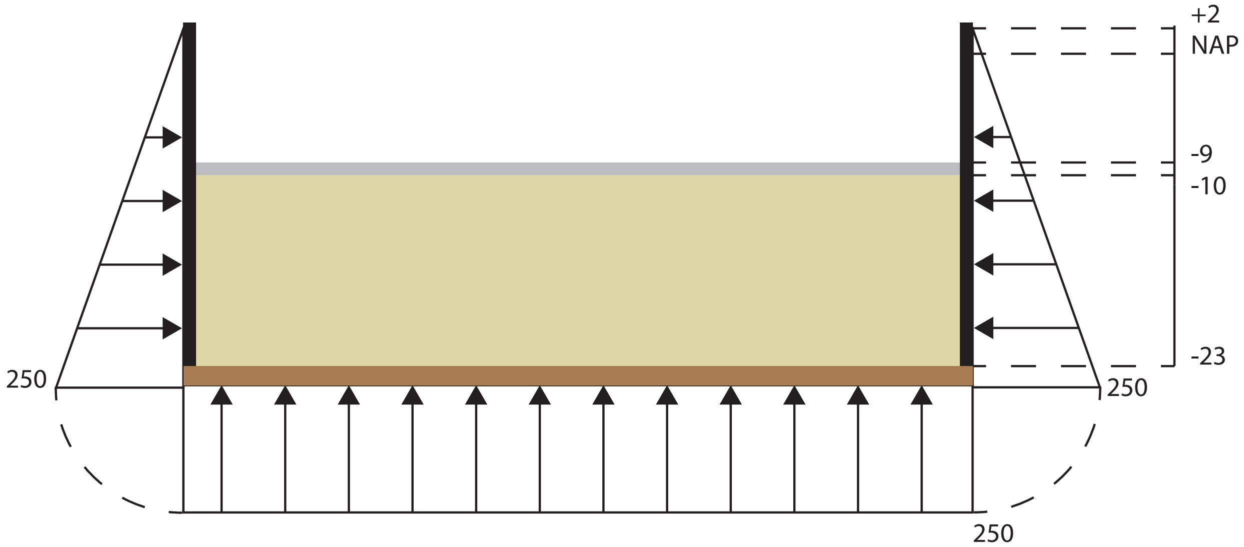

Figure S.1: Building pit pump structure.

Assumptions:

- Soil has a saturated weight of 20 kN/m^3;
- Under water concrete (UWC) has a weight of 25 kN/m^3 and is used as foundation slab for the structure;
- Maximum water level outside the building pit lies at NAP +2 m;
- The floor level of the building pit lies at NAP -10 m.

Upward pressure is calculated by the water pressure. The safety factor is obtained from NEN (2012).

$$P_{up} = \gamma \cdot \rho_{water} \cdot g \cdot h_{water} \tag{S.1}$$

$$P_{up} = 1.0 \cdot 10 \cdot 25 = 250 \quad kPa$$

Downward pressure is calculated by summation of the (saturated) weight of the soil layers above the loam layer. The safety factor is obtained from NEN (2012).

$$P_{down} = \gamma \cdot (\rho_{UWC} \cdot g \cdot h_{UWC} + \rho_{sand} \cdot g \cdot h_{sand}) \tag{S.2}$$

$$P_{down} = 0.9 \cdot (25 \cdot 1 + 20 \cdot 13) = 256.5 \quad kPa$$

S.2 Culvert structure

For the culvert structure, a similar calculation is carried out. The building pit has a smaller depth than the building pit for the pump structure: the floor lays at NAP -7.5 m. The upward water pressure remains the same.

$$P_{down} = 0.9 \cdot (25 \cdot 1 + 20 \cdot 14,5) = 283.5 \quad kPa$$

S.3 Conclusion

Both building pits provide sufficient resistance against uplift as can be seen in table S.1.

Table S.1: Summary uplift calculations.

Structure	Upward [kPa]	Downward [kPa]	Unity Check
Pump	250	256.5	1.03
Culvert	250	283.5	1.13

T

Ecological impact

Appendix K has shown the impact of the new inlet structure on the tidal range in the river. Section T.1 will look at the tidal range at each of the intertidal areas in more detail. In order to do so, the tidal ranges will be compared for September 2016. After that, the possibility for fish to migrate is looked into (section T.2), followed by a summary of the findings (section T.3).

T.1 Tidal ranges

Table T.1 shows the tidal ranges, Habitat Suitability Indices (HSIs) for the Driekantige Bies of each of the current intertidal areas. The higher the location number, the more downstream the area is. Due to resonance, the tidal range near Gouda is larger, resulting in a higher HSI. It can be seen that the suitability reaches high values. Table T.2 shows the situation after the inlet structure is implemented (situation 5 of table K.10). Except for areas 1, 2 and 3, the old intertidal areas were expanded and merged to form the new intertidal areas (location 25 - 35). While tidal ranges are almost 90% of the original, the HSI in the river drops drastically. Reason for this is the high sensitivity of the suitability index, as can be seen in figure 4.6. Between 1.6 and 1.0 m tidal range, the index drops from 1 (Excellent suitability) to 0 (Not suitable). However, the available area has increased (to almost the size of the city center of Delft) and is less fragmented.

Table T.1: Tidal range and Habitat Suitability Index (Driekantige Bies) of the intertidal flats, current situation.

Location	Area [m^2]	Tidal range	HSI
1	6,330	1.64	1
2	7,429	1.64	1
3	2,907	1.62	1
4	5,937	1.62	1
5	27,340	1.61	1
6	1,272	1.61	1
7	9,864	1.61	1
8	1,607	1.61	1
9	5,204	1.59	0.98
10	7,551	1.59	0.98
11	6,024	1.58	0.97
12	3,085	1.57	0.96
13	9,741	1.57	0.96
14	13,604	1.57	0.95
15	18,307	1.57	0.95
16	8,188	1.56	0.94
17	6,835	1.56	0.93
18	1,662	1.55	0.91
19	19,547	1.53	0.88
20	2,347	1.51	0.86
21	53,358	1.49	0.81
22	4,981	1.49	0.81
23	21,623	1.43	0.72
24	5,231	1.39	0.66
Total	249,974	1.54	0.90

Table T.2: Tidal range and Habitat Suitability Index (Driekantige Bies) of the intertidal flats, new situation.

Location	Area [m^2]	Tidal range	HSI
1	6,330	1.43	0.71
2	7,429	1.42	0.71
3	2,907	1.42	0.70
25	64,800	1.42	0.69
26	6,250	1.42	0.68
27	2,750	1.41	0.68
28	90,000	1.39	0.64
29	164,900	1.35	0.58
30	9,750	1.33	0.55
31	27,000	1.33	0.55
32	102,375	1.32	0.53
33	28,000	1.31	0.52
34	57,000	1.21	0.51
35	92,500	1.21	0.52
Total	691,091	1.35	0.58

To compare both situations, one could multiply the available area by the average HSI:

$$Score = A_{total} \cdot HSI \tag{T.1}$$

Table T.3: Summary of current and future intertidal flats.

Situation	Area	HSI	Score
Current	249,974	0.90	224,741
New	691,091	0.58	381,998

T.2 Fish migration

In chapter 4, two aspects were introduced that are important for fish migration: the cross-section and flow velocity. The impact of the barrier on both aspects is discussed in the paragraphs below.

T.2.1 Cross-section

For fish migration, cross-sections should have a width above 1.0 m (safe side, see chapter 4). Water depths should be greater than 0.5 m. The projected culverts have dimensions well above the minimum values: the square culverts have sides of 2.7 m. It is therefore concluded that the cross-sections are large enough for fish to migrate.

T.2.2 Velocity

Besides water levels, SOBEK also provides discharges. Average velocities in the culverts can be obtained by dividing the discharge by the total cross-section. The results for begin September 2016 are shown in figure T.1. Fish can migrate with flow velocities smaller than 0.8 m/s, see chapter 4. It was found that values are below 0.8 m/s about 67% of the time. Fish are therefore able to migrate, although not continuously. The effects of this ‘migration window’ could be a subject for further research.

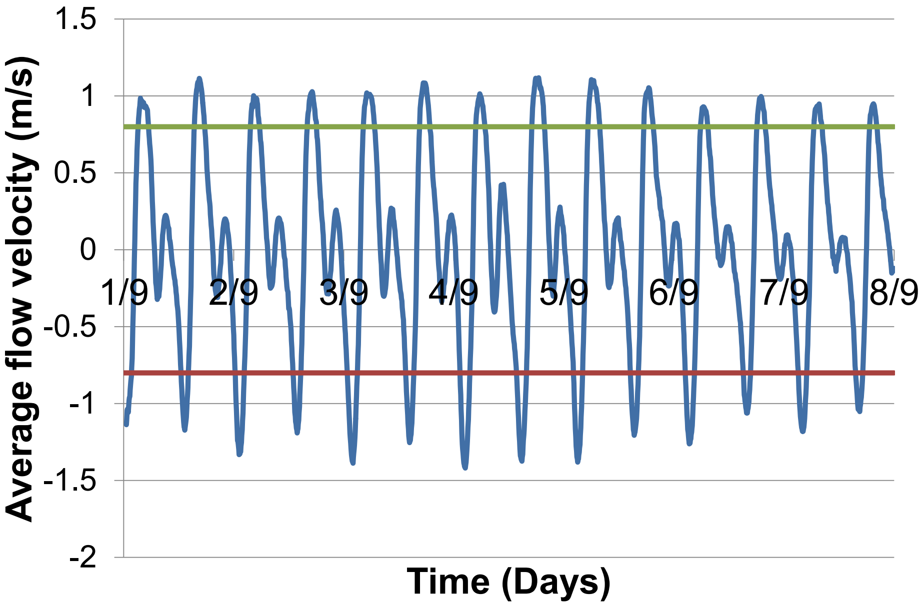

Figure T.1: Average flow velocity through the barrier (blue) and advised flow velocities for fish migration (red and green).

T.3 Summary

Although almost 90% of the tidal range has been preserved, the HSI for the Driekantige Bies is drastically reduced. The play field is very narrow: the suitability drops from 1.0 for tidal ranges larger than 1.60 m to 0 for tidal ranges below 1.0 m. Although the suitability decreased, the available area increased in the proposed design. One could argue that the decrease in tidal action is compensated by the increase in intertidal area. Furthermore, the size of the cross-sections is large enough for fish to migrate. The flow velocities in the barrier allow fish to migrate about 67% of the time.

U

Construction cost

In this appendix the direct cost of construction are calculated. In the summary, building cost and budget/investment cost will be presented. The unit prices mentioned throughout this Appendix were discussed with cost experts within Royal HaskoningDHV (Meinderts, 2017; Bakker, 2017). The cost mentioned are 'all-inclusive' and include e.g. cost for raw materials, delivery, man hours and removal.

U.1 Cost factors

In cost estimations four different cost can be distinguished:

1. Direct construction cost;
2. Building cost;
3. Budget or investment cost (Excl. VAT);
4. Budget or investment cost (Incl. VAT).

The building and investment cost are calculated by multiplying the direct construction cost by a factor (*opslag-factoren*):

$$C_{building} = C_{direct} \cdot c_{building} \tag{U.1}$$

$$C_{budget,noVAT} = C_{building} \cdot c_{budget} \tag{U.2}$$

$$C_{budget,VAT} = C_{budget,noVAT} \cdot VAT \tag{U.3}$$

Where, $c_{building}$ = 1.5, c_{budget} = 1.4 and (Meinderts, 2017) and VAT = 21%.

U.2 Building pit

To calculate the direct construction cost of the building pit, various unit rates have been assumed (see table U.1). To calculate the cost of the sheetpiles, a number of extra assumptions were made:

- Volumetric weight steel: 7,850 kg/m^3;
- Thickness sheet pile: 20 mm;
- Price sheetpiles: 3,500 €/t (struts included).

Table U.1: Direct construction cost building pit (2017).

	Quantity		Cost/unit rate		Cost [€] (2017)
Foundation layer (sand)	2,763	$[m^2]$	40	[€/m^3]	110,500
Sheet piles	85	$[m^3]$	27,475	[€/m^3]	2,321,638
Underwater concrete	5,525	$[m^3]$	175	[€/m^3]	966,875
Dewatering	5,525	$[m^2]$	7.5	[€/m^2]	41,438
Mobilisation/Demob.	1	[-]	500,000	[€]	500,000
Total					3,940,450

U.3 Construction

To calculate the direct construction cost of the construction, various unit rates have been assumed (see table U.2). To calculate the cost of reinforcement steel a number of extra assumptions were made:

- Volumetric weight steel: 7,850 kg/m^3;
- Percentage reinforcement in construction: 150 kg/m^3;
- Price construction steel: 1,100 €/t.

Table U.2: Direct construction cost structure (2017).

	Quantity		Cost/unit rate		Cost [€] (2017)
Dredging	7,260	[m^3]	7.5	[€/m^3]	54,450
Scour protection	8,000	[m^2]	40	[€/m^2]	320,000
Concrete	25,667	[m^3]	140	[€/m^3]	3,593,380
Reinforcement steel	25,667 (concrete)	[m^3]	8,635	[€/m^3]	4,235,055
Crane track	67	[m^3]	750	[€/m^3]	52,250
Total					8,253,135

U.4 Valves

The culvert structure needs valves to close the barrier in case of storm surge and when the pumps are operational. Every culvert is closed off by a pivot valve. Furthermore, an extra closure mechanism is needed as emergency closure mechanism and for maintenance purposes. The lower culverts are closed off by hydraulic valves (two at each side of the culvert. The upper culverts are closed by stop locks. The price for the valves is calculated based on expert judgement (Bakker, 2017). The direct construction cost are tabulated in table U.3.

Table U.3: Direct construction cost valves (price level 2017).

	Quantity	Cost/unit rate	Cost [€] (2017)
Pivot valves	31		
- Valves	31	60,000	1,860,000
- Hydraulic system	31	25,000	775,000
Flap valves	60		
- Valves	60	90,000	5,400,000
- Hydraulic system	60	30,000	1,800,000
Stop locks	64	2,187	139,968
Total			9,974,968

U.5 Pumps

Royal HaskoningDHV provided a cost estimate of pumps, see figure U.1. It was found that, for 2014, the budget cost for the size of the pumps (≈20 m^3/s) lays around €450,000 per m^3/s. This leads to direct construction cost of €213,470 per m^3/s in 2017.

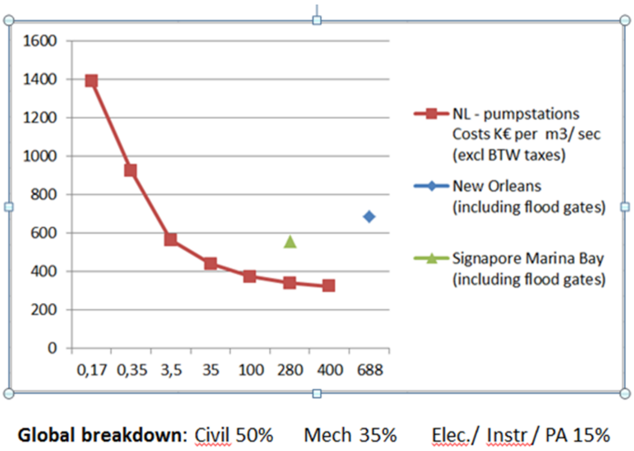

Figure U.1: Investment cost based on pumping capacity. Source: RHDHV.

Roughly, the cost are broken down into three aspects:

1. Civil construction: 50%
2. Mechanic construction: 35%
3. Electronics, Instrumentation & Process Automatisation (E& I, PA): 15%

The civil construction cost have been taken into account in section U and are therefore not taken into account again. Table U.4 shows the direct construction cost of the pump.

Table U.4: Direct construction cost for pumps (price level 2017).

	Cost per pump [€]	Total cost [€]
Mechanic construction	1,494,290	8,965,740
Electronics, Instrumentation & Process Automatisation	640,410	3,842,460
Total	2,134,700	12,808,200

U.6 Intertidal areas

In this section a rough estimate on the cost of creation of extra intertidal areas is provided. The locations are shown in figures P.1 & P.2. For the cost calculation, the intertidal areas are schematised in figure U.2.

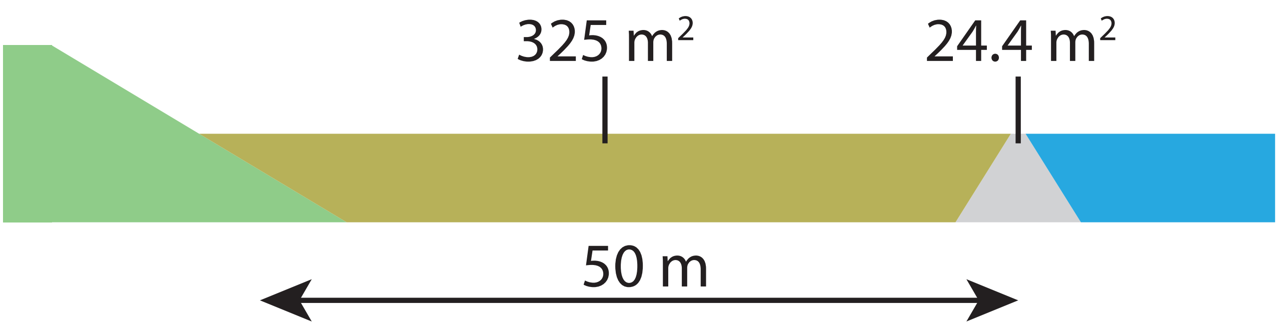

Figure U.2: Schematisation of the intertidal areas.

For the cost estimation, it is assumed that the intertidal areas cover approximately 75% of the river length. In the new situation 691,091 m^2 is available (table T.2). The current areas have a total surface of 249,974 m^2 (table T.1). So, approximately 64% of the area still needs to be realised. The length over which the cross-section depicted in figure U.2 needs to be realised is calculated by:

$$L = L_{river} \cdot coverage \cdot 0.64 = 18,000 \cdot 0.75 \cdot 0.61 = 8,617 \quad m \tag{U.4}$$

Furthermore, the following assumptions were made:

- Weight stone: 2.7 t/m^3 (Schiereck, 2012, p.337);
- Void ratio riprap: 0.5.

The direct construction cost of the new intertidal areas (as if realised in 2017) are presented in table U.5.

Table U.5: Direct construction cost realisation intertidal areas (price level 2017).

	Quantity [m^3]	Cost/unit rate [€/m^3]	Cost [€] (2017)
Sand	2,687,951	30	84.015,021
Riprap	151,197	40.5	6,379,891
Total			90,394,912

Summary cost 2017
The direct construction cost mentioned in tables U.1 - U.5 are accumulated in table U.6. The factors mentioned in section U.1 were used to translate the direct construction cost to building and investment cost.

Table U.6: Summary of cost (price level 2017).

	Direct cost	Building cost	Investment cost (excl. VAT)	Investment cost (incl. VAT)
Building pit	3,940,450	5,910,675	8,274,945	10,012,683
Construction	8,253,135	12,379,703	17,331,584	20,971,216
Valves	9,974,968	14,962,452	20,947,433	25,346,394
Pumps	12,808,200	19,212,300	26,897,220	32,545,636
Intertidal areas	90,394,912	135,592,368	189,829,315	229,693,471
Total	125,371,665	188,057,498	263,280,497	318,569,401

U.7 Values for sensitivity analysis

Tables U.7 - U.12 show the Present Value (cost) of the different aspects when the barrier is realised in either 2030, 2050 or 2070. A difference was made between the DRUK/STOOM scenario and the RUST/WARM scenario. In the scenarios different economic growth/inflation is assumed. For DRUK/STOOM this is 2.5% per year, for RUST/WARM, this is 1% per year.

Table U.7: Present value (cost) building pit.

	2030	2050	2070
RUST/WARM	5,681,282	2,375,881	993,580
DRUK/STOOM	6,881,431	3,864,621	2,170,376

Table U.8: Present value (cost) construction.

	2030	2050	2070
RUST/WARM	11,899,248	4,976,199	2,081,019
DRUK/STOOM	14,412,917	8,094,313	4,545,777

Table U.9: Present value (cost) valves.

	2030	2050	2070
RUST/WARM	18,466,673	7,722,660	3,229,574
DRUK/STOOM	22,367,685	12,561,721	7,054,679

Table U.10: Present value (cost) pumps.

	2030	2050	2070
RUST/WARM	18,466,673	7,722,660	3,229,574
DRUK/STOOM	22,367,685	12,561,721	7,054,679

Table U.11: Present value (cost) intertidal areas.

	2030	2050	2070
RUST/WARM	130,330,042	54,503,303	22,792,980
DRUK/STOOM	157,861,755	88,655,369	49,788,972

Table U.12: Accumulated Present value (cost).

	2030	2050	2070
RUST/WARM	184,843,917	75,592,416	31,612,330
DRUK/STOOM	218,943,640	122,959,036	69,053,956