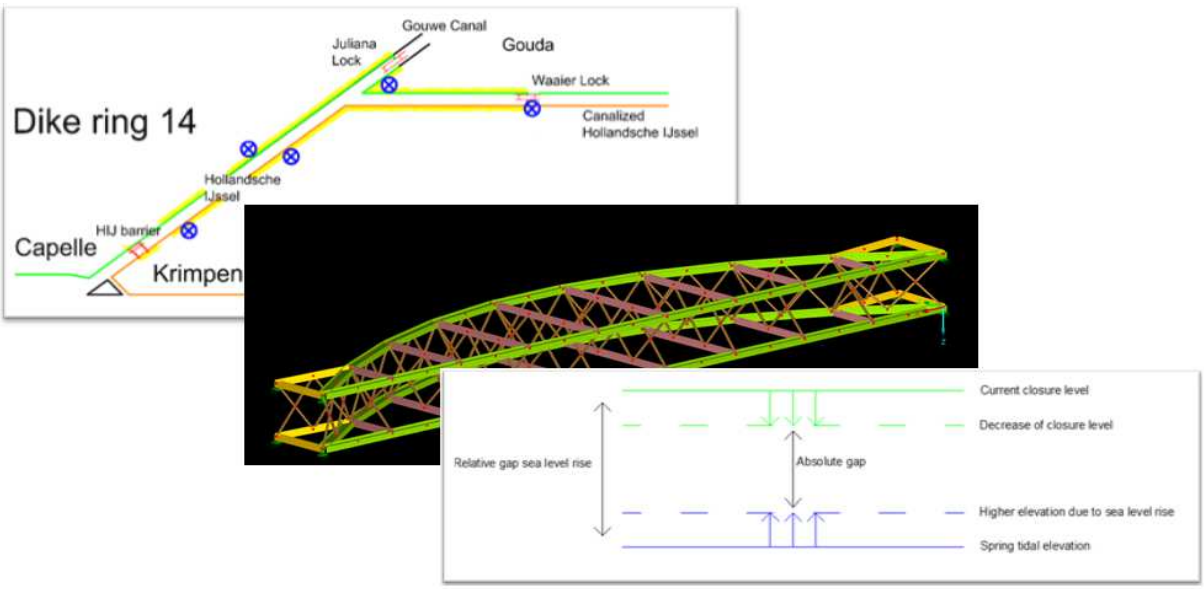
MASTER THESIS REPORT

ADAPTATION STORM SURGE BARRIER HOLLANDSCHE IJSSEL

Matthieu Welsink

Rotterdam, 21 June 2013



GRADUATION COMMITEE

Prof. dr. ir. S.N. Jonkman
Ing. M.Z. Voorendt
Ir. W.F. Molenaar
Ir. P. Ravenstijn
Prof. dr. ir. F.S.K. Bijlaard

TU Delft Delft University of Technology

Witteveen + Bos

This page is intentionally left blank

Colophon

Graduate

M.W.J. Welsink BSc
Johan Wagenaarlaan 29
2343 ND Oegstgeest
Tjeuwelsink@gmail.com
+31(0) 61 08 39 052
Student number: 1333860

Graduation committee

Chairman graduation committee
Prof. dr. ir. S.N. Jonkman
S.N.Jonkman@tudelft.nl
+31(0) 15 27 85 278

Supervisor from the TU Delft
Ing. M.Z. Voorendt
M.Z.Voorendt@tudelft.nl
+31(0) 15 27 83 340

Supervisor from the TU Delft
Ir. W.F. Molenaar
W.F.Molenaar@tudelft.nl
+31(0) 15 27 89 447

Supervisor from Witteveen+Bos consulting engineers
Ir. P. Ravenstijn
P.Ravenstijn@witteveenbos.nl
+31(0) 10 24 42 827

Supervisor from the TU Delft section: Steel, hybrid and composite structures
Prof. dr. ir. F.S.K. Bijlaard
F.S.K.Bijlaard@tudelft.nl
+31(0) 15 27 84 581

Trademarks

MATLAB R2010b is a registered trademark of The Math Works, Inc.

MatrixFrame is a registered trademark of Matrix Software B.V.

AutoCAD Civil 3D 2012 is a registered trademark of Autodesk

RSTAB 8.01 is a registered trademark of Ingenieur-Software Dlubal GmbH

Google Maps is a registered trademark of Google Inc.

Preface

The thesis is part of the master Hydraulic Engineering, specialization Hydraulic Structures, and marks the end of my study at the Delft University of Technology (faculty of Civil Engineering).

In this thesis the situation in the Hollandsche IJssel is studied. The situation in the Hollandsche IJssel focuses on the many problems associated with climate change, traffic and safety of the flood defences. The center of this thesis is the storm surge barrier at the mouth of the Hollandsche IJssel River which protects the hinterland against flooding.

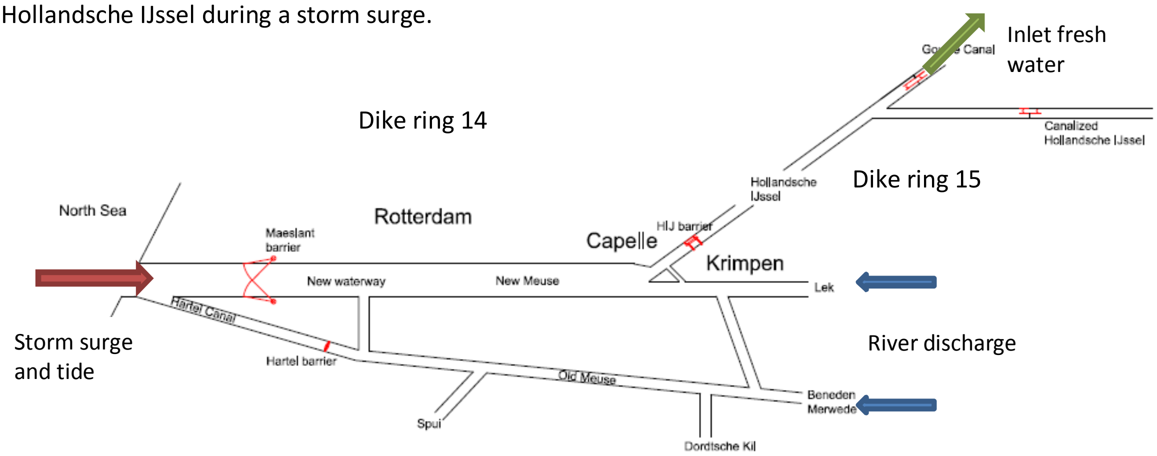
I would like to thank Witteveen+Bos consulting engineers for providing a workplace at their Rotterdam office. Furthermore I would like to thank the members of my graduation committee, my friends and my colleagues at Witteveen+Bos for their help and support during my master thesis.

Rotterdam, June 2013

M.W.J. Welsink

Summary

The Hollandsche IJssel is a river which flows from Nieuwegein to Capelle were it ends at the New Meuse. In the twentieth century the river was divided into two parts; the canalized part which flows from Nieuwegein to Gouda and the tidal part which flows from Gouda to Capelle. After the flood disaster of 1953 the Government decided to build a storm surge barrier in the mouth of the Hollandsche IJssel closing of the tidal part of the Hollandsche IJssel during a storm surge.



Schematization Rijnmond and Hollandsche IJssel system

After the 2008 report of the second Delta Committee the Delta programs studied the flood protection and effects of climate change in the Rijnmond and Hollandsche IJssel system and concluded that [1];

- the overall safety of the dike rings protecting Central Holland and the Krimpenerwaard is very low,
- the fresh water inlet near Gouda would need to stop because of salt intrusion during low river discharges.

Due to climate change it is expected that the sea level rises and the average discharge during summer months decreases, therefore the mentioned problems will probably increase in the future. The climate studies do not provide a clear picture of climate change due to large uncertainties in the studies of the KNMI and IPCC.

Overall safety

The overall safety of the two dike rings is low because the levees along the Hollandsche IJssel have a steep, unstable inner slope during governing conditions.

- The third nationwide safety assessment (comparable to an APK for cars) concluded that the levees along the Hollandsche IJssel and the Hollandsche IJssel storm surge barrier are not up to the current standards.
- The program Safety in the Netherlands concluded that the risks due to a flood in dike ring 14 and 15 were too high. The risk of a flood is the likelihood that a breach occurs during a storm surge multiplied with the consequences (economic damage and loss of life) of that breach.

Fresh water inlet

The fresh water inlet near Gouda is needed for the flushing of Central Holland. This flushing ensures that the canals of Central Holland do not become brackish. The canals need to maintain fresh water in order to grow crops and maintain kettle. Salt intrusion entering the system due to the tide and low discharges prevents the inlet of water because the water in the Hollandsche IJssel becomes salt. Due to this the inlet stops during salt intrusion.

Objective

A study which looks at the integrated system of the Hollandsche IJssel and solves the aforementioned problems in combination with aspects like morphology, ecology, limited budget and the local surrounding is necessary.

The objective of this study is the development and (conceptual) design of the preferred strategy for the important aspects (overall safety, salt intrusion and climate change) in the Hollandsche IJssel. The preferred strategy is cost-effective and exists of a technically and societally feasible design.

Strategies

In the first part of this study different strategies to solve the problems in the Hollandsche IJssel system are compared, these strategies are;

- Maintaining the existing situation in the Hollandsche IJssel,
- Adaptation of the existing storm surge barrier and construction of a dam or new storm surge barrier when necessary,
- Construction of a dam in the Hollandsche IJssel which closes off the entire Hollandsche IJssel,
- Construction of a new storm surge barrier at the mouth of the Hollandsche IJssel.

Adaptation of the existing Hollandsche IJssel storm surge barrier and postponing of a dam or new storm surge barrier is preferred because:

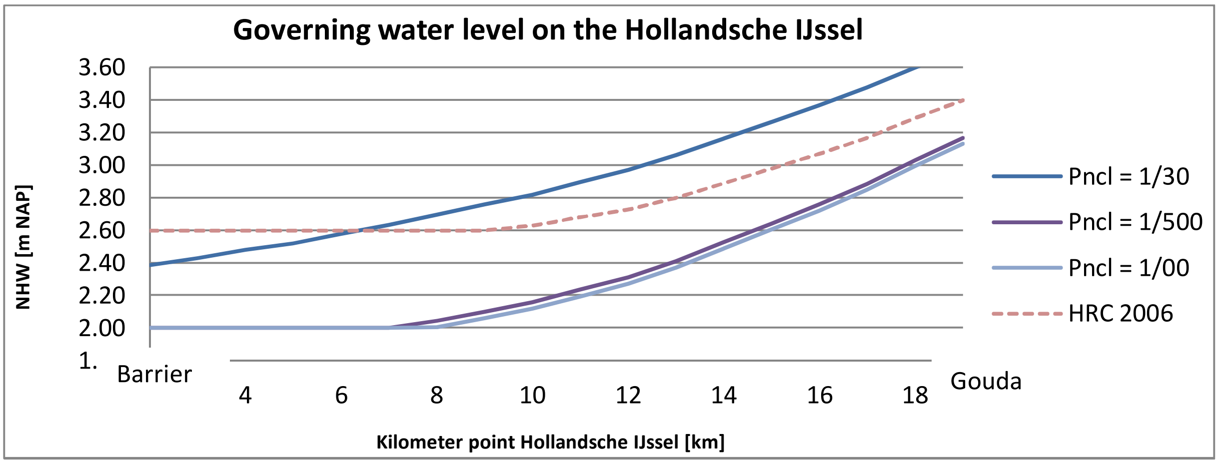
- The uncertain sea level rise influences the necessity and effectiveness of a dam or new storm surge barrier. When the existing storm surge barrier is adapted the sea level rise can be monitored.
- The open connection to the sea is important for tidal nature and shipping in the Hollandsche IJssel.
- The investments needed for a new structure are large while adaptation of the existing storm surge barrier is relatively cheap.

Closure scheme

The table shows the new closure scheme that is introduced for the Hollandsche IJssel storm surge barrier. The closure level of the Hollandsche IJssel storm surge barrier reduces from +2.25 m NAP to +1.75 m NAP this results in a water level decrease of approximately 0.50. Closure because of a storm surge occurs during the ebb slack period because of the lower water levels, closure because of salt intrusion occurs during the ebb slack flood slack because this increases the storage of water on the Hollandsche IJssel.

New closure scheme Hollandsche IJssel storm surge barrier

	Storm surge	Salt intrusion
Closure level	+1.75 m NAP	250 mg/l
Closure period	During ebb slack	During flood slack
Pump/inlet stop level	+2.00 m NAP	-0.5 m NAP

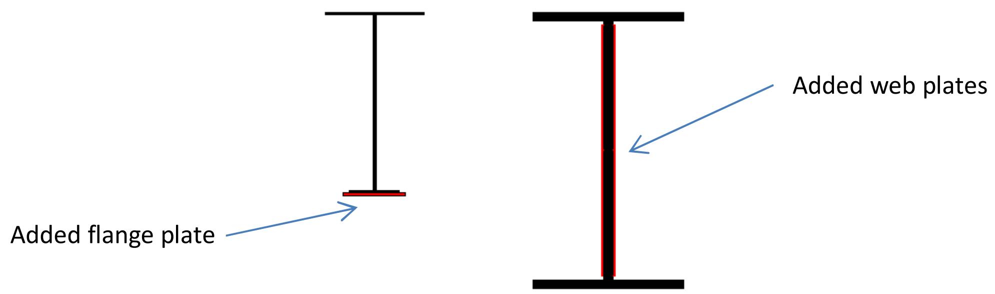


Governing water levels along the Hollandsche IJssel, exceedance probability safety level 1/ 10 000

The decrease of the governing water levels due to the reduction of the closure level is threatened because of the low closure reliability of the Hollandsche IJssel storm surge barrier. The closure reliability is expressed in the non-closure probability, which is the likelihood that the storm surge barrier does not close when there is a closure request. The non-closure probability of the existing barrier is 1/30 per event and results in an increase of the governing water levels. A non-closure probability of 1/30 means that one out of the thirty closure request results in an open barrier and therefore high water levels on the Hollandsche IJssel. The targeted decrease that should be reached to maintain the decrease of 0.50 meter is 1/500. The effect of the non-closure probability then increases the water levels with approximately 0.05 meter (shown in the figure below). The HRC 2006 line shows the current governing water levels on the Hollandsche IJssel.

Structural adaptations

Elements of the storm surge barrier need to be adapted to withstand the increased loads or guarantee the use of the storm surge barrier during salt intrusion. The capacity of elements within the storm surge barrier needs to be increased to withstand the increased loads. Flange plates, shown in red, should be welded to the flange of the transverse girder, web plates should be welded to the web of the curved arch.

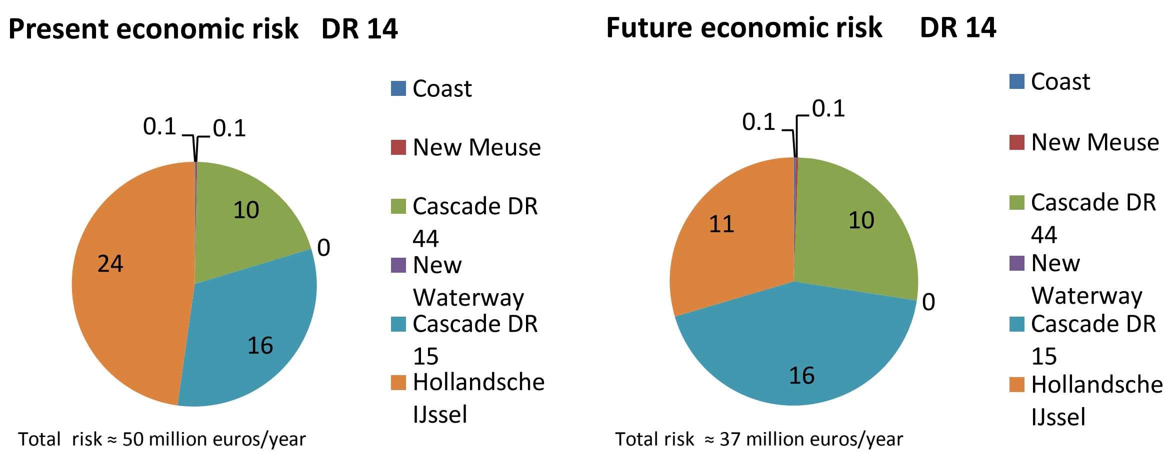


Adaptation welded profiles; transverse girder (left) and curved arch (right)

Scour protection is needed directly behind the storm surge barrier to prevent scour holes migrating under the barrier. Construction of a vertical slot fish passage is necessary to let fish pass the storm surge barrier during closure. The independency of the two lift gates should be increased to increase the closure reliability of the storm surge barrier and therefore reduce the effect of the non-closure probability.

Effects of the adaptations

The adaptation of the storm surge barrier will introduce a new closure scheme and increase the closure reliability to decrease the governing water levels. The decrease of the governing water levels results in an increase of the overall safety. Due to the decrease of the governing water levels with 0.50 meter the risk contribution of the Hollandsche IJssel levees reduces from 24 million euros per year to 11 million per year. The reduction of the levee reinforcement costs is small due to the high safety level that should be reached.



Result of the decreased water levels [million euros/year]

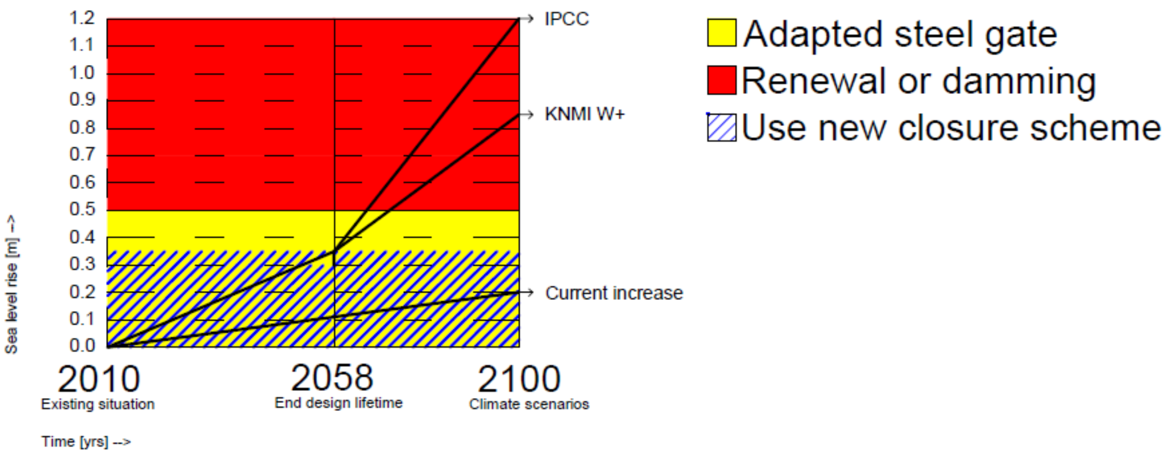
The adapted storm surge barrier will close during salt intrusion and therefore prevent salt intrusion reaching the inlet near Gouda. During closure water from the New Meuse cannot enter the Hollandsche IJssel therefore the inlet uses the discharge from the canalized Hollandsche IJssel and small scale water supply to guarantee the water needed for the inlet (shown in the figure ‘source of water’). The small scale water supply is used to reroute water to Central Holland using small canals. Closure of the adapted storm surge barrier is limited to one month because of the water quality and tidal nature in the system.

Source water canalized Hollandsche IJssel

Conclusion

The overview of the different adaptation shows that the use of the new closure scheme is possible until 0.35 meter sea level rise has occurred, after 0.35 sea level rise the number of closures increases to fast and a new closure scheme should be adopted.

The existing steel gate cannot be used after introduction of the new closure scheme. The adapted steel gate is used until 0.50 meter sea level rise is reached, after 0.50 meter sea level rise the choice between renewal and damming should be taken, this increase depends predominantly on the rate of change of the sea level rise. If the rate of change is high damming is preferred, otherwise renewal is preferred. The black sloping lines show the expected sea level rise that has occurs during the different climate studies.



Overview possible use different adaptations based on calculation conducted in this study

The overall conclusion is that adaptation of the storm surge barrier is indeed the preferred strategy and is possible within the aspects that have been studied in this study. The overall safety increases, levee reinforcements cannot be prevented. The first assessments show that the adapted storm surge barrier can withstand the increased loads. Important recommendation is that a study into the concrete elements of the Hollandsche IJssel storm surge barrier is needed before adaptations are executed.

Content

1 Introduction

In this chapter the Hollandsche IJssel is introduced. In section 1.1 the background of the Hollandsche IJssel is given, in section 1.2 the purpose of this study is explained in section 1.3 the structure of this report is given. At the end of this chapter is should be clear what the purpose and objective of this study are.

1.1 Background

The Hollandsche IJssel River starts in the province of Utrecht near Nieuwegein and flows into the New Meuse in South-Holland shown in Figure 1. Originally the Hollandsche IJssel was a branch of the river Lek. However in 1285 count Floris V decided that part of the original river needed to be dammed; this dam would ensure that the Hollandsche IJssel could better drain the water from the peat land of central Holland [2]. In the twentieth century the river was divided into two parts, the canalized and tidal part. The two parts are separated at Gouda using a lock.

Figure 1 - Basin Hollandsche IJssel, source; Wikipedia

During the storm surge of February 1953 the levees along the tidal part of the Hollandsche IJssel partly collapsed. The gap was however closed in time preventing the flooding of central Holland. After the storm surge the Dutch government decided to instate a committee researching the options to increase the safety against flooding. This committee, called the Delta Committee, advised to dam many of the main river branches. The first branch which was partly closed off was the Hollandsche IJssel; this closure was executed in 1958 with the construction of a movable storm surge barrier at the mouth of the Hollandsche IJssel.

In the 1995 Flood Defence Act (previously part of the Delta Act and since 2009 part of the Water Act) the safety against flooding was legally anchored [3]. In article 9 of this Act the manager of the flood defence is obliged to report (at this moment every 12 years) to the government what the current state of the flood defence is. In 2011 the “third nationwide safety assessment” reported that large parts (more than 80%) of the levees along the Hollandsche IJssel were not up to standard, also the storm surge barrier in the Hollandsche IJssel was not up to standard because the probability of a non-closure event was too high [4].

The Second Delta Committee instated in 2007 was tasked to study the safety situation up to 2100. In 2008 the committee concluded that the safety should be increased and a larger sea level rise should be expected [1]. Since the third nationwide assessment and the second Delta Committee the situation in the Hollandsche IJssel is part of different programs.

1.2 Purpose of this study

Important aspect in this study is the safety (expressed in economic damage and casualties) of the hinterland which depends for a large part on the flood defence system of the Hollandsche IJssel. The Hollandsche IJssel is a complicated system where different functions and problems are related. Important aspects that directly affect the safety are the levees and storm surge barrier which do not meet the standards and the expected higher water levels due to climate change.

Other functions and problems that are related to the system of the Hollandsche IJssel are the salt intrusion due to climate change, the congestion during rush hours, the ecology of the system, the morphology of the system and local developments.

Objective

The objective of this study is the development and (conceptual) design of the preferred strategy for the important aspects (overall safety, salt intrusion and climate change) in the Hollandsche IJssel. The preferred strategy is cost-effective and exists of a technically and societally feasible design.

1.3 Structure of this report

In the first part of the report (analysis system) the Hollandsche IJssel and Rijnmond system is defined. The system of Rijnmond is defined because it affects the boundary conditions in the Hollandsche IJssel (shown in Figure 2). The system of the Hollandsche IJssel is defined because the strategies and closure scheme of the storm surge barrier focus on the problems in this region. After the definition of the system the current situation, problems in the Hollandsche IJssel system and the boundary conditions affecting the system are described (chapter 2).

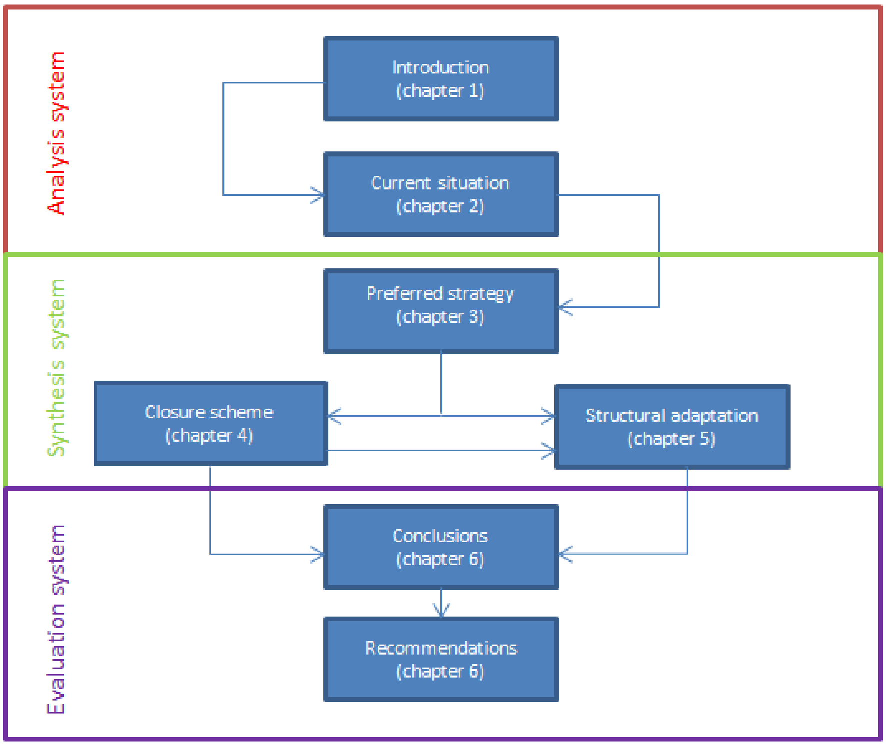


Figure 2 - Flowchart report outline

The second part of this study (synthesis system) focuses on the design of a solution for the multiple problems, identified in chapter 2. The design of the solution starts with the definition of the different strategies that can be used to solve the problems in the Hollandsche IJssel; these strategies are evaluated on criteria (introduced in chapter 2) relevant to the Hollandsche IJssel (chapter 3). The preferred strategy is elaborated in two separate chapters. In chapter 4 the choice for the strategy is substantiated and the effects of the preferred strategy are calculated. Chapter 5 focuses on the structural adaptations that are needed to withstand the effects calculated in chapter four and elements that are needed in the preferred strategy.

The last part of this study (evaluation system) focuses on the conclusions and recommendations concerning the objective of the preferred strategy mentioned in section 1.2 (chapter 6).

Important symbols, abbreviations and technical terms are found in the glossary. When the reader is not familiar with the philosophy behind flood protection in the Netherlands it is advised to read appendix A.

2 Analysis of the situation in the Hollandsche IJssel

In this chapter the aspects that describe the system of the Hollandsche IJssel are analyzed, this analysis is conducted based on the following aspects;

1. Schematization of the Hollandsche IJssel and Rijnmond system (section 2.1).
2. Analysis of the relevant developments important for the situation in the Hollandsche IJssel (section 2.2).
3. Description of the Hollandsche IJssel and Maeslant storm surge barrier that affect the (hydrological) boundary conditions in the Hollandsche IJssel and Rijnmond system (section 2.3).
4. Description of the fresh water that is needed to flush the brackish canals of Central Holland (section 2.4).
5. Description of the levees in the Hollandsche IJssel River (section 2.5.1).
6. Description of the morphology in Hollandsche IJssel River (section 2.5.2).
7. Description of the ecology in Hollandsche IJssel River (section 2.5.3).
8. Analysis of the surrounding around the Hollandsche IJssel storm surge barrier (section 2.6).

At the end of this chapter the problems and different aspects are summarized (section 2.7).

2.1 Rijnmond and Hollandsche IJssel system

The Rijnmond system is the area between the Maeslant barrier located near the North Sea and the Lek located just behind the mouth of the Hollandsche IJssel. In the south the boundary of the system is located at the mouth of the Spui, Dordtsche Kil and Beneden Merwede. The existing storm surge barriers in the system are the Maeslant barrier, the Hollandsche IJssel barrier and the Hartel barrier. The Hartel barrier is not considered in this study because the Maeslant barrier is larger and known to be a weak link in the system. The Rijnmond system is schematized in Figure 3.

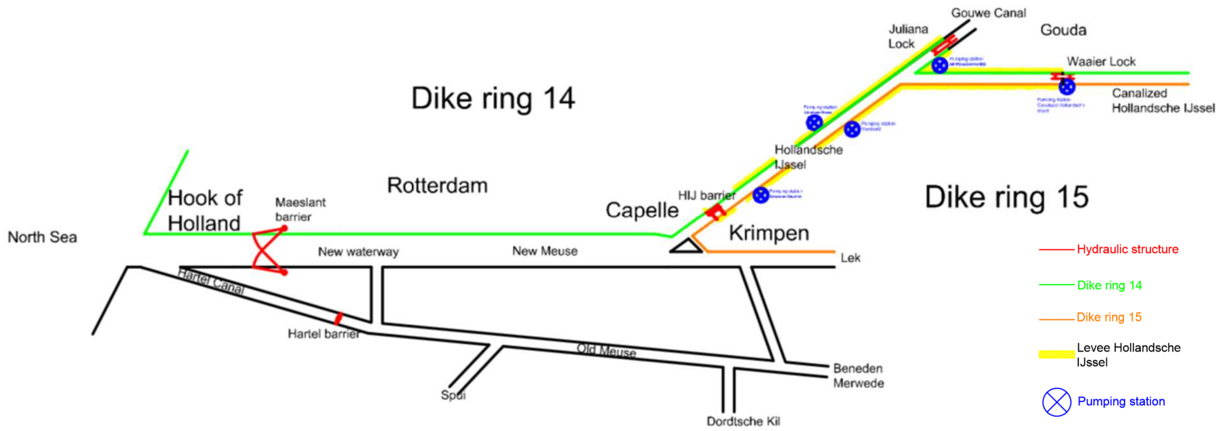


Figure 3 - Schematic overview Rijnmond and Hollandsche IJssel system

The system of the tidal Hollandsche IJssel is located between the Hollandsche IJssel storm surge barrier, the Juliana Locks and the Waaier Lock. The Waaier Lock near Gouda marks the transition between the tidal and canalized part. The Juliana Locks mark the transition between the tidal part and the Gouwe Canal. The tidal part of the Hollandsche IJssel flows from Capelle to Gouda, the canalized part from Gouda to Nieuwegein. The two important dike rings are 14 (Central Holland) and 15 (Krimpenerwaard). A dike ring is a closed system of levees and structures which protect lower lying areas. The system of the Hollandsche IJssel is shown in Figure 4 and schematized in Figure 5.

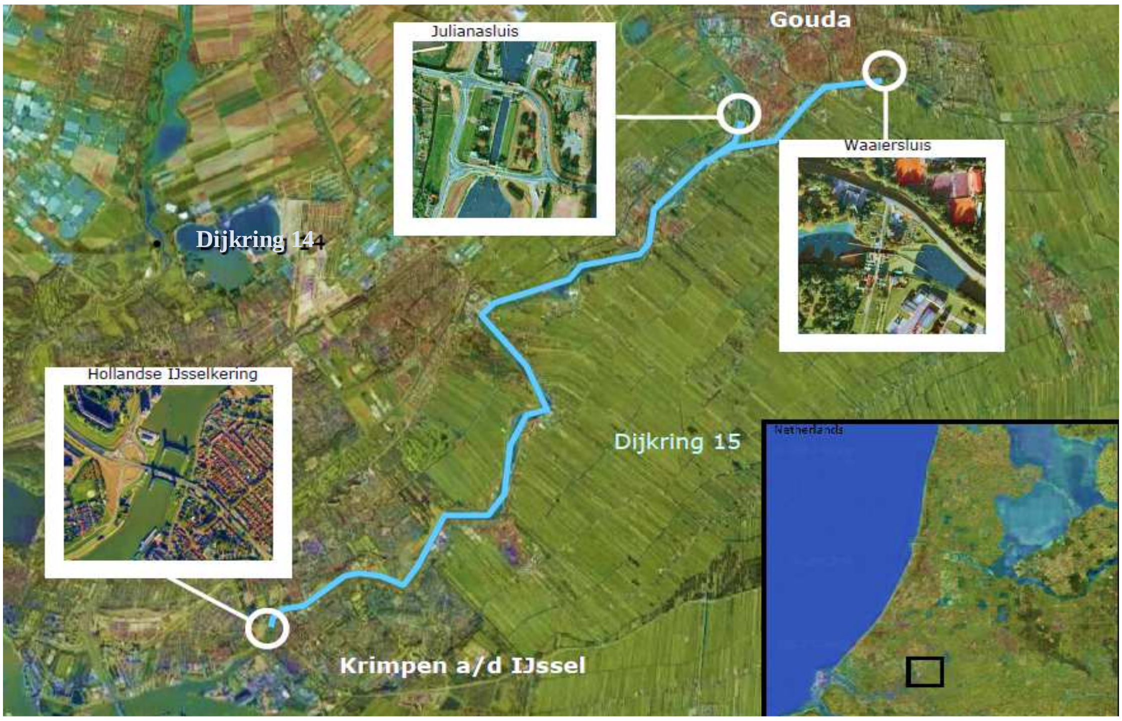


Figure 4 - Tidal part of the Hollandsche IJssel, source; Google Maps

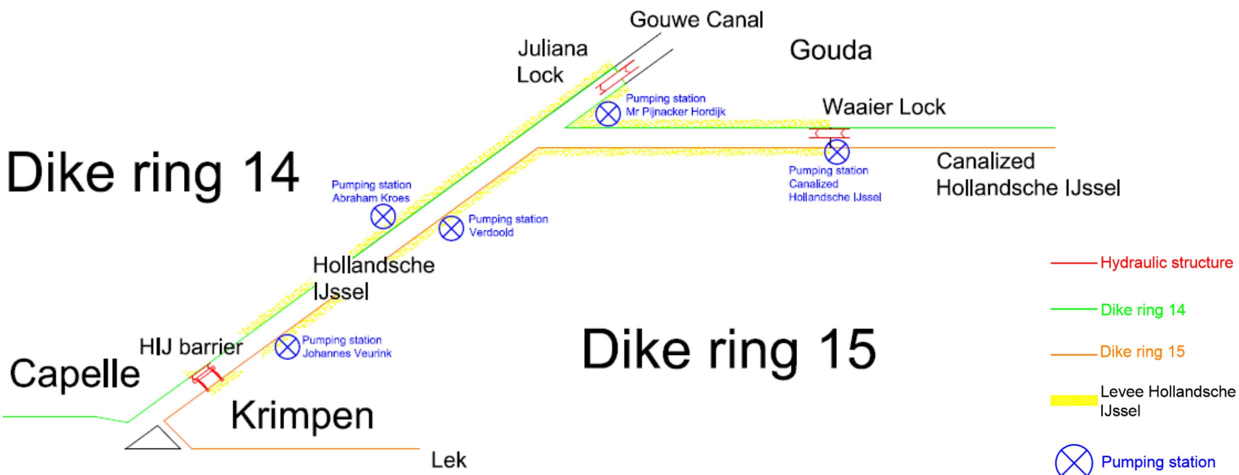


Figure 5 - Schematic overview Hollandsche IJssel system

2.2 Relevant developments for the Hollandsche IJssel system

As a result of the Second Delta Committee the Government of the Netherlands decided to instate different programs each tasked with their own region or subject. The important programs for the Hollandsche IJssel are the Delta Program, new flood defence program, multi layered safety and safety in the Netherlands, schematized in Figure 6. The Delta Program Rijnmond and Drechtsteden (R&D) studied the different topics that are related to flooding of the R&D area. These topics are; Hollandsche IJssel, Open Closable Rijnmond (AOR) and urban river fronts.

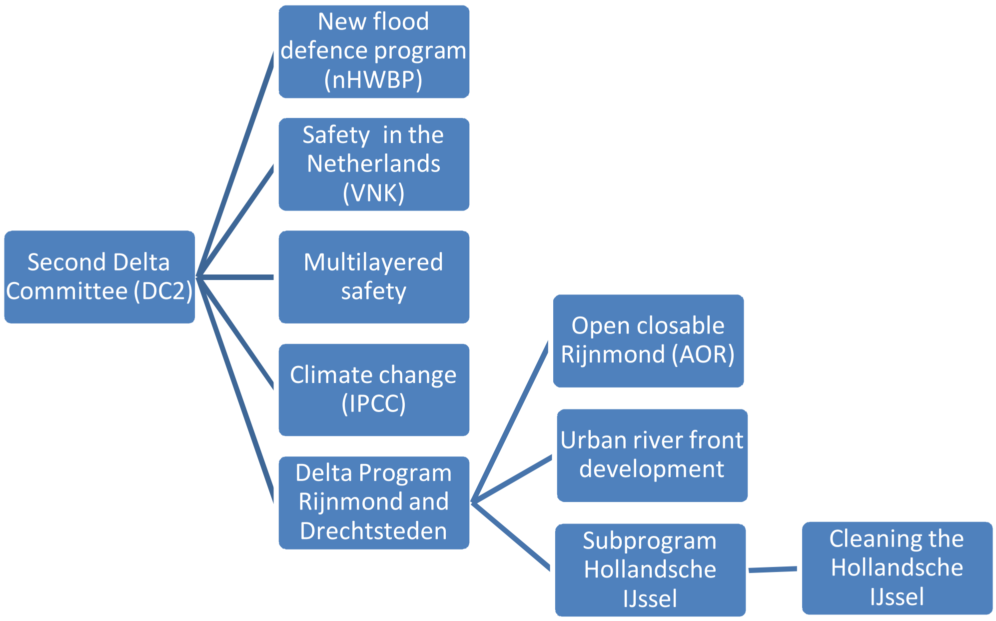


Figure 6 - Relevant developments Rijnmond and Drechtsteden

2.2.1 New flood defence program (nHWBP)

The nHWBP program is the execution of the safety philosophy in the Netherlands laid down in the Water Act. This safety philosophy is developed after the 1953 flood disaster. The new flood defence program started after the results of the third nationwide safety assessment were published. Task of this program is the reinforcement of all the levees and structures that were not up to the standard in the last assessment.

The levees along the Hollandsche IJssel and the storm surge barrier are part of the nHWBP because they were not up to standards according to the last assessment. The budget reserved for this program is limited because the Government tries to economize as much as possible. The number of reinforcements that need to be executed limits the budget per project. Only the costs that are needed for the reinforcements are covered by the nHWBP budget. The strategy which is the most cost-effective is preferred from the position of the nHWBP. A brief description of the safety philosophy and the developments is given in appendix A.

2.2.2 Safety in the Netherlands (VNK)

The program Safety in the Netherlands (VNK 1 and 2), started after the Second Delta Committee, investigates the overall safety of all dike rings in the Netherlands. The overall safety of a dike ring is increased when the risks due to flooding are decreased. Risk is defined as the consequences multiplied with the failure probability of the flood defence and expressed in economic damage and casualties per year. A decrease of the risks is possible when the consequences (economic damage and casualties) are reduced or the failure probability (of elements in the dike ring) is decreased.

Dike ring 14 and 15 are analyzed in the first and second analysis round of the VNK program. In this analysis the failure probability of a levee section is coupled to inundation due to a breach in that levee section. For each inundation the economic damage and casualties are calculated. The multiplication of the failure probability of the levee and the consequences of the inundation results in an expected economic damage and casualties per year. If this analysis is conducted for every levee section Figure 7 is obtained.

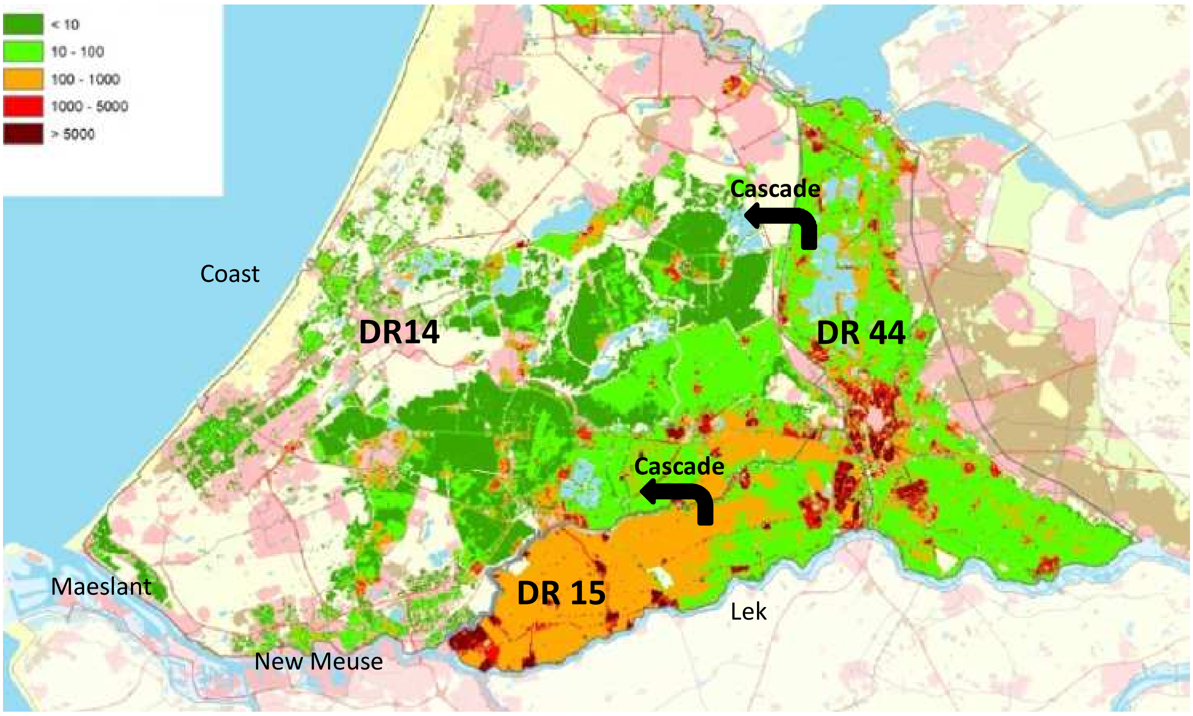


Figure 7 - Expected economic damage due to flooding [euros/year per hectare], source; VNK2

There are six contributors to the expected economic damage of dike ring 14 [5], these are:

- Contribution due to a breach at the coast.
- Contribution due to a breach of the levees along the New Meuse.
- Contribution due to failure of the Maeslant barrier and consequently higher water levels behind the barrier.
- Contribution due to failure of the Hollandsche IJssel flood defence system (barrier and levees).
- Contribution due to the cascade effect. The cascade effect is the effect that a breach in another dike ring leads to inundation of the dike ring because this dike ring lies lower than the inundated dike ring. When the levees along the Lek are breached dike ring 15 will flood, when the water reaches the Hollandsche IJssel it breaches the levees and creates a cascade into dike ring 14.
    - Cascade effect of dike ring 15 to 14.
    - Cascade effect of dike ring 44 to 14.

The contribution to the total expected economic damage per year is shown in Figure 8.

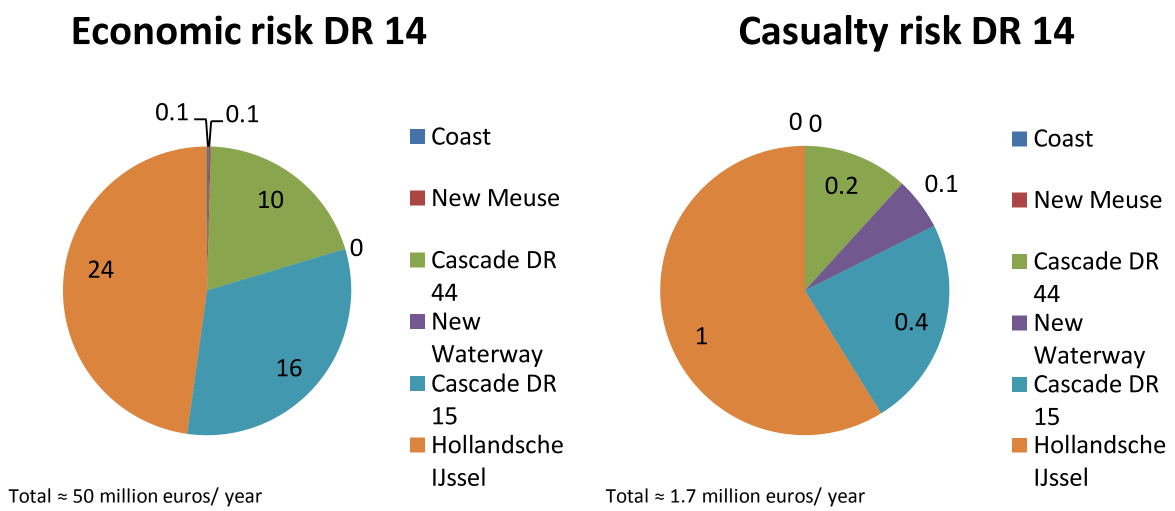


Figure 8 - Economic risk of the different contributions [million euros/year and casualties/year], source; VNK2

2.2.3 Multilayered safety

Before flooding of the major rivers in 1993 and 1995 flood protection was only focused on flood prevention (first layer), a more integrated approach was developed after 1995 resulting in different layers. Multilayered safety is introduced in the Nation Water Plan of 2009-2015. In multi layered safety the flooding risks are spread over different layers, three official layers and one unofficial layer [6]. These different layers are;

1. Prevention, the first layer consists of the flood defences and measures to give room to the river (failure probabilities). An example is the Hollandsche IJssel storm surge barrier.
2. Spatial planning, the second layer consist of all the measures that reduce the effects of flooding (consequences). An example is the construction of floating houses or the construction of compartment levees.
3. Evacuation and disaster management, the third layer implements evacuation and disaster plans for dike rings (consequences). An example is an evacuation plan for the Krimpenerwaard.
4. Insurance, the fourth unofficial layer. This is not part of any legislation and therefore not treated.

This thesis focuses on the first layer (prevention) of a flood. It is expected that measures in the second and third layer are not cost-effective for the situation in the Hollandsche IJssel. When the Hollandsche IJssel levees are breached the deep low lying polders are filled within a couple of hours. The evacuation time is very short and large structures would be needed (because of the deep polders) to reroute water, therefore these measures are not cost-effective.

2.2.4 Sea level rise due to climate change

The studies conducted by the Intergovernmental Panel on Climate Change (IPCC) and Koninklijk Nederlands Meteorologisch Instituut KNMI) show that the level of the sea will rise due to the climate change (Figure 8 and appendix F.1) [1, 7].

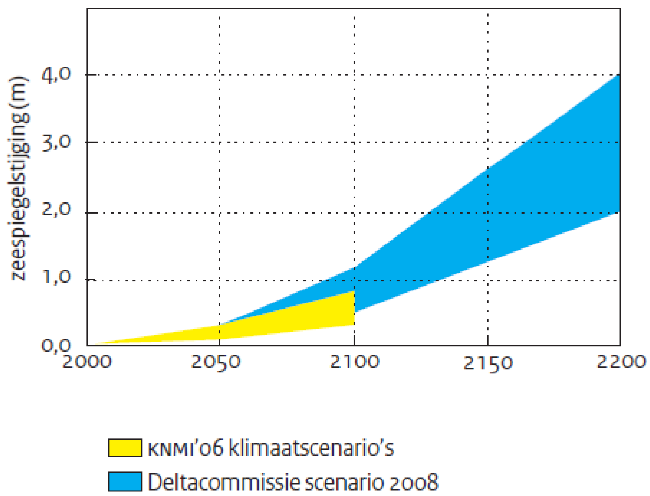


Figure 9 - Relative sea level rise, source; second Delta Committee

The KNMI and IPCC studies predict a sea level rise of 0.85 - 1.2 m in the year 2100 while the average sea level rise of the last few centuries was 0.2 m per century (shown in Table 1). The sea level rise influences the number of closures per year but not directly the governing water levels in front of the Hollandsche IJssel barrier. The governing water levels in front of the barrier are determined by the closure level of the Maeslant storm surge barrier and discharge from the Rhine and Meuse, described in section 2.3.

Table 1 - Expected sea level rise 2050 and 2100

	2050	2100
Current situation	0.10 m	0.20 m
KNMI W+	0.35 m	0.85 m
IPCC	0.35 m	1.20 m

The predictions made by IPCC and the KNMI and shown in table 1 are not validated; this makes it difficult to estimate a design sea level rise for the Hollandsche IJssel system [7, 8]. The system, evaluated in the next chapters, should take the variable sea rise into account. The solution should vary according the relative sea level rise because the construction of a new storm surge barrier when it is always closed behave like a dam, construction of a new storm surge barrier is then a waste of money.

2.2.5 Open closable Rijnmond system (AOR)

The open closable Rijnmond (AOR) system is an alternative for the future safety situation in the Rijnmond region. In this alternative the Rijnmond area is protected by a series of barriers which closes during high water. The closure of the barriers creates a polder within the barriers in which the water levels stay the same. The high discharge is rerouted to the South-western part of the Netherlands (shown in Figure 10) [9].

Figure 10 - Overview of the AOR system, source; Deltaprogramma 2013

The Hollandsche IJssel is not part of the AOR system because the governing water levels on the Hollandsche IJssel are lower than the governing water levels in the AOR system, which depend on the closure level of the Maeslant barrier. The Hollandsche IJssel storm surge barrier will therefore always close when the AOR barriers close, governing water level in front of the Hollandsche IJssel storm surge barrier will depend on the water level introduced in the AOR system, this influences the hydrostatic water pressure acting on the storm surge barrier.

The most important barrier for the Hollandsche IJssel is the Maeslant barrier because the closure level of the Maeslant barrier determines the governing water levels in front of the Hollandsche IJssel storm surge barrier. Governing situation is a closed Maeslant and Hollandsche IJssel barrier. Water levels on the New Meuse are then affected by the closure level of the Maeslant barrier (or other AOR barriers). Other aspects like salt intrusion are not affected through the AOR system because the AOR system is only introduced during storm surges which exceed the closure level of the Maeslant barrier.

2.2.6 Urban river fronts along the Hollandsche IJssel

As part of the Delta program the urbanization of the Rijnmond area up to 2100 was studied [10]. This study expects that large parts of the Hollandsche IJssel will become an urban river front (shown in Figure 11). An urban river front is a bank of the river where the functions living, working and flood defence are integrated.

Figure 11 - Future urban developments in red (left), urban river development (right), source; Delta Program R&D

The development of urban river fronts limits the space available for a new storm surge barrier or dam. When the design lifetime of the existing storm surge barrier is extended the available space for new structures becomes limited.

2.2.7 Delta program Rijnmond and Drechtsteden – sub program Hollandsche IJssel

Part of the Delta program R&D studies the system of the Hollandsche IJssel; this subprogram studies the future safety situation in the Hollandsche IJssel. The subprogram, guided by the province South-Holland, presented their results at the beginning of 2013 [11]. Important conclusions from this report are;

- The reinforcement costs could be reduced when the governing water levels are decreased,
- The storm surge barrier could be used to prevent of salt intrusion reaching the barrier.

In combination with the Delta program R&D a suitable solution is made for the total Rijnmond and Drechtsteden system including the Hollandsche IJssel. Part of the conclusion from this report is confirmed in the calculations conducted as part of chapter 4.

2.3 Storm surge barriers affecting the Hollandsche IJssel

There are two storm surge barriers which influence the Hollandsche IJssel system; the Maeslant storm surge barrier and the Hollandsche IJssel storm surge barrier. The Maeslant barrier is located at the mouth of the New Waterway while the Hollandsche IJssel storm surge barrier is located at the mouth of the Hollandsche IJssel. The Hollandsche IJssel barrier has two lift gates (shown in Figure 11); one of the gates is lowered when a storm surge is expected. The Maeslant barrier has two sector gates which are floated into the waterway when a storm surge is expected (shown in Figure 11). Table 2 presents the main features of the storm surge barriers.

Figure 12 - Maeslant (left) and Hollandsche IJssel (right) storm surge barrier, source; omroepwest

Table 2 -Current features storm surge barriers [1, 11]

	Maeslant barrier	Hollandsche IJssel barrier
Type	Sector gate	Lift gate
Completion	1997	1958
Costs (price level 2012)	571 million euros	103 million euros
Where	New Waterway	Hollandsche IJssel
Width	600 m	82 m
Exceedance probability	1/10 000 per year	1/10 000 per year
Requirement non-closure probability	1/1 000 per request	1/1 000 per request
Current non-closure probability	1/100 per request	1/30 per request
Design water level	+5.5 m NAP	+4.5 m NAP
Closure level	+3.0 m NAP	+2.25 m NAP

The Maeslant storm surge closes when the expected water levels are above +3.00 m NAP. When the discharge is below 6 000 m^3/s the storm surge barrier closes when the +2.00 m NAP water level is reached. When the discharge is more than 6 000 m^3/s the storm surge barrier closes during the ebb slack period preceding the expected high water. Closure of the Maeslant barrier takes approximately 2:30 h.

The Hollandsche IJssel barrier closes when the expected water levels are above +2.25 m NAP. The storm surge barrier closes during the ebb slack period preceding the high water. The closure of the Hollandsche IJssel storm surge barrier takes approximately 0:25 h when closed during the ebb slack period, otherwise it takes 1 hour because a translation wave might be created in the Hollandsche IJssel.

Closure of the Maeslant barrier means that the Hollandsche IJssel barrier is also closed because the governing water levels on the New Meuse are higher than in the Hollandsche IJssel. The Hollandsche IJssel barrier can be closed when the Maeslant barrier is open, because the closure level of the Hollandsche IJssel barrier is lower.

The safety assessment of storm surge barriers consists of two parts. The first part assesses the structural safety of the barrier; the second part assesses the non-closure probability of the storm surge barrier. The non-closure probability (P_{ncl}) is the likelihood that the gate does not close when the gate should close due to expected high water. In the last nationwide safety assessment both storm surge barriers passed the assessment on the first part (stability, turning, piping etcetera) [4]. The requirement for the second part (reliability closure) is given by the following formula [12]:

$$P_{ncl} \leq 1/10 * norm\ for\ the\ exceedance\ probability$$

For both storm surge barriers the norm for the exceedance probability is 1/10 000 and therefore the requirement for the non-closure probability is 1/100 000, which is not met for both storm surge barriers. When this requirement is not met, the safety of the levees behind the storm surge barriers should be assessed using higher governing water levels. The governing water levels behind the storm surge barrier are increased due to the effect the non-closure probability and high water levels if the barrier was open. The increase of the governing water levels reduces the safety of the levees in the hinterland of the storm surge barriers.

2.4 Flushing of the canals in Central Holland

Large parts of the canal system in Central Holland need to be flushed in order to maintain fresh water. Salt water from sea seeps into the aquifers and slowly progresses landwards because of pressure differences. Brackish water than enters the canal system because the canals create a leak between the brackish aquifer and the surface water (shown in Figure 13).

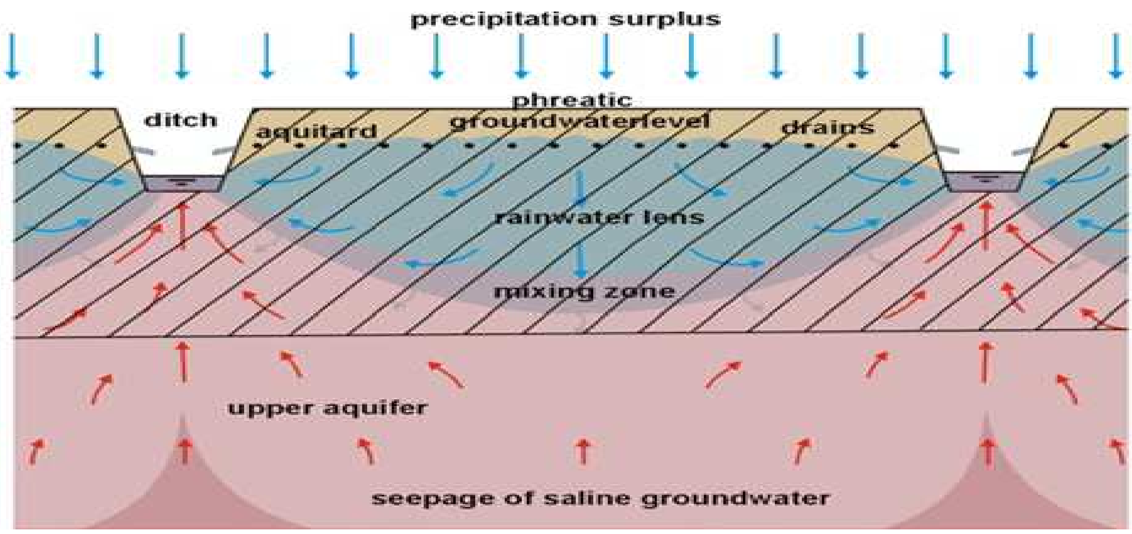


Figure 13 - Salt seepage in Central Holland, source; STOWA

The agriculture in Central Holland needs fresh water in the canals to grow crops and maintain kettle, without fresh water there is a lot of economic damage [13]. Pumping station Mr Pijnacker Hordijk drains water from the Hollandsche IJssel and discharges this water in the Gouwe Canal and therewith the system of Central Holland. The water that is drained needs to be fresh, it should not have a chloride concentration higher than 250 mg/l. When the concentration is higher the inlet near Gouda should stop, this happen predominantly during periods with a low discharge [14]. During low discharges the "Small-scale water supply (KWA) takes over, this discharge is however not high enough to fulfill the requirements [13].

Figure 14 - Inlet water, normal inlet (green arrows), KWA (orange arrows), source; Unie van Waterschappen

The Haringvliet pumping stations regulate the outflow distribution of the Rhine during low discharges. This regulation is done to maintain a high discharge through the New Waterway which prevents salt intrusion reaching the mouth of the Hollandsche IJssel [15]. Based on the discharge measured near Lobith the outflow of the Haringvliet is determined. When the discharge through the New Waterway falls below 1250 m^3/s salt intrusion is not prevented in the mouth of the Hollandsche IJssel. The inlet of water needed to stop three times during the last decade. It is expected that more closures will occur due to prolonged periods of low discharge [16].

2.5 Hollandsche IJssel River

The tide enters the Hollandsche IJssel through the New Waterway and New Meuse (shown in Figure 3). There is a tidal difference in the Hollandsche IJssel because the Delta Committee decided to build a storm surge barrier instead of a dam. The average tidal difference in the Hollandsche IJssel is 1.51 meters. The tidal part of the Hollandsche IJssel is 19 kilometers and has an average width of 135 m. The river ends at the and Waaier locks near Gouda. The management of the Hollandsche IJssel (HIJ) lies with the department of Public Works district New Waterway, management of the levees lies with the water board Schieland and Krimpenerwaard (HHS&K) (shown in Figure 15).

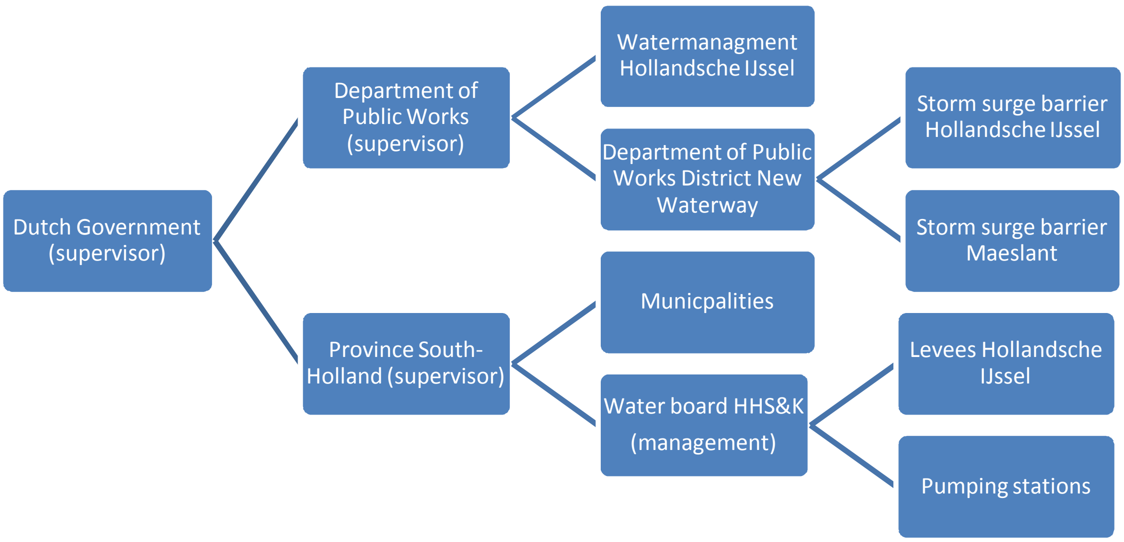


Figure 15 - Management situation Hollandsche IJssel

2.5.1 Levees along the Hollandsche IJssel

The levees along the Hollandsche IJssel are part of two dike rings. Dike ring 14 protects Central Holland and needs to withstand water levels with an exceedance probability of 1/ 10 000 year. Dike ring 15 protects the Lopiker and Krimpenerwaard and needs to withstand water levels with an exceedance probability of 1/ 2 000 per year. The exceedance probability is the probability that a certain water level is exceeded. The Water Act specifies three different flood defences:

1. a-defences, protect directly against outside water
2. b-defences, connect two different a-defences
3. c-defences lie behind a b-defence.

The Hollandsche IJssel storm surge barrier is a b-defence because it connects the levees of dike ring 14 and 15 with each other. The levees along the Hollandsche IJssel lay behind a storm surge barrier and are therefore c-defences.

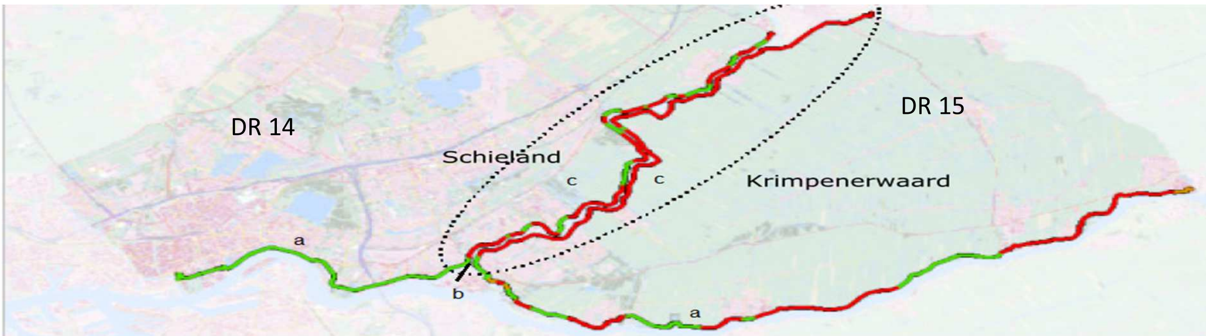


Figure 16 - Third nationwide safety assessment (red is not up to the standards), source; water board HHS&K

In the third nationwide safety assessment, executed between 2006 and 2011, c-defences were assessed for the first time. The result of this assessment is that eighty percent (28 kilometers) of the levees along the Hollandsche IJssel are not up to the standards (shown in Figure 16) [4]. This is predominantly because the steep inner slope of the levees is unstable during governing high water levels (NHW). The factors influencing the governing high water levels are;

- Closure level Hollandsche IJssel and Maeslant storm surge barrier,
- Non-closure probability of the Hollandsche IJssel and Maeslant storm surge barrier,
- Pump stop of the pumping stations discharging water on the Hollandsche IJssel,
- Wind set-up in the Hollandsche IJssel.

Figure 17 shows the flood defence system of the Hollandsche IJssel and Rijnmond region.

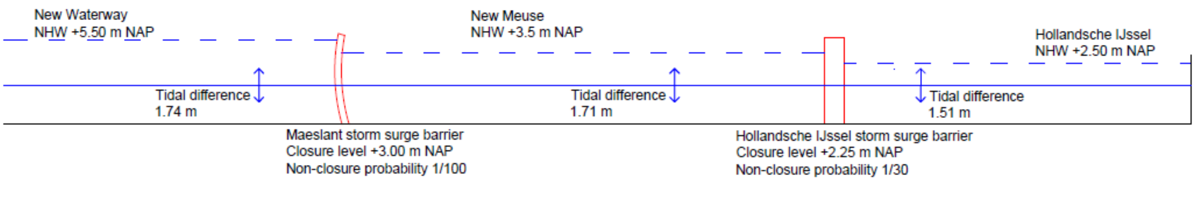


Figure 17 - Cross-section flood defence system

The third nationwide safety assessment is conducted using the hydrological boundary conditions from the Hydrological Boundary Conditions 2006 (HRC 2006) [17]. The HRC 2006 is a report in which the governing water levels for all levees in the Netherlands are presented. The governing water levels on the Hollandsche IJssel were calculated for the first time in this report because c-defences were assessed for the first time. The effect of the non-closure probability was not accounted for in this report because the department of Public Works did not know how to process these effects. The wind speed used to calculate the governing water levels on the Hollandsche IJssel was 20.5 m/s [17, 18]. Since the HRC 2006 no new HRCs have been made because the time between individual safety assessments is increased from six to twelve years, the report of the fourth nationwide review will be presented to the government in 2023. The time between the reports is increased because the water boards need more time to reinforce all the levees that were not up to the standards in the previous assessment.

In the last century there has been a lot of economic activity along the Hollandsche IJssel, different industries, houses and other buildings have been built along the Hollandsche IJssel (shown in Figure 18). Result of these activities is that different municipalities have developed on and along the levees, so called ribbon development. This ribbon development makes levee reinforcements difficult.

Figure 18 - Levee in Moordrecht (left), urban developments on the levee (right)

2.5.2 Existing morphological situation

As can be seen from Figure 19 the tidal rise is faster than the tidal fall. The slack period during ebb is much larger than the slack period during flood (shown in Figure 19). This means that the velocity during a tidal rise is larger than the velocity during a tidal fall, because an equal amount of water enters and leaves the basin of the Hollandsche IJssel [19]. In theory the system has a net import of sediment from other rivers because a larger velocity means more sand import, sand transport (S) is related to the velocity (u) to the power four [19].

$$S \sim u^4$$

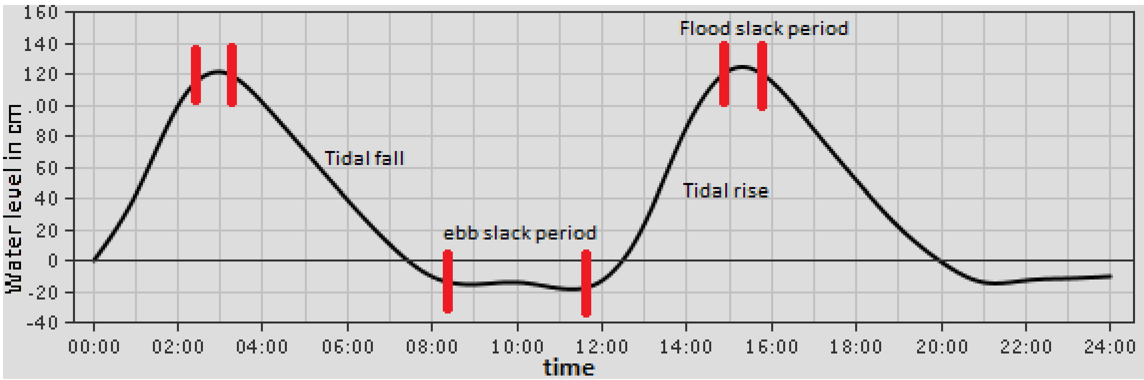


Figure 19 - Tidal wave Krimpen aan de IJssel, source; Rijkswaterstaat

The small slack period during flood prevents sedimentation of the fine material because the time for the small particles to settle on the river bottom is limited. The closure of the storm surge barrier will preferably happen during the ebb slack period; because this period lasts two hours and during that time there are no substantial velocities. The effect of closure during the ebb slack period is a net export of sediment because the system is not balanced with the return of the sediment in the following flood period, the barrier is closed.

The effect of an ebb slack period closure is not a problem because it partly counteracts the import of sediment. The closure during the high water slack period needs to be prevented as much as possible, because the input of sediment amplifies and fine sediment will settle in the course of the river. Because of the net import of sediments it is probably needed to dredge the Hollandsche IJssel occasionally.

2.5.3 Ecology in the Hollandsche IJssel

The ecology in the Hollandsche IJssel is restored during the last two decades. The program “Cleaning the Hollandsche IJssel”, started by the province and water boards, cleaned all the contaminated soil in the forelands and created new nature. The tidal nature in the Hollandsche IJssel is unique because there are not a lot of fresh water tidal rivers remaining in Europe. Due to this program fish returned to the Hollandsche IJssel, before the program the river was dead due to polluted forelands. Nowadays anglers fish in the Hollandsche IJssel to catch bass, roach, pike, eel and bream [20]. Most of the fish spawn their eggs in the summer near the banks of the river.

The Hollandsche IJssel is part of the Ecological Main Structure (EHS). The EHS is a nationwide project primarily managed by the provinces. The EHS system consists of three parts; nature reserves, connections and designated reserves. The river is a connecting part; some of the forelands are nature reserves. For a Civil Engineering project it needs to be demonstrated that it does not threaten a part of the EHS. When the project threatens the EHS other solutions need to be found or important reasons must be given why part of the EHS may be lost. The lost nature always needs to be compensated.

2.6 Surroundings Hollandsche IJssel storm surge barrier

The surrounding of the storm surge barrier is of interest in this study because developments and limitations in the local surrounding influence the system of the Hollandsche IJssel.

2.6.1 Municipalities near the storm surge barrier

The two municipalities in the direct surrounding of the storm surge barrier are Krimpen and Capelle. There is also an industrial area called Stormpolder. Other municipalities upstream have no direct impact on the storm surge barrier. These municipalities are important for the local reinforcement of the levees (ribbon development).

Figure 20 - Overview regions, source; Google Maps

Stormpolder is an industrial area which deteriorated during the last decades. This industrial area used to accommodate chemical industry, shipyards and garages. Now only the shipyards, a few garages and a youth prison are left in this industrial area. The municipality of Krimpen has plans to demolish large parts of the industry and introduce the functions living and working (shown in Figure 21) [21]. In the last few years no investors have been found to implement these plans. The municipality of Krimpen lies between the Hollandsche IJssel and the Lek. The neighborhoods in the direct surrounding of the storm surge barrier are built just after or just before the Second World. These neighborhoods may be demolished in the coming decades to make room for new urban development programs [10]. Stormpoldervloedbos is a nature reserve in the corner of industrial area Stormpolder, (shown in Figure 21)

Figure 21 - Master plan Stormpolder, source; municipality Krimpen

The municipality of Capelle lies between the city of Rotterdam and the Hollandsche IJssel. The municipality expanded from Rotterdam to the Hollandsche IJssel. The neighborhoods on the Northen banks are therefore relative new (1995) and are therefore not demolished.

2.6.2 Surrounding near the storm surge barrier

The system of the Hollandsche IJssel storm surge barrier also has a bridge and a lock, which were constructed in 1958 (shown in Figure 22). In the rush hours there is congestion near the bridge because it has only one lane per direction. For commuting traffic from the Krimpenerwaard to Rotterdam this connection is important. The bridge also has a bascule bridge above the lock to accommodate ships that need a large vertical clearance. The office of Public Works district New Waterway is located next to the bridge.

Figure 22 - Algera corridor, source; Google maps

The Algera Locks are used when the storm surge barrier is closed or when there are ships that need a large vertical clearance. During the summer the Hollandsche IJssel is part of the “Staande Mast Route” (route for ships which need a high vertical clearance) which runs from the South-western part of the Netherlands to Lake IJssel, therefore the lock is used intensive by sailboats in this period [22]. The result of this combination is waiting time for both cars and ships during the use of lock and bridge.

2.7 Summary of the important problems

The important aspects and problems in the Hollandsche IJssel are described in the preceding sections of this chapter and summarized in Table 3.

Table 3 - Summary problems Hollandsche IJssel

Analyzed aspects	Problem	Description
1 Schematization	-	-
2 Relevant developments	Uncertain sea level rise Overall safety dike ring 14 and 15	Sea level rise and in particular the uncertainties in the sea level rise creates a lot of possible scenarios that should be taken into account. The overall safety of dike ring 14 and 15 is threatened due to the high failure probability of the levees along the Hollandsche IJssel.
3 Storm surge barriers	Results nationwide safety assessment	The results of the third nationwide safety assessment shows that the storm surge barriers do not meet the standards.
4 Fresh water supply	Climate change and salt intrusion	During low discharges the water near the inlet becomes brackish and cannot be used for flushing. Low discharges and consequently salt intrusion will increase due to climate change.
5 Levees	Results nationwide safety assessment	Large investments are needed to reinforce the levees that are not up to the standards.
6 Morphology	Morphological balance	The morphological balance is threatened when new boundary conditions are introduced.
7 Ecology	Recovered ecology	The ecology that has just recovered due to the program Cleaning the Hollandsche IJssel is threatened when the system is changed.
8 Surrounding	Delayed traffic flows Urban river developments and ribbon development	The traffic flows in the system are delayed due to the extensive use of the Algera Bridge and Lock. The development of urban river fronts and ribbon development limit the available space for reinforcements or other structures.

3 Strategies for the Hollandsche IJssel

The solution to the problems in the Hollandsche IJssel system is directly related to the choice made at the mouth of the Hollandsche IJssel; the barrier affects the boundary conditions, governing water levels, the solution to salt intrusion and the ecology in the river. This chapter investigates the preferred strategy for the storm surge barrier and river based on the following sections;

1. Description and composition of the strategies (section 3.1).
2. Evaluation of the strategies based on specified criteria (section 3.2).
3. Possible secondary functions related to the new storm surge barrier or dam (section 3.3).
4. Conclusion for the situation in the Hollandsche IJssel (section 3.4).

At the end of this chapter it should be clear what a strategy is, on which criteria the strategies are evaluated and which strategy is preferred and elaborated.

3.1 Description and composition of the strategies

This section describes and composes the strategies. The description of the strategies focuses on the explanation of different solutions that were created after the Second Delta Committee; the analysis of the important aspects is directly related to the composition of the strategies. The important aspects are related to the problems described in the summary of section 2.7. The different solutions that were created are used as the basis of the strategies; other elements have been added in this study to create a feasible strategy.

3.1.1 Description of the strategies

After the second Delta Committee different programs and authorities studied the Hollandsche IJssel system and developed a solution. These studies studied in particular the safety of the system which is not up to the standard, there are however other problems in the Hollandsche IJssel that are also important. Therefore the different solutions that focus on the safety of the system are expanded. The different solutions that are developed by the Delta Programs and expanded at the end of this section are [11, 9];

- <u>Doing nothing</u> is the solution where the original situation is maintained and nothing is done to increase the safety or solve other problems in the Hollandsche IJssel.
- <u>Reinforcement of the levees</u> is the solution where all the levees along the Hollandsche IJssel storm surge barrier are reinforced. A lot of structures, like coffer dams and diaphragm walls for example, will be needed to reinforce the levees along the Hollandsche IJssel.
- <u>Adaptation storm surge barrier</u> is the solution where the existing storm surge barrier is adapted to the changing boundary conditions.
- <u>Renewal storm surge barrier</u> is the solution where the storm surge barrier is completely rebuilt according to the changed boundary conditions.
- <u>Damming of the Hollandsche IJssel</u> is the solution where the Hollandsche IJssel is closed off. Pumping stations in the dam discharge the water from the Hollandsche IJssel in the New Meuse; locks connect the Hollandsche IJssel to the New Meuse.

The term strategy is introduced because a combination and expansion of these solutions should create a situation in the Hollandsche IJssel that solves multiple problems and increases the public support for the total solution.

3.1.2 Aspects related to the compilation of the strategies

Three aspects are important because they determine the choice for a specific strategy or add elements that should be part of the strategies. The aspects described below are used to compose the strategies and are elaborated in chapter four.

- General effect of the sea level rise on the barrier,
- Levee reinforcement in the Hollandsche IJssel,
- Tipping points that are important for the strategies.

General effect sea level rise on the system of the Hollandsche IJssel

Chapter two (section 2.2.4) describes that the sea level rise is an important but uncertain parameter that varies according to different climate studies. The sea level rise is important because the rate of change of the sea level rise determines the effectiveness of the strategies. When the sea level rises fast a newly constructed storm surge barrier becomes a dam in theory because the barrier closes at high waters which occur more often. The construction of a storm surge barrier is then a waste of money. The effectiveness of the strategies considering the sea level rise should therefore be taken into account.

The possible relative sea level rise is the gap between the current spring tidal elevation (shown in appendix F.2) and the current closure level. The spring tidal level is used instead of the average tidal level because this would not cause immediate closure during an average tide. The relative sea level rise possible is +2.25 m NAP (current closure level mentioned in Table 2) minus +1.36 m NAP (spring tidal elevation in 2012) is approximately 0.9 meter (shown in Figure 23). To decrease the governing water levels on the Hollandsche IJssel it should however be taken into account that the closure level needs to be decreased. The absolute gap (sea level rise) is therefore less than the relative sea level rise which includes the possible decrease of closure level. The elaboration of the sea level rise is conducted in section 4.4.

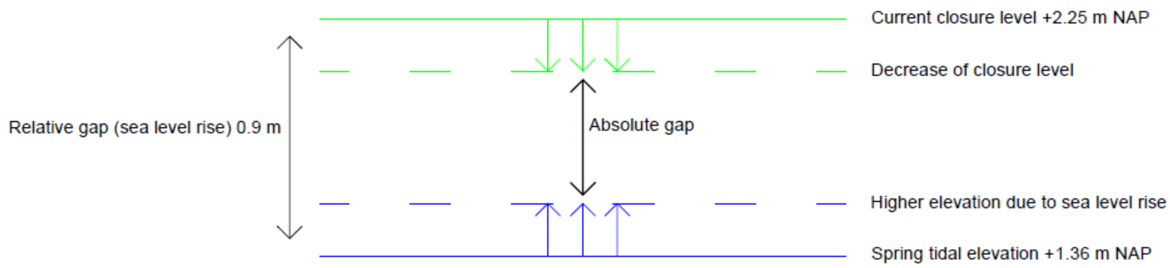


Figure 23 - Sea level rise and closure level

Levee reinforcement as part of the strategies

The actual strength of the levees depends amongst others on the governing water levels on the Hollandsche IJssel. The closure level of the Hollandsche IJssel barrier determines the water levels on the Hollandsche IJssel. When higher water levels are predicted the barrier closes. A lower closure level will consequently ensure that the barrier closes earlier and thus introduces lower water levels on the Hollandsche IJssel. Governing water levels on the Hollandsche IJssel are the important load on the levees. When this load is reduced the relative strength of the levees increases and reinforcement of the levees might not be necessary.

Only a decrease of the governing water levels is not enough to prevent the reinforcements (described in section 4.1.1). The inner slope stability depends not only on the governing water level but also on the steepness of the inner slope, structure of the soil, ground level of the polder and other loads like for example precipitation (described in appendix E.1). Due to this strong dependence only a decrease of the governing water levels is not enough to prevent reinforcements, therefore reinforcement of a part of the levees should always be included in the strategy. The elaboration of the levee reinforcement is conducted in section 4.1.2.

Important tipping point

Tipping points are important because these points can suddenly change the boundary conditions and therefore change the reasoning behind certain choices. When these tipping points are considered in advance the strategies can account for the fact that a tipping point might occur. There is one important tipping point in the system, for strategies that adapt part of the storm surge barrier there are two tipping points. A tipping point is described as [23];

The event of a previously rare phenomenon becoming rapidly and dramatically more common.

This explanation is used to describe the effect of the sea level rise. Due to the sea level rise closure of the storm surge barrier (rare event) becomes rapidly more common. The tipping point lies around the relative sea level rise of 0.9 meters [7, 8]. When the number of closures increases rapidly the Hollandsche IJssel system will change, due to the daily closures the barrier becomes a dam. The effect of the sea level rise is elaborated in section 4.4.

After the end of the design life time there is the possibility that a new structure is needed in the Hollandsche IJssel due to this structure the system will abruptly change. The end of the design lifetime does not directly mean that a new storm surge barrier should be built.

3.1.3 Composition of the strategies

The different solutions mentioned in section 3.1.1 are used to create the strategies described in this section, when necessary other elements are also included in the strategies. The strategies are described in combination with the important aspects mentioned in the preceding section. In each strategy (except strategy 0) levee reinforcement is part of the strategy because not all the reinforcements can be prevented. The tipping points in combination with the sea level rise are described in each strategy using the situation before and after the tipping point. The evaluation and multi criteria analysis of the composed strategies is conducted in section 3.2.

Strategy 0: Doing nothing

In this strategy the original situation is maintained and nothing is done to enhance the safety or solve other problems. The tipping point is reached when the storm surge barrier closes too much due to the sea level rise. The effects of other strategies are compared to the situation when nothing is done.

Strategy 1: Adaptation

In this strategy the existing barrier is adapted to withstand; the new boundary conditions and the change of closure scheme. Due to the changes, the loads acting on the barrier will increase and some adaptations might be necessary to maintain the safety. The closure scheme is changed to reduce the governing water levels and stop salt intrusion. A lower governing water level will increase the overall safety and reduce the reinforcement costs; reinforcements that are necessary will be executed. The use of the adapted storm surge barrier has its limits because the exiting storm surge barrier is used, eventually the structure needs to be demolished and make way for a new structure.

<u>Strategy 1A: Damming</u>
When the storm surge barrier should be renewed or when the absolute gap between closure level and spring elevation becomes too small the system will be dammed. The dam will introduce a fixed low water level on the Hollandsche IJssel; no reinforcements due to the sea level rise are necessary. The feasibility of this strategy depends on the sea level rise that occurs.

When the Hollandsche IJssel is dammed the inlet near Gouda should change because the water that is let in comes through the Hollandsche IJssel barrier.

<u>Strategy 1B: Renewal</u>
When the storm surge barrier should be renewed or when the absolute gap between closure level and spring elevation becomes too small a new storm surge barrier will be constructed. To ensure an open system the new storm surge barrier should have a higher closure level (otherwise the barrier should close to often), result of this new closure level is a set of inevitable levee reinforcements due to the higher governing water levels on the Hollandsche IJssel. The feasibility of this strategy depends on the raise of the new closure level and consequently the sea level rise.

When a new storm surge barrier is built in the system the inlet near Gouda can be retained, the source of water is not blocked.

Strategy 2: Renewal

In this strategy a new storm surge barrier is constructed in the near future. The closure scheme of the new barrier will change to decrease the governing water levels and stop salt intrusion into the Hollandsche IJssel. Besides the construction of the new barrier part of the levees are also reinforced because the decrease of the governing water levels does not prevent all the reinforcements.

When the absolute gap between closure level and spring elevation becomes too small the tipping point is reached. The open system can be retained when the closure level of the barrier is increased and consequently the absolute gap is increased (Figure 23), result of this increase is a lower number of closures per year. The side effect of this increase is however that the governing water levels on the Hollandsche IJssel increase and consequently levee reinforcements are necessary due to the higher loads acting on the levees.

When a new storm surge barrier is built in the system the inlet near Gouda can be retained, the source of water is not blocked.

Strategy 3: Damming

In this strategy the system is dammed in the near future. The dam will introduce a fixed low water level in the Hollandsche IJssel and therefore reduce the reinforcement costs and increase the overall safety. Due to the dam there is no tipping point for the sea level rise because the system is cut off from the sea.

When the Hollandsche IJssel is dammed the inlet near Gouda should change because the water that is let in comes through the Hollandsche IJssel barrier.

3.1.4 Summary of the composed strategies

The composed strategies are summarized in Table 4 and evaluated on the aspects treated in the next section.

Table 4 - Summary composed strategies

#	Strategy	Before tipping point	After tipping point
0	Doing nothing	• Nothing is done	• Nothing is done
1A	Adaptation and damming	• Adaptation existing barrier • New closure scheme • Reinforcement levees	• Damming river • Change inlet Gouda
1B	Adaptation and renewal	• Adaptation existing barrier • New closure scheme • Reinforcement levees	• New storm surge barrier • Change closure scheme • Reinforcement levees
2	Renewal	• New barrier • New closure scheme • Reinforcement levees	• Change closure scheme • Reinforcement levees
3	Damming	• Damming river • Change inlet Gouda • Reinforcement levees	

3.2 Evaluation of the strategies

In this section the strategies are evaluated based on the different problems mentioned in the summary of section 2.7. The criteria that are evaluated are;

1. Possible decrease of the governing water levels to;
    - Increase the overall safety. The reduction of the risks in the first layer (prevention) increases the overall safety of dike ring 14 and 15.
    - Reduce the costs of levee reinforcements. The decrease of the governing water levels on the Hollandsche IJssel should result in a decrease of the costs needed for levee reinforcements.
2. Effect of the sea level rise on the strategies. Sea level rise influences the effectiveness of the different strategies and is therefore important.
3. Continuity of the fresh water inlet near Gouda. The use of the closed storm surge barrier should stop salt intrusion reaching the inlet and therefore ensure the use of the inlet.
4. Costs of the different strategies. The costs of the different strategies are important because there is not a lot of money available in the budget of the new flood defence program to increase the safety.
5. Effect of the strategies on the morphological balance. The morphological balance of the system is important because scour holes could threaten the stability of levees, the storm surge barrier or ships cannot pass due to sediment piling up in the river.

6. Effect of the strategies on the ecology. The ecology is important because the system is a unique nature reserve in Europe (fresh tidal estuary).
7. Effect of the strategies on shipping. Shipping is important because the strategies affect the delay of the ships going through the Hollandsche IJssel.
8. Effect of the strategies on vehicular traffic. Traffic crossing the Algera Bridge is important because it is hampered due to use of the lock (during closure of the barrier) and the narrow bridge.
9. Effect of the surrounding on the strategies. The surrounding is important because developments in the surrounding limit the space available for new structures.

The secondary functions that can be assigned to the storm surge barrier or dam are treated in section 3.3 because the conclusions are drawn for secondary functions in general. The strategies are valued compared to strategy 0, which is always valued with a 0.

- A positive influence compared to strategy 0 is valued with a 1 or 2.
- A neutral influence compared to strategy 0 is valued with a 0.
- A negative influence compared to strategy 0 is valued with a -1 or -2.

These values are based on the text in the tables, the text below the tables and the appendices. Based on these values a multi criteria evaluation with weighed criteria will be conducted in section 3.4.

3.2.1 Decrease of the governing water levels

The decrease of the governing water level increases the safety in the hinterland and reduces the costs to reinforce the levees, which were not up to the standards in the last safety assessment. In the current situation the governing water levels are affected by the closure level, non-closure probability, pumping stations and other hydrological boundary conditions. Table 5 shows the possibility to decrease the water levels.

Table 5 - Possibility to decrease the governing water levels on the Hollandsche IJssel

#	Strategy	Possibility to decrease the water levels	Score
0	Original situation	The water levels are not decreased.	0
1A	Adaptation and damming	Before the tipping point is reached the governing water levels can slightly decrease. The decrease is however limited due to the high non-closure probability (which can be decreased) and the tidal elevation because of the open connection. After construction of the dam a considerable decrease of the water levels is possible. Due to the dam the river is closed off and a fixed lower water level is introduced.	2
1B	Adaptation and renewal	Idem as first paragraph strategy 1A. After construction of the barrier the decrease of the water levels is limited; the open connection remains. Due to a good design the non-closure probability of the new storm surge barrier should be low and the increase of the governing water levels limited. The tide in the river is also damped when the flow area (cross-section of the flow at the storm surge barrier) is reduced.	1
2	Renewal	The decrease of the water levels is limited because the open connection remains and therefore there is a tidal elevation in the system. Due to a good design the non-closure probability of the new storm surge barrier should be low and the increase limited. The tide in the river is also reduced when the flow area (cross-section of the flow in the storm surge barrier) is reduced.	1
3	Damming	After construction of the dam a considerable decrease of the water levels is possible. Due to the dam the river is closed off and a fixed low water level is introduced.	2

The preferred strategy for the decrease of the governing water levels is damming of the Hollandsche IJssel. Due to damming most of the hydrological boundary conditions, like the tide, do not influence the governing water levels on the Hollandsche IJssel. If damming is not possible in the near future (strategy 3) it is still preferred after the adaptation of the barrier (strategy 1A).

3.2.2 Effect of the sea level rise on the strategies

In this study the sea level rise is an uncertain parameter which affects the choice between the strategies. The KNMI W+ study expects 0.35 m sea level rise in 2050 and 0.85 m in 2100, there is however no evidence that the sea level rise goes faster than the 0.2 meters of the last centuries. In Table 6 the strategies are compared to the effect the sea level rise has on the strategies.

Table 6 - Effect of the sea level rise

#	Strategy	Effects	Score
0	Original situation	When there is a normal increase (0.2 meter per century) the different problems concerning climate change becomes larger over time.	0
1A	Adaptation and damming	Before the end of lifetime the barrier is adapted, no large structural changes are executed. The sea level rise is monitored and action can be taken when the tipping point is reached. When the tipping point is reached the choice between renewal and adaptation will be made. This choice depends on the sea level rise that has occurred and the actual strength of the levees.	2
1B	Adaptation and renewal	Idem as first paragraph strategy 1A. Idem as second paragraph strategy 1B.	2
2	Renewal	The construction of a new storm surge barrier is normally constructed for a design lifetime of 100 years. The design therefore needs to account for the possible sea level rise that occurs in 2100, without knowledge of this rise it is better to postpone the renewal of the barrier. When the tipping point is reached the closure scheme of the barrier will have to change. The number of closures will have to decrease to prevent the creation of a dam; this is possible when the closure level of the barrier is raised. Disadvantage of this raise is that the governing water levels on the Hollandsche IJssel increase and that reinforcement of the levees is necessary.	-2
3	Damming	The construction of a dam is a permanent solution for the situation in the Hollandsche IJssel and solves all the problems concerning sea level rise; the system is cut off from the sea. Without knowledge of the sea level rise it not sure whether damming is necessary, it is therefore better to postpone the construction of a new dam.	-1

Large investments can better be postponed as long as actual knowledge of the expected sea level rise is lacking; adaptation of the barrier is therefore preferred (strategy 1A or 1B). When the adapted barrier does not fulfill the requirements a surge barrier or dam can be built according to the occurred sea level rise.

3.2.3 Continuity of the fresh water inlet near Gouda

The inlet of water stops when water in the Hollandsche IJssel becomes brackish or when there is no fresh water available. Closure of the Hollandsche IJssel storm surge barrier prevents salt intrusion entering the Hollandsche IJssel. The system behind the inlet (and closure of the Hollandsche IJssel) is described in section 2.5 and 4.2. Table 7 shows the effects of the different strategies on the continuity of the inlet.

Table 7 - Effects of the different strategies on the water inlet

#	Strategy	Effects	Score
0	Original situation	Closure of the barrier does not result in a continuation of the fresh water inlet because the source of water is cut off (New Meuse through the Hollandsche IJssel).	0
1A	Adaptation and damming	Before the tipping point is reached salt intrusion is prevented when the adapted storm surge barrier is closed. The inlet of water after closure is limited to the storage of the Hollandsche IJssel and the small-scale water supply (KWA). After construction of a dam salt intrusion is prevented because water in the system becomes fresh. The source of water should change permanently, because the inlet is cut off from the source of water.	1
1B	Adaptation and renewal	Idem as first paragraph strategy 1A. After renewal of the storm surge barrier salt intrusion is prevented when the storm surge barrier is closed. Under normal circumstances the New Meuse is used as source otherwise the KWA is used.	2
2	Renewal	Salt intrusion is prevented when the storm surge barrier is closed. During normal circumstances the New Meuse is used as source otherwise the KWA is used.	2
3	Damming	After construction of the dam salt intrusion is prevented because water in the system becomes fresh. The source of water should change permanently because the inlet is cut off from the New Meuse.	1

Besides the possibilities mentioned above it is also possible to slow salt intrusion in the New Meuse or New Waterway using a bubble screen or salt stair [24]. All options prevent the stop of the inlet in theory; the implementation and continuity of the different strategies will therefore be decisive. The implementation and continuity of strategies 1A and 3 is low because they permanently rely on the small-scale water supply (KWA mentioned in 2.5). The other strategies use the open connection to the New Meuse as supply and only rely on the KWA when the adapted or new storm surge barrier is closed.

3.2.4 Costs of the alternative strategies

The costs for the alternative strategies are important because the available budget is limited. The program nHWBP only reimburses money that is used for the increase of the overall safety; other aspects like tidal nature are not reimbursed. The costs of the alternative strategies only study the costs that need to be made for salt intrusion and safety because this are the two problems directly related to the storm surge barrier. Construction of a bridge or road is not part of the costs because this can be constructed without the use of the storm surge barrier.

Description costs for the alternative strategies

The total costs of the different strategies looks at the costs that need to be made until the second part of the adaptation strategies is executed. The costs for this second part are calculated back to the base year using the net present value. The net present value (NPV) uses the actual value of the money and calculated money spent in the future back to the present day, the actual calculation of the NPV and costs are presented in appendix C.

<u>Strategy 0</u>
Strategy 0 only executes the levee reinforcements that are necessary according the results of the nationwide safety assessment. The costs that are needed for the levee reinforcements are studies by the water board Schieland and Krimpenerwaard and published in the report of the sub program Hollandsche IJssel [4, 11]. The total costs for the levee reinforcements are 495 million euros according to these studies.

<u>Strategy 1</u>
In strategy 1 the levee reinforcements are executed and the storm surge barrier is adapted. The costs for the levee reinforcements are lower (318 million euros) because the governing water levels are reduced and the reinforcements are optimized [11]. The costs for adaptation of the Hollandsche IJssel barrier are expected to be

50 million euros. The costs to decrease the non-closure probability are expected to be 25 million according to a brain storm session at the department of Public Works; this value is doubled because of other structures like a fish passage [18]. The costs needed for the change of the small scale water supply and canalized Hollandsche IJssel (to guarantee the continued inlet of water during salt intrusion) is approximately 20 million euros according to the thesis of F.Bulsink (UT Twente) [13]. These costs are included in all the strategies except strategy 0.

Adaptation strategy A and B construct a new storm surge barrier or dam in the future, the obtained costs for the construction are calculated back using the net present value of the money. When the lifetime of the adapted storm surge barrier is extended beyond 2060 (end of design life time) this further reduces the costs. For the calculation in appendix C the end of the design lifetime is used as time that the money is spent.

Strategy 2
In strategy 2 the costs for the levee reinforcements are the same as the costs for the adaptation because a larger decrease is only possible with the construction of a dam. The costs needed for the construction of the Hartel barrier are used to estimate the costs of the new Hollandsche IJssel storm surge barrier. The Hartel barrier is used because the dimensions and boundary conditions are comparable to that of the Hollandsche IJssel storm surge barrier.

Strategy 3
In strategy 3 the costs for levee reinforcement are lower because the dam introduces a fixed low water level on the Hollandsche IJssel. The costs obtained from the sub program Hollandsche IJssel are 166 million. The costs for the new dam are 400 million [11]. The costs are high because the new dam should also accommodate a pumping station to discharge water out of the Hollandsche IJssel on the New Meuse and a of new lock chambers because the capacity of the old lock chamber is too low (described in appendix B.3).

Conclusion; costs of the alternative strategies

Conclusion of this section is that adaptation of the storm surge barrier is preferred because the costs needed for adaptation are lower than the costs needed for the counterpart (1A compared to 3 and 1B compared to 2). When possible a new storm surge barrier is preferred above a dam, because locks are not necessary. This is possible when the sea level rise is slow. The different costs are shown in Table 8.

Table 8 - Costs alternative strategies [million euros]

Strategiy	Total costs	Reinforcement	Adaptation	New storm surge barrier	Dam + new locks	Salt intrusion	Score
0 Doing nothing	495	495	-	-	-	-	0
1A Adaptation and damming	543	318	50	-	155*	20	1
1B Adaptation and renewal	448	318	50	60*	-	20	2
2 Renewal	492	318	-	154	-	20	-1
3 Damming	586	166	-	-	400	20	-2

** Net present value with the price level of 2012*

3.2.5 Effect of the strategies on the morphological situation

The morphological situation of the Hollandsche IJssel River is important because both sedimentation and erosion of the river could become a problem. Sedimentation of the river hampers shipping on the Hollandsche IJssel. Erosion of the Hollandsche IJssel threatens the stability of forelands and hydraulic structures (liquefaction and shearing). Erosion deepens the river and therefore increases the steepness of the slope which eventually causes slope instability. Due to both mechanisms the morphological situation should ideally remain the same.

Just behind the Hollandsche IJssel storm surge barrier scour holes of approximately 11 meters depth have developed over time. Assumed is that closure during periods other than the slack tide created these scour holes, the water velocities were too high and sand eroded. Scour holes can threaten the stability of the storm surge barrier; therefore a solution to this problem is preferred. Table 9 describes the effects of the strategies on sedimentation and erosion of the Hollandsche IJssel.

Table 9 - Effect on the morphological situation

#	Strategy	Effects	Score
0	Original situation	When nothing is done the system imports sediment, occasional dredging of the waterway prevents sedimentation of the Hollandsche IJssel.	0
1A	Adaptation and damming	Adaptation of the storm surge barrier does not change the morphological situation, during normal conditions the cross section remains the same. Sedimentation in the Hollandsche IJssel will probably slow down because the number of closures increases; closures during ebb counteract the import of sediment. To prevent the increase of the depth of scour holes the adaptation needs to involve scour protection in front and behind the barrier. When the dam is constructed there is no movement of sand because the dam prevents the tide from entering the basin. There are no other sand fluxes other than movement of sediment caused by ship.	1
1B	Adaptation and renewal	Idem as first paragraph strategy 1A. Renewal of the storm surge barrier will probably change the cross-section of the flow. Whether the cross-section is increased or decreased depends on the design of the new storm surge barrier. The morphological balance will change due to an increase or decrease of the cross-section.	0
2	Renewal	Renewal of the storm surge barrier will change the cross-section of the flow. Whether the cross-section is increased or decreased is not known, the morphological balance will however change.	-1
3	Damming	Damming will ensure that there is no movement of sand because the dam prevents the tide entering the basin. There are no other sand fluxes in the system other than movement of sediment caused by ships.	1

The morphological situation is part of the Hollandsche IJssel system; change in this system is not preferred because changing sediment fluxes could threaten the stability of forelands or hamper shipping. Adaptation of the storm surge barrier or damming of the river is preferred because the original situation is retained or the velocities in the Hollandsche IJssel are small due to the dam.

3.2.6 Effect of the strategies on the ecology

The ecology in the system is affected by the strategies because the tide is dampened and the storm surge barrier closes during low discharges. The ecology in the Hollandsche IJssel is formed by the tidal nature, ecological main structure (EHS) and the fish migration into the Hollandsche IJssel. Table 10 describes the effects of the different strategies on the ecology.

Table 10 - Effect on the ecology

#	Strategy	Effects	Score
0	Original situation	When nothing is done the ecology in the system is preserved.	0
1A	Adaptation and damming	When the barrier is adapted the ecology is preserved. When the barrier closes because of salt intrusion this needs to be monitored. Long closures will result in low oxygen levels and consequently a "dead" river. Values created by the program Cleaning the Hollandsche IJssel are not lost (section 2.2). When the system is dammed the tidal nature is lost because there is no tide in the system.	-2
1B	Adaptation and renewal	Idem as first paragraph strategy 1A. When a new barrier is built part of the tide can still enter the system. The tidal nature in the forelands of the Hollandsche IJssel is preserved.	0
2	Renewal	During normal circumstances the tide can still enter the river therefore the tidal nature is preserved.	0
3	Damming	The tidal nature is lost because the tide cannot enter the Hollandsche IJssel.	-2

There is no strategy that directly enhances the ecology in the Hollandsche IJssel. It is however possible to enhance the nature in the system when the reinforcement of the levees also creates space for tidal nature. The strategies which do not dam the system are preferred because the fresh tidal estuary is not threatened (strategies 0, 1A and 2).

3.2.7 Effect of the strategies on shipping

The Hollandsche IJssel is a river predominantly used for the shipping of containers and other raw materials. Ships from the Rotterdam harbor use the New Meuse, Hollandsche IJssel and Gouwe Canal to reach the Alphen aan de Rijn container terminal (shown in Figure 24). There are 120 ship movements per day in the Hollandsche IJssel it is expected that this grows to 200 ship movements in the future, because of the increase of the container terminal and Heineken brewery located near Alphen aan de Rijn [25].

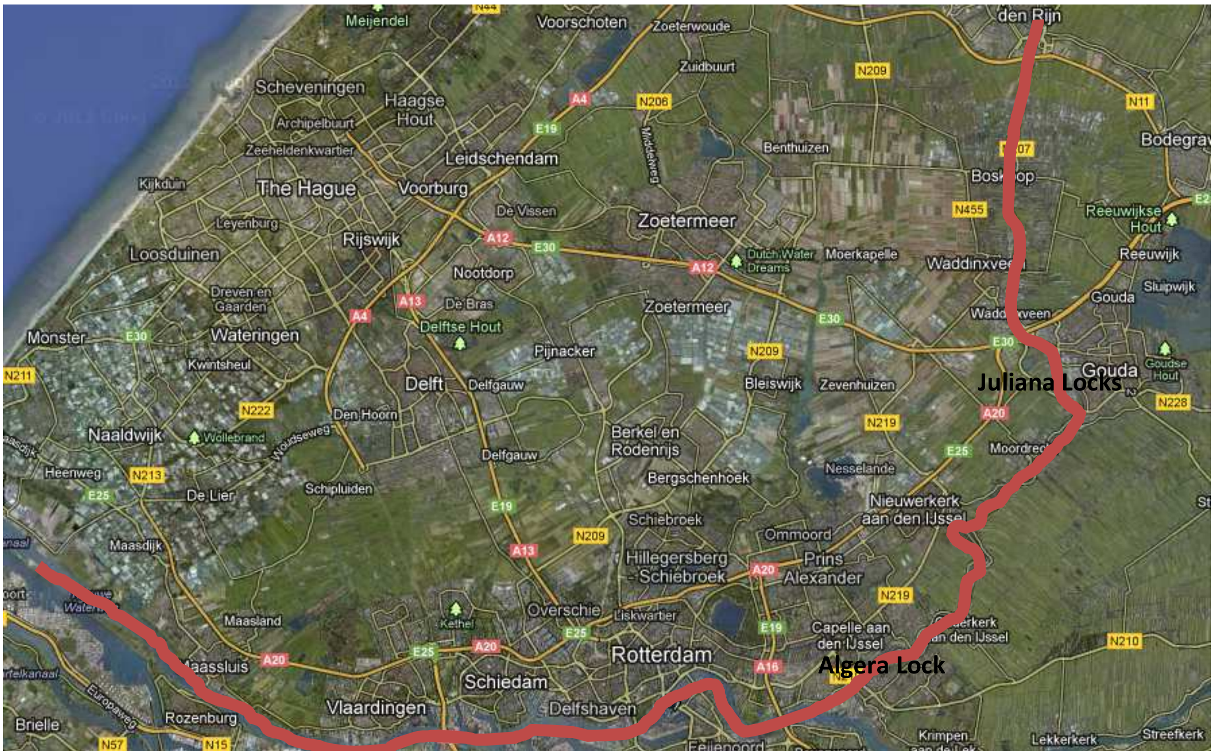


Figure 24 - Route of container ships, source; Google maps

Existing situation shipping

The shipping of containers is a continuous business on the Hollandsche IJssel and makes use of the locks when the barrier is closed or when the containers are stacked too high (4 stacks). Governing ship in the Hollandsche IJssel is a container ship loaded with 4 stacks of containers; the dimensions of this ship are shown in Table 11. The main dimensions of the structures that are passed along the route to the container terminal are also shown in this table.

Table 11 - Dimensions normative ship storm surge barrier and locks

Dimensions	Governing ship	HIJ barrier	Algera lock	Juliana lock (new)
Draft	3.0 m	-4.5 m NAP	-4.5 m NAP	-4.0 m NAP
Width	13.0 m	82 m	24 m	14 m
Length	90 m	-	112 m	115 m

With the construction of the new Juliana lock chamber the dimensions of container ships are limited to this lock chamber [26]. The minimum width, depth and vertical clearance are calculated in appendix B1 and shown in Table 12.

Table 12 - Minimum dimensions channel through the storm surge barrier

Dimensions	One channel
Width	47 m
Depth	-4.4 m NAP
Vertical clearance 3 stacks	+8.8 m NAP
Vertical clearance 4 stacks	+10.9 m NAP

The width and depth of the channel that is needed to pass the Hollandsche IJssel storm surge barrier is calculated according to the guidelines provided by the department of Public Works [27]. The existing storm surge barrier fulfills the requirements with respect to the minimum dimensions of the channel for governing ships. The only disadvantage of the existing barrier is that container ships with four stacks cannot pass the storm surge barrier at the moment.

Future situation shipping

The situation in the Hollandsche IJssel changes when one of the strategies is executed. Due to a change in closure scheme or construction of a dam the ships experience delays. Due to the low capacity of the lock situated next to the Hollandsche IJssel storm surge barrier ships need to wait. Calculation of the economic damage in appendix B.3 shows that the economic damage of ships during a closure is limited when the number of closures is low. When the system is dammed at least two lock chambers are needed to limit the economic damage. The effect of the different strategies on the shipping is shown in Table 13.

Table 13 - Effect of the strategies on shipping

#	Strategy	Effect on shipping	Score
0	Original situation	When the original situation is maintained the economic damage is not high. After the tipping point is reached the economic damage will increase because the number of closures increases.	0
1A	Adaptation and damming	When the closure scheme is changed (lower closure level and closure during salt intrusion) the barrier will close more and cause more economic damage due to delay of ships that need to wait. After construction of the dam ships need to use the lock which leads to extra delay and consequently more economic damage. The expected economic should result in the demand to construct a second lock chamber.	-1
1B	Adaptation and renewal	Idem as first paragraph strategy 1A After construction of the new storm surge barrier shipping will benefit from the open connection because the economic damage is minimal	1

		compared to the dam.	
2	Renewal	Shipping will benefit from the open connection because the economic damage is minimal compared to the economic damage created by the dam. When the new barrier has a high vertical clearance ships with 4 stacks can also pass the barrier without use of the locks.	2
3	Damming	When the Hollandsche IJssel is dammed ships need to use the locks which results in economic damage and the demand to construct a new lock chamber.	-2

In the future the renewal of the barrier (strategies 1B and 2) is preferred because the economic damage is limited due to the open connection. Damming (strategies 1A and 3) is not preferred because ships need to use the lock, which causes delays and consequently economic damage.

3.2.8 Effect of the strategies on vehicular traffic

During the morning and evening rush hours 3 000 vehicles per direction pass the (1*1 lane) Algera Bridge, this causes congestion on the traffic junctions around the Algera Bridge [28]. In studies conducted as part of "Master plan Rotterdam Vooruit" different alternatives are examined to solve the congestion in the region East Rotterdam. One of the alternatives shows that the construction of a 2*2 connection fulfills the requirements laid down by the department of Public Works up to 2040. The situation after 2040 is not studied but an increase seems probable. To maintain a good connection it is therefore desirable to create an extension to a 3*3 connection.

Figure 25 - Location new Algera Bridge, source; Google Maps

Within the current configuration of the storm surge barrier it is possible to construct a bridge (between the two lift gates shown in Figure 25). The width between the lift gate and the existing Algera Bridge is approximately 80 meters the width of a 2*2 bridge is 30 meters which leaves space for construction (the calculations are presented in appendix B.2).

There are possibilities to combine the two functions (bridge and storm surge barrier) but this is not preferred, because different functions within a storm surge barrier complicate the design, reduce the functionality and threaten the safety. Table 14 describes the effect of the different strategies on the construction of the new Algera Bridge and the connection as a whole.

Table 14 - Effect of the strategies on the traffic flows

#	Strategy	Effect on vehicular traffic	Score
0	Original situation	Vehicular traffic is already hampered due to the narrow bridge. When the numbers of closures increase the delay of traffic also increases, because more ships need to make use of the lock and bridge combination.	0
1A	Adaptation and damming	When the barrier is adapted traffic will only benefit when a new bridge is constructed between the two lift gates. Decrease of the closure levels results in more closures and consequently more delay due to use of the lock. In the long term traffic will benefit from the dam because of a road on the crest of the dam.	1
1B	Adaptation and renewal	Idem as first paragraph strategy 1A Vehicular traffic will only benefit when a new bridge is constructed, adaptation or renewal do not increase the traffic flow.	-1
2	Renewal	Vehicular traffic will only benefit when a new bridge is constructed, adaptation or renewal do not increase the traffic flow.	-2
3	Damming	Vehicular traffic will benefit from the dam because a road will be constructed on the crest of the dam.	2

Damming (strategies 1A and 3) is always preferred because there is the possibility to construct a road on top of the dam. The adaptation of the barrier only reduces the congestion when a new bridge is built; this is possible within the two lift gates of the storm surge barrier. Renewal of the barrier allows for the design of both barrier and bridge. The construction of a new bridge is however costly compared to the construction of a road on the dam.

3.2.9 Effect of the surrounding on the strategies

There are two factors that consider the surrounding of the storm surge barrier. These factors are linked to the existing and future available space around the storm surge barrier.

Limitations in the existing space around the storm surge barrier

In sections 2.2 and 2.6 the surrounding of the storm surge barrier and the relevant developments are described. This section analyses the possible locations for the construction of a storm surge barrier, these locations are limited because;

- the traffic junctions or structures limit the change of location,
- the activity in this area is difficult to remove,
- the area is not preferred because there is important nature or history.

An important reason for the limitation in the location of the storm surge barrier is the existing Algera Lock, the lock is designed before the Maeslant barrier was build and is therefore designed to withstand higher governing water levels. Locks are needed because ships need to be able to pass the closed storm surge barrier. Construction of new lock is expensive; it is therefore preferred to maintain the existing locks. The traffic junctions mentioned in Figure 26 limit the change of location because this road is one of the few roads into the Krimpenerwaard; relocation of this road connection is not easy because of the limited space and the new bridge needed in another location.

Figure 26 - Location map; activity (red), nature (green) and traffic connections (yellow), source; Google Maps

There are a few activities in the area which are difficult to remove. There is a large youth prison in the industrial area, with the current shortage of cells it is not expected that this youth prison will be removed. The neighborhood 'S Gravenland Oost is new and therefore this neighborhood is maintained if the storm surge barrier is built in the near future. The other neighborhoods (including industrial area Stormpolder) are old and could be demolished if there are no other "good" alternatives.

Limitations in the future space around the storm surge barrier

The development of urban river fronts (described in section 2.2.1 influences the choice for the preferred strategies in two ways;

- Space in the region becomes limited due to the urban river front developments, therefore large structures should be built in the near future.
- Lower and fixed water levels reduce the height of the levees. Buildings are then built closer to the water line and there is no levee blocking the view.

Influence surrounding on the strategies

The effect of the surrounding is relevant for the near future and the situation after 2050; Table 15 describes the effects of the surrounding.

Table 15 - Effects surrounding on the strategies

#	Strategy	Effects	Score
0	Original situation	When the original situation is maintained nothing happens because no new structures are built.	0
1A	Adaptation and damming	In the near future nothing happens because the adaptation of the storm surge barrier is conducted within the existing storm surge barrier. Additional structures like a fish ladder do not require a lot of space. Damming of the Hollandsche IJssel ensures a fixed water level on the Hollandsche IJssel; this makes the banks of the Hollandsche IJssel interesting for urban developments, this is possible because there is no threat from high water. Therefore the levees could be much lower. There is not much space required for a simple dam, additional functions like; fish ladders, locks, roads and power stations increase the required space. Due to urban river front developments the space available for a large dam might be limited.	1
1B	Adaptation and renewal	Idem as first paragraph strategy 1A In the long term a new storm surge barrier would be built on the same general location (mouth of the Hollandsche IJssel) as the existing barrier. Due to urban river front developments the space available for a new storm surge barrier might become limited.	1
2	Renewal	The new storm surge barrier would be built on the same location as the existing barrier. Main reason for the same location is the use of the same lock, due to the open connection the locks are only used during closure. When the new storm surge barrier is constructed in the near future there is enough space available.	1
3	Damming	Damming of the Hollandsche IJssel ensures a fixed water level on the Hollandsche IJssel which makes the banks interesting for urban developments. There is not much space required for a simple dam, additional functions like; fish ladders, locks, roads and power stations increase the required space.	2

Damming of the Hollandsche IJssel is preferred because the space required for a dam is not that much, besides that there are options for river front developments along the Hollandsche IJssel. Due to the decreasing space it is preferred to construct a structure in the near future.

3.2.10 Summary of the evaluated criteria

The summary of the evaluated criteria is presented in Table 16. The strategies 1A and 1B only give the conclusion for the adaptation part of the strategy, the second part after the strategy is the same as the two counterparts (strategy 2 and 3). The costs shown for strategy 1A and 1B are however related to the whole strategy.

Table 16 - Overview evaluation criteria

	Decrease governing water levels	Sea level rise	Continuity fresh water inlet	Morphological balance	Ecology	Surroundings	Costs
0 Doing nothing	No decrease	Sea level rise is monitored, no intervention is executed.	Continuity not guaranteed	The morphological balance is not affected.	Ecology is not affected.	No new structures are built.	495 million euros
1A Adaptation + damming	Decrease possible	Intervention is executed when the tipping point is reached.	Continuity guaranteed	Morphological balance is not affected at first.	Adaptation does not affect the ecology	Adaptation does not affect the surrounding	543 million euros
1B Adaptation + Renewal	Decrease possible	Intervention is executed when the tipping point is reached.	Continuity guaranteed	Morphological balance is not affected at first.	Adaptation does not affect the ecology	Adaptation does not affect the surrounding.	448 million euros
2 Renewal	Decrease possible	Construction of a new barrier in the near future is not preferred	Continuity guaranteed	The morphological balance is affected due to the changed conditions.	A new storm surge barrier can affect the ecology.	A new barrier affects the local surrounding.	492 million euros
3 Damming	Large decrease possible	Construction of a new dam in the near future is not preferred	Continuity guaranteed	The morphological balance is affected because the tide is diminished.	Tidal nature is destroyed due to the dam.	A dam affects the local surrounding and river fronts.	586 million euros

3.3 Secondary functions of the storm surge barrier or dam

The important functions of the storm surge barrier are described in the preceding sections. Other functions can be added to the structure because the combination of functions reduces the costs or increase the benefits. In this section the possible secondary functions are considered, these functions may not threaten the performance of the storm surge barrier or dam. General drawback of secondary functions is the governance and finance which are not clear and have to be agreed upon in advance. The secondary functions are discussed based on five groups described in appendix D. Table 17 gives a description of the different groups.

Table 17 - Secondary functions

Groups	Description
1 Economic development	Group 1 focuses on development of houses and offices to generate money.

	Economic development is useful when the costs of the storm surge barrier can be reduced. The obvious location for these developments is in the tower of the barrier or core of the dam. Regulations for houses and offices affect the design of the storm surge barrier.
2 Added benefit	Group 2 focuses on development of an exhibition space, watchtower or restaurant which has added benefits but does not generate money. These functions are possible when there is free space which can be used without threatening the performance of the barrier or dam. The change of the design should be limited in this group of functions because the costs would otherwise increase too much.
3 Small scale use	Group 3 focuses on small scale use like billboards or climbing walls. These functions are possible when the outside can be used without changing the design.
4 Large scale use	Group 4 focuses on large scale use of the storm surge barrier or dam like the storage of containers, parking garages and fish farms. Large scale use is useful when the costs of the storm surge barrier can be reduced. These functions will use the hollow space inside the storm surge barrier. Different regulations may affect the design of the storm surge barrier.
5 Ecology options	Group 5 focuses on the maintaining or enhancing of the ecology in the Hollandsche IJssel with the use of a tidal power station for example. Especially the hollow part of the dam can be used for this kind of structures. The design of the barrier may be altered if it can prevent large opposition against the construction of the storm surge barrier. For example the construction of a fish ladder which may convinces environmental groups.

Because of the current economic situation it is not expected that a secondary function from either group one or four will reduce the costs of the storm surge barrier. Group two and three are both secondary functions which do not reduce the costs of the new storm surge barrier. It is expected that the costs cannot be reduced with group 1, 3 and 4. There are no added benefits from group 2.

With the construction of a dam group five becomes interesting. Part of the tidal elevation in the Hollandsche IJssel can be restored when pumping stations connect the Hollandsche IJssel and New Meuse. Group 5 is not interesting for the design of the new storm surge barrier because the open connection (under normal circumstances) ensures a tidal elevation on the Hollandsche IJssel, effect of the power stations is limited when there are not many closures.

3.4 Conclusion; preferred strategy

The different criteria are evaluated in the first part using a multi criteria evaluation (MCE). The second part treats the preferred strategy that is chosen.

3.4.1 Multi criteria evaluation of the strategies

The multi criteria evaluation (MCE) uses the different criteria treated in the preceding section to value the alternative strategies. Each criterion is weighed using the relative importance between the criteria; in Table 18 the relative importance is given. When a criterion in the row is more important (or has equal importance) then the criterion in the column the value 1 is given. When the value in the row is less important a value 0 is given. In the last column all the values in the row are summed up and give the importance of a criterion.

Table 18 - Relative importance criteria

	Water level	Continuity inlet	Sea level rise	Morphology	Ecology	Shipping	Vehicular traffic	Surrounding	Cost	SUM
Water level		1	1	1	1	1	1	1	1	8
Continuity inlet	0		1	1	1	1	1	1	1	7
Sea level rise	0	0		1	1	1	1	1	0	5
Morphology	0	0	0		0	0	1	1	0	2
Ecology	0	0	0	1		1	1	1	0	4
Shipping	0	0	0	1	1		1	1	0	4
Vehicular traffic	0	0	0	1	0	1		1	0	3
Surrounding	0	0	0	0	0	0	1		0	1
Cost	0	0	1	1	1	1	1	1		6

In Table 19 the value of each strategy is presented using the importance and the score that is given per criterion. The value between brackets gives the score given per strategy. The value without brackets gives the multiplication of the score with the importance per strategy and criterion.

Table 19 - Summary results MCE

Criteria	Importance	Strategies 0	1A	1B	2	3
Decrease governing water levels	8	0 (0)	16 (2)	8 (1)	8 (1)	16 (2)
Sea level rise	5	0 (0)	10 (2)	10 (2)	-10 (-2)	-5 (-1)
Continuity of the inlet near Gouda	7	0 (0)	7 (1)	14 (2)	14 (2)	7 (1)
Costs	6	0 (0)	6 (1)	12 (2)	-6 (-1)	-12 (-2)
Morphological balance	2	0 (0)	2 (1)	0 (0)	-2 (-1)	2 (1)
Ecology	4	0 (0)	-8 (-2)	0 (0)	0 (0)	-8 (-2)
Shipping	4	0 (0)	-4 (-1)	4 (1)	8 (2)	-8 (-2)
Vehicular traffic	3	0 (0)	3 (1)	-3 (-1)	-6 (-2)	6 (2)
Surrounding	1	0 (0)	1 (1)	1 (1)	1 (1)	2 (2)
MCE value = ∑ (importance * value)	-	0	33	46	7	0

Conclusion of this section is that adaptability is preferred above a structure that is created in the near future. The conclusion is predominantly based on the unknown sea level rise in combination with the possibilities to decrease the governing water levels and continue the inlet near Gouda. Secondary functions are not added because section 3.3 shows that secondary functions do not result in a reduction of the costs. Adaptation focuses on the minimum of changes needed to guarantee the use of the existing storm surge barrier.

The sea level rise is the important criterion determining the choice for strategy 1A or 1B because it changes the number of closures dramatically. Strategy 1B is preferred when the sea level rise behaves slower than the climate change studies; 1A is preferred when the sea level rise behaves faster than expected. Turning point between the two strategies lies in the absolute gap and relative sea level rise (described in Figure 23). This turning point is analyzed in section 4.4.

3.4.2 Specification of the strategy adaptation

In the following chapters of this thesis the new closure scheme, changed water balance and adaptation of the existing storm surge barrier are described (strategies 1A and 1B). The other strategies are not elaborated; the results of the next chapters should however substantiate the choice for adaptation of the existing storm surge barrier and postponing of the new storm surge barrier or dam. The choice between damming and renewal after the adaptation of the barrier is evaluated on the basis of the sea level rise at the end of chapter four. The new dam or barrier is not designed within the scope of this study.

The solution to the problems described in section 2.7 is presented in Table 20. The problems which are not directly solved with adaptation of the storm surge barrier are shown as a -, these parts are related to the surrounding or to the reinforcement of the levees.

Table 20 - Summary problems Hollandsche IJssel

Analyzed aspects	Problem	Solution
1 Schematization	-	-
2 Relevant developments	Uncertain sea level rise	Adaption of the storm surge barrier, monitoring of the sea level rise and the postponing of large new structures.
	Overall safety dike ring 14 and 15	Decrease of the governing water level on the Hollandsche IJssel and a decrease of the non-closure probability.
3 Storm surge barriers	Results nationwide safety assessment	Assessment of the storm surge barrier and in particular the steel gate and non-closure probability.
4 Fresh water supply	Climate change and salt intrusion	New closure scheme to guarantee the continuity of the inlet.
5 Levees	Results nationwide safety assessment	Reinforcement of the levees, change of the closure scheme and decrease of the governing water levels.
6 Morphology	Morphological balance	Construction of scour protection near the storm surge barrier.
7 Ecology	Recovered ecology	Construction of a fish passage that can be used during closures.
8 Surrounding	Delayed traffic flows	-
	Urban river development and ribbon development	-

In this study the decrease of the governing water levels, reinforcement of the levees, new closure scheme and changed water balance are described in chapter 4. The structural assessment and adaptation of the existing steel gate is described in chapter 5. The preliminary analysis of the non-closure probability is presented in section 5.3.1; the preliminary design of the scour protection and fish passage is presented in section 5.2.2 and 5.2.3. The problems related to the surrounding of the storm surge barrier are not described in this study.

4 A new closure scheme in combination with climate change

This chapter deals with the adaptation of the storm surge barrier, introduces a new closure scheme and reinforces the levees if necessary. This chapter also substantiates the reasons (given in chapter three) to adapt the storm surge barrier and sets up the new closure scheme and water balance that determines the adaptations that are needed to withstand the increased sea level rise. The different aspects of this chapter are described in the following sections;

1. The effect of the governing water levels on the reduction of the costs needed for levee reinforcement and increase of the overall safety (section 4.1). The objectives related to this section are;
   a. Economize 35% of the reinforcement costs,
   b. Decrease the risks of flooding in dike rings 14 and 15 with 50%.
2. The use of the storm surge barrier to prevent salt intrusion reaching the inlet (section 4.2).
3. Water balance and new closure scheme of the Hollandsche IJssel (section 4.3).
4. Effect of the sea level rise on the use of the new closure scheme and choice for damming or renewal (section 4.4).

At the end of this chapter it should be clear what the new closure scheme is, if the objectives are reached, what the solution to salt intrusion is and what the effect of the sea level rise is (section 4.5).

4.1 Decrease of the governing water levels

A decrease of the governing water levels on the Hollandsche IJssel is possible when the closure scheme is changed. This means that the storm surge barrier should close at lower high water levels. The two related reasons for a decrease of the governing water levels are:

- Costs (section 4.1.1); a decrease of the governing water levels results in the prevention of levee reinforcements along the Hollandsche IJssel. The height is for example just sufficient due to the lower water levels.
- Increase of the safety (section 4.1.2); a decrease of the governing water levels means that the failure probability of the levees becomes lower, therefore the contribution of the levees along the Hollandsche IJssel to the overall risk of flooding becomes lower. The contribution of the Hollandsche IJssel is calculated when the failure probability of the levees is multiplied with the consequences of flooding when the levee is breached.

4.1.1 Reduction of the costs for the levee reinforcements

The delta program Rijnmond and Drechtsteden subprogram Hollandsche IJssel did research into the reduction of the reinforcement costs. The conclusion of this study was that the reinforcement costs could be reduced from 495 million euros to 318 million euros, which is a reduction of 35%. The objective to reduce the reinforcement costs with 35% is derived from this study [11].

Governing failure mechanisms

The levee system behind the Hollandsche IJssel barrier consists of 36 kilometer of levees. Twenty-eight kilometers of this system is not up to the standards (shown in Figure 16). Predominantly the failure mechanisms “inner slope stability” and “overflow/overtopping” did not meet the requirements in the Hollandsche IJssel system. A failure mechanism that occurred in a few situations was piping. All the assessments were conducted according to the guidelines drafted by the department of Public Works [12].

The levee fails when a large part of the inner slope becomes unstable and slides down. There are two load combinations that cause an unstable slope;

- Due to high (ground) water levels in the river (and levee) the slope becomes unstable,
- Due to extreme precipitation the ground water level in the levee becomes high and the inner slope unstable.

The levee fails due to overflow or overtopping when large volumes of water are discharged over the crest of the levee. This occurs due to high water levels and wind waves on the river. Piping is a phenomenon that occurs when water is able to flow through or under the levee and transfer sand particles. Piping occurs when there is a large difference between the water levels on the river and in the polder.

The important parameters influencing these failure mechanisms are shown in Table 21. The influence of the water levels on other failure mechanisms is not included because these failure mechanisms are not governing. Almost all failure mechanisms have a positive influence on the assessment (described in appendix E.1) when the governing water level is reduced. Therefore other failure mechanisms do not become governing. In theory only the outer slope could become unstable when the water levels are decreased. This decrease implies that the water levels are decreased below the daily water levels on the Hollandsche IJssel, which can only happen when a dam is constructed and a fixed low water level is introduced. Therefore instability of the outer slope this is not governing for the adaptation of the storm surge barrier.

Table 21 - Important failure mechanisms

Failure mechanism (load)	Influencing parameters
Inner slope stability (high water)	• Gradient of the inner slope • Ground water level in the levee • Governing water level on the HIJ • Soil parameters
Inner slope stability (extreme precipitation)	• Ground water level in the levee • Precipitation • Soil parameters • Gradient of the inner slope
Overflow/overtopping (high water)	• Height of the levee • Governing water level on the HIJ • Dimensions of the outer slope • Cover (clay) layer
Piping	• Structure of the soil (permeability) • Governing water level on the HIJ • Ground water level in the polder

In theory all three failure mechanisms are prevented if the governing water levels on the Hollandsche IJssel are decreased. There are however a few issues that limit this theory:

- It is not certain when extreme precipitation becomes or is governing.
- The steep inner slope of almost all the levees limits the increase of the stability factor, creating a gentle inner slope is often not possible due to limited space (ribbon development).
- Decrease of the water levels is also limited because the tidal elevation and closure levels of the Hollandsche IJssel storm surge barrier should not be too close to each other. Otherwise the barrier would need to close twice a day (during every flood).
- The permeability of the soil is a governing factor in the assessment of piping.

It is assumed that extreme precipitation is not governing when the decrease of the governing water levels is limited. The deep polders on the inner side of both levees ensure that it takes a long time for the ground water levels in the levee to rise to high levels due to precipitation only. The crest height of most levees is around +3 m NAP the level of the polder is around -3 m NAP. Extreme precipitation is a problem when the height of the levees is low relative to the surrounding polder level (shown in appendix E.1).

In this study only the inner slope stability due to high water and overtopping due to high water are governing. Piping is not studied due to the lack of data; it is however certain that piping is only governing for a couple of hundred meters and has therefore no influence on the total result [29].

Reduction of the costs due to a decrease of the governing water levels

Consulting agency Van der Kraan analyzed the two different failure mechanisms [29]. Result of this analysis is the division of the levee sections into different failure classes as shown in Figure 81 and described in appendix E.2. Failure class 1 shows all the sections that have an unstable inner slope, the difference between the failure classes 1.1 - 1.5 is the deficit of the stability factor F. The stability factor F is the determining parameter for the stability of the inner slope and is given as;

$$F = \frac{Resisting\ force}{Driving\ force} \geq 1.17$$

Failure class 2 shows all the levee sections that fail due to overflow/ overtopping. The determining parameter of overflow/ overtopping is the height of the levee and the allowed discharge (overtopping) over the levee.

The water board recalculated some of the stability factors for lower water levels (this recalculation was conducted for the subprogram Hollandsche IJssel) [30]. The results of this recalculation are linearized against the possible decrease of the governing water levels in appendix E.2. The result of this linearization is that the stability factor F increases with 0.05 per meter. This low increase confirms the influence of the steep slope that limits the effectiveness of the decrease.

Table 22 - Failure classes levees

Failure class	VTV assessment	Deficit[-]	Length [km]	Solution
Failure class 1.1	STBI	ΔF= 0.00-0.09	8.3 (29%)	Water levels need to be decreased with more than 1.00 meter.
Failure class 1.2	STBI	ΔF= 0.10-0.19	9.9 (34%)	Small reinforcements are needed.
Failure class 1.3	STBI	ΔF= 0.20-0.29	5.1 (18%)	Large reinforcements are needed.
Failure class 1.4	STBI	ΔF= 0.30-0.39	3.2 (10%)	Large reinforcements are needed.
Failure class 1.5	STBI	ΔF= > 0.40	1.0 (4%)	Large reinforcements are needed.
Failure class 2	HT	ΔH= 0.00-0.50	1.4 (5%)	The levees will be up to the standards when the governing water levels are decreased with (more than) 0.50 meters.
Failure class 3	Other	-		
		Total:	28.8 (100%)	

The solution per failure class is shown in Table 22. The levees in failure class 1.1 will be up to the standard when the water level is decreased with more than 1.0 m; the stability factor increases with 0.05 for a decrease of 1.0 m. Small reinforcements are structures like a drainage system or the removal of a bad soil layer. Large reinforcements are solutions like a diaphragm wall or the construction of a levee with a gentle inner slope. The levees in failure class 2 will be up to the standards when the water level is decreased with more than 0.5 m because the deficit in height is not more than 0.5 m.

The analysis shows that the reduction of the costs is not possible with only a decrease of the governing water levels; the decrease needs to be combined with other aspects to increase the effectiveness. Reinforcements in general are not prevented but major structures are prevented when the decrease of the water levels is combined with a smart design of levee reinforcements. The prevention of high reinforcement costs is especially possible with the large reinforcement because most of the budget is needed for the large structures. The removal of a bad soil layer does not cost much while the construction of a diaphragm wall is expensive. The elaboration of the design of these reinforcements is not part of this master thesis.

4.1.2 Decrease of the flood risk for dike rings 14 and 15

The decrease of the flood risks is possible when the governing water levels are decreased. The decrease of the risks is possible even when it is concluded that the reduction of the reinforcement costs is limited. The reduction of the risks is possible because the risk approach studies the entire dike ring, while the reduction of the costs only studies the levee section. The nationwide safety assessment is stricter for a particular levee section then the risk approach which studies the entire dike ring.

The overall safety of a dike ring is related to the risks of flooding. The risk of flooding is the likelihood that a certain area floods multiplied with the consequences of that flood.

$$Risk = Probability * Consequence$$

Probability is the likelihood that the levee fails; in this case it is assumed that inner slope stability is the only failure mechanism that can occur. This assumption is plausible because the inner slope stability is low and governing in nearly all the levees that are not up to the standards according to the last safety assessment [4]. The consequence is the "damage" that is caused by flooding of a region when a breach due to inner slope stability has occurred; the damage is expressed in casualties and economic damage. The risk is expressed in economic damage and casualties in a year.

The program Safety in the Netherlands (VNK) studied the risks due to flooding for dike ring 14 and 15, described in section 2.2.2. The results of this study are used to calculate the effect the reduction of the governing water levels has. The reduction of the governing water levels predominantly affects the failure probability of the levees (lower extreme load acting on the levees). In theory the lower governing water levels also affect the consequences because less water flows into the polder, in this study it is assumed that the reduction only affects the failure probability of the levees.

The stability factors that were recalculated by the water board (and also used in the preceding section) and a note written by W. ter Horst on the current failure probability of the levees are used to estimate the effect of the reduction [30, 31]. The estimated failure probability of the governing levee stretches along the Hollandsche IJssel is 1/100. The stability factors (F_d) that were recalculated are used to show the effect of the decrease on the reliability index β, which is linked to the failure probability. The reliability index β is obtained with an assumption that is used for Dutch levees [32, 33];

$$F_d = 1 + 0.13 * (\beta - 4) \, for \, \beta$$

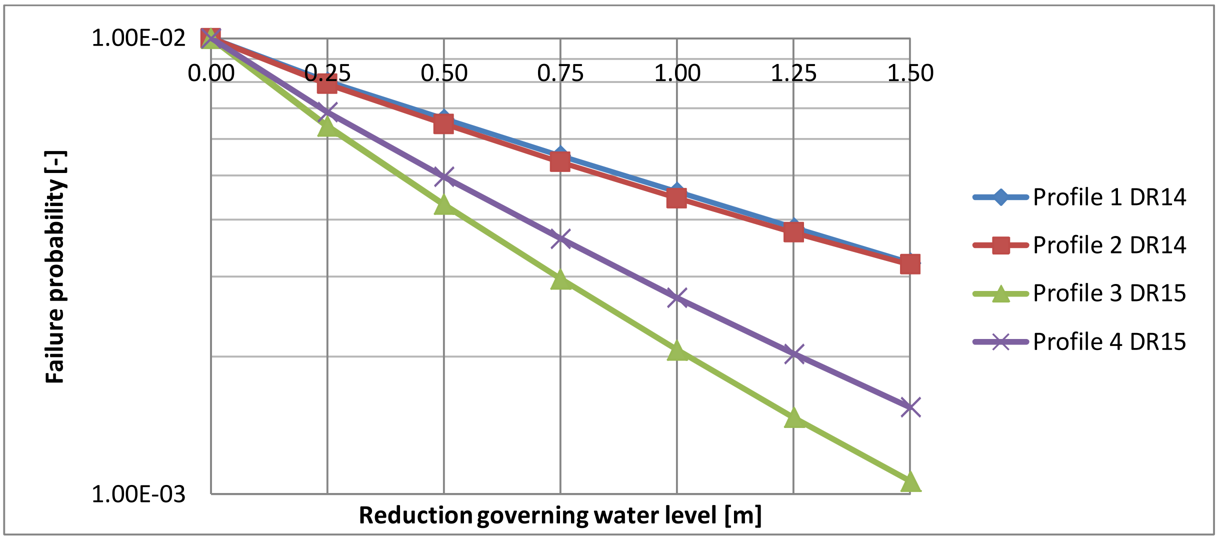


Figure 27 - Normalized failure probabilities

Presented in Figure 27 is the normalized failure probability for the representative profiles along the Hollandsche IJssel (the location of the profiles is shown in E.3 Figure 82). The failure probabilities correspond with the results from the preceding section, the reduction of the governing water levels decreases the failure probability but the assessment criterion per levee section is not met (1/10 000 per year).

The failure probabilities combined with the consequences of a breach result in the risk contribution. The casualties and economic damage due to a breach in one of the stretches are obtained from the calculations that were conducted by the program Safety in the Netherlands [5]. Figure 28 and Figure 29 show the reduction of the risks when the governing water levels on the Hollandsche IJssel are decreased. The figures show the contribution of the levees along Hollandsche IJssel to the total risks of dike ring 14 and 15, not the total risks of the dike rings. The entire calculation of the risks is presented in appendix E3.

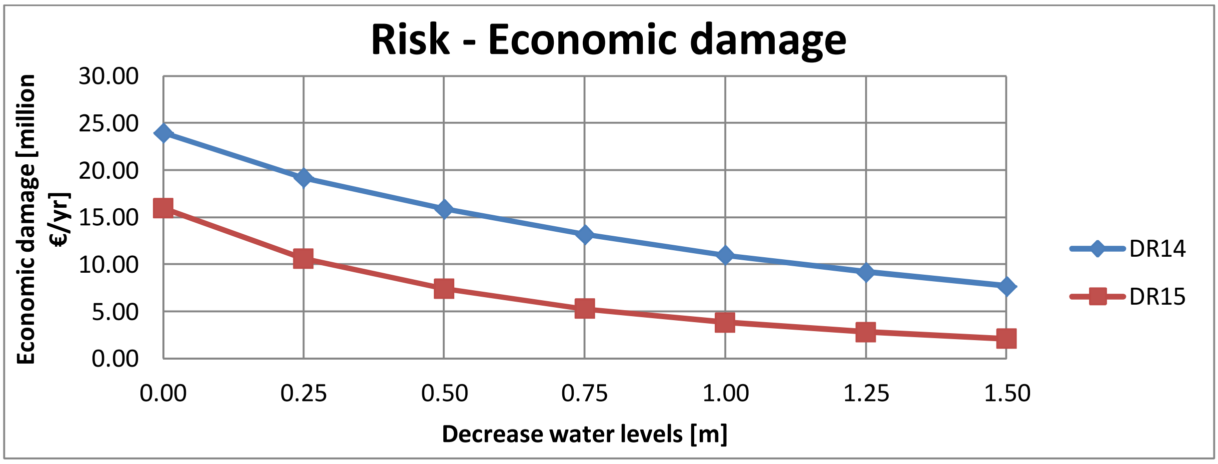


Figure 28 - Reduction of the risks due to a decrease of the governing water, economic damage

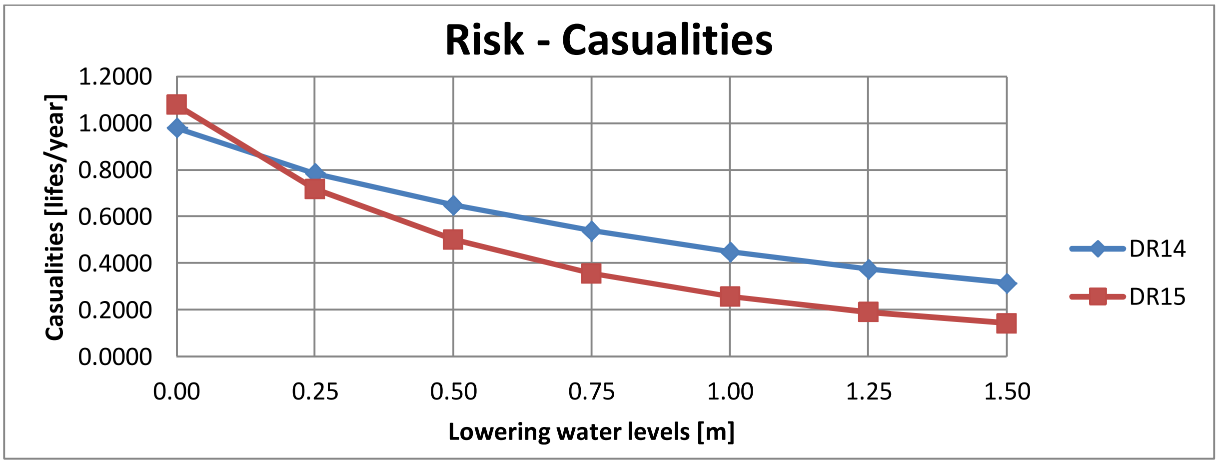


Figure 29 - Reduction of the risks due to a decrease of the governing water, casualties

Both dike rings show a decrease in the economic damage and casualties per year. The effect on the total risks of dike ring 14 is however larger because the risk contribution of the levees along the Hollandsche IJssel is a major part of the total risk. For dike ring 14 stretch one is governing. For dike ring 15 stretch 3 and 4 have nearly the same influence.

4.1.3 Aspects influencing the (decrease of the) governing water levels

The governing water levels on the Hollandsche IJssel are influenced by a lot of different aspects; therefore the decrease of the governing water levels is difficult. The aspects that influence the governing water levels on the Hollandsche IJssel are described in this section, shown in Figure 30 and summarized in the resume given below;

- Hydrological boundary conditions that affect the governing water levels during an extreme event;
    - Storm surge
    - River discharge
    - Sea level rise
    - Tidal elevation
- Influence of storm surge barriers in the system
    - Closure level and time
    - Non-closure event

Other aspects that influence the governing water levels, like morphology, shipping and ecology for example are not treated because these aspects do not influence the governing situation during an extreme situation. These aspects are however important during normal circumstance which is also the case for salt intrusion, which is treated in sections 4.2 and 4.3.

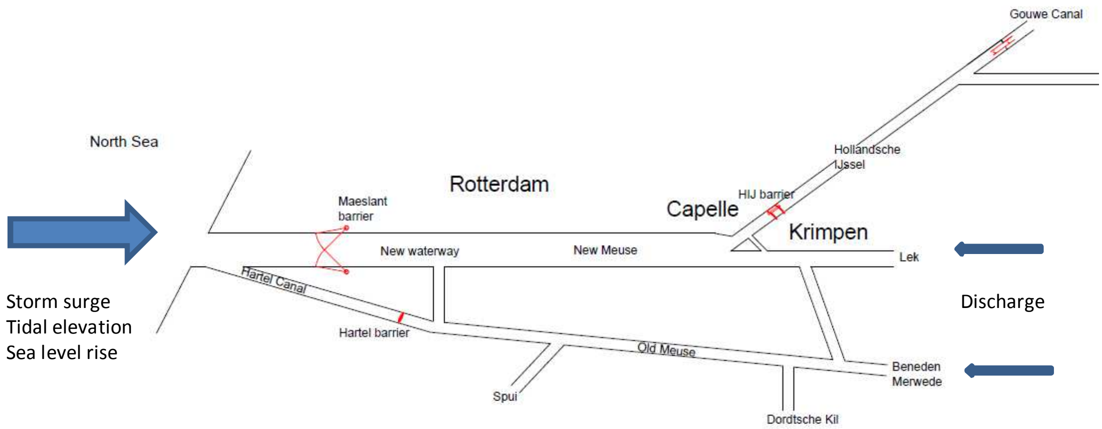


Figure 30 - Overview of the Rijnmond system and aspects influencing the governing water levels

Hydrological boundary conditions

The water levels at the mouth of the Hollandsche IJssel are determined by four independent parameters; discharge of the Rhine at Lobith (1), storm surge at sea (2), sea level rise (3) and the tidal elevation (4). Both the discharge and storm surge are independent and have their own Weibull distribution, therefore the joint distribution function of the two variables is constructed. An extreme distribution is a distribution which is used to estimate the occurrence of situations with a very low probability (extreme situations). The tidal elevation and sea level rise do not have an extreme distribution and are therefore added to the results of the combined extreme distribution.

The extreme distribution of the discharge at Lobith is calculated using discharge data obtained from the website of Rijkswaterstaat [34] (described in appendix F.3) (1). The extreme distribution of the storm surge is calculated using storm surge data obtained from the TU Delft (described in appendix F.4) (2) [35]. The water levels in the Hollandsche IJssel are estimated using the formula given below [36];

$$h_{basin} = h_{sea} + \left(\frac{Q}{\mu A}\right)^2 * \frac{1}{2g}$$

For which Q is the extreme distribution of the discharge, h_{sea} is the extreme distribution of the storm surge, μA is a parameter for the outflow, g is the gravity acceleration and h_{basin} is the water level in the Hollandsche IJssel. The given formula that describes the water levels in the basin is used in a Monte Carlo simulation to obtain the distribution of h_{basin}. The Monte Carlo simulation is a simulation that makes use of random sampling to obtain numerical results. The entire calculation of the joint distribution function for h_{basin} is presented in appendix F.5.

The sea level rise obtained from different studies is shown in appendix F.1 and added after analysis of the water levels (3). The tidal elevation in the Hollandsche IJssel is measured by the department of Public Works who maintains a record (described in appendix F.2) [37]. The tidal elevation in a river system can change when the lay-out of the river changes (4). The lay-out of the Hollandsche IJssel is fixed (levees on both sides) and therefore the tidal elevation will not change significantly. The average tidal difference is used because the extreme distributions (storm surge and discharge) used to calculate the high water levels at the mouth predict water levels with an exceedance probability of 1/10 000 per year. It is not sure which tidal maximum (spring, neap, average) happens at the same time as the high water levels. It is however certain that during high water at least one tidal maximum occurs because the period of high water is longer than the rise of the tide. The water levels in Figure 31 are shown for the different sea level rises.

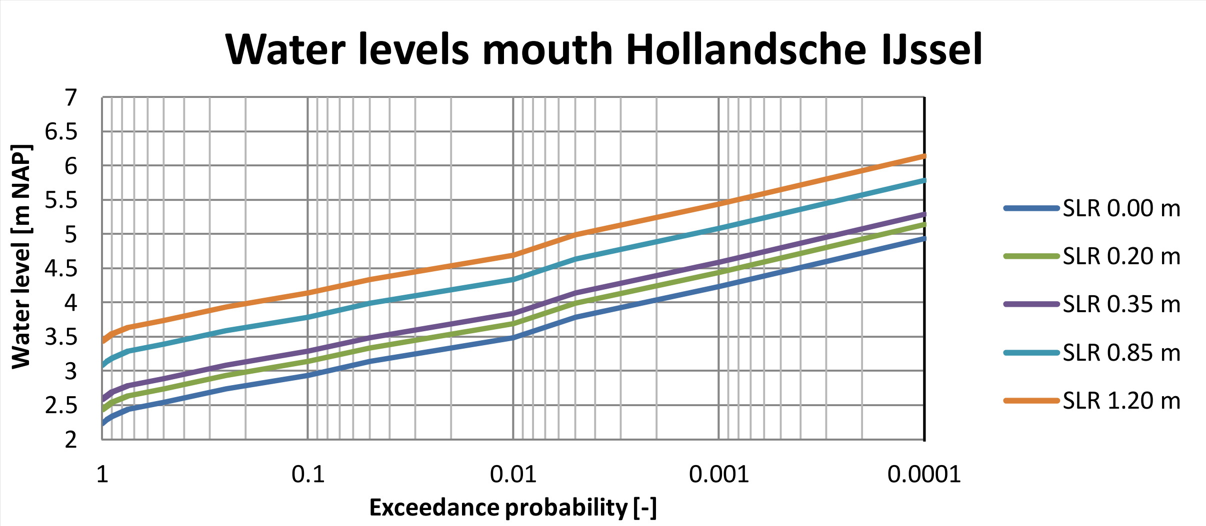


Figure 31 - Water levels Hollandsche IJssel, without influence of the storm surge barriers

Influence of the storm surge barriers in the system

The governing water levels in the lower parts of the rivers are predominantly affected by the tide and storm surge levels at sea, high discharges from the rivers do not often result in high water in the lower lying areas near the coast. Therefore the closure scheme of the Maeslant storm surge barrier is an important factor determining the water levels on the New Meuse and near the Hollandsche IJssel storm surge barrier (shown in Figure 30); the closed barriers ensure that the storm surge does not enter the river system.

<u>Closure level Maeslant storm surge barrier</u>
The Maeslant barrier closes when the expected water levels from sea are too high; closure level of the Maeslant barrier is consequently the important factor determining the governing water levels behind the barrier. The closure level of the barrier could be increased when the sea level rise causes too many closures or the closure level could decrease to increase the reliability of the levees. A study conducted by Witteveen+Bos showed that the limited increase of the closure level is acceptable. The increase of the governing water levels behind the storm surge barrier is low when the closure level is increased with 0.2 m [38].

The number of closures in a year should be limited to a maximum of 1 - 2 according to the port of Rotterdam [39, 40]. When the barrier closes more than 2 times a year it is expected that the economic damage and especially the loss of reputation will threaten the competitiveness of the harbor. The barrier may therefore close once a year due to a storm surge and once a year for the yearly maintenance and testing. The closure level that is needed to maintain the once a year closure is shown in Table 23. These values are obtained with the use of Figure 95, in this figure the water levels near Hook of Holland are calculated for different sea level rises.

Table 23 - Closure level Maeslant barrier

Sea level rise	Climate study	Closure level*
0.00 m	Current situation	+2.60 m NAP
0.20 m	Current increase	+2.80 m NAP
0.35 m	KNMI 2050 W+	+2.95 m NAP
0.50 m	-	+3.10 m NAP
0.85 m	KNMI 2100 W+	+3.45 m NAP
1.00 m	-	+3.60 m NAP
1.20 m	IPCC 2100	+3.80 m NAP

** This is the closure level needed to maintain a closure of once a year given the sea level rise*

According to Table 23 (and appendix F.6) it is not needed to change the closure level of the Maeslant barrier because the current closure level of the Maeslant barrier (+3.00 m NAP) will not result in a once a year closure until more than 0.35 m sea level rise is reached.

Closure Maeslant storm surge barrier

The governing situation on the New Meuse and Hollandsche IJssel is the combination of storm surge and high discharge (shown in Figure 30). The governing exceedance probability in the system is 1/10 000 per year, the combination of the two aspects (storm surge from sea $P_{storm\ surge}$ and discharge from the hinterland $P_{discharge}$) should result in the governing exceedance probability P_{norm}, which is 1/10 000 for the storm surge barriers in the system.

The influence of the storm surge is however limited, when the Maeslant barrier is closed the rise of the water levels on sea does not affect the water levels inside. When the Maeslant storm surge barrier is closed the water levels on the New Meuse and Hollandsche IJssel only increase due to the water discharged through the Lek and Beneden Merwede. The discharged water accumulates behind the barrier because the barrier is closed. The water levels on the Hollandsche IJssel and New Meuse are therefore equal to the water level just after closure of the barrier plus the effect of the discharge.

The water level just after closure is always the same because the Maeslant barrier closes during the ebb slack period preceding the high water that reaches +3.00 m NAP. When the water level after closure is always the same the maximum discharge is important, this creates the rise of the water levels on the river. The maximum discharge that can occur during a closure should be maximized because this results in the largest increase of water level on the river. The governing situation occurs when the barrier has just closed (when the closure level +3.00 m NAP is predicted) because $P_{storm\ surge}$ is minimal and $P_{discharge}$ increases to reach the governing exceedance probability P_{norm}. When the governing discharge is known, the governing water levels on the Hollandsche IJssel are calculated. The total calculation of the exceedance probability and governing water levels is conducted in appendix F.5-7 the results are presented in Table 24.

Table 24 - Governing water levels on the New Meuse

Sea level rise*	Exceedance probability of the closure level	Exceedance probability governing discharge	Governing discharge [m^3/s]	$h_{governing}$ [m NAP]
0.00	1/9	1/12	8 100	+3.50
0.10	1/7	1/16	8 500	+3.58
0.20	1/5	1/22	9 000	+3.65
0.35	1/3	1/37	9 700	+3.82

*sea level rise higher than 0.35 is not treated because then the closure level of +3.00 m NAP should change

When the exceedance probability of the closure level increases, the Maeslant storm surge barrier experiences more closures per year. When the number of closures increases the norm which should be met (P_{norm}) decreases. To maintain the norm it is necessary to decrease the exceedance probability $P_{discharge}$ and therefore increase the governing discharge should.

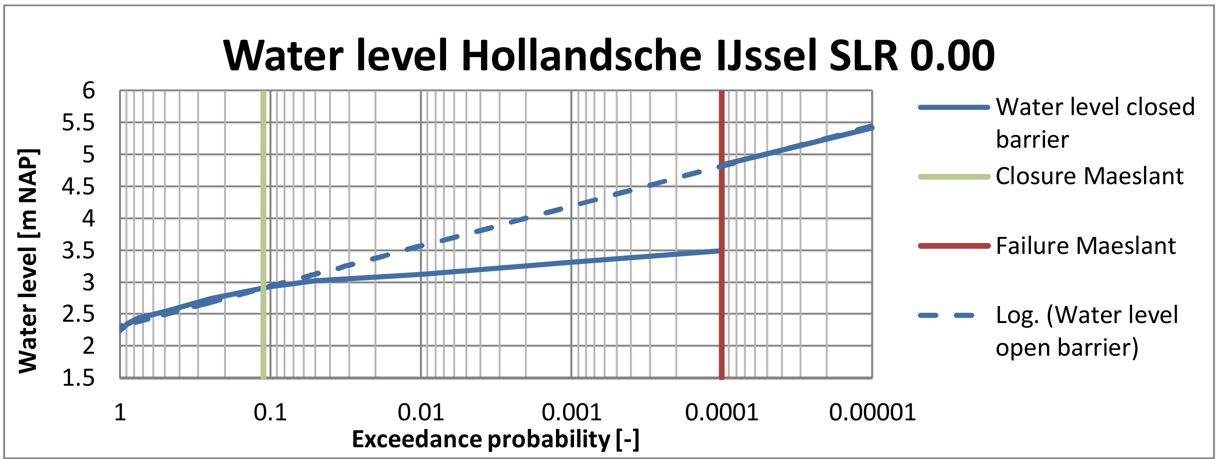


Figure 32 - Water levels at the mouth of the HIJ, with the influence of storm surge barriers

Figure 32 shows the governing water levels at the mouth of the Hollandsche IJssel. When the Maeslant barrier closes (green line) the governing water levels inside the basin do not follow the water levels of the open barrier. When the Maeslant barrier fails (red line) the high water levels of the open barrier are restored in the

Hollandsche IJssel. In between these two events the governing water levels are dependent of the discharge. The design situation in the Hollandsche IJssel is reached just before the Maeslant and Hollandsche IJssel barrier theoretically fail.

Non-closure event of the Hollandsche IJssel storm surge barrier

The non-closure probability is the probability that the storm surge barrier does not close when the barrier should close. Non-closure happens when the lift gate is jammed or when the storm surge barrier does not close due to a human error for example. The non-closure probability is important because this probability affects the water levels on the Hollandsche IJssel. When the non-closure probability is too high the decrease of the water levels is counteracted by the effect of the non-closure probability.

The effect of the non-closure probability is calculated for the situation directly behind the storm surge barrier and the effect on the governing water levels along the Hollandsche IJssel. The effect of the non-closure probability directly behind the barrier depends on the closure level of the barrier and the governing water level that would be introduced when the barrier failed. The effect of the non-closure probability near Gouda depends on the wind set-up that increases due to the open connection with the New Meuse. The increase directly behind the storm surge barrier gives a general idea of the magnitude in the entire Hollandsche IJssel and is described in this section; the increase along the Hollandsche IJssel and near Gouda is part of the water balance which is described in section 4.3.2.

The effect of the non-closure probability (P_{ncl}) needs to be calculated for the design conditions in front of the storm surge barrier and for the water levels on the Hollandsche IJssel. The governing water level ($h_{governing}$) on the Hollandsche IJssel has to be increased by the effect of high water in the river when the barrier is not closed. In formula form this is given as:

$$h_{governing} = n * P_{ncl} * h_{open} + (1 - n * P_{ncl}) * h_{closed}$$

There are two parts in the formula, the part $1\text{-}n*P_{ncl}$ which is the closed part and the $n*P_{ncl}$ part which is the non-closure part. The probability that the storm surge barrier does not close is the non-closure probability per event multiplied with the number of closures in a year ($n*P_{ncl}$). When the storm surge barrier is closed the water level behind the barrier is h_{closed}, when the barrier is open the water level behind the barrier is h_{open}. The non-closure probability is given as the probability per event, the non-closure probability per year is therefore the non-closure probability multiplied with the number of closures n. The total probability of $1\text{-}n*P_{ncl} + n*P_{ncl}$ should be one because the total probability should always be one. In the calculation of the governing water levels the following assumptions are made:

- The Hartel barrier has no influence.
- The Maeslant barrier is closed during governing conditions; during closure discharge from the Rhine is governing for water levels on the New Meuse.

Table 60 shows an example calculation for the situation that the storm surge closes once a year. This means that the non-closure probability per event is the non-closure probability per year. The effect of the non-closure probability is given as the governing water level minus the water level that occurs when the barrier is closed.

Table 25 - Example calculation normative water level

	Non-closure probability (P_{ncl})	Water level open barrier (h_{open})	Water level closed barrier (h_{closed})	Governing water level ($h_{governing}$)
Example 1	1/10	+5.0 m NAP	+2.0 m NAP	+2.3 m NAP

$$h_{governing} = 1 * 0.1 * 5.0 + (1 - 1 * 0.1) * 2.0 = +2.3\ m\ NAP$$

Figure 33 shows the effect of the non-closure probability at the mouth of the Hollandsche IJssel for the reduced closure level of 1.75 m NAP. The effect of the non-closure probability changes two times. The first change occurs with an exceedance probability of 1/9 per year this is due to closure of the Maeslant barrier (from then on the increase depends on the discharge). The second change occurs with an exceedance probability of 1/10 000 per year and is due to the fact that the Maeslant and Hollandsche IJssel storm surge barrier theoretically fail.

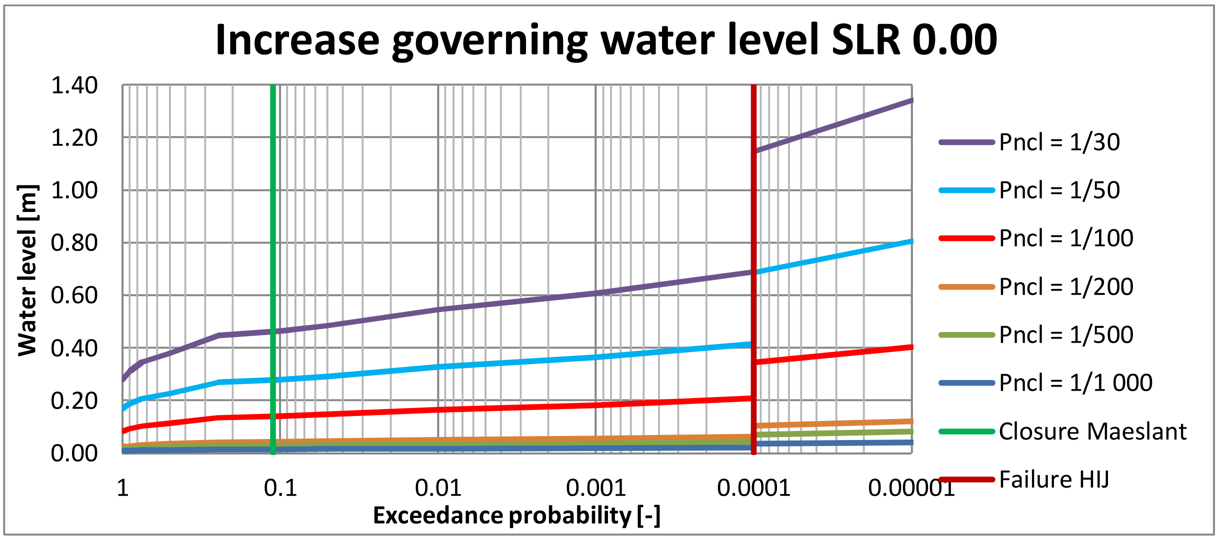


Figure 33 - Increase of the water levels behind the barrier due to non-closure, closure level 1.75 m NAP

4.1.4 Conclusion; decrease governing water levels

The conclusion of this section is divided in two parts. In the first part the objectives mentioned at the start of this section are discussed. In the second part the results of the aspects that influence the water levels are given.

Objectives to substantiate the adaptation of the barrier

It is possible to reduce the risks and costs for the reinforcements when the governing water levels on the Hollandsche IJssel are reduced there are however certain drawbacks. Most of the reinforcements are not prevented because the steep inner slope of the levees limits the effectiveness of the decrease. When the decrease of the governing water levels is combined with the smart reinforcement of the levees it is possible to reduce the costs. The safety of dike ring 14 and 15 increases considerably when the governing water levels are decreased, especially the safety in dike ring 14 increases because the contribution of the levees along the Hollandsche IJssel is large.

The considerable increase of the overall safety compared to the minimal decrease of reinforcement costs when only the water levels are decreased is caused by the low actual strength of the levees (the estimated failure probability of the levees is 1/100) [31]. The decrease of the failure probability to 1/200 decreases the risks with a factor 2 but the assessment of the levee section would still result in a levee that is not up to the standards because 1/10 000 should be reached.

The objectives mentioned at the beginning of this section are feasible when a reduction between 0.5 and 1.0 meter is reached;

- The risks due to the levees along the Hollandsche IJssel are reduced with 40-60% (data from Figure 28 and Figure 29),
- Ten too twenty percent of the smaller reinforcements is prevented (failure class 1.1 and 2 shown in Table 22),
- The larger reinforcements are not prevented but the reduction of the water levels in combination with other aspects should make it possible to achieve a 35% reduction [11].

Aspects influencing the decrease

The study of the hydrological boundary conditions showed that the Hollandsche IJssel is a complicated system, which is affected by a lot of different aspects. The sea level rise ensures that the governing water levels and the number of closures of the Maeslant barrier increase. The current closure level of the Maeslant barrier (+3.00 m NAP) is maintained until 0.35 meter rise has occurred, when the sea level rise continues the closure level should be raised. This raise is needed to maintain the economic position of the harbor of Rotterdam according to the harbor of Rotterdam.

The relative increase of the governing water levels just behind the Hollandsche IJssel is high when the non-closure probability is not decreased. This probability should preferably be lower than 1/500 because the increase is then limited to 0.04 meters in the governing situation (shown in Figure 33). In between closure and failure of the Maeslant barrier the contribution is limited because the governing water levels on the New Meuse only increase due to accumulated water behind the Maeslant barrier (discharge).

The exact effects of the sea level rise and reduction of the governing water levels are presented at the end of this chapter because salt intrusion (section 4.2) and water balance (section 4.3) also affect the system.

4.2 Use of the storm surge barrier during salt intrusion

Flushing of the canal system in Central Holland, as mentioned in section 2.5, is essential for the agriculture. Therefore continuity of the inlet needs to be guaranteed. Especially during droughts salt intrusion reaches further land inwards as the discharge through the rivers is low. This section treats the following aspects;

- Action that could guarantee the continuity of the inlet (section 4.2.1).
- Limitations in the use of the storm surge barrier (section 4.2.2).
- Conclusion in the use of the storm surge barrier (section 4.2.3).

In the existing situation salt intrusion is slowed in the New Meuse using the salt stair (described in G.3.1) when salt intrusion eventually reached the inlet near Gouda the measured salt concentration is too high and the inlet closes. After closure the small scale water supply provides some discharge into the system. In the last decade the inlet near Gouda closed three times, during the closure the agriculture in Central Holland experienced a lot of economic damage (8.8 million euros per drought period [13]).

4.2.1 Actions that could guarantee the continuity of the inlet

There are two types of actions that could guarantee the continuity of the inlet (shown in Figure 34). The first type focuses on the change of the source. The original source (New Meuse) is cut off and another source is used for the inlet. The second type focuses on the slowing of the salt intrusion (on the New Waterway and New Meuse). Salt intrusion will eventually reach the inlet but the time that the inlet closes is decreased. The possible actions that could guarantee the continuity of the inlet are described in Table 26.

Figure 34 - Overview type of actions, source; Google maps

Table 26 - Actions to guarantee the continuity

Action	Explanation
1: Stop of the inlet during droughts	The stop of the inlet is not possible because agriculture in Central Holland will experience a lot of economic damage. The master thesis of F. Bulsink (UT Twente) showed that the expected economic damage due to the salt intrusion would be enormous when there is no flushing in Central Holland [13]. There is no other inlet with the capacity of the Gouda inlet which can take over for a long period [16].
2: Change the inlet point or source • **Change to Lake IJssel** • **Move to canalized Hollandsche IJssel**	When the inlet changes to Lake IJssel salt intrusion of the Hollandsche IJssel is no problem. The structures that are needed to let water from Lake IJssel into the polders are however expensive, therefore the change of the inlet is only executed when there are no other options. The inlet can be moved to the canalized Hollandsche IJssel, part of the water entering Lobith then needs to be rerouted to this canal. The rerouting of water already happens on a smaller scale (KWA mentioned section 2.5) [13].
3: Slow the intrusion on the New Meuse • **Salt stair** • **Bubble screen (mobile)**	There are different options to slow the salt intrusion in the New Meuse. There is a salt stair (trapjeslijn) that is already situated in the New Meuse. The salt stair situated in the New Meuse (briefly described in appendix G.3.1) has eroded over time due to the tidal flow, the renovation of the salt stair is currently executed [41]. Due to the salt stairs the sea water is forced upwards, this introduces turbulence and therefore mixing of the fresh and salt water. A (mobile) bubble screen in the New Meuse can be used to increase the turbulence in the New Meuse and therefore mix fresh and salt water. Problem with a bubble screen is however the effectiveness which is not more than 50%. The maintenance which is needed to prevent clogging of the air vents is also a problem (described in appendix G.3.2)
4: Use of the storm surge barrier during droughts	The storm surge barrier can be used during salt intrusion to close off the Hollandsche IJssel and therefor cut off the only possibility of salt water to reach the inlet near Gouda.

The use of the salt stair and storm surge barrier is preferred to guarantee the continuity of the inlet because it uses structures that are already constructed in the system (or renovated). The salt stair delays salt intrusion in the New Meuse. When the delayed salt intrusion eventually reaches the mouth of the Hollandsche IJssel the lift gate of the storm surge barrier is lowered into the river preventing salt intrusion reaching Gouda. The limitations that prevent the use of the storm surge barrier are;

- The ecology in the Hollandsche IJssel depends on the tide, closure of the storm surge barrier means that the tide cannot enter the system of the Hollandsche IJssel. Tidal nature might not survive without the tide and oxygen levels in the river descend because there is no flow of water in the system.
- The inlet uses water from the New Meuse, when the storm surge barrier is closed this source is cut off.
- The closed storm surge barrier delays the shipping in the Hollandsche IJssel; ships need to use the lock next to the storm surge barrier.

4.2.2 Limitations in the use of the storm surge barrier

The three limitations mentioned in the preceding section are discussed in this section, when needed a solution is presented to solve or mitigate these limitations.

Ecology in the Hollandsche IJssel

The Hollandsche IJssel is characterized as a fresh tidal river. The open connection to the sea is therefore important aspect for the diverse ecology in the system. The diverse ecology in the Hollandsche IJssel is characterized with;

- The open connection that allows fish to migrate into and out of the Hollandsche IJssel.
- The diverse tidal nature in the forelands of the Hollandsche IJssel.
- The water quality of the Hollandsche IJssel that is restored by the program "Cleaning the Hollandsche IJssel".

<u>Open connection</u>
With the project "Cleaning the Hollandsche IJssel" (section 2.4.2 and 2.5) fish migrated into the Hollandsche IJssel. When the barrier is closed during long periods of low discharge fish are not able to pass the storm surge barrier. Fish pass the barrier to spawn eggs in sheltered areas along the banks of the Hollandsche IJssel. Fish should therefore be able to pass the closed barrier using a fish passage that is constructed next to the adapted storm surge barrier.

<u>Tidal nature</u>
The nature in the Hollandsche IJssel is part of the Ecological Main Structure (EHS). The tidal nature in the Hollandsche IJssel is categorized according to the categories laid down in the articles of the EHS [42, 43]. The Hollandsche IJssel is a connecting part within the system of the EHS, the tidal nature in the forelands is categorized in appendix G.2. There is one category which is directly affected by the tide, N05.01. This category is a swamp/marshy area which adapts very fast to new circumstances. Closure of the storm surge barrier during low discharge is therefore no problem for the tidal nature in the Hollandsche IJssel.

<u>Water quality</u>
The water quality of flowing water is higher than the water quality of stagnant water. Flora and fauna living in water use the available oxygen from the water to survive, when this water is not refreshed the oxygen levels decrease and flora and fauna will perish. Closure of the storm surge barrier during floods will also increase sedimentation in the Hollandsche IJssel, mud will settle in locations with low velocities where fish spawn their eggs [44]. Due to the aforementioned effects the closure of the Hollandsche IJssel storm surge barrier is limited to one month, this time period is chosen after consultation of an ecologist at Witteveen+Bos (ir. B. de Jong).

Water inlet near Gouda

When the storm surge barrier is closed the inlet near Gouda is cut off from the normal source of water. During a flood fresh water coming from the Rhine is pushed into the Hollandsche IJssel due to the tide, this results in the inlet of water near Gouda. When the storm surge barrier is closed this source of fresh water is not available, therefore other sources of water should be used. The two only sources of water that are available (without the change of location) during the closure of the HIJ barrier are the storage of water on the Hollandsche IJssel and the water discharged through the canalized Hollandsche IJssel (shown in Figure 35). Closure of the storm surge barrier during the flood slack period increases the amount of water stored in the Hollandsche IJssel. This water ensures that the inlet can continue for two days (described in appendix B3), after that the canalized Hollandsche IJssel and small scale water supply should be used. The use of the storage on the Hollandsche IJssel is needed because the discharge trough the canalized Hollandsche IJssel (and small scale water supply) should be increased. The rerouting of water to the canalized Hollandsche IJssel takes time.

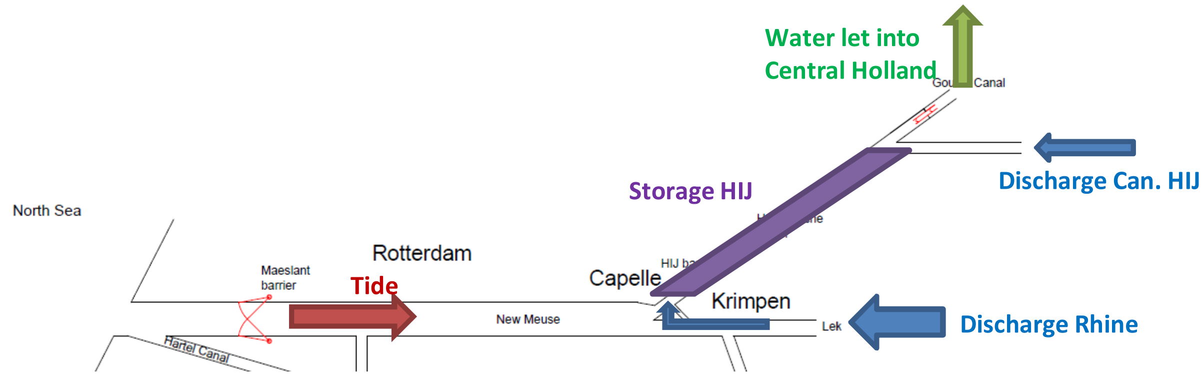


Figure 35 - Overview source of water into and out of the Hollandsche IJssel

The use of the small scale water supply (KWA) is not sufficient for the need of water into Central Holland the supply of water through the canalized Hollandsche IJssel should therefore increase to fulfill the supply that is needed [13]. The supply that is needed according to the thesis of F.Bulsink (UT Twente) is 24 m^3/s. The small scale supply accommodates 10 m^3/s which means that the canalized Hollandsche IJssel should supply 14 m^3/s; this is possible because the maximum discharge through the canalized Hollandsche IJssel is 21.5 m^3/s (shown in appendix H.3).

Shipping in the Hollandsche IJssel

Shipping on the Hollandsche IJssel is delayed when the storm surge barrier is closed. Given the current return periods of salt intrusion events (shown in appendix G.5 and G.6) it is not expected that closure happens every year (the inlet needed to stop three times in the last decade). When the closure of the storm surge barrier due to salt intrusion remains limited, ships can use the lock situated next to the storm surge barrier.

4.2.3 Use of the storm surge barrier during design salt intrusion

The closure of the storm surge barrier during salt intrusion is possible when the closure of the gate can be limited to one month. The total solution and operation to salt intrusion is described using the design salt intrusion.

Total solution salt intrusion

In appendix G.5 the salt intrusion periods that occurred during the last decades have been analyzed, the longest period of salt intrusion occurred in 1990, during this period there were 60 salt days [41]. A salt day is a day in which the measured salinity near Krimpen is higher than 250 mg/l. Table 27 described the action that are taken to use the Hollandsche IJssel storm surge barrier and limit the number of salt days (and therefore the closure period of the storm barrier) to 30 days (one month).

Table 27 - Actions to maintain the inlet near Gouda

Action	When	Description	Effect/ *consequence*
Action 0	Before adaptation of the storm surge barrier.	• Renovation of the salt stair in the New Meuse.	Slows salt intrusion on the New Meuse and therefore reduced the number of salt days (it lasts longer to reach the Hollandsche IJssel). *According to Deltares this action reduces the salt days to 30* [41].
Action 1	During adaptation of the storm surge barrier.	• Adaptation of the storm surge barrier • Construction of a fish passage • Optimization of the canalized Hollandsche IJssel and small scale water supply	When the storm surge barrier is adapted to withstand the new load combination the barrier can close when salt intrusion reached the mouth of the Hollandsche IJssel. A fish passage is to let fish pass the barrier during closure. The optimization of the KWA and canalized HIJ increases the possible discharge through the canals.

			This action does not reduce the number of salt days but ensures that the barrier can be closed for a month.
Action 2	After exceedance of the closure period.	• Construction of a (mobile) structure (bubble screen or an alternative)	A structure in the New Meuse is needed to slow the salt intrusion and maintain the closure period of one month. *This action reduces the number of salt days; it is not sure what the exact reduction is. The effectiveness of a bubble screen is not more than 50%* [24].
Action 3	After the closure period is exceeded again.	• Relocation of the inlet to Lake IJssel	The inlet near Gouda is abandoned and structures are constructed near Lake IJssel to let water in from Lake IJssel. *This action guarantees the inlet of water as long as the discharge to Lake IJssel is guaranteed. Due to the high investment costs (new structures) this action is prevented as long as possible.*

Functioning system during salt intrusion

In the current situation the inlet stops and the small scale water supply provides a small part of the needed water to flush the system. After completion of action 0 and 1 (adaptation of the existing storm surge barrier) salt intrusion is slowed in the New Meuse using the renovated salt stair. When salt intrusion reaches the barrier, the barrier closes. After closure of the storm surge barrier the storage in the Hollandsche IJssel is used for 2 days (described in appendix G.4), during that time water is rerouted to the canalized Hollandsche IJssel and KWA to supply the inlet. When the salt concentration at the storm surge barrier becomes lower than 250 mg/l or when the closure lasts longer than a month the Hollandsche IJssel barrier opens.

After completion of action 2 a bubble screen is used during period where salt intrusion is expected. This bubble screen will not function during normal circumstances because the operation of a bubble screen is rather expensive and needs a lot of maintenance. Action 3 is not described.

4.2.4 Conclusion; salt intrusion

Actions 0 and 1 are executed as part of the adaptation of the Hollandsche IJssel storm surge barrier and are needed to reduce the number of salt days or make it possible to use the storm surge barrier. Action 2 is not needed to prevent the design salt intrusion but might be necessary when the salt intrusion periods last longer due to climate change. According to the different KNMI studies the average monthly discharge will decreases during the summer months and increases during the winter (shown in Figure 36). When this occurs action 2 should be necessary.

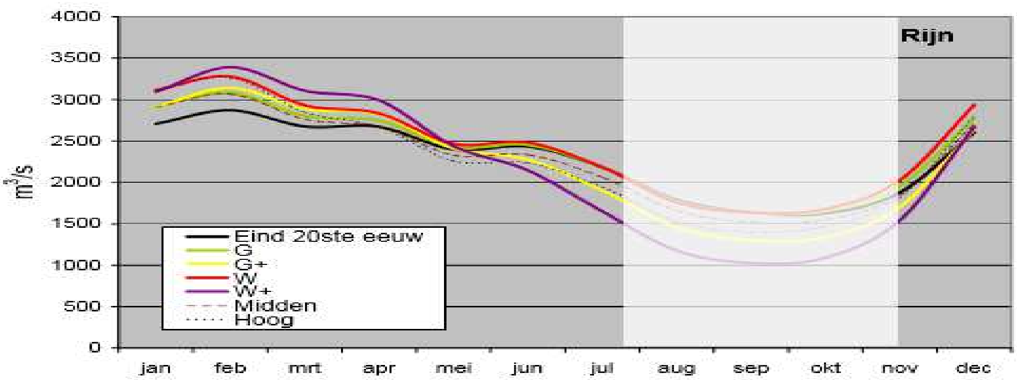


Figure 36 - Average monthly discharge, source; KNMI

4.3 Water balance of the Hollandsche IJssel

The water balance of the Hollandsche IJssel changes when the closure scheme of the Hollandsche IJssel is changed due to salt intrusion and decrease of the governing water levels. Table 28 shows the two different closures and the different factors that influence the water balance. Figure 37 and Figure 40 sketch the different discharges into and out of the Hollandsche IJssel during a storm surge and salt intrusion.

Table 28 - Factors affecting the water balance in the Hollandsche IJssel

Influence factor	Closure storm surge (section 4.3.1)	Closure salt intrusion (section 4.3.2)
Precipitation	X	
Discharge pumping stations (DR14 and DR15)	X	
Overtopping	X	
Pump stop level	X	
Non-closure probability	X	
Inlet stop level/ shipping		X
Inlet flushing		X
Discharge canalized Hollandsche IJssel		X
Closure level	X	X

4.3.1 Closure and water balance during a storm surge

The water balance during a storm surge is predomintantly affected by aspects directly related to the exteme conditions and the assessment of the storm surge barrier. Due to the extreme conditions there is precipitation directly into the Hollandsche IJssel and discharge of pumping stations due to precipitation in the surrounding polders. When the water levels in the Hollandsche IJssel become too high the pumping stations need to stop discharging water on the Hollandsche IJssel. The storm surge barrier influences the water balance because the non-closure probability, closure level and height of the gate increase the water level on the Hollandsche IJssel during a storm surge.

The inlet near Gouda is not used during a storm surge because precipitation in the surrounding area will provide enough fresh water to prevent salt water in the canal system of Central Holland, therefore the inlet stop level, inlet flushing and the discharge of the canalized Hollandsche IJssel are not important for the water balance during a storm surge. Ships use the locks during a storm surge but do not influence the water level on the Hollandsche IJssel.

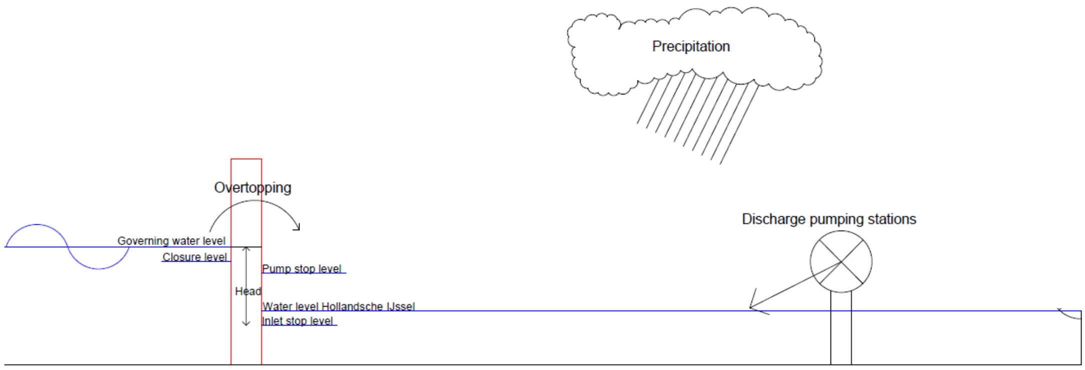


Figure 37 - Water balance Hollandsche IJssel during storm surge

Precipitation

Due to climate the change the intensity and duration of precipitation increases [45]. Precipitation directly on the Hollandsche IJssel increases the water level. In appendix H.1 the precipitation during a 12 hour storm is calculated using data from the KNMI W+ scenarios. Result of this calculation is that the precipitation of a once in ten thousand year storm is 70 mm/12 hours. The twelve hour storm is used because the closure of the Hollandsche IJssel barriers lasts 12 hours.

Discharge pumping stations

The pumping capacity (including expected increase of the capacity) of the pumping stations increases the water level on the Hollandsche IJssel with 0.14 m every hour (described in appendix H.3).

Overtopping

During design conditions the overtopping discharge due to wave action over the Hollandsche IJssel barrier is 10 l/s/m, this means that the water levels on the Hollandsche IJssel will increase with approximately 0.01 meter during the total closure time of the barrier (described in appendix H.2).

Closure level storm surge

The existing closure level of the storm surge barrier is +2.25 m NAP. The current average high tidal level is +1.36 m NAP and the spring tidal level is 1.46 m NAP. The sea level rise that is expected at the end of the design lifetime of the storm surge barrier is between 0.10 (normal increase) and 0.35 (KNMI W+ and IPCC) meter. Figure 38 shows that the new closure level and the spring tidal elevation lay close to each other when the sea level raises fast.

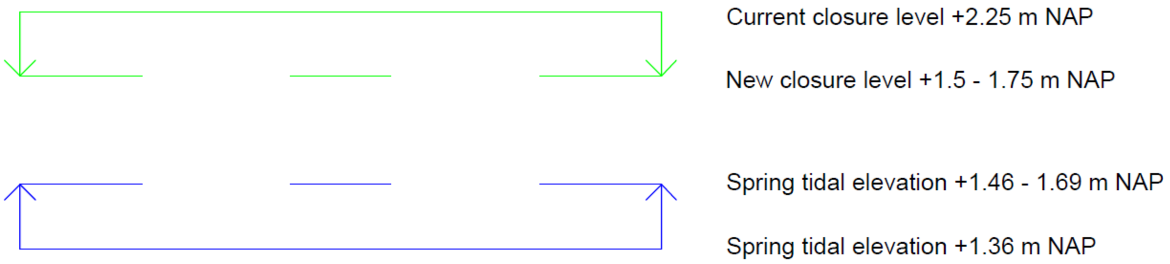


Figure 38 - Increase of the spring tidal level due to the sea level rise

The closure level should not lie to close to the spring tidal level because this would result in many closures when the discharges are slightly higher than the average discharges on which the tidal levels of appendix F.2 are based. As the average tidal level and spring tidal level lie close to each other (difference of 0.12 meters) there are often water levels closes to the spring tidal level. The new closure level of +1.75 m NAP should limit the number of closures. Closure (because of a storm surge) in the new situation will happen during the ebb slack period because:

- The discharge during the ebb slack period is nearly zero.
- The ebb slack period is longer than the flood slack period (described in section 2.5.3).
- The storage for water in the Hollandsche IJssel is maximized (precipitation, pumping station discharge and overtopping).
- Closing during the ebb slack period will not cause sedimentation of the Hollandsche IJssel (described in section 2.5.3).

Pump stop level

The pump stop level is the water level at which the pumping stations should stop with the discharge of water onto the Hollandsche IJssel; this level is measured at the Hollandsche IJssel storm surge barrier. The pump stop level of the pump stations along the Hollandsche IJssel should be nearly the same as the governing water levels. A higher pump stop level will reduce the decrease of the governing water levels. A lower pump stop level decreases the possible storage for;

- Precipitation in the Hollandsche IJssel,
- Discharge of the pumping stations,
- Overtopping over the storm surge barrier.

The total storage on the Hollandsche IJssel should at least equal the contribution of the three aspects mentioned. Therefore the storage is;

$$Storage = Precipitation + Discharge\ pumping\ stations + Overtopping$$
$$Storage = 0.07 + 0.14 * 12h + 0.01 = 1.76\ m$$

The pump stop level should be equal to the water level just after closure plus the minimum storage that is needed. The storm surge barrier closes in the ebb slack period preceding the high water; therefore the water level just before closure is the closure level minus the average tidal elevation.

$$Pump\ stop\ level = (closure\ level - average\ tidal\ elevation) + storage$$
$$Pump\ stop\ level = (1.75 - 1.51) + 1.76 = 2.00\ m\ NAP$$

The current pump stop level on the Hollandsche IJssel is +2.60 m NAP, this water level has not been reached in the last decades [46]. With the new pump stop level of +2.00 m NAP a reduction of 0.60 meter is obtained without the effect of the non-closure probability.

Non-closure probability

The non-closure probability just behind the storm surge barrier is only affected by the introduced high water levels. The water levels at the end of the Hollandsche IJssel are also affected by the wind that sets up the water levels at the end of the basin. When there is a non-closure event the storm surge barrier is open and the wind can set up water from the New Meuse to the end of the Hollandsche IJssel near Gouda.

The open and closed water levels in the Hollandsche IJssel are used to calculate the effect of the non-closure probability as is done in section 4.2.2 and appendix F.8. The current water levels are obtained from the HRC 2006. In the HRC the effect of the non-closure probability was not accounted for and a lower governing wind speed was used [17].

Figure 39 shows the governing water levels (NHW) along the axis for different non-closure probability (P_{ncl}) of the barrier. The lines showing the water levels do not become lower than +2.00 m NAP because the governing water levels cannot become lower than the pump stop level which is introduced in the preceding section. When the current non-closure probability (1/30) is compared with the non-closure probability of 1/500 (or lower) it is shown that the influence of the non-closure probability on the governing water levels is quite high. The results of Figure 39 are comparable to the increase calculated in Figure 33.

The effect of the non-closure probability is calculated for the exceedance probability of 1/ 2 000 and 1/ 10 000 because the two dike rings along the Hollandsche IJssel should withstand water levels with a different exceedance probability. The results of both the calculations (presented in appendix F.8) are comparable because the wind speed and water level are not much lower in the 1/ 2 000 situation.

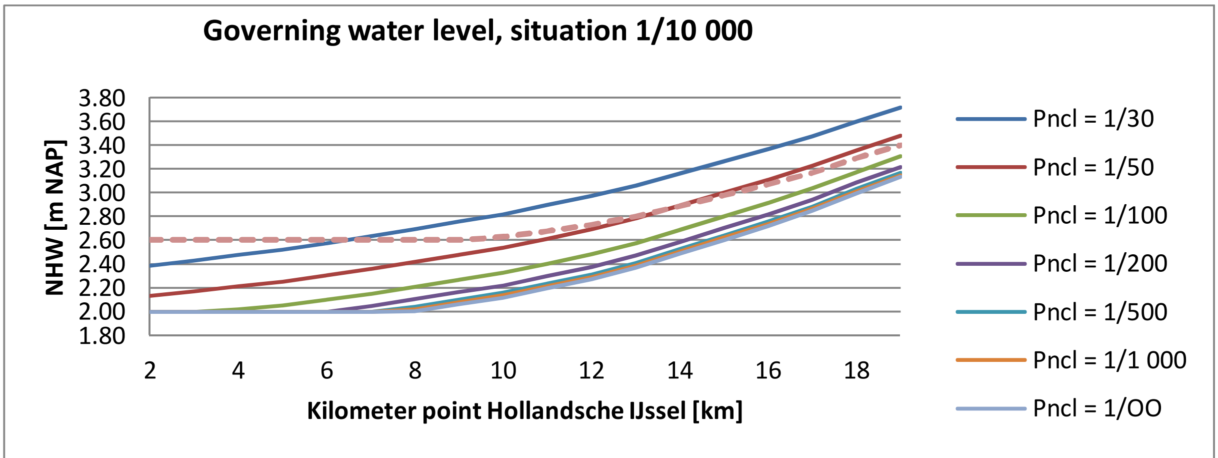


Figure 39 - Governing water levels on the Hollandsche IJssel, SLR 0.00 m, exceedance probability 1/ 10 000

Water balance

Just after closure the water levels in the Hollandsche IJssel are approximately +0.25 m NAP, after closure the water levels on the Hollandsche IJssel will slowly increase until the pump stop level of +2.00 m NAP is reached. After the pump stop the levels on the Hollandsche IJssel will not increase any further. The effect of the non-closure probability shows a large increase of the governing water levels therefore the non-closure probability of the storm surge barrier should be decreased.

4.3.2 Closure and water balance during salt intrusion

The water balance during salt intrusion is predominantly affected by the aspects directly related to the inlet of water needed for flushing. The closure level, inlet flushing and discharge of the canalized Hollandsche IJssel are aspects that influence the water balance on the Hollandsche IJssel.

Aspects related to a storm surge (overtopping, pump stop level, non-closure probability discharge pumping stations and precipitation) are not important because the system experiences low water levels.

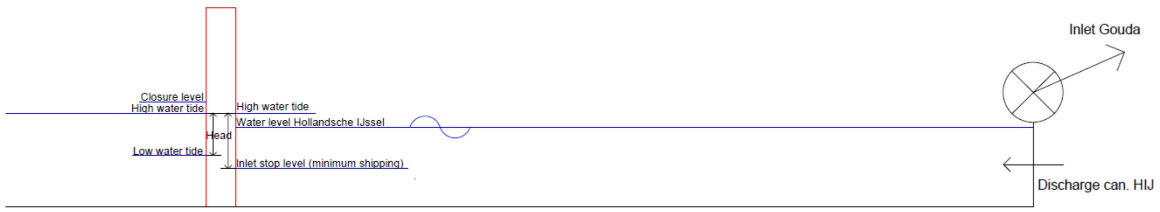


Figure 40 - Water balance Hollandsche IJssel during salt intrusion

The different factors related to the water balance during salt intrusion (presented in Table 28) are treated in the next paragraphs. The total water balance is given in het last paragraph of section 4.3.2.

Inlet stop level/shipping

The inlet stop level is the water level at which the inlet near Gouda should stop with the intake of water from the Hollandsche IJssel; this level is measured at the Hollandsche IJssel storm surge barrier. This level is related to the draft of container ships which need a minimum water depth. The inlet of water should stop when the water level near the Hollandsche IJssel becomes -0.50 m NAP. With water levels lower than -0.50 m NAP shipping is not possible on the Hollandsche IJssel [37].

Inlet and discharge canalized Hollandsche IJssel

When the storage of the Hollandsche IJssel is used the small scale water supply and canalized Hollandsche IJssel are used for the supply of water to the inlet. These aspects do not change the water balance on the Hollandsche IJssel because the inlet near Gouda and the outflow of the canalized Hollandsche IJssel are situated close to each other.

Closure level salt intrusion

The closure level due to salt intrusion is not linked to a certain water level but to the salt concentration measured near Krimpen aan de IJssel. The inlet near Gouda stops when the salt concentration becomes higher than 250 mg/l because the salt concentration of the water used for flushing should be lower than 250 mg/l [15]. The storm surge barrier will open when the salt concentration near Gouda is lower than 250 mg/l during an entire tidal cycle or when the barrier is closed longer than a month.

Closure of the storm surge barrier will happen during a high tide because the possible storage for the inlet is maximized. The side effects of the closure during a flood tidal slack period are;

- Sedimentation of the Hollandsche IJssel (described in section 3.2.4). If the salt intrusion closure is limited the sedimentation can be mitigated.
- High flow velocities during closure because there is only a small flood tidal slack period in which the storm surge barrier cannot be closed. The structural design (scour protection) should account for the occurring flow velocities during a flood slack closure.

Figure 41 - Configuration inlet Gouda, source; Google maps

Water balance salt intrusion

Just after the flood slack closure the water levels on the Hollandsche IJssel are approximately +1.00 m NAP (described in appendix G.4), after closure the water levels decrease to -0.5 m NAP (measured at the barrier) in the next two days. During the remaining closure period the water levels on the Hollandsche IJssel stay approximately -0.5 m NAP.

4.3.3 Conclusion; water balance

The total water balance for storm surge and salt intrusion is summarized in Table 29. The non-closure probability should decrease because otherwise the decrease of the governing water levels is not possible. The effect of the non-closure probability is high but diminishes relative fast when the non-closure probability is decreased.

Table 29 - Important parameters water balance Hollandsche IJssel

	Storm surge	Salt intrusion
Closure level	+1.75 m NAP	250 mg/l
Closure period	Ebb slack	Flood slack
Pump/inlet stop level	+2.00 m NAP	-0.5 m NAP
Storage	1.76 m (storage necessary)	1.51 m (storage available)

4.4 Effect of the sea level rise

In chapter three (section 3.1 and 3.4.1) it was mentioned that the choice between a dam and a new barrier on the absolute gap between the decreased closure level and the increased sea level. The choice between the two choices does not directly depend on the end of the (design) lifetime of the storm surge barrier. The Hollandsche IJssel storm surge barrier, constructed in 1958, is designed for a period of 100 years; the end of the design lifetime is therefore in 2058. This does however not mean that the storm surge barrier cannot be used after 2058. A lot of structures (storm surge barriers and dams which are part of the Delta Works and bridges from that time) reach the end of the design life time in that period. Inspection and maintenance of these structures should extend the lifetime beyond the design period. The Hollandsche IJssel storm surge barrier is assessed for the sea level rise that might occur during the coming decades.

Table 30 - Expected levels in this study

Sea level rise [m]	Spring tidal level Hollandsche IJssel [m NAP]	Closure level Hollandsche IJssel barrier [m NAP]	Closure level Maeslant barrier [m NAP]	Governing water level New Meuse [m NAP]
0.00	1.36	1.75 (2.25**)	3.00	3.50
0.10	1.46	1.75	3.00	3.60
0.20	1.56	1.75	3.00	3.70
0.35	1.71	1.75	3.00	3.85
0.65*	1.96*	2.00*	3.20*	4.10*
0.85	2.21	2.25	3.40	4.35
1.20	2.56	2.60	3.80	4.70

*Interpolation
**Current closure level

The sea level rise predominantly affects the number of closures of the Hollandsche IJssel and Maeslant storm surge barrier. To limit the number of closures the closure level should be increased and the levees should be reinforced to withstand the higher governing water levels. The reinforcements due to higher closure levels are not necessary if eventually the system is dammed and a fixed low water level is introduced. The water levels as result of the different representative sea level rises are shown in Table 30 . The spring tidal levels at the mouth of the Hollandsche IJssel are obtained using the current spring tidal water levels and adding the representative sea level rise. The new and current closure levels are described in section 4.3.1. Closure levels higher than +1.75 m NAP in Table 30 are obtained using a value slightly higher than the spring tidal level. The closure level of the Maeslant barrier and governing water level on the New Meuse are described in appendix F.6 and F.7

4.5 Conclusion; new closure scheme in combination with climate change

It is possible to decrease the water levels on the Hollandsche IJssel with 0.5 meter; a further decrease of the water levels is not possible because this would result in too many closures of the Hollandsche IJssel storm surge barrier. The reduction of half a meter is possible when the closure level of the Hollandsche IJssel barrier decreases from 2.25 m NAP to 1.75 m NAP and when the non-closure probability is decreased to a value lower than 1/500 (shown in Figure 42).

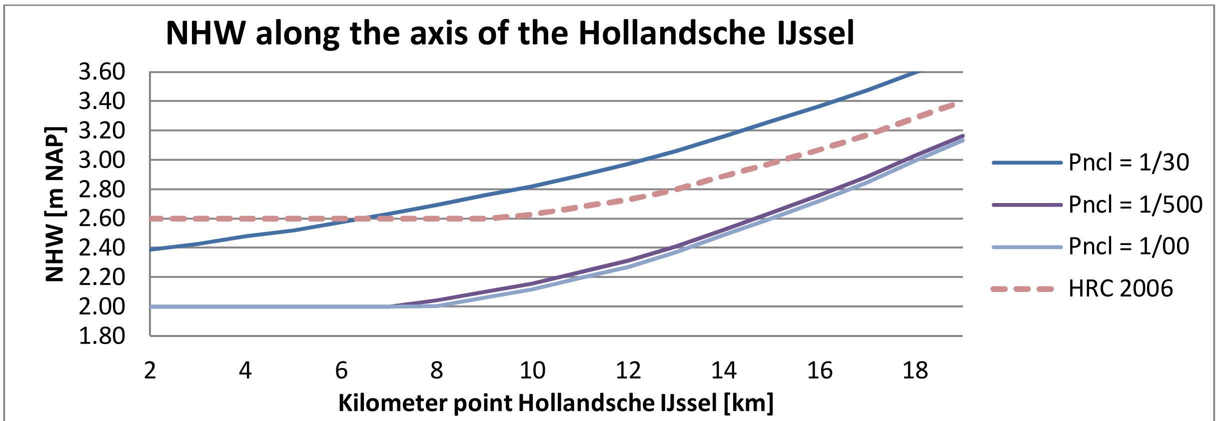


Figure 42 - Governing water levels along the Hollandsche IJssel, exceedance probability 1/ 10 000

Effects of the reduced closure level

The objectives mentioned at the beginning of this chapter were that the contribution of the Hollandsche IJssel to the total risks of dike ring 14 and 15 should be reduced with 50 % and that the reinforcement costs would need to decrease with 35%.

The graphs shown in Figure 43 and appendix E.4 show that the contribution of the Hollandsche IJssel to the risks in dike ring 14 and 15 are reduced with approximately 50% when the governing water levels are reduced with 0.5 meter. The economic damage in dike ring 14 is reduced from 24 to 11 million euros for example (shown in Figure 43). The total reduction of the risks in dike ring 14 is larger because a large part of the risks are contributed to the Hollandsche IJssel, in dike ring 15 the influence of the levees along the Lek is governing.

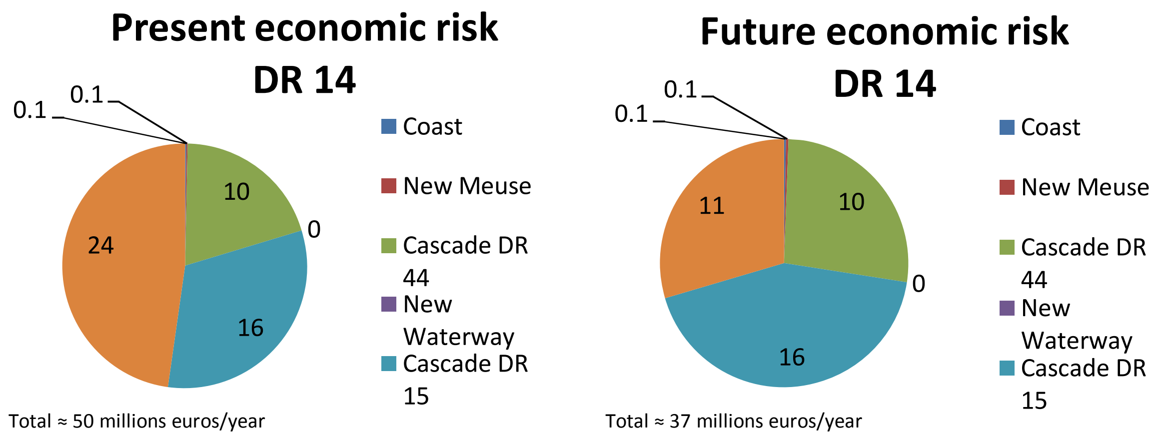


Figure 43 - Result decrease water levels [million euros/year]

The reduction of the water levels on the Hollandsche IJssel prevents 10% of the reinforcement that are necessary (failure class 2: height and a quarter of failure class 1.1). Due to the decrease the height of all the levees is sufficient (no levees have a deficit in the height larger than 0.5 meter). The objective to reach reduction in the costs 35% reduction seems possible because there is influence of the governing water levels on the reduction of the stability factor F. A decrease of the governing water levels combined with smart design to reduce the effect of the steep inner slope will decrease the reinforcement costs.

Salt intrusion

During the design salt intrusion it is possible to use the closed storm surge barrier. The closure is limited to one month because of the tidal ecology and water quality of the Hollandsche IJssel. Action 0 and 1 (described in Table 33 and shown in Figure 44) renovate the salt stair in the New Meuse and adapt the storm surge barrier (construction of a fish passage, optimization of water supply during closure and adaptations needed to withstand the new load condition) to limit the closure to one month. When, due to climate change, the drought periods last longer a bubble screen is needed to further slow salt intrusion in the New Meuse. When the drought period further increases and limitation of the closure period is not possible the inlet near Gouda should be abandoned and moved to Lake IJssel (the relocation to Lake IJssel is not studied).

Table 31 - Summary salt intrusion

Action	When	Description
Action 0	Before adaptation of the storm surge barrier.	• Renovation of the salt stair in the New Meuse.
Action 1	During adaptation of the storm surge barrier.	• Adaptation of the storm surge barrier. • Construction of a fish passage. • Optimization of the canalized Hollandsche IJssel and small scale water supply.
Action 2	After exceedance of the closure period.	• Construction of a (mobile) structure (bubble screen or an alternative).
Action 3	After the closure period is exceeded again.	• Relocation of the inlet to Lake IJssel.

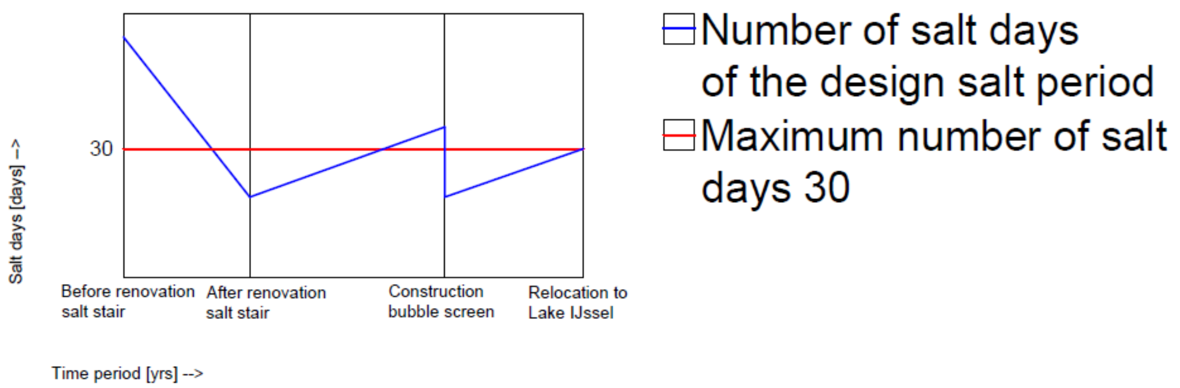


Figure 44 - Overview effect lower discharge and adaptation

In Figure 44 the qualitative increase and decrease of the salt days is given as function of the lower average discharge and actions that are taken in the system. In the current situation the number of salt days is 60 during the design salt period, due to the renovation of the salt stair the number of salt days reduces. Due to the eroding salt stair and the decreasing average discharge during the summer months the number of salt days slowly increases until the maximum number of salt days is increased again. The next step uses a bubble screen to reduce the number of salt days again. When the maximum number of salt days is exceeded once again the inlet is moved to Lake IJssel.

Water balance

The water balance in the Hollandsche IJssel changes when the closure level of the Hollandsche IJssel storm surge barrier decreases. When a storm surge occurs the storm surge barrier closes in the ebb slack period preceding the expected exceedance of the closure level +1.75 m NAP. During a storm surge and precipitation the pumping stations along the Hollandsche IJssel will discharge water on the Hollandsche IJssel, when the pump stop level is reached (at the barrier) the pumping stations stop the discharge. During salt intrusion the storm surge barrier closes in in the flood slack period preceding the expected exceedance of 250 mg/l. During the first days of the closure the storage of water on the Hollandsche IJssel will be used for the inlet of water, the inlet stop level is reached when ships cannot sail on the Hollandsche IJssel. Table 32 summarizes the water balance in the Hollandsche IJssel.

Table 32 - Summary water balance Hollandsche IJssel

	Storm surge	Salt intrusion
Closure level	+1.75 m NAP	250 mg/l
Closure period	Ebb slack	Flood slack
Pump/inlet stop level	+2.00 m NAP	-0.5 m NAP
Storage	1.76 m (storage necessary)	1.51 m (storage available)

Sea level rise

The new closure level of the Hollandsche IJssel storm surge barrier is +1.75 m NAP. The possible sea level rise before the storm surge barrier should close to much is 0.35 meter. When 0.35 meter sea level rise is reached the spring tidal level during average discharge is +1.71 m NAP (shown in Table 30). The relative sea level rise is then approximately 0.9 meter as described in section 3.1.2 (sea level rise 0.35 plus decrease closure level 0.50 meter). Given average discharge the expected number of closures would be 24 (2 spring tides per month). The new closure scheme is therefore used until 0.35 meter, after that a new structure is built (shown in Figure 45).

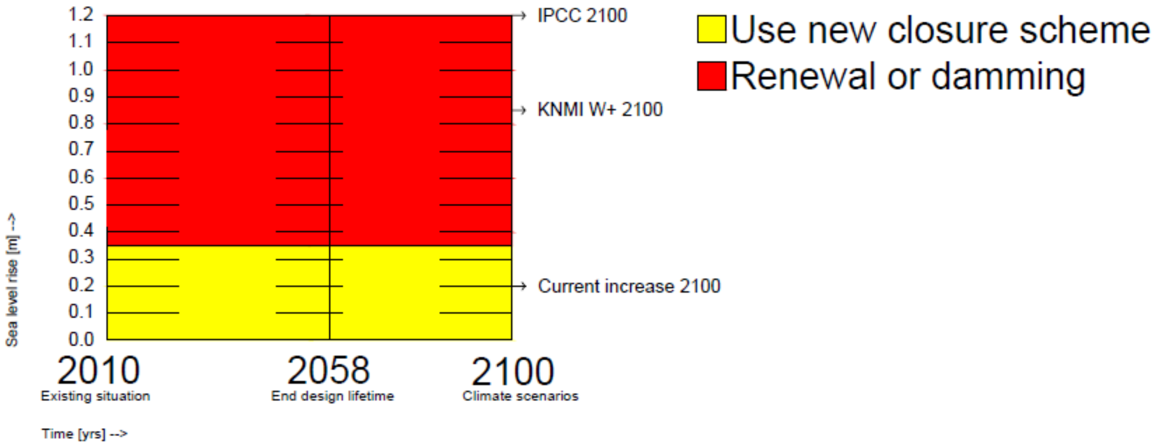


Figure 45 - Absolute sea level rise closure scheme

The possible sea level rise explained in this paragraph does not take the structural adaptation of the storm surge barrier into account. Chapter five describes the structural adaptation and assessment of the storm surge barrier. Chapter five also includes the adaptations that are needed due to salt intrusion or reduction of the non-closure probability.

5 Adaptation of the existing Hollandsche IJssel storm surge barrier

In this chapter the adaptation of the existing storm surge barrier is studied based on the results of chapter three and four. In chapter three it is concluded that adaptation of the storm surge barrier is preferred. The results of chapter four endorse the conclusions of chapter three.

The solution to the different problems described in section 3.4.2 shows which assessments or adaptations of the storm surge barrier are necessary. Adaptations directly related to the structural safety (and use during salt intrusion) of the storm surge barrier are described in chapter 5. A preliminary design of the adaptations related to the storm surge barrier is presented (fish passage, scour protection etc.). The solutions related to the surrounding of the storm surge barrier are not described in this study; this study focuses on the adaptation of the storm surge barrier.

1. Assessment of the existing storm surge barrier (section 5.1).
2. Detailed assessment and adaptation of the steel gate (section 5.2).
3. Preliminary analysis into the solutions to decrease the non-closure probability (section 5.3.1).
4. Preliminary design of the scour protection that is needed to prevent scour holes (section 5.3.2).
5. Preliminary design of the fish passage needed to pass the closed storm surge barrier (section 5.3.3).

At the end of this chapter it should be clear if the storm surge barrier can be adapted to accommodate the new closure scheme (section 5.4).

5.1 Assessment of the existing Hollandsche IJssel storm surge barrier

The existing Hollandsche IJssel storm surge barrier constructed in 1958 and adapted for the first time in 1978 (second gate) experienced different boundary conditions during the decades. The lay-out (described in section 5.1.1) and existing boundary conditions are assessed in the third nationwide safety assessment conducted between 2006 and 2011 (described in section 5.1.2) [4]. The introduction of the closure scheme and climate change introduces new load combinations; therefore the Hollandsche IJssel storm surge barrier should be assessed once again (section 5.1.3). This section concludes with the elements that should be assessed (section 5.1.4).

5.1.1 Lay-out Hollandsche IJssel storm surge barrier

The total storm surge barrier consists of three different parts; connecting parts (A and D), lock gates (B) and gates (C). The lay-out of the Hollandsche IJssel storm surge barrier is presented in Figure 46. All the elements lie within the core zone of the barrier which is laid down in the ledger of the flood defence. The core zone of the levee is the part that is primary used for the safety, the zone that lies outside the core zone is the protection zone, this zone is not directly needed for the safety but secondary effects in this area might affect the safety.

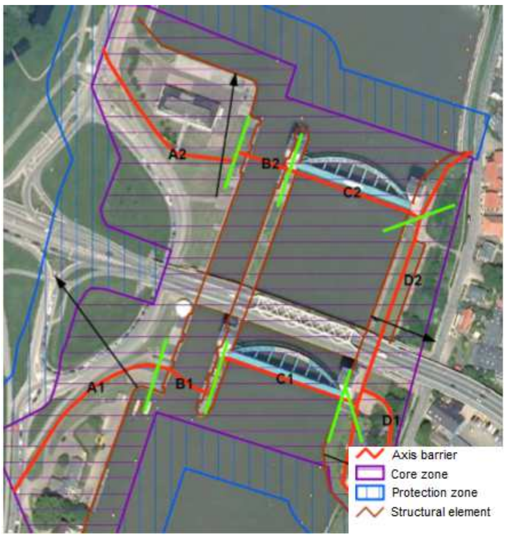


Figure 46 - Lay-out storm surge barrier, source; Rijkswaterstaat

In 1958 only the first part (..1) of the storm surge barrier was completed, the second part (..2) was completed in 1978. The first part was built as a result of the recommendations made by Delta Committee (instated because of the 1953 flood disaster) and had a design lifetime of 100 years. The Hollandsche IJssel storm surge barrier was the first of many dams and barriers to close off rivers. Due to misalignment the gate got stuck in 1978, after which the water boards demanded the construction of the second barrier which was originally intended. This barrier increases the closure reliability, when a gate got stuck the second gate could be used. When gate C1 closes the primary flood defence runs over A1, B1, C1 and D1, when gate C2 closes the primary flood defence runs over A2, B2, C2 and D2.

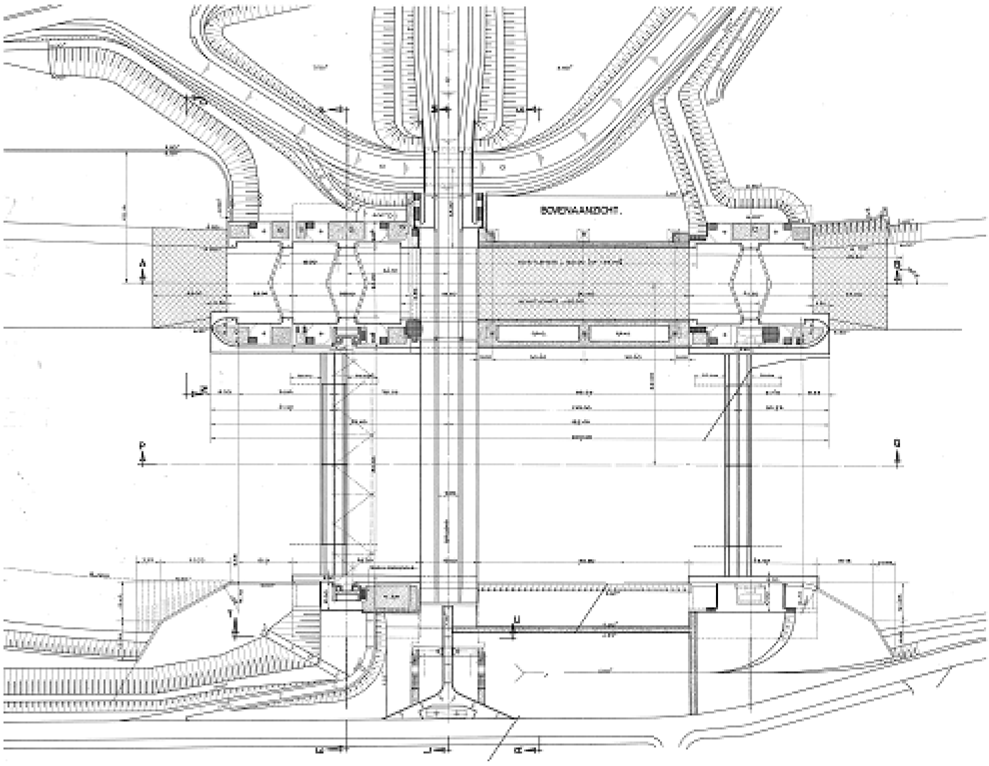


Figure 47 - Technical drawing top view Hollandsche IJssel barrier, source; Rijkswaterstaat

The connecting parts A and D connect the levees of dike ring 14 and 15 to the storm surge barrier and ensure that the system is closed off. The lock gates B1 and B2 only close during closure of the storm surge barrier, during normal lock cycles other lock gates are used. The steel gates of the storm surge barrier are shown as C1 and C2. Other structural elements in the Hollandsche IJssel storm surge barrier are not directly used for the protection against high water but are necessary to maintain the structural integrity of the storm surge barrier, the walls and sheet piling in the middle (located between the lock and the actual barrier) for example. More information about the Hollandsche IJssel storm surge barrier is shown in Figure 47 and presented in appendix J and O.

5.1.2 Third nationwide safety assessment Hollandsche IJssel storm surge barrier

In the third nationwide safety assessment the Hollandsche IJssel storm surge barrier was assessed by Witteveen+Bos [47]. In an assessment the structure or levee is assessed for the period until the next assessment (this period is currently 12 years). The plan period during design is different because the design takes more conditions into account, for hydraulic structures the plan period is normally 50 or 100 years. In an assessment not the whole structure should be calculated, a simple calculation that shows that the governing loads are lower than the design loads is often sufficient.

The result of the third nationwide safety assessment for the Hollandsche IJssel storm surge barrier is that some elements of the total storm surge barrier do not meet the standards. The elements that are not up to the standards are;

- Stability of the grass cover (STBK) is not guaranteed in part A1, D1, A2 and D2.
- Stability of the outer slope (STBU) is not guaranteed in part A2.
- The closure reliability (BS) is too low

After the assessment the problems concerning STBK and STBU were solved, the closure reliability is investigated by the department of Public Works. The structural safety (STCO) of the steel barrier and lock gates was guaranteed because the design loads were higher than the governing loads. Due to a change in closure scheme it is not certain whether the total storm surge barrier is safe therefore a new assessment is necessary.

5.1.3 Assessment structural safety Hollandsche IJssel storm surge barrier

The structural assessment of the storm surge barrier analyses the important load combinations on the steel gate and the results of the structural analysis conducted in appendix K. The design load on the total storm surge barrier takes a governing water level of +4.5 m NAP into account, the governing water levels after introduction of the new closure scheme become +3.5 m NAP [4]. High water levels are therefore not governing during design conditions but the difference (hydraulic head) between the water levels is (shown in Figure 48). Due to lower governing water levels on the Hollandsche IJssel the hydraulic head increases.

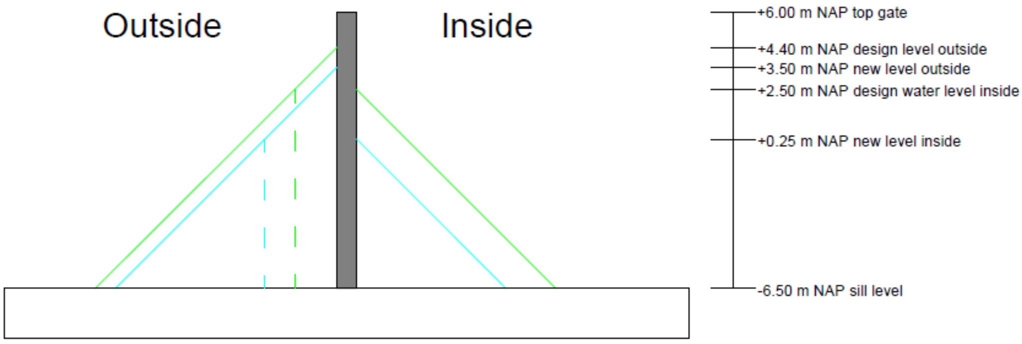


Figure 48 - Schematic overview assessment existing gate, design level (green), new level (cyan)

The structural elements which are used for the transfer of the loads due to the hydraulic head need to be assessed, other elements are not assessed. The connecting parts (A and D shown in Figure 46) are needed to withstand the governing water levels (not the hydraulic head) therefore assessment is not necessary. The lock gates (part B) are not governing because multiple gates are used; therefore the load is spread amongst the gates [47]. The gates (part C1 and C2) needs to transfer the loads generated by the difference in water levels therefore the elements that are part of the gates should be checked. The different load combinations are analyzed in the next section; the gates (part C1 and C2) are assessed in the important load combinations.

Load combinations

The change of closure scheme and water balance, described in chapter three, ensures that the hydrological boundary conditions change. The load combinations are described for the period directly after adaptation of the storm surge barrier and change of the closure scheme. The first assessment only studies the possibility to change the closure scheme, the detailed calculation for a longer plan period (including) sea level rise is executed in section 5.2. The different load combinations are briefly described in this section, the load combinations that are assessed are detailed in section 5.2.1.

1. <u>Storm surge</u>

During the design storm the Maeslant and Hollandsche IJssel storm surge barrier are closed. Governing water levels (due to the closure level and discharge from Lobith) and waves in front of the Hollandsche IJssel barrier will occur (shown in Figure 49). Due to the new closure level the governing water levels on the Hollandsche IJssel will be lower and therefore the hydraulic head higher. The storm surge barrier is assessed for the load combination storm surge because the boundary conditions change when the sea level rises.

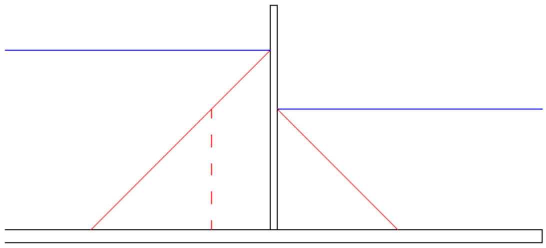


Figure 49 - Schematic overview load combination storm surge

2. Salt intrusion

During salt intrusion the Hollandsche IJssel barrier will close to prevent salt intrusion into the Hollandsche IJssel. The governing situation will occur when the water levels on the Hollandsche Ijssel are high (just after closure) and there is a low tide in front of the Hollandsche IJssel barrier (shown in Figure 50). The existing storm surge barrier is not designed for a negative hydraulic head (or changes in the head and consequently fatigue). The storm surge barrier is assessed for the load combination because this load combination is new.

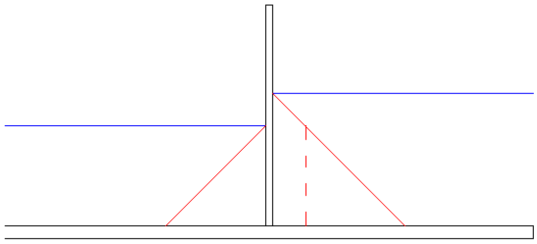


Figure 50 - Schematic overview load combination salt intrusion

3. Wind load

During normal conditions the gate is raised above the water and is exposed to a wind load acting on the steel gate. The wind pressure and fluctuating wind speeds apply (harmonic) loads to the steel gate. The storm surge barrier is not assessed for the load combination wind load because the extreme wind speeds do not change much due to climate change.

4. Up and down movement of the gate

When the gate closes or raised the gate is loaded due to the water flow at the underside of the barrier, other loads during movement of the gates are due to the steel cables that are fixed to the mechanism raising and lowering the gate. The storm surge barrier is not assessed for the load combination movement because the up and down movement of the gate does not change.

5. Transport and construction

The transport and construction phase of a structure is often determining for the profiles because other loads are applied to the structure. The storm surge barrier is not assessed for the load combination transport and construction because this does not occur.

6. Extreme events

Extreme events have a low probability to occur but could lead to large damage. Likely extreme events for the Hollandsche IJssel storm surge barrier are: failure Maeslant barrier and the collision of a ship into the gate. The storm surge barrier is not assessed for the different extreme events because the probability of these events does not change significantly due to the changing boundary conditions.

7. Closure reliability

Non-closure of the storm surge barrier is a load combination that occurs when there is a closure request but the barrier does not close. When the Hollandsche IJssel storm surge barrier does not close the water levels on the New Meuse are also introduced on the Hollandsche IJssel which consequently leads to failure of the levees because these water levels are higher than the governing water levels. The storm surge barrier is assessed for the load combination non-closure because the last nationwide assessment showed that the closure reliability was too low.

The elements within the storm surge barrier are assessed for the increased load combination storm surge and fort the new load combination salt intrusion. A preliminary analysis into the closure reliability is presented in section 5.3.1. The different elements within the storm surge barrier were designed for the same load combinations. If it is shown that one of the elements does not fulfill the changed load combinations it can be expected that also other elements within the storm surge barrier do not fulfill the load combinations. Reasons should be given why the specific elements would fulfill the requirements of the different load combinations.

Structural safety analysis storm surge and salt intrusion

The structural safety of the storm surge barrier should be assessed given the load combinations storm surge and salt intrusion. The structural safety of the storm surge barrier is assessed for the introduction of the closure scheme, it needs to be demonstrated that the storm surge barrier is able to withstand the changed boundary conditions. The different structural elements within the storm barrier are;

- Steel gate that is directly used to withstand the hydraulic load (STCO),
- Supports that transfers the forces to the towers (STCG),
- Towers that transfer the forces to the foundation (STCG),
- Pile foundation that transfers the forces to the soil (STCG),
- Sheet piles that prevent piping (STPH).

The assessment STCO is used for elements that directly withstand water on either side of the structure, the assessment STCG studies the elements that are used to transfer the different forces but does not directly retain water. The normal stability checks for storm surge barriers are not possible because these checks focus on the use of shallow foundations, the Hollandsche IJssel has as pile foundation. The technical information that is available on the original construction of the storm surge barrier is limited. There are technical overview drawings of the entire storm surge barrier and there are detailed technical drawings of the steel gates constructed in 1978 after one of the gates got stuck.

In Table 33 the different assessment are described and the result of the assessment is given. The calculations conducted as part of the assessment are presented in appendix K.

Table 33 - Result preliminary analysis structural elements storm surge barrier Hollandsche IJssel

Element	Assessment	Result
Steel gate (STCO)	The preliminary assessment of the steel gate compares the design load to the new load. When the design load is higher than the new load the gate is safe. The load only uses the hydrostatic pressures created by the governing water levels (shown in Figure 49 and described in appendix K.1).	The assessment shows that the design load per running meter is 186 kN and the new load 263 kN per running meter. The gate is therefore not safe when the new closure scheme is introduced
Supports (STCG)	The preliminary assessment of the supports studies the transfer of the support forces to the concrete tower. The balance of forces is used to schematize the loads in the supports. The tensile strength of concrete is much lower than the compressive strength. When a tensile load is introduced the structure is not safe when a compressive load is introduced it is expected that the load can be introduced (described in appendix K.2).	The schematization of the forces shows that the steel gate always introduces a compressive force into the structure. Due to a storm surge the gate is pushed to the back of the tower, due to salt intrusion the gate is pushed to the front of the tower (shown in Figure 51).
Towers (STCG)	The assessment of the tower is not possible because there is no knowledge of the structural integrity. It is assumed that the transfer of the loads is possible when a compressive force is introduced.	The supports introduce a compressive force therefore the towers are assumed to be safe.
Pile	The preliminary assessment of the pile foundation	Due to the load combination storm

foundation (STCG)	calculates the changed loads due to the load combination storm surge and salt intrusion. When the increase is limited the pile foundation should be able to transfer the loads. Due to higher loads some settlements might occur because more resistance is activated (described in appendix K.3).	surge the loads on the pile foundation increase with approximately 10%. The pile foundation should be able to transfer these forces because the capacity of a pile foundation increases when more settling is allowed. The transfer or the negative loads due to bending of the foundation piles is possible.
Sheet piles (STPH)	The preliminary assessment of the sheet piles analyses piping under the barrier. The results of the third nationwide safety assessment are used to assess piping (described in appendix K.4) [47].	Piping is not a problem because the aquifers are closed off by sheet piling into the second clay layer.

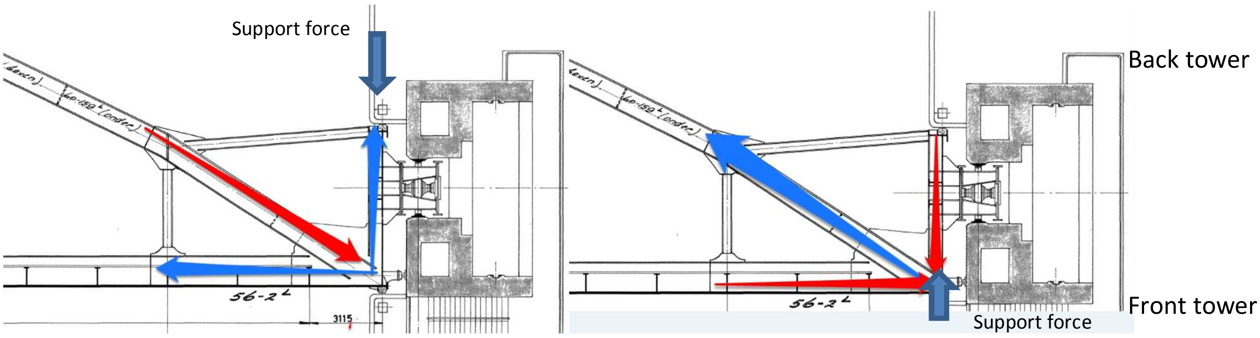


Figure 51 - Forces near the supports; storm surge (left), salt intrusion (right); compressive (blue), tensile (red).

5.1.4 Conclusion; preliminary assessment

Conclusion of the preliminary assessment is that the storm surge barrier cannot transfer the forces due to the changed load combinations. The transfer of the negative loads due to salt intrusion does not seem a problem because the loads are lower; point of attention is the transfer of the negative horizontal load through the pile foundation. The transfer of the increased storm surge loads is a problem; the steel gate cannot transfer the increased loads according to the preliminary assessment. The structural integrity of the towers, sill and pile foundation is not known and therefore a point of attention, in this study it is assumed that compressive forces do not result in problems.

In this study the steel gate which cannot transfer the loads in the preliminary assessment is assessed in detail in the section 5.2. This is also the most important part of the structure because it directly withstands water from either side (STCO). The different elements within the gate are assessed and the effect of the sea level rise is included in the assessment, when needed adaptations to the steel gate are made to increase the resistance. The concrete part of the storm surge barrier is not analyzed in this study. Research into the original design of concrete and reinforcement is needed to produce a complete assessment of the concrete elements within the storm surge barrier.

5.2 Detailed assessment and adaptation of the steel gate

In this section the detailed assessment and possible adaptation of the steel gate is conducted. In part 5.2.1 of this section the steel gate and load combinations are schematized. In part 5.2.2 of this section the detailed assessment of the gate is conducted. In part 5.2.3 of this section the adaptations that might be needed are described.

5.2.1 Schematization of the steel gate

The schematization of the steel gate is based on the technical drawings shown in appendix O and the figures of the steel gate shown in appendix P. The steel gate (shown in Figure 52) is schematized as a framework in software program RSTAB, this software program is used to calculate the transfer of the forces in steel frameworks. The green profiles show the the arches, the brown profiles show the x-bracing, the red profiles show the horizontal struts and the yellow profiles show the stiff end supports. An overview of the schematization is presented in appendix L.1 and Q. The plate wall is not directly schematized in this program, in the assessment of the plate wall the forces in the two green girders are divided over the elements within the plate wall.

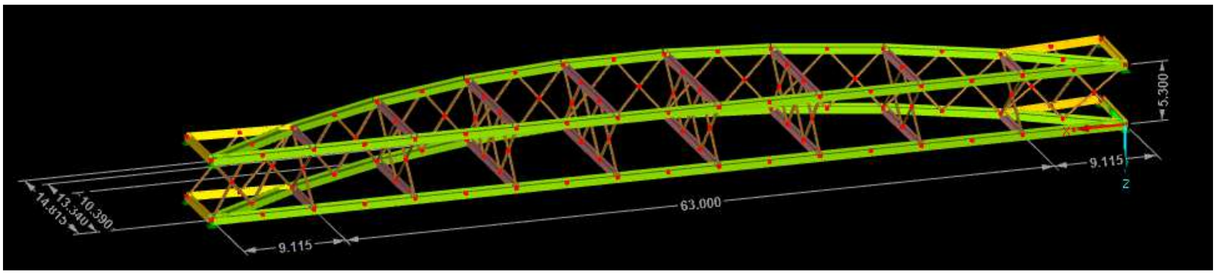


Figure 52 - Schematization framework steel gate [dimensions m]

Storm surge

The governing situation during a storm surge occurs at the end of the closure because the water levels on the New Meuse are maximal. The water levels on the New Meuse increase due to accumulation of water behind the storm surge barrier, when the Maeslant barrier is closed the discharge of the Meuse and Rhine accumulates in the New Meuse, Haringvliet and Hollands Diep Meuse. The water levels in the Hollandsche IJssel do not increase when there is no precipitation or pump discharge into the Hollandsche IJssel. The head over the Hollandsche IJssel barrier is maximal with high water levels on the New Meuse and no precipitation or discharge of pumping stations into the Hollandsche IJssel (shown in Figure 53).

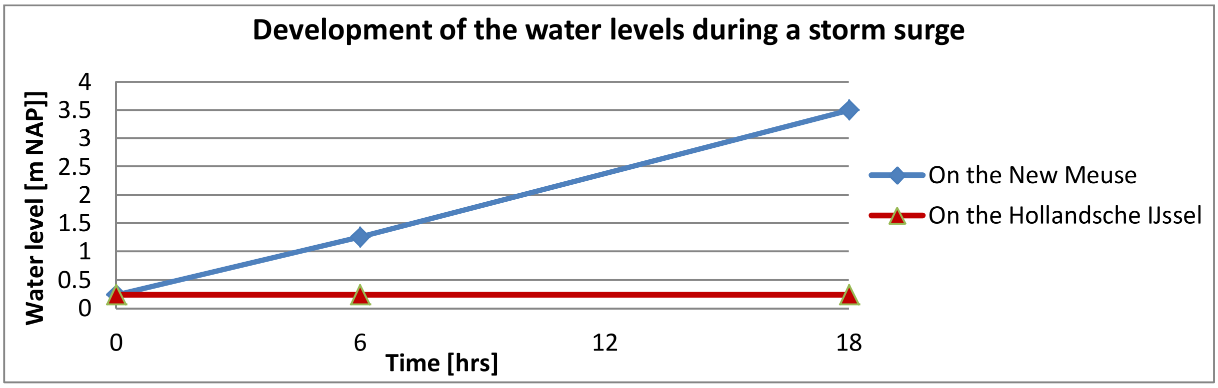


Figure 53 - Development of the water levels during a storm surge

The waves acting on the structure are generated on the New Meuse and travel in a straight line to the Hollandsche IJssel and the storm surge barrier. These waves increase the pressure acting on the steel gate (the wave height is calculated in appendix I.2). Sea level rise increases the water levels on the North Sea and therefore higher water levels occur more often. When high waters occur more often the number of closures of the Maeslant storm surge barrier increases which increases the exceedance probability of closure of the storm surge barrier (described in section 4.1.3 paragraph influence storm surge barriers). The increase of the exceedance probability means that the governing discharge increases and therefore the governing water level at the end of the closure period.

The governing situation during a storm surge is shown in Figure 54; the associated pressure distribution is described in appendix I.4. The situation that occurs directly after the change of closure scheme is given by a water level of +3.00 m NAP, when the sea level rise increases the governing water levels on the Hollandsche IJssel increase to +3.82 m NAP for 0.35 m sea level rise.

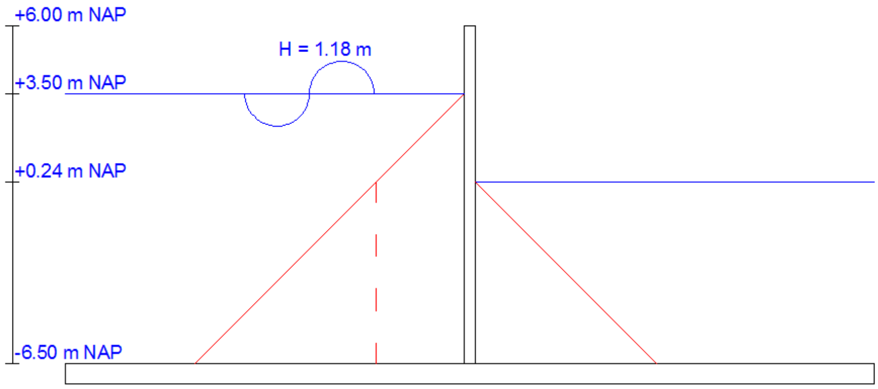


Figure 54 - Detailed overview load combination storm surge after introduction of the new closure scheme

Salt intrusion

The governing situation during salt intrusion occurs a few hours after closure of the storm surge barrier. The water levels on the Hollandsche IJssel are high due to the flood slack closure and the water levels on the New Meuse are low because of the tidal fall on the New Meuse (shown in Figure 55). The water levels on the Hollandsche IJssel will decrease after this situation, because the inlet near Gouda uses the storage of the Hollandsche IJssel. There is a tide on the New Meuse because the Maeslant barrier is only closed during a storm surge not during salt intrusion.

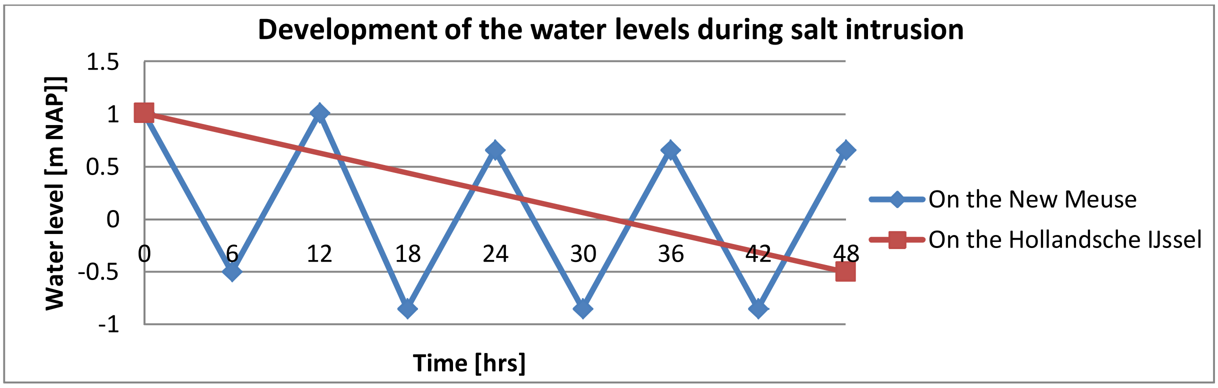


Figure 55 - Development of the water levels during salt intrusion

The sea level rise does not affect the governing situation because the governing situation occurs with low water levels on the New Meuse. When the water levels on the New Meuse increase (due to sea level rise) the negative head reduces and therefore the load on the gate reduces. The governing situation during a storm surge is shown in Figure 56; the associated pressure distribution is described in appendix I.4.

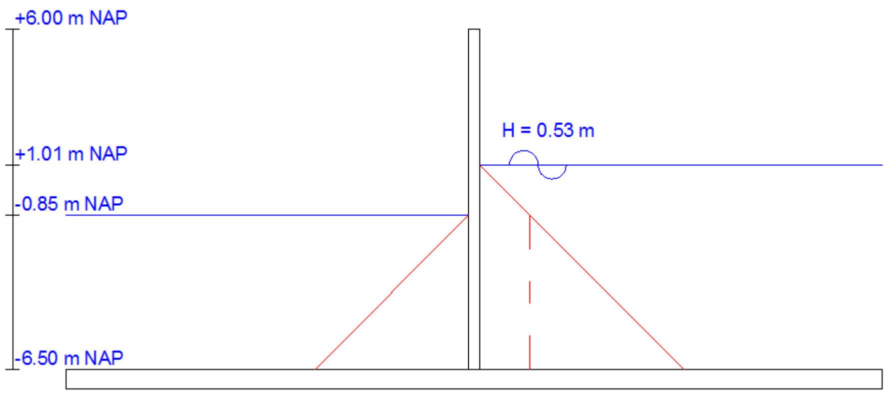


Figure 56 - Detailed overview load combination salt intrusion

Overview load combinations due to sea level rise

The assessment of the steel gate is conducted using the governing water levels obtained from Table 59. The outside water levels are on the New Meuse the water levels on the inside are from the Hollandsche IJssel. The water levels up to 0.35 meter sea level rise are obtained from appendix F.7 and I.3. The governing water levels that correspond to the sea level rise of 0.50 and 1.00 meter are obtained from Table 57 because the closure level of the Maeslant barrier changes to 3.10 m NAP and 3.60 m respectively. The difference between the old and new closure level, of the Maeslant barrier, is added to the governing water levels. The governing discharge does not increase because the closure level of the Maeslant barrier is increased to maintain the closure of once a year.

Table 34 - Overview governing water levels load combinations

Sea level rise	Storm surge [m NAP]			Salt intrusion [m NAP]		
	Outside	Inside	Head	Outside	Inside	Head
0.00 m	+3.50	+0.24	3.26 m	-0.85	1.01	-1.86 m
0.10 m	+3.61	+0.24	3.36 m	-0.85	1.01	-1.86 m
0.20 m	+3.69	+0.24	3.46 m	-0.85	1.01	-1.86 m
0.35 m	+3.82	+0.24	3.57 m	-0.85	1.01	-1.86 m
0.50 m*	+3.92	+0.24	3.68 m	-0.85	1.01	-1.86 m
1.00 m*	+4.42	+0.24	4.28 m	-0.85	1.01	-1.86 m

*Obtained with the increase of the closure level of the Maeslant barrier from Table 23

5.2.2 Assessment of the steel gate

The assessment of the gate is divided in three parts, the first part conducts the assessment of the steel profiles that are schematized in RSTAB, the second part conducts the assessment of the schematized plate wall and the last part assesses the possibility that fatigue occurs. All the assessments are conducted according to the governing Euro codes; in some assessment the Dutch codes are used.

Assessment steel profiles

The assessment of the structural elements in the steel gate focuses on the elements which are expected to be the first to fail for the load combination storm surge or salt intrusion. The steel gate is designed to withstand high water levels during storm surge, it is therefore not necessary to assess every part of the steel gate. Important aspect in the assessment of steel structures is the transfer of tensile forces which is easier than the transfer of compressive forces. The disadvantage of a compressive force is that a steel profile tends to buckle before the yield stress is reached. Buckling is characterized by a sudden failure (bending) of a structural member subjected to a compressive force. In design codes for steel the resistance of the profile is multiplied with a model factor less than 1 to decrease the actual strength of the steel profile. This factor depends on the unsupported length, steel grade and dimensions of the steel profile.

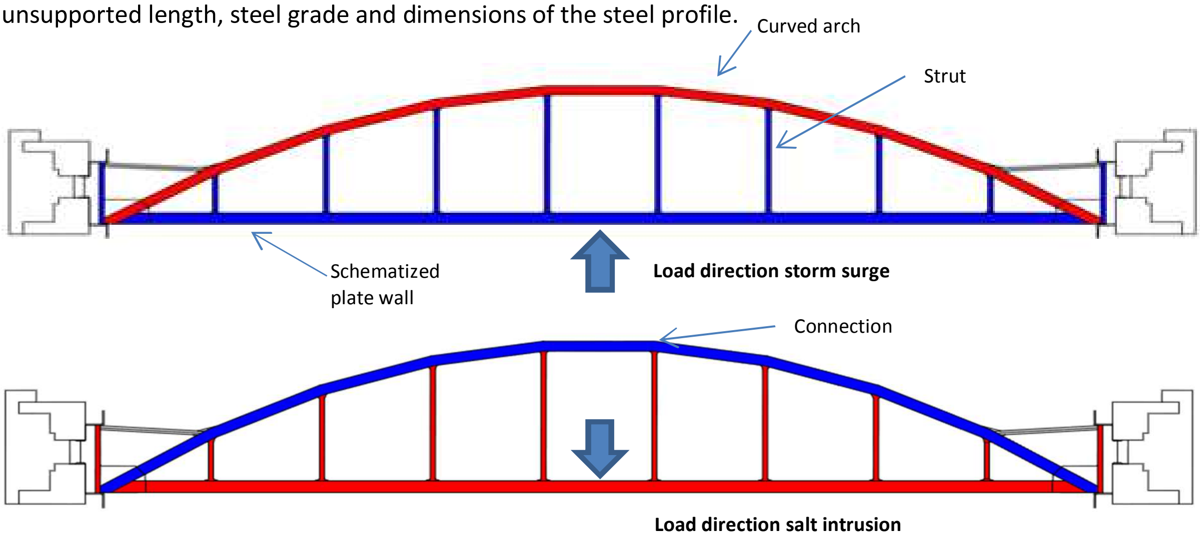


Figure 57 - Forces in the gate during the two load combinations; tensile (red) and compressive (blue)

Figure 57 shows the different forces in the gate when either the load combination storm surge or salt intrusion occurs. The assessments that are needed in combination with buckling are;

- Assessment of the normal compressive force in combination with buckling in the strut during a storm surge. The strut located in the middle is governing because the unsupported buckling length is the longest.
- Assessment of the normal compressive force in combination with buckling in the curved arch located in the underside. The arch located in the underside should be assessed because the applied load results in higher forces in the underside.
- Assessment of the tension force in the curved arch located in the underside of the steel gate. This assessment is needed because the tensile force in this arch is higher than the compressive force.
- Assessment of the connection between the strut and the arch. In the design of the steel gate the connection between the strut and the arch is designed to transfer a compressive force. In the new load combination salt intrusion the strut transfers a tensile force, which means that the strut and arch are pulled apart. The connection between the strut and arch therefore needs to be assessed for the transfer of a tensile force.

The results of the assessment are presented in Table 35 the entire calculation is presented in appendix L.2 and L.3.

Assessment plate wall

The schematized wall shown in Figure 58 is divided into multiple elements that together create the plate wall that withstands the loads applied on the plates.

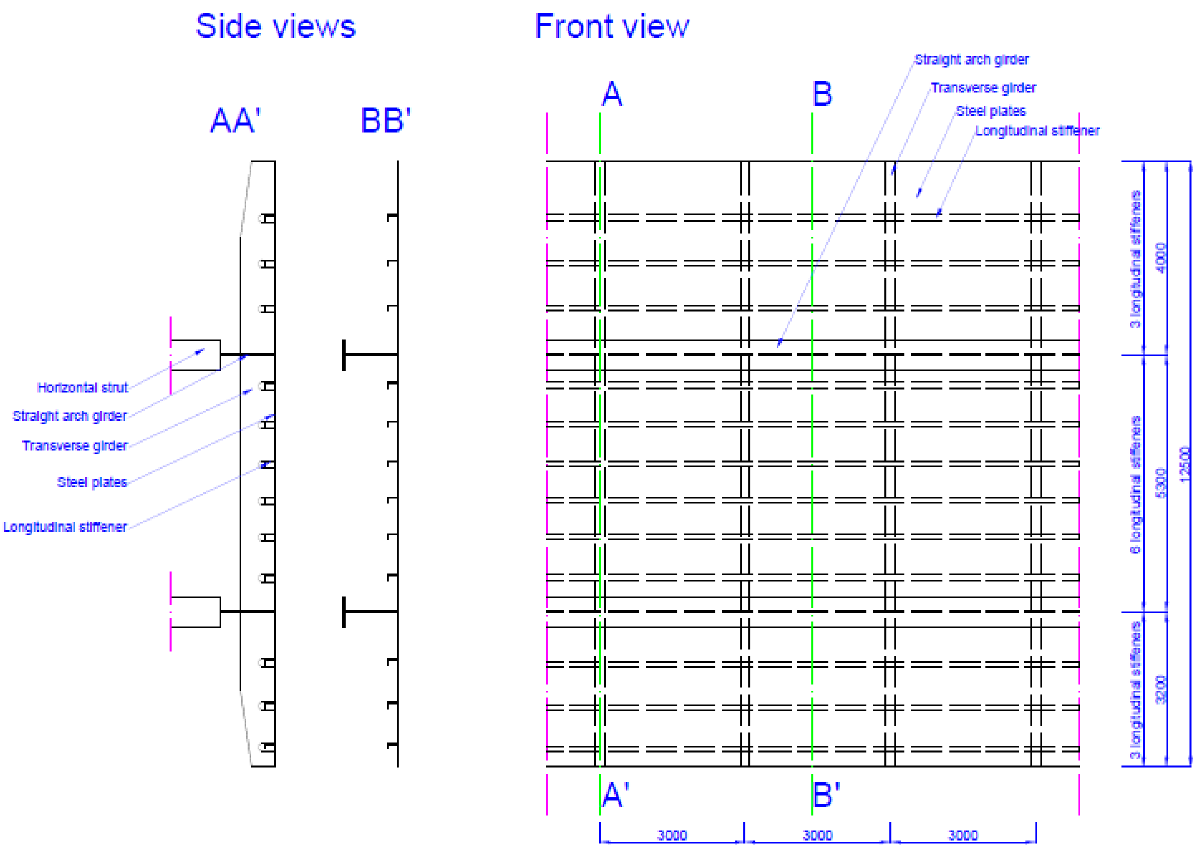


Figure 58 - Schematized plate wall (dimensions mm)

The plate wall consists of different structural elements and is loaded in multiple directions. In general the plate wall is loaded with a compressive force during a storm surge and a tensile force during salt intrusion (shown in

Figure 57). Locally the plates and the longitudinal stiffeners transfer the distributed loads to the transverse girders; the result of this is local bending in the plates and stiffeners (shown in Figure 59)

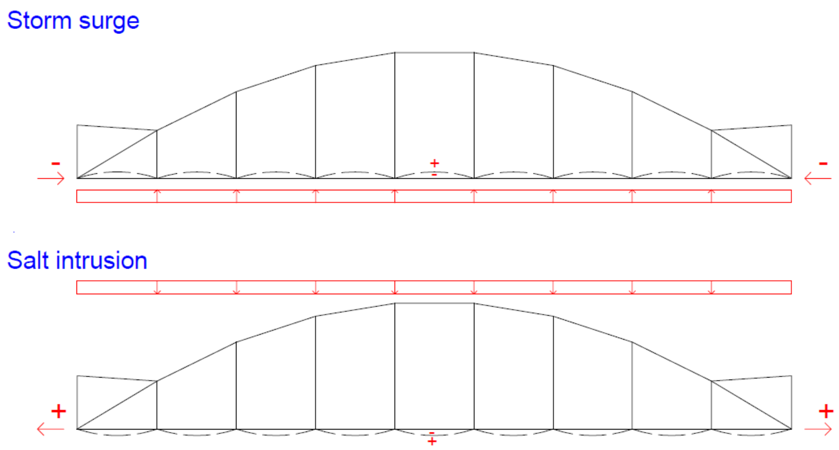


Figure 59 - Schematization load combinations, tensile (+) and compressive (-)

Due to the different possible new and increased loads the plate wall (transverse girder and longitudinal stiffener) needs to be assessed. The assessments that are needed are;

- Assessment of the bending moments in the transverse girder due to a storm surge. The forces applied to the transverse girder are transferred to the arch using bending moments therefore the assessment should compare the design moments to the resistance of the elements.
- Assessment of the compressive force and bending moments in the longitudinal stiffener due to a storm surge. When both forces are applied to the stiffener the stiffener bends in the middle due to the applied loads, this local bending increases the arm of the normal force and therefore introduces a secondary bending moment.
- Assessment of the torsional capacity of the longitudinal stiffener. The torsional capacity of the longitudinal stiffener should be assessed because the stiffener is open. Open profiles are susceptible to torsion because angular displacement is easier than in a closed profile.

The results of the assessment are presented in Table 35 the entire calculation is presented in appendix L.4.

Assessment fatigue

The Hollandsche IJssel storm surge barrier is closed for a prolonged period during salt intrusion. Due to this long closure a fixed low water level is introduced on the Hollandsche IJssel, the water levels outside will still change because of the tidal influence. This change results in the possibility that fatigue occurs. Fatigue is characterized as the local damage (cracks) that occur because of cyclic loading. The gate therefore needs to be assessed for fatigue.

In the assessment of fatigue the number of cycles that occur are compared to the number of cycles that should theoretically occur before failure occurs. The results of the assessment are presented in Table 35 the entire calculation is presented in appendix L.5.

Results detailed assessment

In Table 35 the results of the calculations conducted in appendix L are given, the results are shown as unity checks (UC). In a unity check the design force is divided through the capacity of the element. When the unity check is less than 1 the resistance of the element is higher than the load and the element is safe. When the unity check is more than one the resistance is lower than the load and the element is not safe.

$$UC = \frac{Design\ load}{Resistance\ element} \leq 1$$

Due to the secondary effect of the sea level rise the governing water levels on the New Meuse increase and therefore the loads on the storm surge barrier during a storm surge increase. When the sea level rise is higher than 0.35 meter the effect of the changed Maeslant closure scheme is also accounted for. The increased loads due to the sea level rise are calculated in appendix L.6.

Table 35 - Summary unity check detailed assessment

Element	Unity check salt intrusion	Unity check storm surge					
		SLR 0.0	SLR 0.10	SLR 0.20	SLR 0.35	SLR 0.50	SLR 1.00
Strut	-	0.52	0.54	0.55	0.58	0.59	0.64
Curved arch	0.74	1.18	1.22	1.24	1.30	1.32	1.45
Connection	0.92	-	-	-	-	-	-
Transverse girder	-	1.06	1.08	1.11	1.16	1.18	1.29
Longitudinal stiffener	-	0.83	0.85	0.88	0.93	0.95	1.06
Fatigue	0.00	-	-	-	-	-	-

Table 35 shows that the curved arch and the transverse girder fail during a storm surge, the load combination salt intrusion is not governing. Due to the sea level rise eventually also the longitudinal stiffener fails, it is however not expected that the storm surge barrier still functions with a sea level rise of 1.00 m.

5.2.3 Adaptation of the elements within the steel gate

The conclusion of the last section is that the curved arch in the underside of the steel gate and the transverse girder in the plate wall cannot transfer the loads applied during a storm surge. The other elements in the steel gate are able to transfer the loads and can resist the increased loads due to sea level rise.

Adaptation of the steel gate is possible when steel plates are added to the flange and web of the profiles. Due to the two steel gates one of the gates can be adapted while the other gate functions. The adaptation of the gate should occur in a controlled environment therefore the steel gate will be lifted out of the towers using a heavy lift vessel; the gate is then moved to the steel constructor Hollandia which is located on the industrial area Stormpolder. In the production hall the steel profiles can be cleaned and the flange and web plates can be welded to the profiles that need to be adapted.

The curved arch fails because the tensile force and bending moment in the arch too high, due to the higher tensile force a part of the bending moment is redistributed therefore the capacity of the normal force needs to be increased.

$$N_{R;d} = f_y * A$$

When two web plates (12*1000 mm) are welded to each side of the web the arch is safe.

The transverse girder fails because the bending moments around the supports connected to the arch are too high. The moment resistance of the cross-section is related to the section modulus which is increased when a flange plate is welded to the outer fibre of the profile.

$$M_{R;d} = W_y * f_y$$

When a flange plate (250*10) is welded to the outer fibre the transverse girder is safe.

When the capacity of the profiles is increased the capacity of the connections connecting the elements should also be checked, these connections are often weak links, this is not checked.

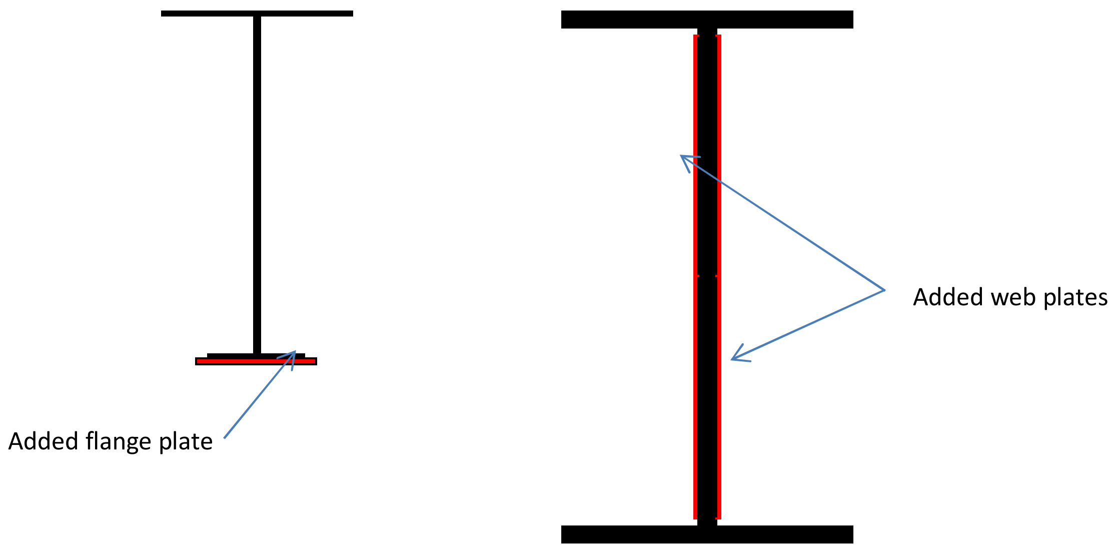


Figure 60 - Adaptation welded profiles; transverse girder (left) and curved arch (right)

When one gate is adapted the other gate still functions when one gate is finished the other gate can be adapted. The costs for the adaptation of the steel profiles in the gate are 112 500 euros (calculated in appendix L.7.3).

5.3 Preliminary analysis and design adaptations

The adaptation of the storm surge barrier does not only focus on the steel gate but also on the other elements that are needed. There are adaptations needed to decrease the non-closure probability, a first analysis of these elements is described (section 5.3.1). Scour protection is needed because scour holes have developed on the riverside of the storm surge barrier, these scour holes could threaten the structure (section 5.3.2). A fish passage is needed in order to use the storm surge barrier during salt intrusion (section 5.3.3).

5.3.1 Preliminary analysis closure reliability

After the gate got stuck in 1976 the water boards insisted that the reliability of the storm surge barrier should be increased. The construction of a second gate behind the existing gate was executed in 1978 as was originally intended in 1958. Due to the two gates the system became in theory a parallel system, both barriers should fail totally independent of each other and therefore greatly enhance the reliability of the Hollandsche IJssel storm surge barrier. The third nationwide safety assessment concluded that the closure reliability of the total storm surge barrier is too low (1/30 per closure event) [4, 18]. The department of Public Works studies the entire fault tree of a non-closure event, in this section a preliminary analysis of the adaptations that are needed to increase the non-closure tree is presented.

The failure tree of the Hollandsche IJssel storm surge barrier presented in appendix M shows that the closure reliability does not only rely on the two gates but also on other aspects not directly related to the structural part of the barrier. The fault tree of the Hollandsche IJssel storm surge barrier (and especially the non-closure part) shows that the two barriers are not independent of each other. Three of the four aspects (high water alarm system, mobilization and controls) that determine the closure reliability do not depend on the two barriers but occur before the signal is given to close one of the two gates.

In appendix M (Figure 156) the technical part of the preliminary fault tree for the closure reliability is presented. The result of this preliminary fault tree is that the two gates form a parallel system but are completely dependent of each other (common cause failure). Common cause failure occurs when there is no power or signal. On a larger scale the reliability of closure can predominantly be increased when the different action that needs to be taken are streamlined.

Common cause failure no power or signal

The critical points that affect the closure reliability of the storm surge barrier due to the aspect no power are;

- The system needs power from both Capelle and Krimpen to lower the gates
- Both gates use the same infrastructure to transfer power to the towers
- There is one emergency aggregate for both gates.

The critical point that affects the closure reliability of the storm surge barrier due to the aspect no signal is the failure of the connection to tower 2. The connection (and all other important connections) to the tower that is situated on the other side of the Hollandsche IJssel is reached using the fixed part of the Algera Bridge that crosses the Hollandsche IJssel. When a ship collides with the bridge the system fails and the gate can only close manually.

Conclusion; closure reliability

It is not possible to create an independent system for both gates. The important improvements that are needed to decrease the dependency and therewith the closure reliability are;

- The power infrastructure should make it possible that a power outage of Capelle or Krimpen does not result in immediate failure.
- Separate power lines connecting each of the towers.
- The installation of an extra emergency aggregate so that each barrier has its own aggregate.
- Relocation of the important connections that cross the fixed Algera Bridge to a concrete box girder that lies on the bottom of the river for example.

The analysis of the non-closure probability shows that the two gates in the Hollandsche IJssel storm surge barrier are a parallel system but are completely dependent of each other. Adaptations that are executed should reduce the dependency of the two gates. Both gates should have their own power and signal infrastructure.

5.3.2 Preliminary design scour protection

In the last nationwide safety assessment it became known that scour holes have developed at the riverside of the storm surge barrier. These scour holes have a depth of approximately 11 meters according to the department of Rijkswaterstaat District New Waterway [48], but do not directly threaten the stability. The bearing capacity of the storm surge barrier is not threatened because the barrier is founded on piles, when the barrier had a shallow foundation it would be a problem because scour holes would directly reduce the bearing capacity. Scour holes may however not develop too far under the sill because otherwise an open connection is created between the inner and outer side of the barrier. When the storm surge barrier is adapted the problems concerning the scour protection should be solved.

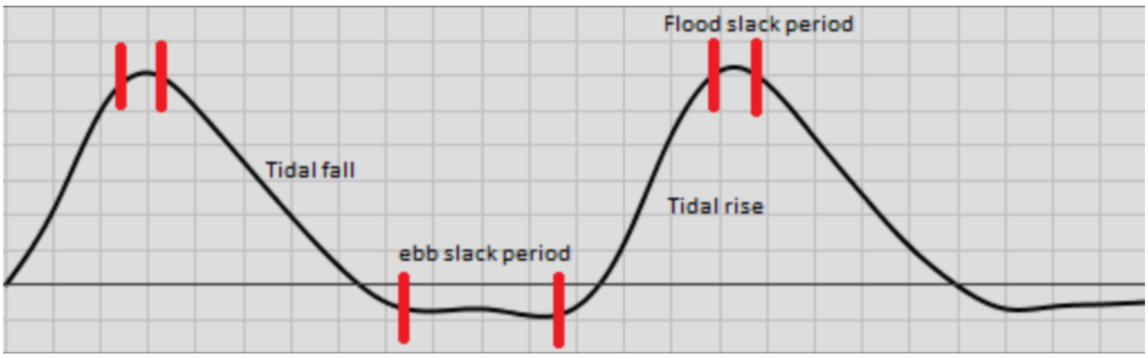


Figure 61 - Tidal form Hollandsche IJssel near Krimpen, source; Rijkswaterstaat

The tidal cycle shown in Figure 61 shows four separate parts of the tide; tidal fall, ebb slack period, flood slack period and tidal rise. The storm surge barrier normally closes during the ebb slack period which does not cause scour because the tidal velocity is nearly zero, when the velocities are nearly zero the closure will not result in a translation wave therefore the closure lasts 25 minutes (shown in Figure 62). During a flood slack closure (because of salt intrusion) the barrier closes during the tidal fall or tidal rise because the flood slack period is small, during this closure there is discharge into or out of the system due to the tide. Closure of the storm surge barrier will take 60 minutes because a translation wave will be created when the barrier is closed to fast.

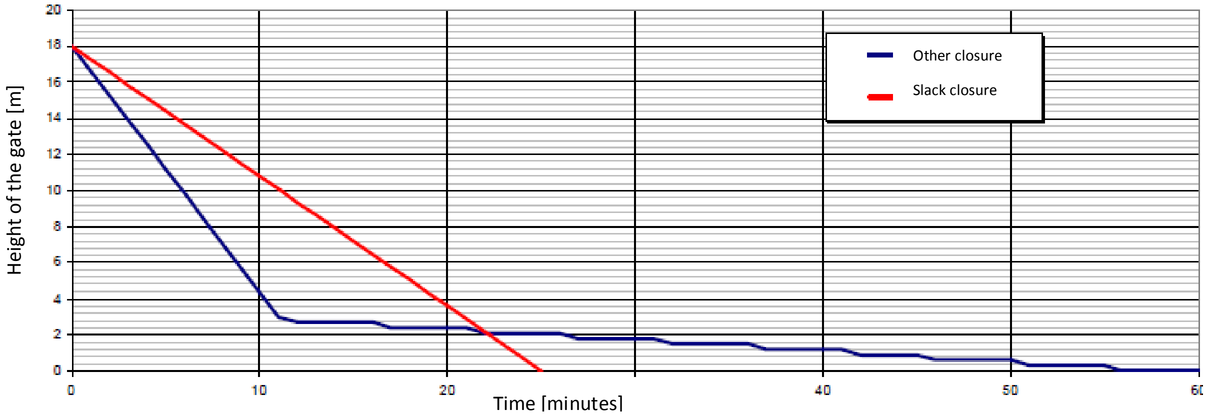


Figure 62 - Closure Hollandsche IJssel barrier, source; HKV Lijn in water

A translation wave is a wave that is created due to an abrupt disturbance of the boundary conditions. When the barrier closes to fast during the tidal rise (which is the governing situation) a positive wave will travel into the basin of the Hollandsche Ijssel a negative wave will travel into the New Meuse. Due to the height of the wave and resonance with other sources the wave can cause a lot of trouble, therefore this wave should be prevented as much as possible.

The governing situation occurs during the tidal rise this period is short therefore velocities and discharge are high and a lot of sediment can erode. The calculations are executed for the situation that the gate closes in steps from 2 meter above the sill to the sill (shown as the blue line in Figure 62), for the calculations a static opening of 1 meter is assumed (average between 2 meters and the closed situation).

The discharge into the Hollandsche IJssel during the tidal rise is 275 m^3/s; the discharge is calculated using the time the tidal rise lasts and the volume of water that enters the Hollandsche IJssel (surface HIJ multiplied with the tidal elevation). The velocity of the water that enters the Hollandsche IJssel increases as the gated lowers into the river (discharge divided through the cross-sectional area of the flow).

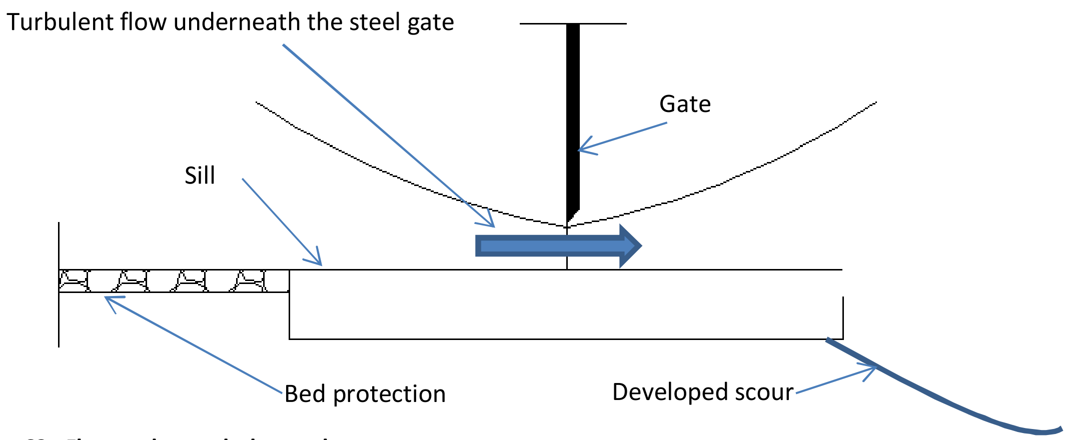


Figure 63 - Flow underneath the steel gate

The velocity and turbulence of the water that flows over the bed determine the scour behind the barrier. Adaptation of the underside of the steel gate to limit turbulence is not possible. Turbulence increases when the flow has no fixed release point (which is the case at rounded ends), the steel gate has a fixed release point (sharp edge at the end) and therefore the turbulence cannot be decreased (shown in Figure 63). The calculations in appendix N show that the developed scour hole will be equal to 11.1 meters. The nominal stone diameter of the scour protection should be 0.14 m which is equal to stone class 60-300kg. The length of the bed protection should be so long that failure of the bed protection will not result in failure of the barrier, the average slope after failure is 1:15 [49]. The length of the bed protection should therefore be equal to 170 m. When ships pass the bed protection they do not damage the bed protection.

5.3.3 Preliminary design fish passage

A fish passage is needed when the storm surge barrier is closed for a prolonged period, therefore closure of the storm surge barrier during a storm surge is not governing and closure during salt intrusion is. The boundary conditions related to the fish passage are predominantly due to the conditions on both the New Meuse and Hollandsche IJssel;

- The fish passage should account for a fixed water level on the Hollandsche IJssel (introduced after two days) and a fluctuating water level on the New Meuse because of the tide.
- The discharge through the fish passage should be limited because not too much salt water should enter the system.
- Fresh and salt water fish like the bass, roach, pike and bream should be able to pass the fish passage; these fish are not able to overcome large water level differences.

The fish passage is located on the side of Krimpen because there is space available to construct a fish passage parallel to the barrier (the space on the Capelle side is limited due to the lock). The inlets of the fish passage should be located such that fish know that there is a way to pass the barrier. A good inlet is located within the migration line of fish and has a good attract flow which makes fish swim to the entrance of the fish passage. The attract flow is created by the location and opening of the entrance. The migration line is the line in which fish will look for another way to pass the barrier [50]. The migration line of most fish is located close to the storm surge barrier because the velocities are zero due to the closed barrier; therefore the entrance to the fish passage is located close to the barrier. The possible use of both steel gates ensures that 4 entrances to the fish passage are needed (shown in Figure 64).

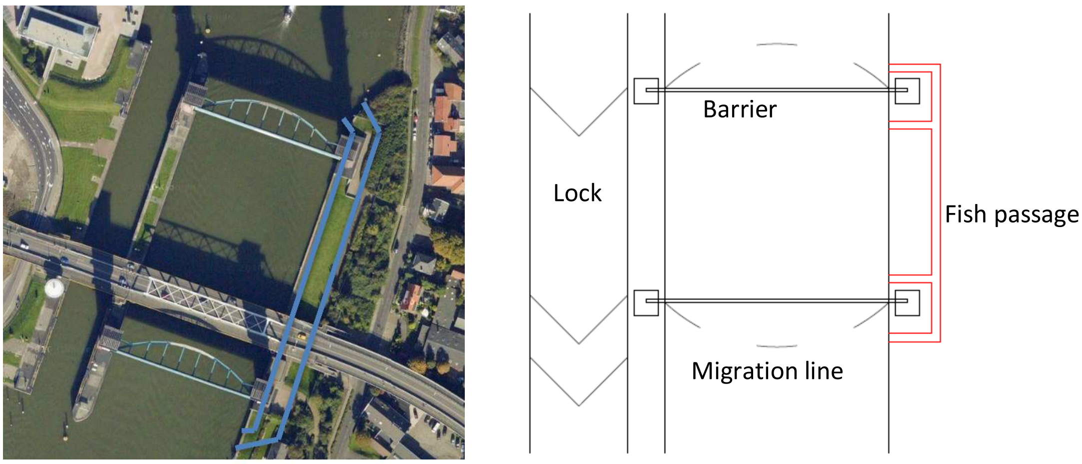


Figure 64 - Location fish passage

The vertical slot passage in a culvert (under water or partly under water) is preferred because;

- The discharge needed is small due to the small vertical slots.
- The fluctuation of the water levels is possible.
- Most fish are able to pass through the vertical slot.

The vertical slot passage is schematized in Figure 65. Other alternatives are not preferred because the fish passage does not function with fluctuating water levels (the Wit fish passage), are specific for one fish (eel passage), require a lot of space (all alternatives designed as a wild river) and require a lot of structure (siphon passage and fish locks).

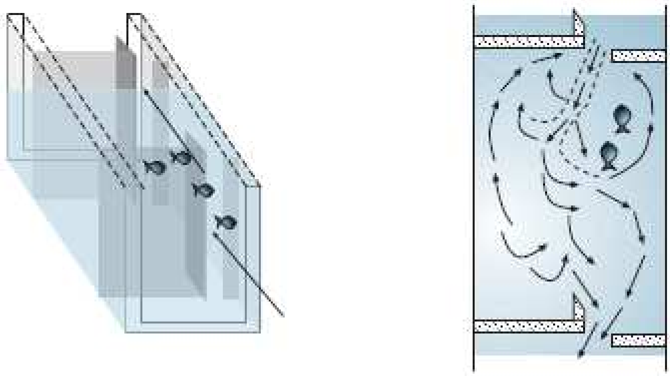


Figure 65 - Schematization vertical slot fish passage, source; Handboek vismigratie 2004

Only the preliminary design of the fish passage is described this study, the exact design of the fish passage is not part of this study.

5.4 Conclusion; adaptation storm surge barrier

The preliminary assessment of the structural elements within the storm surge barrier (section 5.1) shows that the steel gate needs to be adapted because the governing loads during a storm surge are too high. There is limited information available about the other elements (towers, foundation, supports); the preliminary analysis does not show that these elements fail when the loads increase. It is however possible that this occurs because the original design loads, on which the entire storm surge barrier is designed, are exceeded during the new governing conditions.

Steel gate

In the detailed assessment of the steel gate the different elements within the gate are assessed for the situation directly after the introduction of the new closure scheme. The assessment shows that increased loads due to the storm surge are governing; the capacity of the profiles is not exceeded due to the loads during salt intrusion. During storm surge the combination of the tensile force and bending moment exceeds the capacity of the curved arch. The capacity of the transverse girder is exceeded due to the bending moments during a storm surge. Fatigue due to cyclic loading is not governing; the number of cycles to failure is much lower than the number of cycles that will occur in the lifetime of the gate.

Due to the sea level rise the loads due to a storm surge increase, the loads due to salt intrusion do not increase because the difference between the water levels in the governing situation becomes small due to sea level rise. The combination of the plates and longitudinal stiffeners will fail between 0.5 and 1.0 meter sea level rise. The struts that connect the plate wall and the curved arch do not fail due to the increased loads. Elements within the steel gate will be adapted to introduce the new closure scheme and account for the possible sea level rise. The elements that will be adapted when the gate is lifted out of the tower and shipped to the production hall of Hollandia are;

- Curved arch. The cross-sectional area of the curved arch is increased to transfer the increased tensile forces. Web plates (12*1000) are welded to both sides of the web. Only the curved arch located in the underside of the steel gate needs to be adapted, experiences the highest loads.

- Transverse girder. The section modulus of the transverse girder is increased to transfer the bending moments around the supports. A flange plate (10*250 mm) is welded to the flange at the inside of the steel gate. All transverse girders experience the same loads and are therefore all adapted.

The adapted elements ensure that the steel gate is able to withstand the governing loads due to introduction of the new closure scheme and 0.50 meter sea level rise. In theory the storm surge barrier should account for 0.35 meter sea level rise because the conclusion of chapter four shows that the adapted barrier can be used until 0.35 meter is reached. The capacity increase that is needed to account for 0.50 meter sea level rise instead of 0.35 meter is limited because only the increase due to the raised closure level of the Maeslant barrier needs to be taken into account. The governing discharge does not increase because the Maeslant closure level increase and therefore the exceedance probability of closure of the Maeslant barrier remains the same.

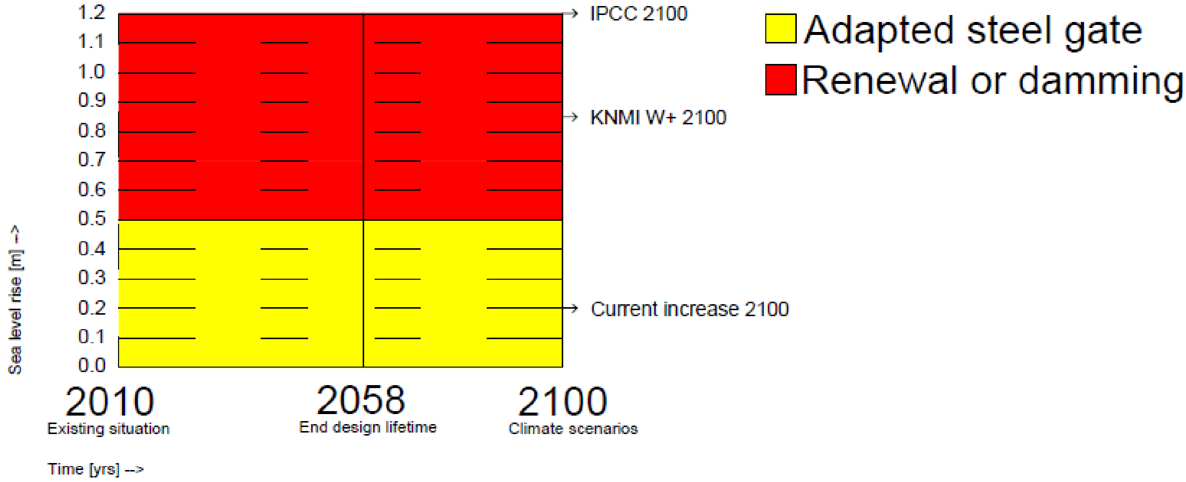


Figure 66 - Absolute sea level rise closure scheme

Preliminary design other adaptations

The preliminary analysis of the non-closure tree (described in section 5.3.3 and appendix M) shows that there are four aspects that influence the closure reliability; alarm system fails, mobilization fails, control error and technical failure closure. Only the last aspect is linked to the civil engineering elements of the storm surge barrier, the other aspects are influenced by management and software. The two gates are linked on multiple occasions and do therefore not fail independent of each other. The power and signal connections need to be separated within the storm surge barrier to increase the independency of the two gates and therefore lower the non-closure probability.

The preliminary analysis and design of the scour (protection) shows that scour holes develop on the inner side of the storm surge barrier because the water velocity during a flood slack closure is too high. Due to the prevention of translation waves in the Hollandsche IJssel the gate closes slowly in the last two meters, this extended period ensures that large water velocities occur. The scour protection that is needed to prevent this scour has a nominal diameter of 0.14 meter.

The preliminary analysis of the fish passage shows that a vertical slot fish passage is preferred because the discharge needs to be small, fluctuation of the water levels is possible and most of the fish are able to pass the barrier. The fish passage is located at the Krimpen side of the storm surge barrier because there is space available for the structure. The entrances to the fish passage are located close to the gates to increase the number of fish using the fish passage.

6 Conclusion and recommendations

In this chapter the conclusion and recommendations of this study are presented. The conclusion (section 6.1) explains the objective described in section 1.2 based on the problems studied in the intermediate chapters. The recommendations (section 6.2) focus on the further research that is needed to study the effects of the adaptation and design of the adapted storm surge barrier.

6.1 Conclusion

The first part of the conclusion describes the objective of this study; the second part of the conclusion focuses on the adaptations that are needed in the preferred strategy.

Objective

In the introduction of this report the background and purpose of this study are described. The objective that is presented in this section is given as;

The objective of this study is the development and (conceptual) design of the preferred strategy for the important aspects (overall safety, salt intrusion and climate change) in the Hollandsche IJssel. The preferred strategy is cost-effective and exists of a technically and societally feasible design.

Preferred strategy

The preferred strategy for the system of the Hollandsche IJssel is strategy 1; this strategy consists of the adaptation of the storm surge barrier, reinforcement of the levees and introduction of a new closure scheme. The construction of a new storm surge barrier (1A) or dam (1B) is postponed until the adapted barrier does not fulfill the requirements. The adaptation is preferred above other strategies because;

- The postponing of large structures like a dam or storm surge barrier is more expensive than the investments needed for the adaptation of the storm surge barrier.
- Renewal would not result in lower reinforcement costs compared to adaptation because the effect of the steep inner slope is large.
- Damming is not necessary as long as the sea level rise is low.

Overall safety

The overall safety in the Hollandsche IJssel is a problem because the contribution of the Hollandsche IJssel system to the total risks of especially dike ring 14 is too high and the storm surge barrier and levees of the Hollandsche IJssel were not up to the standards in the third nationwide safety assessment conducted between 2006 and 2011. The overall safety in the system is increased the governing water levels on the Hollandsche IJssel are decreased, this is possible when;

- the closure level of the Hollandsche IJssel storm surge barrier is decreased and,
- the closure reliability of the Hollandsche IJssel storm surge barrier increased.

The adaptations that are needed to decrease the closure level and increase the closure reliability are;

- The introduction of a new closure scheme which reduces the closure level from +2.25 m NAP to +1.75 m NAP, reduces the pump stop level from +2.60 m NAP to +2.00 m NAP and closes during the preceding ebb slack period.
- The adaptation of the steel gate to allow the introduction of the new closure scheme. The capacity of the curved arch and transverse girders is not high enough therefore the gate is lifted out of the barrier and transported to the production hall of Hollandia, there flange and web plates are welded to the curved arch and transverse girder.
    - Two web plates (12*1000 mm) with steel grade S235 are welded to the web of the curved arch.
    - One flange plate (10*250) with steel grade S235 is welded to the flange of the transverse girder.

- The increase of the independency between the two parallel storm surge barriers to reduce the non-closure probability. The power and signal connections to each tower should be divided from each other.
- The construction of scour protection to prevent the erosion of sand directly behind the storm surge barrier. Directly behind the storm surge barrier scour holes develop because of the slow closure during a flood slack. Scour holes behind the storm surge barrier could threaten the stability or create a connection between the outer and inner side of the barrier.

Direct effects
The effect of the adaptations is that the risk contribution of the Hollandsche IJssel flood defence system reduces with approximately 50%, 10% of the levee reinforcement is directly prevented. The reduction of the governing water levels does not prevent all the levee reinforcements therefore the cost reduction should combine the decrease of the water levels and the smart design of the levee reinforcements.

The effect of the non-closure probability should be reduced from 1/30 per event to 1/500 otherwise the decrease of the governing water level diminishes. The increase of the independency is a first step more adaptations should probably be necessary to reach the targeted non-closure probability.

Effects due to climate change
Due to the sea level rise the governing water levels in front of the Hollandsche IJssel increase. The loads on the Hollandsche IJssel storm surge barrier and the number of closures increase. In Figure 67 the overview of the different adaptions are given. The sea level rise that occurs according to the climate studies is given as a black line. It is possible to use the new closure scheme until 0.35 meter is reached, after that the number of closure is too high (more than 24) and the storm surge barrier behave like a dam. It is possible to use the adapted steel gate until 0.50 meter is reached. The increase of the loads between 0.35 and 0.50 meter is not large; therefore the steel gate is adapted to withstand governing water levels that occur with 0.50 meter.

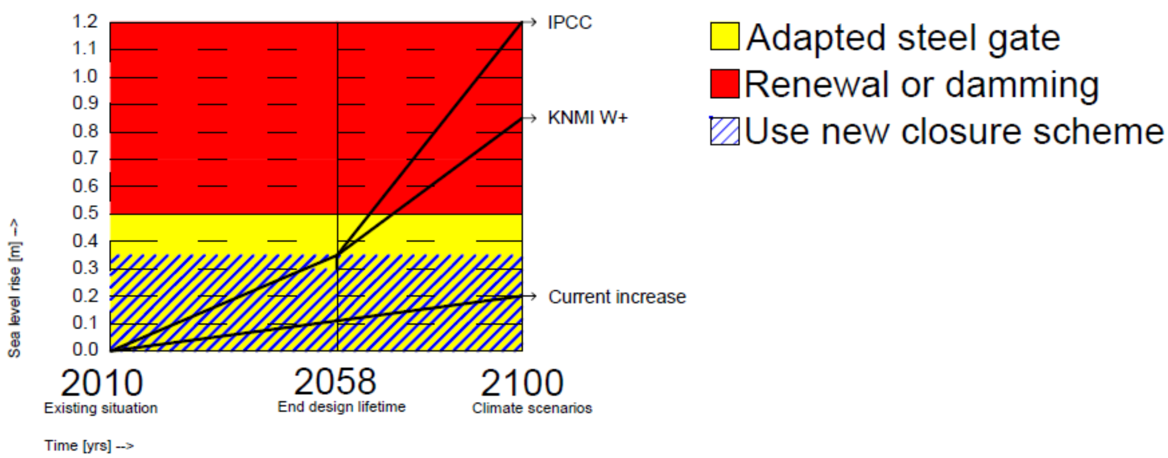


Figure 67 - Overview possible use different adaptations based on calculation conducted in this study

When the 0.35 meter sea level rise has occurred the closure scheme of the Hollandsche IJssel storm surge barrier should be increased to reduce the number of closures. After 0.50 meter sea level rise the adapted storm surge barrier does not fulfill the given requirements. The choice between adaptation and damming should be taken when this level is nearly reached. The choice between adaptation and damming will depend on the costs needed for the additional levee reinforcement and the construction costs of the dam and new storm surge barrier. The rate of change of the sea level rise is also important because this affects the effectiveness of the storm surge barrier.

Salt intrusion

Salt intrusion is a problem because salt intrusion in the Hollandsche IJssel prevents the inlet of water into the canals of Central Holland. Salt intrusion in the Hollandsche IJssel is prevented when;

- The Hollandsche IJssel storm surge barrier is closed during salt intrusion periods,

- The source of water into the canal system is guaranteed.

The adaptations that are needed to prevent salt intrusion in the Hollandsche IJssel are;

- Renovation of the salt stair in the Hollandsche IJssel which increases the turbulence in the fresh and salt water layers and therefore introduces more mixing and slowing of the salt intrusion (already in progress).
- The introduction of a new closure scheme which closes the storm surge barrier when salt intrusion reaches the mouth of the Hollandsche IJssel. The closed storm surge barrier prevents the tide pushing the saline water into the Hollandsche IJssel. The closure level of the Hollandsche IJssel should be the salt concentration which may not be higher than 250 mg/l measured at the mouth. The inlet stop level on the Hollandsche IJssel is -0.5 m NAP, which is needed to maintain shipping on the Hollandsche IJssel. The possible discharge of the canalized Hollandsche IJssel and small scale water supply should increase from 12 m^3/s to 24 m^3/s to guarantee the inlet of water during the period that the storm surge barrier is closed.
- The construction of a vertical slot fish passage which allows fish to pass the closed storm surge barrier.

<u>Direct effects</u>
The effect of the adaptations is that the number of salt days in the governing situation is reduced from 60 to 30 days. During the 30 salt days the storm surge barrier is closed and the small scale water supply and canalized Hollandsche IJssel are used to guarantee the inlet.

During the closure the tide cannot enter the Hollandsche IJssel therefore the water quality decreases and the tidal nature adapts to the fixed water levels. The closure of the storm surge barrier is limited to one month to prevent a "dead" river.

<u>Effects due to climate change</u>
Due to the longer and lower low discharges the number of salt days increases. When the number of salt days exceeds the closure of one month additional methods need to be taken. The preferred method described in this study is additional slowing in the New Meuse using a bubble screen, when this no longer works the inlet should be relocated. The additional slowing and relocation are not studied in this thesis.

Other aspects

The adaptation of the storm surge barrier seems societally feasible and cost effective because;

- the adaptations are executed within the current configuration of the storm surge barrier,
- the ecology is no threatened with the adaptation of the storm surge barrier,
- the costs for the adaptation of the profiles within the steel gate are low.

The local surrounding and traffic flows are analyzed at the start of this study but are not part of the strategy evaluated in the other chapters of this report.

Overall conclusion

The overall conclusion is that adaptation of the storm surge barrier is indeed the preferred strategy and is possible within the aspects that have been studied in this study. The overall safety increases, levee reinforcements cannot be prevented. The first assessments show that the storm surge barrier can withstand the increased loads, more research is needed on the structural integrity of the concrete elements.

6.2 Recommendations

The recommendations at the end of this study focus on the adaptation of the storm surge barrier and choice that needs to be made between renewal and damming.

1. The climate change should be closely monitored and studied because the sea level rise affects the time at which a new structure is needed. Climate studies (KNMI and IPCC) should be validated to the

sea level that has occurred, because the rate at which the sea level rise changes affects the choice between damming and renewal.

2. Adaptation of the storm surge barrier prevents only a small part of the reinforcements. The decrease of the governing water levels in combination with smart reinforcement should be researched to reduce the costs for levee reinforcement. The smart reinforcements should focus on the possibility to use simpler structures because of the decreased water levels. Instead of a diaphragm wall a sheet pile might also suffice.

3. The structural adaptation of the storm surge barrier focused on the assessment of the steel gate because the steel gate directly retains the water and there was information available about the steel gate. The transfer of the forces through the towers, sill and foundation is only briefly discussed. At first the structural integrity of the storm surge barrier should be analyzed. Based on this analysis the new loads due to introduction of the new closure scheme and sea level rise should be assessed. Results of this study should show which adaptation might be necessary.

4. The study which determines the exact fault tree and corresponding failure probability of each aspect is in progress. After that a study into the possible adaptations of the storm surge barrier is necessary to achieve the targeted non-closure probability of 1/500.

5. A model of the salt intrusion into the Hollandsche IJssel should be made to investigate for which boundary conditions salt intrusion occurs and what the effect of the different solutions is. This model should use the system of the Rijnmond and Hollandsche IJssel the input of the boundary conditions should result in a prediction of the salt intrusion.

6. The vertical slot fish passage and scour protection should be designed. In this study it is shown that it is needed and a preliminary design is made. Design calculations should result in a final design that can be constructed by a contractor.

7. The effect of the adapted storm surge barrier and levee reinforcements on the local surrounding should be studied. This study should increase the public support for the levee reinforcements that are not prevented. This study could also provide a better insight in the choice between damming and renewal that eventually should be taken.

8. In the current assessment of the steel gate only one important connection and the steel profiles are assessed. Eventually only the steel profiles are adapted, normally connections prove to be governing. Therefore a detailed assessment of the different connections including the stiffeners to introduce the forces into the profiles should be needed. In the first step of the assessment it is also necessary to map the different connections based on the current steel gate, the steel gate is adapted a few times since the technical drawings made in 1978.

9. The length of the closure period of the Hollandsche IJssel storm surge barrier determines the effectiveness of the closure during salt intrusion. The closure period, which is assumed to be one month, depends on the ecology, fish migration, shipping etcetera. A study into the ecology is needed to specify and categories the exact effects of the closure period.

Bibliography

[1] C. Veerman, „Samen werken met water," Hollandia printing, Den Haag, 2008.

[2] Wikipedia, „Hollandsche IJssel," 2012. [Online]. [Geopend 2012].

[3] Rijkswaterstaat, „Helpdeskwater," 2013. [Online]. Available: http://www.helpdeskwater.nl/onderwerpen/wetgeving-beleid/waterwet. [Geopend juni 2013].

[4] Rijkswaterstaat, „Derde toets primaire waterkeringen, landelijke toets 2006-2011," Inspectie verkeer en waterstaat, Den Haag, 2011.

[5] VNK2, „Overstromingsrisico van dijkringgebieden 14, 15 en 44," Projectbureau VNK2 , Utrecht, 2012.

[6] M. Voorendt, „Policy and Legislation on Flood Protection," Delft , [Not published].

[7] KNMI, „Klimaatverandering in Nederland, Aanvullingen op de KNMI'o6 scenario's," Drukkerij Koninklijke Broese en Peereboom B.V., Brede, 2009.

[8] KNMI, „Sea level rise for 2050 and 2100," 2006.

[9] Deltaprogramma, „Deltaprogramma 2013, Werk aan de delta," 2013.

[10] Deltaprogramma, „Developments Rotterdam for 2050-2100," 2010.

[11] S. Krol, „Deltaprogramma Rijnmond-drechtsteden; Gebiedsrapportage Hollandsche IJssel," Den Haag, 2013.

[12] Rijkswaterstaat, „Voorschrift toetsen veiligheid primaire waterkeringen," 2007.

[13] F. Bulsink, „Zoetwaterverkenning Midden-West Nederland," Universiteit Twente, Enschede, 2010.

[14] KWR Watercycle Research Institute, „Klimaatbestendigheid in de Zuidwestelijke Delta," 2008.

[15] Rijkswaterstaat, „Watemanagment in the Netherlands," 2011.

[16] Alterra, Deltares and the bakelse stroom, „Handelingsruimte Zoutmanagment," 2012.

[17] Rijkswaterstaat, „HRC2006, Hydrologische randvoorwaarden," Den Haag, 2006.

[18] HKV lijn in water, „Faalkans Hollandsche IJssel kering," 2010.

[19] J. Bosboom en M. Stive, „Coastal Dynamics 1 CT4305," 2011.

[20] Visstekkengids Anglermaps , „Anglermaps," 2013. [Online]. Available: www.anglermaps.nl/visstekken/rivieren-kanalen/hollandse-ijssel/. [Geopend 2013].

[21] Krimpen, „Masterplan Stormpolder," Krimpen, 2005.

[22] Rijkswaterstaat, „Staande Mast Route," Libertas, Bunnik, 2012.

[23] Wikipedia, „Tipping point," Wikipedia, 1 May 2013. [Online]. Available: www.en.wikipedia.org/wiki/Tipping_point. [Geopend 2013].

[24] Delft Hydraulics, „Volkerak-Zoommeer, Effectiviteit zoutbestrijdingsmaatregelen bij sluizen," 2007.

[25] Alphen aan de Rijn, „Bestemmingsplan Steekterpoort I, Deel 1, - Toelichting," Alphen aan de Rijn, 2012.

[26] Van Hattum en Blankevoort, „2de Julianasluis," 2013. [Online]. Available: www.vhbinfra.nl/nl/projecten/detail/julianasluisgouda. [Geopend 2013].

[27] Rijkswaterstaat, „Richtlijn vaarwegen RVW," 2011.

[28] Rotterdam Vooruit, „Herontwerp Brienenoord- en Algeracorridor," 2010.

[29] Consulting agency Van der Kraan, „Inventarisatie situattie primaire waterkeringen Deltadeelprogramma Rijnmond en Drechtsteden," 2012.

[30] HHS&K, „memo effect waterstandsverlaging STBI," 2012.

[31] W. t. Horst en R. Jongejan, „Memo: Inschatting faalkans Hollandse IJssel dijken," 2012.

[32] Rijkswaterstaat, „ADDENDUM BIJ HET TECHNISCH RAPPORT WATERKERENDE GRONDCONSTRUCTIES," Ando bv, Den Haag, 2008.

[33] M. v. d. Meer, „Comparison of the Dutch and American Levee Safety Approach," Sacremento, 2009.

[34] Rijkswaterstaat, „Waterbase," Rijkswaterstaat, 14 mei 2013. [Online]. Available: www.live.waterbase.nl/waterbase_wns.cfm?taal=nl. [Geopend 2013].

[35] P. v. Gelder, *Peak over Threshold dataset,* Delft: TUDelft.

[36] J. Vrijling en P. van Gelder, „Probabilistic Design in Hydraulic Engineering CT5310," TUDelft, Delft, 2006.

[37] Rijkswaterstaat, „Waterstanden," 2010.

[38] Witteveen+Bos, „Evaluatie sluitregime Maeslantkering," Rotterdam, 2009.

[39] Vellinga, „Effecten op de scheepsvaart, deelrapport AOR," TUDelft, Delft, 2010.

[40] F. Bruin en J. Wessels, „Economische consequenties van 'sluitingen andere redenen van de SVK Nieuwe Waterweg," Lloyd's register, 1995.

[41] Deltares, „Herstel van trapjeslijn in Nieuwe Maas (fase 2), Vervolgstudie naar de effecten op de zoutindringing," Delft, 2010.

[42] P. Schipper en H. Siebel, „Index Natuur en Landschap, Onderdeel natuurbeheerstypen," 2009.

[43] Province South-Holland, „Natuurbeheersplan," 2012. [Online]. [Geopend 2012].

[44] Rijkswaterstaat, „Europsese Kaderrichtlijn Water," 2000.

[45] KNMI, „Risicosignalering zware regen," knmi, De Bilt, 2006.

[46] Rijkswaterstaat, Waterakkoord Hollandsche IJssel en Lek, 2005.

[47] Rijkswaterstaat, Witteveen+Bos, „Stormvloedkering en sluis Hollandsche IJssel (VWK10)," VormVijf, Rotterdam, 2010.

[48] L. Hove, Interviewee, *Visit to the Hollandsche IJssel storm surge barrier.* [Interview]. 7 March 2013.

[49] G. Schiereck, „Introduction to Bed, bank and shore protection," VSSD, Delft, 2004.

[50] M. Kroes en S. Monden, „Vismigratie; Een handboek voor herstel in Vlaanderen en Nederland," Ministerie van de Vlaamse Gemeenschap, Brussel, 2004.

[51] Wikipedia, „Wikipedia; Waterschappen," 2013. [Online]. Available: http://nl.wikipedia.org/wiki/Waterschap. [Geopend 2013].

[52] Rijkswaterstaat, „Specifieke aspecten tunnelontwerp," 2005.

[53] Wikipedia, „Julianasluis," Wikipedia, 13 March 2013. [Online]. Available: www.nl.wikipedia.org/wiki/Julianasluis. [Geopend 2013].

[54] Rijkswaterstaat, „Nota Mobiliteit, Naar een betrouwbare en voorspelbare bereikbaarheid," Rijksoverheid, Den Haag, 2004.

[55] Triami Media BV, „inflation.eu worldwide infation data," Triami Media BV, 2013. [Online]. Available: www.nl.inflation.eu/inflatiecijfers/historiche-inflatie/cpi-inflatie-nederland.aspx. [Geopend 2013].

[56] M. Hillen, S. Jonkman, W. Kanning, M. Kok, M. Geldenhuys en M. Stive, „Coastal defence cost estimates," Delft, 2010.

[57] K. Noortwijk en M. Duits, „Bayesian Frequency Analysis of Extreme River Discharges," 2010.

[58] H. Zhong, P. van Overloop, P. van Gelder en T. Rijcken, „Influence of a Storm Surge Barrier's Operation on the Flood Frequency in the Rhine Delta Area," Delft, 2012.

[59] Rijkswaterstaat, „Vaarwegenoverzicht," 2013. [Online]. Available: http://www.rijkswaterstaat.nl/water/feiten_en_cijfers/vaarwegenoverzicht/. [Geopend 2013].

[60] Technische Adviescommissie voor de Waterkeringen, „Leidraad Kunstwerken," Delft, 2003.

[61] W. Molenaar, S. Baars en H. Kuijper, „Manual Hydraulic Structures, CT3330," TuDelft, Delft, 2008.

[62] J. Beersma, B. T.A., S. de Goederen en P. de Jacobs, „Zoet, zouter, zoutst; Statistiek van de externe verzilting in Midden- West Nederland," KNMI, De Bilt, 2005.

[63] T. Buishand, R. Jilderda en J. Wijngaard, „Regionale verschillen in extreme neerslag WR2009-1," KNMI, De Bilt, 2009.

[64] H. J. Verhagen, K. d'Angremond en F. v. Roode, „Breakwaters and closure dams," VSSD, Delft, 2012.

[65] E. Simiu, N. Heckert, J. Filiben en S. Johnson, „Extreme wind load estimates based on the Gumbel distribution of dynamic pressures: an assessment," 2001.

[66] KNMI, „Wind climate assessement of the Netherlands," 2003.

[67] F. Bijlaard en A. Gresnigt, „Staalconstructies 2, Gedeelte plasticiteitsleer, stabiliteit en verbindingen," TU Delft, Delft, september 2012.

[68] CEN, „Eurocode 3 - Design of steel structures - Part 1-5: Plated structural elements," CEN, Brussel, 2006.

[69] Nederlandse Norm, „NEN 6771," NEN, Delft, 2000.

[70] R. Abspoel en F. Bijlaard, „Steel structures 3, Part: Stability of steel plate structures CT4121," TU Delft, Delft, 2005.

[71] H. Kolstein, „Fatigue CT5126," TU Delft, Delft, 2007.

[72] F. Bijlaard, Interviewee, *Costs steel.* [Interview]. 5 june 2013.

[73] HHS&K, „Toetsverslag primaire waterkeringen, toetsing 2010, Dijkring 14 en dijkring 15.," HHS&K, Rotterdam, 2010.

[74] P. Dircke, T. Jongeling en P. Jansen, „An overview and comparison of navigable storm surge barriers," Rotterdam, 2009.

[75] J. Vrijling, P. Gelder en A. Vrouwenvelder, „Probability in Civil Engineering," Faculteit CITG, Delft, 2006.

Glossary

Technical terms

Word	Explanation
AND-port	Used in fault trees to specify that both event should occur before failure
Assessment	The check if a certain failure mechanism does not occur
Aquifer	Water bearing layer of soil
Barrier	See storm surge barrier
Cascade effect	The flooding of a lower lying dike ring via a higher lying dike ring
Casualties	Number of people that die
Closure level	Predicted water level at which the storm surge barrier should close
Closure scheme	The total set of measures that is related to closure of the barrier
Dike ring	A system of connected structures that protect a lower lying area
Ebb slack period	The period during the ebb tide that the velocities are nearly zero
Exceedance probability	Probability that a certain (water) level is exceeded
Failure class	Group of levees for which the failure mechanism is the same
Failure probability	The probability that a structural element fails
Fault tree	Method used to assess which events lead to failure of a top event
Fish passage	A structure that fish can use to pass an obstacle
Flange plate	Steel plate welded to the flange of a profile
Flood slack period	The period during the flood tide that the velocities are nearly zero
Flushing	Use of water to refresh the brackish water of the canals in Central Holland
Governing water level	Water level that occurs in the governing situation (norm)
Head	Difference between the water levels in and outside of the barrier
Hollandsche IJssel system	The system from the HIJ barrier up to the canalized parts near Gouda
Inlet stop level	The water level at which the inlet near Gouda should stop
Load combinations	Different loads that occur together and create a load combination
Levee	An earthen dam located along rivers to protect the hinterland
Migration line	The line in which fish swim to pass the barrier (related to the velocity)
Morphological balance	The balance of sediments in a water system (basin, estuary or river)
Nationwide safety assessment	The assessment of all the levees in the Netherlands
Negative head	See head, but not in the governing load direction.
Non-closure probability	The probability that a non-closure event occurs when there is a request
Norm	Describes a certain safety for a failure mechanism (related to probability)
Occurrence	Number of times that a certain value occurs, related to return period
OR-port	Used in fault trees to specify that failure of one element leads to failure
Overall safety	The total safety in a dike ring expressed in economic damage or casualties
Piping	Flow of water in the aquifers under the structure or levee
Pump stop level	The water level at which the pumping stations stop discharging water
Reinforcement (concrete)	Steel rebars added to concrete to increase the tensile strength
Reinforcement (levee)	Ground or structures added to a levee to increase the strength
Return period	Certain value that occurs one time in a given period, related exceedance)
Ribbon development	Development of buildings predominantly on and along a levee
Rijkswaterstaat	Department of Public Works
Rijnmond system	The region around the mouth of the Rhine
Risk	The probability that something occurs multiplied with the consequences
Salt intrusion	Intrusion of salt water into fresh water rivers and polders along the coast
Sea level rise	The rise of the water levels at sea due to climate change
Secondary function	Function not directly related to the primary function (safety) of the barrier
Small scale water supply	The supply of water to the Rijnmond system using small canals
Storm surge	The piling up of water under a storm due to the low pressure
Storm surge barrier	A barrier that closes to protect the hinterland when a storm surge occurs

Tipping point	The point at which an extreme event becomes common
Urban river front	Front of the river were living, working and flood defence are integrated
Water balance	The total balance of the water in a certain system
Web plates	Steel plate welded to the web of a profile
Weibull distribution	A probability function used for the distribution of extremes

Symbols

Symbol	Unit	Explanation
$\Delta_{closure\ level}$	m	Difference between the old and new closure levels
ΔF	-	Deficit stability factor
$\Delta_{wind\ speed}$	m	Difference between the old and new increase due to the wind
$1-P_{ncl}$	-	Probability that the barrier closes
a	mm	Throat of the weld
A	mm^2	Cross-sectional area of the profile
A_{basin}	m^2	Surface of the Hollandsche IJssel
B	m	Width of the Hollandsche IJssel
C_2	-	Wind friction coefficient
CL	m NAP	Closure level of the storm surge barrier
d	m	Depth of the Hollandsche IJssel
dS/dt	-	Gradient of the water level
E	N/mm^2	Elasticity modulus
F	-	Stability factor
F_d	kN	Design force
f_y	N/mm^2	Yield stress
g	m^3/s	Gravity acceleration
H	kN	Total horizontal force acting on the barrier
h_{basin}	m NAP	Water level in the basin
h_{closed}	m NAP	Water level just after closure of the barrier
$h_{govening}$	m	Governing water level
H_s	m	Significant wave height
h_{open}	m NAP	Governing water level on the HIJ with an open barrier
h_{closed}	m NAP	Governing water level on the HIJ with an closed barrier
h_{sea}	m NAP	Water level at sea
$h_{set\ up}$	m	Set up due to the wind
h_{tide}	m	Added water level due to the tide
H_{wind_max}	m	Maximum wind set-up near Gouda
i	%	Interest per year
I_{lock}	-/hour	Intensity of ships that need to use the locks
I	mm	Moment of inertia of the profile
k	1/m	Wave number
L	km	Length of the tidal part of the Hollandsche IJssel
l_{ctc}	m	Center to center length between the struts
M	kNm	Resulting moment acting on the barrier
$M_{E;d}$	kNm	Design bending moment
$M_{R;d}$	kNm	Bending resistance of the profile
n	-	Number of closures of the Hollandsche IJssel barrier
N	kN	Normal force in the profile
p_0	kPa	Pressure due to the waves around the bottom
p_1	kPa	Pressure due to the waves around the water level
$P_{discharge}$	-	Probability that a certain discharge occurs
P_{ncl}	Per event	Probability that a non-closure event occurs
P_{norm}	-	Probability that a certain (water) level is exceeded

$P_{storm\ surge}$	-	Probability that a certain storm surge occurs
q_d	kN/m^2	Design load on acting on the gate
$Q_{0.10}$	m^3/s	Governing discharge for a sea level rise of 0.10 meter
$Q_{0.20}$	m^3/s	Governing discharge for a sea level rise of 0.20 meter
$Q_{0.35}$	m^3/s	Governing discharge for a sea level rise of 0.35 meter
Q_{Meuse}	m^3/s	Discharge of the Meuse
Q_{Rhine}	m^3/s	Discharge of the Rhine
Q_{tot}	m^3/s	Total discharge accumulating behind the barrier
S	kg	Sediment
t	h	Time
t_f	mm	Thickness of the flange
t_w	mm	Thickness of the web
T_p	s	Peak period of the waves
u	m/s	Wind speed
U_{max}	m/s	Water velocity above the scour protection
V_d	kN	Shear force
V	m^3	Volume Hollandsche IJssel
W	-	Weibull reduced variable
W_y	mm^3	Section modulus of the profile
z_b	m	Distance between the keel of the ships and the bottom
α	-	Shape parameter Weibull distribution
β	-	Shape parameter Weibull distribution
β	-	Reliability index
γ	-	Shift parameter Weibull distribution
λ	-	Ratio used for buckling
μA	m^2	Cross-sectional area New Waterway
ρ	kg/m^3	Density of a material
σ	N/mm^2	Stress in a profile

Abbreviations

Abbreviation	Full description
AOR	Open Closable Rijnmond
BS	Closure reliability
Ca	Capelle
CDF	Cumulative distribution function
CL	Closure level
ctc	Center to center
DC2	Second Delta Committee
DR14	Dike ring 14
DR15	Dike ring 15
EHS	Ecological Main Structure
FV	Future value
HHS&K	Water Board Schieland en Krimpenerwaard
HIJ	Hollandsche IJssel
HRC2006	Hydrological boundary conditions 2006
HT	Height
HW	High water
IPCC	Intergovernmental Panel on Climate Change
KNMI	Koninklijk Nederlands Meteorologisch Instituut
KWA	Small scale water supply
Log.	Logarithmic function
MCE	Multi criteria evaluation

ML	Maeslant
Mo	Moordrecht
NAP	Normaal Amsterdams Peil
NHW	Normative high water
nHWBP	New flood protection program
NWO	Non water retaining object
PDF	Probability density function
PV	Present value
R&D	Rijnmond and Drechtsteden
RP	Return period
SBW	Strength and loads on the flood defences
SLR	Sea level rise
STBI	Inner slope stability
STBU	Outer slope stability
STCG	Strength of the water retaining part of the structure
STCO	Strength of part of the structure
STMI	Micro instability
STPH	Piping stability
STVL	Foreland stability
TOI	Assess and design guidelines
TP	Assessed water level
UC	Unity check
VNK	Safety in the Netherlands
VTV2006	Safety assessment regulations 2006
WTI2017	Legal assessment methods 2017
WV21	Flood safety in the 21st century

Figures and tables

Figures

Tables

Appendices

Appendix A Flood protection philosophy Netherlands [6]

The low lying parts of the Netherlands are always threatened by floods. Flood protection measures have been taken since the middle ages. This appendix gives a brief overview of the history and developments in flood protection.

Development safety philosophy

In the middle ages large part of the lands were not defended against floods that occurred regularly. In the low lying country of the Netherlands mounds were created or high river banks were used to build villages. The water management and flood protection of a region was regulated by local villages. In 1255 earl Willem I decided that different villages needed to work together; this resulted in the first water board, Rijnland. Since then water boards ensure the flood protection of a region and manage the water system [51]. During the centuries there were hundreds of water boards, this number is brought back to 27 water boards in 2010.

Levees were used to defend part of the land from the river or sea. When a flood defence was breached the new levee was build a little higher than the original levee. This new levee was built behind part of the breached levee; scour holes on the original site of the levee made it difficult to rebuild the levee on the exact same spot. This method of reinforcement (little higher and behind the original site) was maintained up to the flood disaster in 1953 which resulted in a lot of casualties and economic damage.

Before the Second World War civil engineer Johan van Veen warned for the low levees in the South-western part of the Netherlands. In 1940 the committee "Stormvloedcommissie" confirmed these warning. The Second World War and rebuilding of the Netherlands prevented large levee reinforcements. After the flood disaster of 1953 the First Delta Committee was instated, this committee researched the safety of the Netherlands and advised to close many of the river branches (shown in Figure 68). The first Delta work was the Hollandsche IJssel barrier in 1958 the last was the Maeslant barrier in 1997.

Figure 68 - Overview Delta works, source; Wikipedia

The delta committee also researched the optimum safety level that should be used in Central Holland. In an econometric study eventually the failure probability of 1/125 000 (per year) was found to be the optimum (in that time). Politics translated this optimum into an exceedance probability of 1/10 000 that causes failure in 10% of the cases. The exceedance probabilities of other regions are all derived from the study conducted in Central Holland, which became dike ring 14 (shown in Figure 69). This derivation is done based on the sea or river threat. A threat from the river is predicted in advance (1-2 weeks) while the threat from the sea is not known that far in advance (24 hours). A dike ring is a series of connected flood defences which protect a lower lying area.

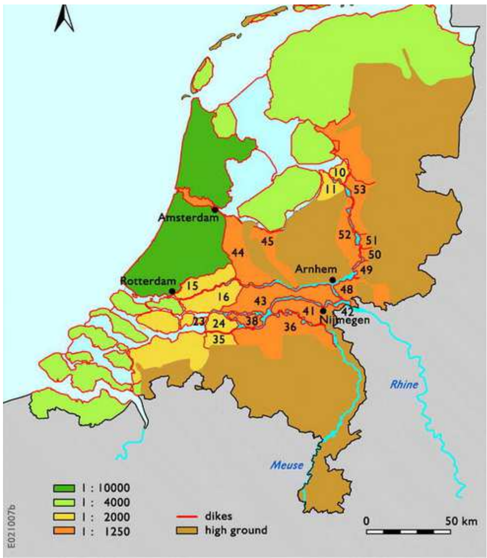


Figure 69 - Dike rings in the Netherlands, source; worldpress.com

After the high discharges and accompanying floods on the Rhine and Meuse in 1993 and 1995 the attention was drawn to the problems in the basins of the rivers. The program Room for the River was instated to increase room for the river. Obstructions in the basins were removed and emergency overflow areas were designated.

The levees in the Netherlands are assessed every 6 years (laid down in the Water Act). This assessment can be seen as the APK for levees. The third nationwide safety assessment was conducted between 2006 and 2011. The report in 2011 showed that a large part of the flood defences does not meet the standards (including the levees along the Hollandsche IJssel). In this assessments the levee are assessed for the period up to the next assessment round (design of new levees happens for a period of 50-100 years). Assessments are based on governing water levels that are compiled before the assessment (HRC 2006). The HRC 2006 compiles the governing water levels for each of the exceedance probability (1/10 000, 1/4 000 etc.). Only the height of the levee is assessed with a probabilistic calculation, all the other failure mechanisms (appendix E1) are assessed using model factors. Model factors include all the uncertainties. All the regulations concerning the assessments are laid down in the VTV2007 [12].

After each assessment round the government creates budget for the reinforcement of the levees that were not up to the standard. The program after the third assessment is the new flood defence program (nHWBP); the budget is limited therefore only the minimal costs are reimbursed.

In 2007 the Second Delta Committee was instated, this committee studied the effects of climate change, in specific the sea level rise, for the next centuries. Result of this study is a list of recommendations for the next century. The Dutch Government adopted large parts of these recommendations. They instated the Delta Program which is the important program behind different developments in the flood protection field. The developments after the second delta committee are described in appendix A.2.

Developments after the Second Delta Committee (DC2)

After the Second Delta Committee different programs were started (shown in Figure 70), the important programs are the Delta Program and the Safety in the Netherlands (VNK) program.

Figure 70 - Schematic overview different programs, source; VNK2

The VNK programs study the situation from a risk perspective. Risk is determined as the failure probability multiplied with the consequences. For the failure probability of a levees all the failure mechanisms need to be assessed with a probabilistic calculation (not just the height). The consequences of flooding are given as the expected economic damage and casualties. Results of the VNK programs are used to prioritize the reinforcements, which reinforcements decrease the risks the most. The Delta program researches all the important regions designated by DC2. The delta program Rijnmond-Drechtsteden studies the situation in this region; different sub programs like the Hollandsche IJssel treat different aspects of this system.

The fourth nationwide safety assessment is conducted between 2011 and 2023; the report is presented to the government in 2023. This safety assessment should take advantage of the developed probabilistic methods.

Appendix B Traffic flows

This appendix studies the traffic flows on and crossing the Hollandsche IJssel. The first part treats shipping; the second part treats traffic crossing the bridge.

Appendix B.1 Channel dimensions

Appendix B.1.1 Width channel

The width of the channel is calculated using the guideline for shipping canals in the Netherlands [27].

The minimum width of a double channel is 3 times the width of the normative ship plus the crosswind surcharge is 3*14+5 = 47 m.

Appendix B.1.2 Depth of the sill

The depth of the sill is calculated using the guideline for shipping canals in the Netherlands [27].

The minimum depth of single channel is the low water level (LLWS - 1.3 times the depth of the normative ship is -0.50- 3*1.3 = -4.4 m NAP. The depth of the sill is located at -6.5 m NAP therefore there are no problems.

Appendix B.1.3 Vertical clearance

The vertical clearance is calculated using the guideline for shipping canals in the Netherlands [27]. The highest water level at the inner side of the Hollandsche IJssel storm surge barrier is equal to the pump stop level which is +2.0 m NAP.

The height of a container ship with stacks of three containers is 7 m, for a ship with 4 stacks it is 9.1 m.

The vertical clearance for 3 stacks becomes +9.0 m NAP. The vertical clearance for 4 stacks becomes +11.1 m NAP.

Appendix B.2 Bridge dimensions

The dimensions of the bridge (or tunnel) are calculated using the guideline for tunnel design in the Netherlands [52].

Table 36 - Bridge part dimensions, source; Rijkswaterstaat

Bridge dimensions	Width			
Bicycle lane	2.5	m		
Strip between cyclists and vehicles	1.0	m		
Lane width	3.5	m		
Emergency lane	2.25	m		
Side stripe	0.2	m		
Dividing stripe	0.15	m		
<u>Type bridge</u>				
2*2 lanes + bicycle lane			26.9	m
3*3 lanes			34.2	m
3*3 lanes + bicycle lane			37.7	m

Appendix B.3 Economic damage container ships

Every day there are 120 ship movements on the Hollandsche IJssel it is expected that this grows to 200 ship movements in the future, because of the increase of the container terminal (and Heineken brewery) [25]. This means that during a closure of the storm surge barrier a large part of this ships need to pass the barrier using the lock situated next to the barrier. It is assumed that 75% of the ships in the Hollandsche IJssel pass the storm surge barrier, this number of ships results in waiting time at the Algera Lock.

The waiting time is large when the barrier is closed because the capacity of one lock chamber is not enough. The economic damage can be calculated using the waiting time for ships and the costs per hour. The economic damage due to delay of a container ship is approximately 175 euros per hour [39].

The average waiting time of a container ship at the locks is calculated using the number of ships per hour (n) and the lock capacity per hour. It is assumed that a total lock cycle lasts approximately 20 minutes. This means that the values per hour are calculated using;

$$i_{lock} = \frac{1\ hour}{one\ lock\ cycle} = \frac{60}{20} = 3\ ships\ per\ hour$$

$$i_{ships} = \frac{0.75 * n}{24\ hours} = \frac{150}{24} = 6.25\ ships\ per\ hour$$

The average waiting time of ships is calculated using the difference in number of ships per hour and the time one lock cycle lasts.

$$t_{waiting} = \left(i_{ships} - i_{locks}\right) * t_{lock\ cycle} = (6.25 - 3) * 20 = 65\ minutes = 1.08\ h$$

The economic damage is then calculated by multiplication of the waiting time per ship, the number of ships passing the barrier and the economic damage per hour.

$$q_{closure\ 24\ hours} = t_{waiting} * 0.75 * n * q_{container} = 1.08 * 150 * 175 = 28\ 350\ euros\ per\ closure$$

Table 37 - Economic damage due to delay of the shipping

	Number of closures [per year]*	Economic damage [million euros per year]
Adaptation	10	0.29
Renewal	10	0.29
Damming	365	10.3

*expected number of closures in 2050 [11]

With approximately 10 closures per year the economic damage is low and there is no need to construct a second lock chamber. If the system is dammed however the economic damage per year is considerable and the construction of a second loch chamber is feasible. The new lock chamber of the Juliana Lock costs approximately 35 million euros [53]. Second reason for the construction of a second lock chamber is that the “Nota Mobiliteit”, written by the department of Public Works, prescribes that waiting times for locks may not be more than 30 minutes [54]. When there are 2 lock chambers the intensity increases to 6 ships per hour and thus an average waiting time of approximately 30 minutes.

Appendix C Costs alternative strategies

The evaluation of the costs per strategies is conducted using the net present value of the money needed. The net present value can be calculated using the following formula:

$$PV = \frac{FV}{(1+i)^t}$$

PV is the present value of money
FV is the future value of money
i is the difference between inflation and interest
t is the number of years

The costs for the Hartel Barrier are used for the construction costs of the new storm surge barrier. The width of the two storm surge barriers is comparable and both barriers use a gate with a large span. The costs for the Hartel Barrier were 143 million euros in 2009, with the inflation of 2010 (1.93%), 2011(2.38%) and 2012(2.90%) this gives 154 million euro [55, 56]. The costs for a dam are expected to be 400 million; this dam is expensive because of the pumping station that is needed and the navigation locks that are needed to let ships pass the dam [11].

The net present value of the construction in the long term is calculated back to the base year (end of 2012). Structures which are postponed are built in 2060 that means that t is 48 years; the interest rate is 4% inflation is 2%. The net present value of both the new storm surge barrier and dam in 2060 are 60 and 154 million respectively.

The costs needed for the reinforcement of the levees is researched for the Delta Program (sub program Hollandsche IJssel), the result of these studies is shown in Figure 71 [11]. These reinforcements are executed in the near future, it is schematized that all the money is spent in 2012 (no calculations of the net present value).

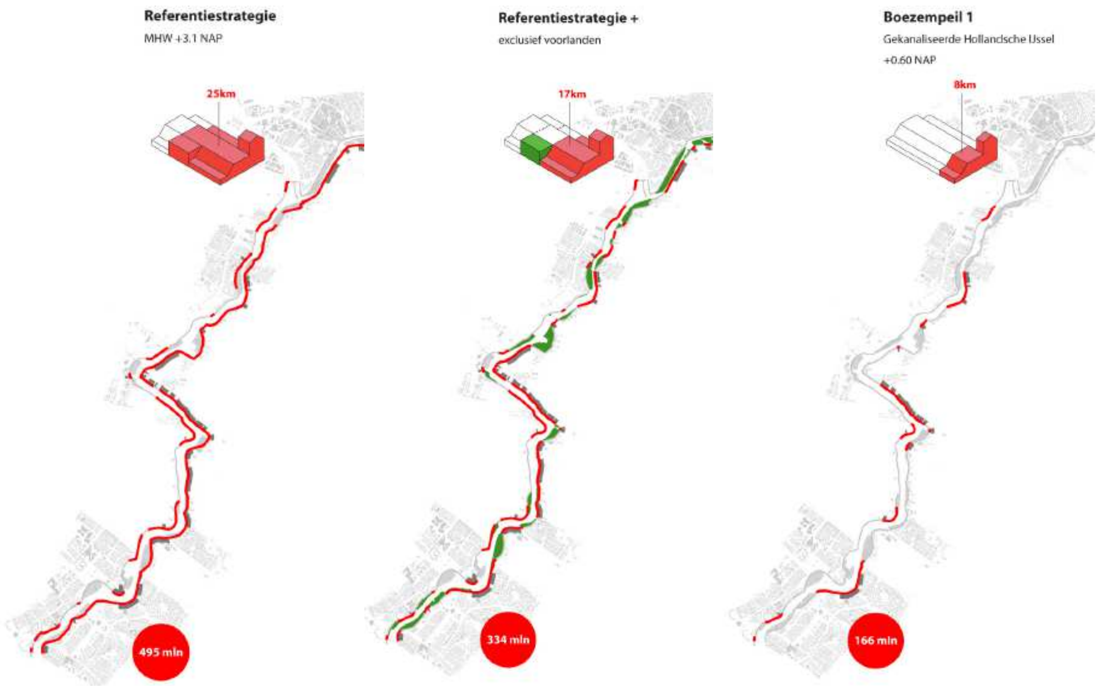


Figure 71 - Overview costs reinforcements, source; Delta Program

The costs for the adaptation of the storm surge barrier are estimated based on the report of HKV which investigated the costs for the adaptation of the storm surge barrier as part of a brain storm session [18]. The costs for the adaptation depend for a large part on the gate. If the gates needs to be replaced this costs a lot of money. To increase the non-closure probability up to 1/500 an investment of 25 million euros is expected, with the possible structures needed for salt intrusion this figure is doubled to 50 million euros for the adaptation of the storm surge barrier.

The adaptation of the canalized Hollandsche IJssel and KWA is expected to cost 20 million euros. In the thesis of F.Bulsink it is expected that the adaptation of the KWA costs 10 million euros, the same amount of money is reserved for the optimization of the canalized Hollandsche IJssel as part of the small-scale water supply [13].

Table 38 - Expected costs in million euros, price level 2012

Costs: Strategies	Total	Reinforcement	Adaptation	New storm surge barrier	Dam + new locks	Salt intrusion
0 Doing nothing	495	495	-	-	-	-
1A Adaptation and damming	543	318	50	-	155*	20
1B Adaptation and renewal	448	318	50	60*	-	20
2 Renewal	492	318	-	154	-	20
3 Damming	586	166	-	-	400	20

**calculated using the NPV with base year 2012 and year spend 2058*

Appendix D Secondary functions

The different secondary functions are divided in five groups. The traffic function of the storm surge barrier is not addressed in this appendix because it is already treated in appendix B.

1 Economic development

The group "economic development" focuses on secondary functions which are economically interesting to build in combination with a storm surge barrier. Different economic development options are; houses, offices and a shopping mall for example. The purpose of these options is to reduce the costs and create a structure which fulfills more functions. The design is affected by both the design aspects of the dam or barrier and of the secondary function. An example of a possible economic development is the Waterslot in't Spui. This Building Engineering master thesis combined a storm surge barrier with houses and offices (shown in Figure 72).

Figure 72 - Waterslot Spui, source; thesis A.Dijk

2 Added benefit

The group "added benefit" focuses on secondary functions which have a benefit to society. The secondary functions are possible to create within the storm surge barrier and add something to the local surrounding of the storm surge barrier. Different added benefit options are; museum, exhibition space, watchtower and a restaurant. The purpose of these options is to increase the added benefit of the storm surge barrier or dam. The costs of the storm surge barrier or dam are not reduced but something other than money is introduced. The design is primarily affected by the performance; the other functions have a limited influence. An example of an added benefit is a restaurant on a special place, shown in Figure 73 for example.

Figure 73 - Restaurant Maldives, source; Hilton

3 Small scale use

The group "small scale use" focuses on secondary functions which can be maintained in the storm surge barrier. Different small scale uses are; billboards and climbing wall. The purpose of these options varies. The billboards are primarily meant to generate money; the climbing wall is more an added benefit. These functions do not affect the design of the storm surge barrier. An example of small scale use is for example the advertisement on the existing Hollandsche IJssel storm surge barrier (shown in Figure 74).

Figure 74 - Advertisement Rijkswaterstaat on the existing storm surge barrier

4 Large scale use

The group “large scale use” focuses on the use of the empty space. Different “large scale uses” are; fish farm, container storage and parking garage. The purpose of these options is the use of the space that is not needed for the performance of the storm surge barrier and can be used for other functions. The inner side of a tower can for example be used to store raw materials. An example is the creation of a fish farm in the towers (shown in Figure 75).

Figure 75 - Fish farm, source; Google

5 Ecology options (power station, pumping station or

The group ecology options focuses on the use of the dam (or storm surge barrier) as option to enhance (or maintain) the ecology in the Hollandsche IJssel. Different “ecology options” are; power station, pumping station and fish ladder. The purpose of these options is to increase the ecology and create energy for example. An example is the construction of a tidal power station in the dam, shown in Figure 76.

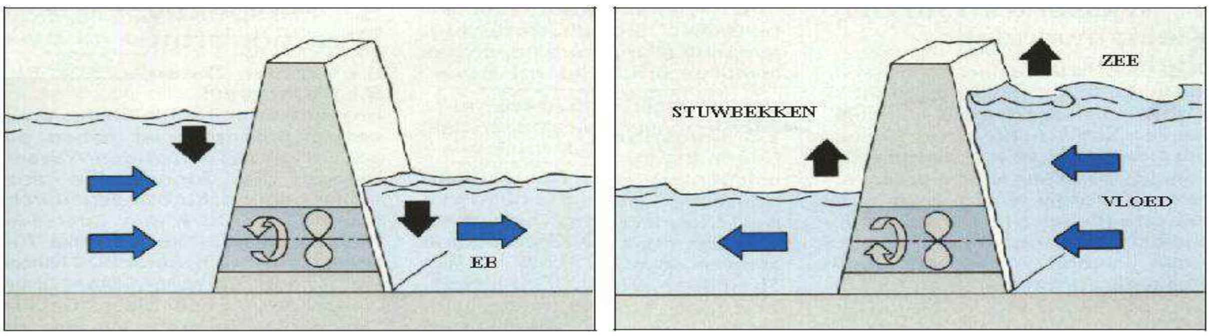


Figure 76 - Tidal power station, source; www.joostdevree.nl

Appendix E Safety analysis flood defences

This appendix describes the safety of the flood defences along the Hollandsche IJssel. The first part describes the failure mechanisms; the second part describes the safety of the flood defences.

Appendix E.1 Voorschrift toetsen veiligheid 2007; failure mechanism

The failure mechanisms that need to be assessed accoring to the guidelines are displayed in Figure 77 and summarized below. The accurate description of all the failure mechanims is found in the VTV guidelines of 2007 [12].

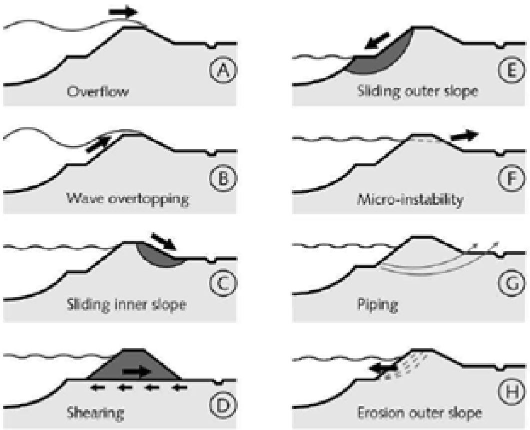


Figure 77 - Failure mechanisms flood defence, source; lecture notes CIE5314

- Overflow and overtopping (HT)
- Piping/heave (STPH)
- Sliding outer slope (STBU)
- Sliding inner slope (STBI)
- Micro instability (STMI)
- Revetment (STBK)
- Foreland (STVL)
- Non water retaining objects (NWO)

In the VTV each failure mechanism is assessed on the important load combinations for that mechanism. In general there are three important load combinations that are schematized and shown in Figure 78;

1. Governing high water on the river (blue line); due to a storm surge or flood wave there are high water levels on the river and therefore in the levee.
2. Fall of the water level after governing high water level (red line); the ground water level in the levee reacts slow to the change of the water level on the river. Especially outer slope stability is affected by this prehenomena.
3. Extreme precipitation (green line); due to precipitation the ground water level slowly rises, when the polder behind the levee lies low it lasts longer before the levee is saturated.

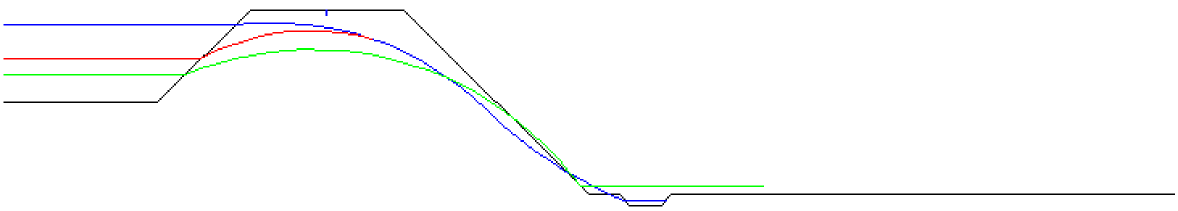


Figure 78 - Load combinations schematized levee

The decrease of the governing water levels is important for load combination 1; the other load combinations are not affected. Table 39 briefly describes the effect of the decrease for the different failure mechanisms.

Table 39 - Influence because of the decrease in governing water levels

Failiure mechanisms	Influence
Overflow and overtopping (HT)	Positve influence because a lower water level means more freeboard for overtopping.
Piping and heave (STPH)	Positive influence because the pressure of the uplifting water reduces.

Sliding outer slope (STBU)	No influence because a low water level influences the stability of the outer slope.
Sliding inner slope (STBI)	Positve influence because lower water levels mean lower horizontal forces and lower water levels inside the levee. The influence is limited due to the other load case extreme precipitation which is independent of the governing water level.
Micro-instability (STMI)	No significant influence the soil structure and drainage are more important.
Revetment (STBK)	No significant influence the revetment is primarily meant for shipwaves on the Hollandsche IJssel.
Foreland (STVL)	Positive influence because the lower water levels mean lower shear stresses inside the foreland.
Non-water retaining objects (NWO)	There can be some influence when the water levels are decreased because the loads on the theoretical profile of the levee are lower. The theoretical profile is the profile without the earth that could possibly be removed by non-water retaining objects, a tree which is uprooted for example.

Table 39 shows that there is no negative influence when the governing water levels are decreased.

Appendix E.2 Analysis failure mechanism inner slope stability

The water board HHS&K investigated the possibilities to reduce the governing water levels. The decrease of the governing water level should increase the stability factor F of the levees and therefore prevent reinforcement of the levees. The VTV guidelines prescribe a minimum stability factor F of 1.17 for the levees along the Hollandsche IJssel.

$$F = \frac{Resisting\ force}{Driving\ force} \geq 1.17$$

The resisting force is the shear force that is created along the slip circle shown in Figure 79. The driving force is the weight of the soil above the slip circle. The water board divided the levees in the Hollandsche IJssel in four stretches study (shown in Figure 82) with each on representative profile. The four profiles have a small outer slope, a very steep inner slope and a large foreland. The representative profile of stretch 1 is shown in Figure 79.

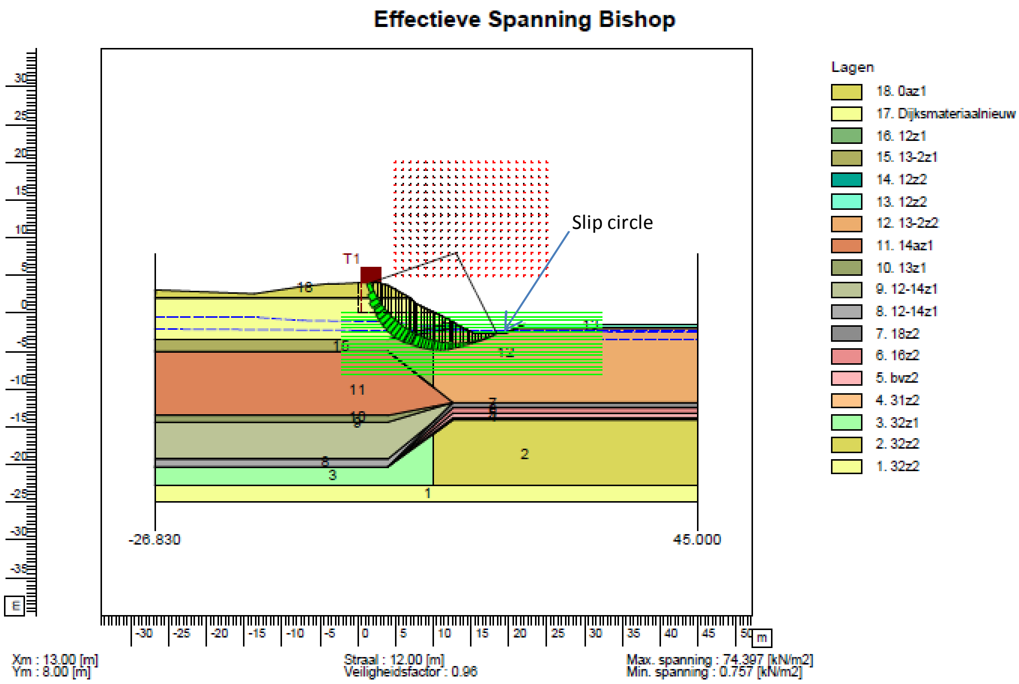


Figure 79 - Representative profile Ca37 stretch 1, source; water board HHS&K

The Water Board HHS&K recalculated the stability factor F for a few new water levels; the results of these calculations are shown in Table 40 [30].

Table 40 - Recalculation stability factor F, source; water boards HHS&K

Profile		NHW	$F_{assessment}$	$F_{recalculation}$ en ΔF							
		[m NAP]		TP -0.6m		TP -1.6m		NAP +0.6m		NAP -0.6m	
1	Ca37	+2.60	0.83	0.86	+4%	0.90	+8%	0.91	+10%	0.96	+16%
2	Mo26	+3.00	1.00	1.03	+3%	1.07	+7%	1.09	+9%	1.12	+12%
3	30.9+73	+2.75	0.69	0.75	+9%	0.82	+19%	0.86	+25%	0.92	+33%
4	36.9+26	+2.95	0.85	0.90	+6%	0.96	+13%	1.00	+18%	1.05	+24%

The stability factors obtained from the water board HHS&K are linearized for each profile. Figure 80 shows the effect of the decreasing water levels. The four representative profiles all show a steep line downward.

$$Profile\ Ca37, strech\ 1 = \frac{1\ meter\ decrease}{slope\ of\ the\ trendline} = \frac{1}{25} \approx 0.04$$
$$Profile\ Mo26, strech\ 2 = \frac{1\ meter\ decrease}{slope\ of\ the\ trendline} = \frac{1}{30} \approx 0.03$$
$$Profile\ 30.9 + 73, strech\ 3 = \frac{1\ meter\ decrease}{slope\ of\ the\ trendline} = \frac{1}{14} \approx 0.07$$
$$Profile\ 36.9 + 26, strech\ 4 = \frac{1\ meter\ decrease}{slope\ of\ the\ trendline} = \frac{1}{18} \approx 0.06$$

The increase of the stability factor F per decrease of the water level is obtained from the formulae shown in Figure 80. The average increase of the stability factor for a decrease of the water levels for 1 meter is 0.05. Therefore a very large decrease of the water level is needed before significant reinforcements can be prevented. It is thought that predominantly the steep inner slopes limit the influence of the water levels. It should be kept in mind however that the linearization of the stability factor against the decrease of the water level is a crude approximation.

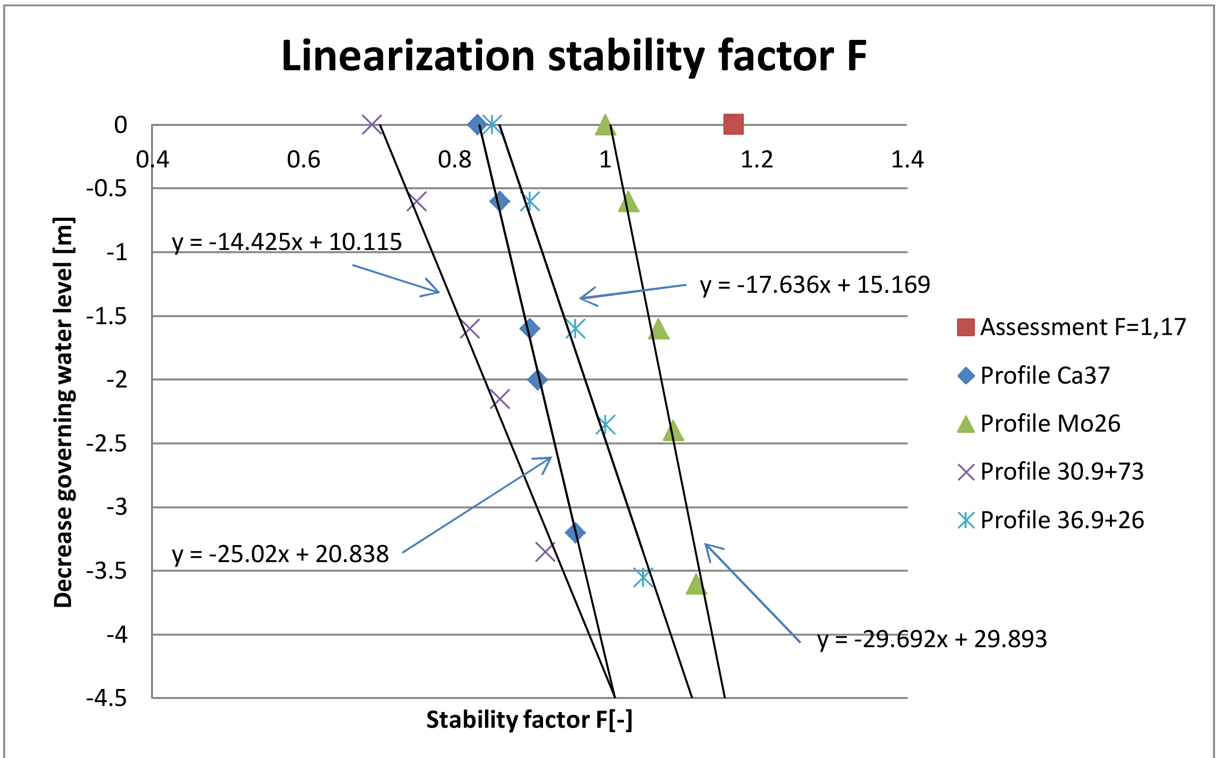


Figure 80 - Linearization Stability Factor

The different failure classes mentioned in section 4.1.1 are based on Figure 81. The deficit of the stability factor is shown in different colors along the Hollandsche IJssel. The black lines indicate that another failure mechanism was governing; in most cases the height was insufficient (deficit of less than 0.5 meters).

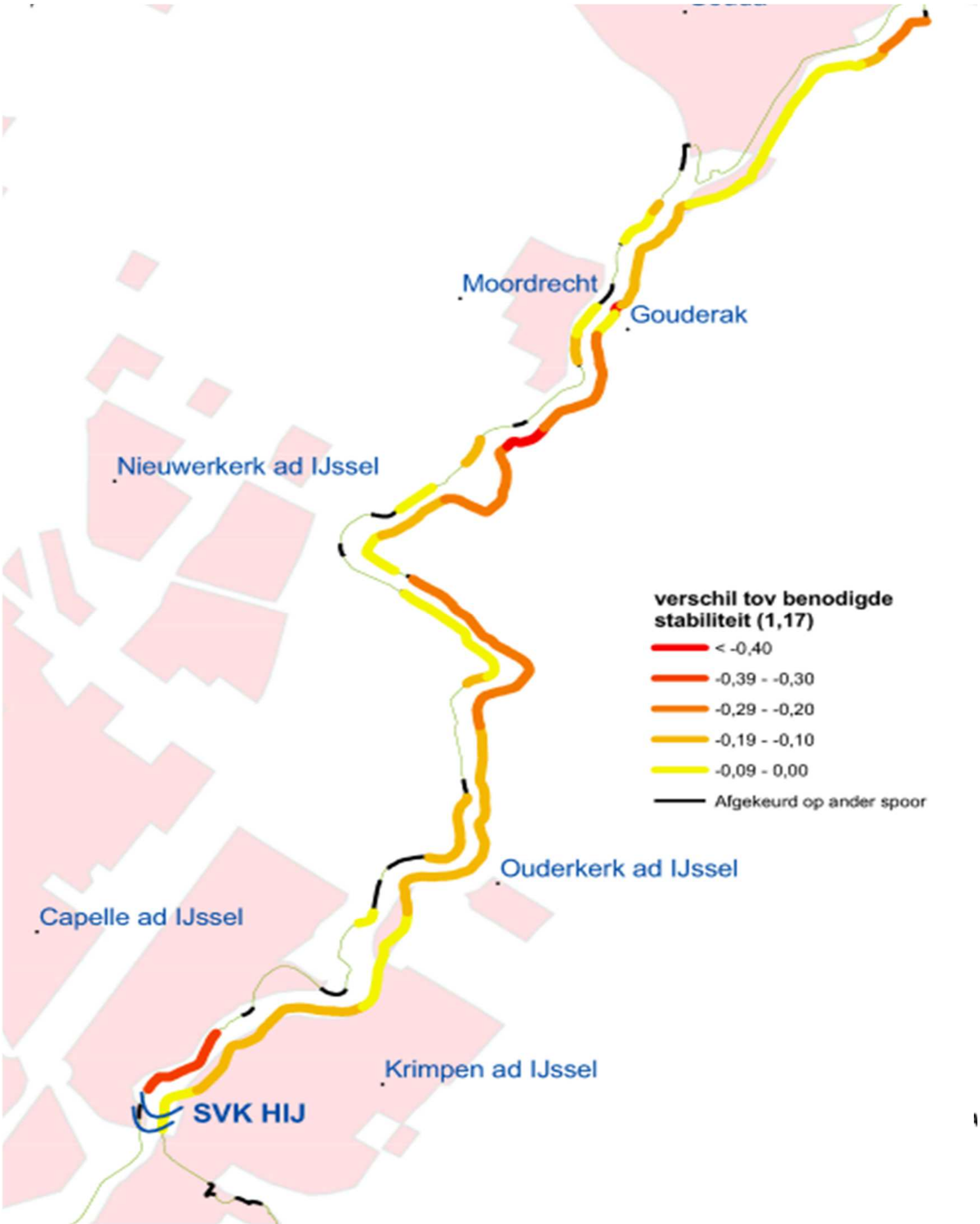


Figure 81 - Deficit stability factor F, source; Van der Kraan

Appendix E.3 Overall safety analysis DR14 and DR15

The improvement of the overall safety of dike ring 14 and 15 is linked to the reduction of the risk of flooding. When the risk of flooding is reduced the expected economic damage and casualties decrease and thus the overall safety in the dike ring increases. The risk of flooding is based on two components:

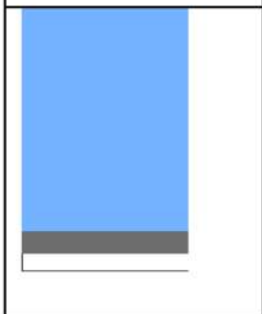
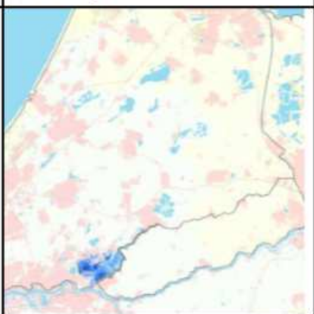
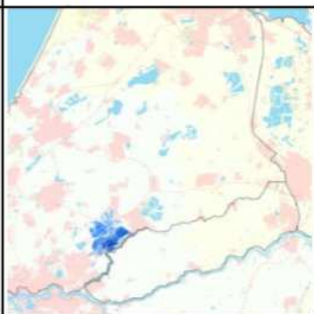
- Consequence; economic damage and casualties
- Probability of failure; likelihood that a section of the dike ring fails

Figure 82 - Four levee sections and profiles Hollandsche IJssel, source; W ter Horst

For the determination of the overall safety in dike ring 14 and 15 the flood defences along the Hollandsche IJssel were divided in four parts (same four parts are used in appendix B.2) and inner slope stability is set as the governing failure mechanism [5]. For each of the four profiles different consequences were calculated. The failure probabilities of levees are hard to calculate, calculations made for VNK showed that the levees should have failed under daily circumstances [5]. Eventually it was chosen to use a failure probability of 1/100 per year based on engineering judgment [31].

The starting point in the calculations is the failure probability of 1/100 and the consequences mentioned in the report of VNK (shown in Table 41).

Table 41 - Consequences breaches levees Hollandsche IJssel, source; VNK [5]

	Breach in stretch 1 near Capelle	Breach in stretch 2 near Nieuwekerk	Breaches in stretch 3 and 4
			
Economic damage [billion €]	2.1	0.3	0.8
Casualties	35 - 145	3 - 13	16 - 68

$$Risk = Probability * Consequence$$

Table 42 - Risks due to a breach in the levees along the Hollandsche IJssel

	Stretch 1	Stretch 2	Stretch 3	Stretch 4
Economic damage [million €/year]	21	3	8	8
Casualties [lifes/year]	0.90	0.08	0.54	0.54

The analysis of appendix B.2 is based on a set of stability factor combined with water levels. The reliability index (which directly relates to the failure probability) can be obtained with the use of a assumption used for Dutch levees [32, 33].

$$F_d = 1 + 0.13 * (\beta - 4)$$

The results of this formula cannot be used because of the same reason as mentioned in the beginning of this section (unrealistic failure probabilities). The difference between the different reliability indexes can however be used to give an estimation of the trend (decrease in governing water level versus reliability index β). This trend can then be used to express the reduction in risks when the water levels are decreased. Table 43 shows the stability factor that is assessed for different water levels.

Table 43 - Stability factor F for different water levels

Profiles	Length [m]	NHW [m NAP]	Safety assessed [F]	NHW -0.6 [F]	NHW -1.6 [F]	NAP +0.6 [F]	NAP -0.6 [F]
Profile 1	10 700	2.60	0.83	0.86	0.90	0.91	0.96
Profile 2	6 700	3.00	1.00	1.03	1.07	1.09	1.12
Profile 3	9 800	2.75	0.69	0.75	0.82	0.86	0.92
Profile 4	8 300	2.95	0.85	0.90	0.96	1.00	1.05

The stability factor F can be converted into the reliability index β with the formula mentioned above, results are shown in Table 44 and Figure 83.

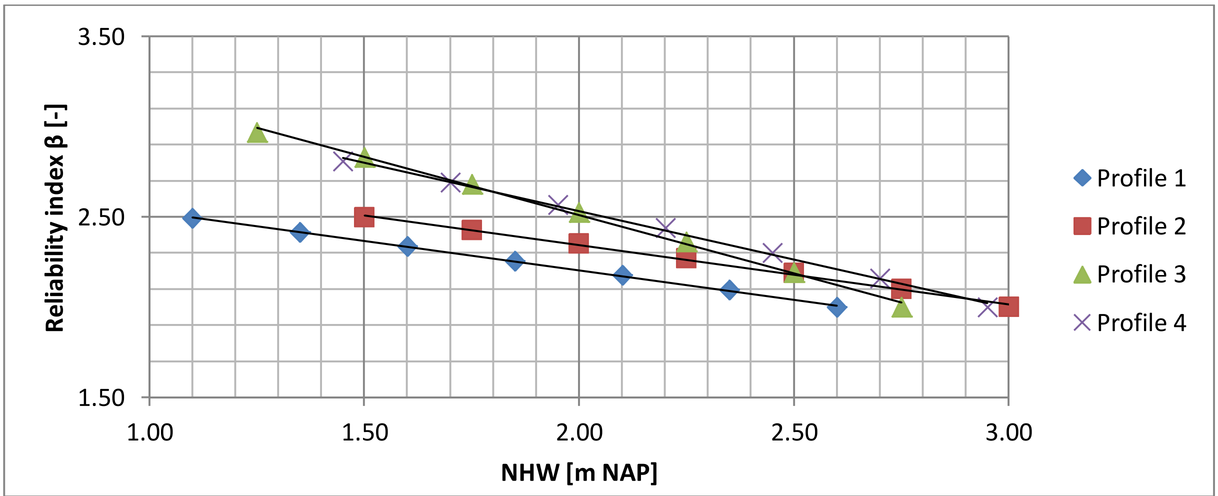


Figure 83 - Reliability index β for different governing water levels

Table 44 - Reliability index β for different water levels

Profiles	Length [m]	NHW [m NAP]	Safety assessed [β]	NHW -0.6 [β]	NHW -1.6 [β]	NAP +0.6 [β]	NAP -0.6 [β]
Profile 1	10 700	2.60	2.69	2.92	3.23	3.31	3.69
Profile 2	6 700	3.00	4.00	4.23	4.54	4.69	4.92
Profile 3	9 800	2.75	1.62	2.08	2.62	2.92	3.38
Profile 4	8 300	2.95	2.85	3.23	3.69	4.00	4.38

The last part of the configuration is that the reliability index for the safety assessed β is set to 2 (failure probability of 1/100), the other β are given as the difference between the old and new index. The safety assessed for profile 1 is 2.69 for NHW -0.6 this is 2.92. The new safety assessed for profile 1 is 2.0 for NHW – 0.6 this becomes 2 + (2.92 – 2.69) = 2.23. The other results are given in Table 45.

Table 45 - Normalized reliability index β for different water levels

Profiles	Length [m]	NHW [m NAP]	Safety assessed [β]	NHW -0.6 [β]	NHW -1.6 [β]	NAP +0.6 [β]	NAP -0.6 [β]
Profile 1	10 700	2.60	2.00	2.23	2.54	2.62	3.00
Profile 2	6 700	3.00	2.00	2.23	2.54	2.69	2.92
Profile 3	9 800	2.75	2.00	2.46	3.00	3.31	3.77
Profile 4	8 300	2.95	2.00	2.38	2.85	3.15	3.54

The reliability index is changed to failure probabilities with the use of the Normal distribution. The decrease of the governing water levels is assumed to be the same along the entire Hollandsche IJssel and is normalized to the NHW.

$$\beta = -\log(P_f)$$

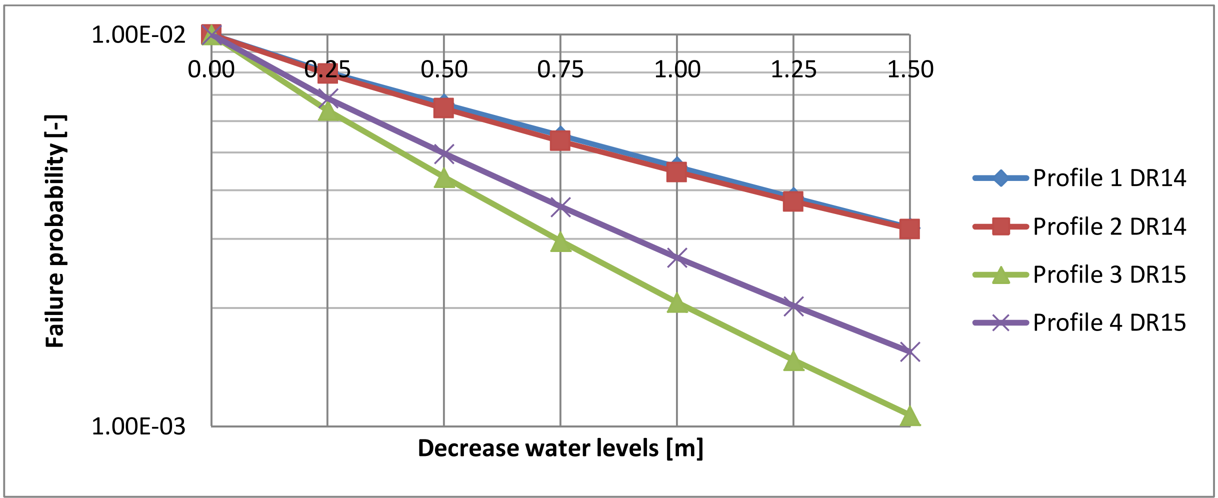


Figure 84 - Normalized failure probabilities

The result of the analysis is shown in Figure 84. The results seem reliable because the decrease of the failure probability is not that big (factor 10). When the failure probability would decrease a lot more reinforcements could have been prevented. The two profiles from dike ring 14 show a similar trend, the profiles from dike ring 15 also show a trend. The dike ring 15 profiles diverge from each other when the water levels decrease more.
When the failure probabilities and the consequences are multiplied the risk reduction is obtained, shown in Table 46 and Figure 29.

Table 46 - Risk reduction due to a decrease in the governing water levels

	Risks DR14		Risks DR15	
Decrease water level	Economic damage/ yr	Casualties/ yr	Economic damage/ yr	Casualties/ yr
0.00 m	24.00	0.980	16.00	1.080
0.25 m	19.22	0.785	10.63	0.718
0.50 m	15.91	0.650	7.43	0.502
0.75 m	13.20	0.540	5.28	0.356
1.00 m	11.00	0.450	3.82	0.258
1.25 m	9.19	0.376	2.80	0.189
1.50 m	7.70	0.315	2.10	0.141

Especially the decrease of the risks in DR14 is large as the Hollandsche IJssel is a major contributor to the total risk; the reduction of the risks in DR15 is rather small compared to the contribution of the Lek levees (shown in Figure 86).

Appendix E.4 Results decrease water levels

The graphs shown in Figure 85 show the results of the calculations conducted for dike ring 14 (by the program VNK) in the left graph and the results of the calculations conducted in this study on the right (when the water levels are reduced with 0.5m).

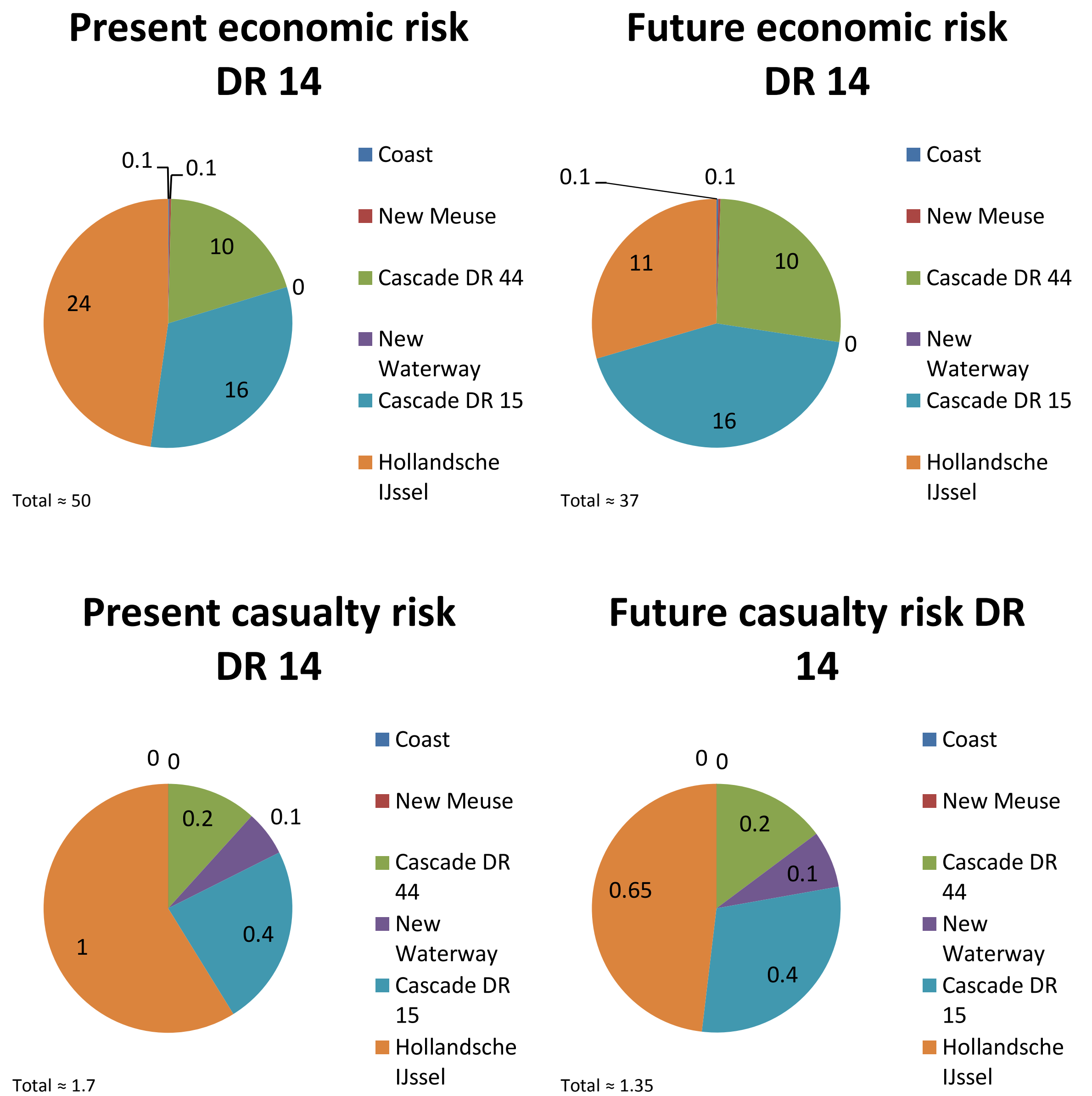


Figure 85 - Contribution to the safety of dike ring 14 [million euros/year and casualties/year], source; VNK2

The graphs shown in Figure 86 show the results of the calculations conducted for dike ring 15 (by the program VNK) in the left graph and the results of the calculations conducted in this study on the right (when the water levels are reduced with 0.5m).

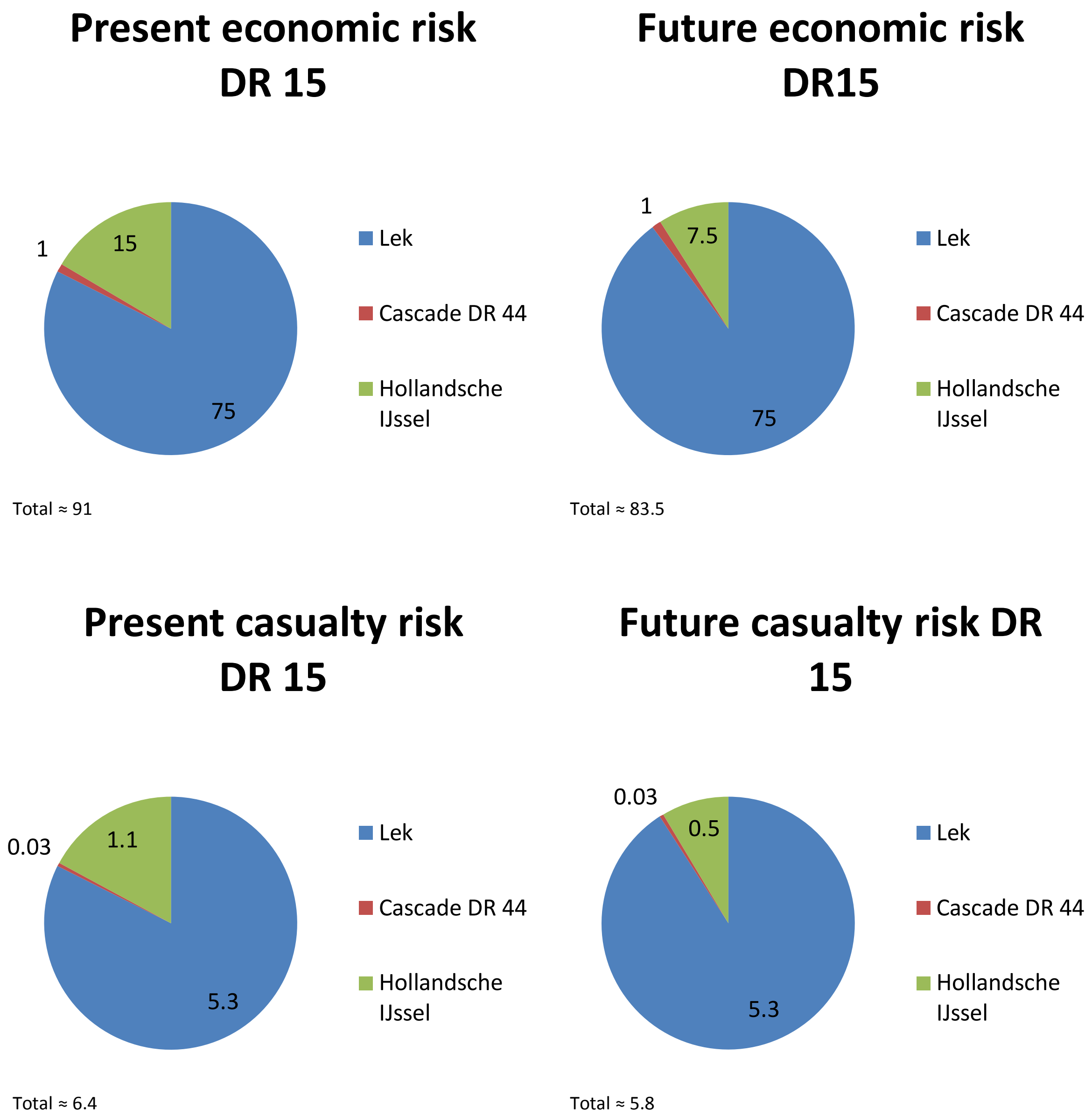


Figure 86 - Contribution to the safety of dike ring 15 [million euros/year and casualties/year], source; VNK2

Appendix F Water level analysis

This appendix describes the water levels in the Rijnmond and Hollandsche IJssel system. The first part describes the water levels on (and in front of) the Hollandsche IJssel; the second part treats the non-closure probability and its relation to the water levels.

Appendix F.1 Climate change

There are different institutions that study the climate change. The reports used in this report are from the KNMI and the IPCC. The sea level rise due to the climate change is described for the situation in 2010, 2050 and 2100 [1, 7, 8]. The uncertain sea level rise is used to study the effects of the different strategies and adaptations.

Table 47 - Expected sea level rise according to the KNMI W+ and IPCC scenarios

Situation	Current increase	KNMI W+	IPCC
2050	0.10 m	0.35 m	0.35 m
2100	0.20 m	0.85 m	1.20 m

Appendix F.2 Tide

The information about the tide is obtained from the website of the department of Public Works [37].

Appendix F.2.1 Hook of Holland

Table 48 - Tidal levels Hook of Holland, average discharge 2 200 m^3/s

Type	High water	Low water	Difference
Average tide	+1.11 m NAP	-0.63 m NAP	1.74 m
Spring tide	+1.30 m NAP	-0.60 m NAP	1.90 m
Neap tide	+0.88 m NAP	-0.60 m NAP	1.48 m
Average water level		+0.07 m NAP	

Table 49 - Tidal duration Hook of Holland, average discharge 2 200 m^3/s

Type	High water	Time	Low water
Average tide	01:32 h		07:10 h
Spring tide	01:30 h		06:47 h
Neap tide	01:35 h		07:37 h
Duration rise		06:47 h	
Duration fall		05:38 h	

Appendix F.2.2 Hollandsche IJssel

Table 50 - Tidal levels Krimpen aan den IJssel, average discharge 2 200 m^3/s

Type	High water	Low water	Difference
Average tide	+1.24 m NAP	-0.27 m NAP	1.51 m
Spring tide	+1.36 m NAP	-0.25 m NAP	1.61 m
Neap tide	+1.08 m NAP	-0.28 m NAP	1.36 m
Average water level		+0.29 m NAP	

Table 51 - Tidal duration Krimpen aan den IJssel, average discharge 2 200 m^3/s

Type	High water	Time	Low water
Average tide	03:13 h		11:27 h
Spring tide	03:22 h		11:36 h
Neap tide	02:52 h		10:05 h
Duration rise		04:11 h	
Duration fall		08:14 h	

Appendix F.3 Discharge

The department of Public Works keeps records of the discharge measured at Lobith. The extreme discharge distribution is estimated using 15 years of measurements; these measurements are analyzed using the Peak over Threshold (PoT) method. The threshold is 3 000 m^3/s the duration of a high wave is 14 days. The extreme discharges are predicted with the use of the Weibull distribution which is often used for high river discharges [57].

The data is categorized in bins of 500 m^3/s, there are 992 measurements. For each bin the cumulative, probability (P), 1-probability (Q) and Weibull Reduced Variable are calculated, shown in Table 52. The Weibull reduced Variable is calculated using the formula;

$$W = \ln\left(\frac{1}{Q}\right)^{\alpha}$$

Table 52 - Part of the discharge calculation

Classification Q	Number of days	Cumulative	P	1-P	W
3000 to 3500	310	310	0.3125	0.6875	0.3905
3500 to 4000	224	534	0.5383	0.4617	0.7812
4000 to 4500	164	698	0.7036	0.2964	1.2061
4500 to 5000	88	786	0.7923	0.2077	1.5421
5000 to 5500	59	845	0.8518	0.1482	1.8579
..	..	..	..	..	..

The Weibull reduced variable is plotted against the discharge (per bin) and a trend line is drawn trough these points. With the use of the created graph the correct value of α is approximated. This approximation is done to change the value compared to the fitting of the graph R^2, when the value does not increase further the correct α is found. Beta and gamma are found using the SLOPE and INTERCEPT function (between Q and W) in Excel. The obtained Weibull distribution is given by the following parameters:

Table 53 - Weibull distribution discharge

Distribution	Alpha	Beta	Gamma
Weibull	1.015	1 467.7	2 814.5

The found distribution is validated with the use of the 1/1 250 exceedance probability which should be in the order of 16 000, in this case 15 150 m^3/s. The exceedance probabilities are transferred to Weibull reduced variables with the formula shown, in which P is the exceedance probability and n is the number of high water waves per year.

$$\mathrm{W} = \ln\left(\frac{P}{n}\right)^{1/\alpha}$$

The number of high water waves in a year is calculated using:

$$n = \frac{total\ number\ of\ measurments}{number\ of\ years * duration\ high\ wave} = \frac{992}{15 * 140} = 4.7$$

The triangle on the x-axis represents the 1/1 250 year exceedance probability. The diamonds present the exceedance probabilities of 1/10 up to 1/10 000.

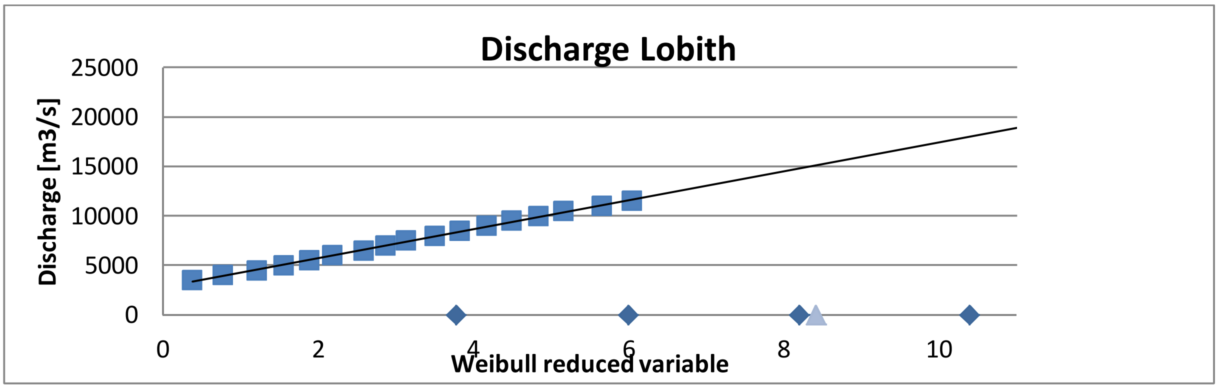


Figure 87 - Discharge Lobith, Weibull distribution

Appendix F.4 Storm surge

In different articles P. van Gelder uses Peak over Threshold data of the storm surge levels near Hook of Holland. This data set (acquired from P. van Gelder lecturer at the faculty of civil engineering) is used to construct the distribution of the storm surge levels near Hook of Holland. The data consists of 1 577 measurements over a period of 98 years. The threshold of this data set is 0.25 m NAP.

The data is categorized in bins of 0.25 m. Water levels lower than 0.25 m +NAP are not considered. For each bin the cumulative, probability (P), 1-probability (1-P) and Weibull Reduced Variable are calculated. The Weibull reduced variable is calculated using the formula:

$$W = \ln\left(\frac{1}{1-P}\right)^{\alpha}$$

Table 54 - Part of the storm surge calculation

Classification h		per bin	cumulative	P	1-P	W
..	..	..	..	..	..	..
1.75	2	7	1 570	0.9956	0.0044	7.4740
2	2.25	4	1 574	0.9981	0.0019	8.8856
2.25	2.5	1	1 575	0.9988	0.0012	9.5744
2.5	2.75	1	1 576	0.9994	0.0006	10.7703
2.75	3	1	1 577	1	0	#DIV/0!

The Weibull reduced variable is plotted against the discharge and a trend line is drawn trough these points. With the use of the graph the correct value of α is approximated. This approximation is done to change the value compared to the fitting of the graph R^2, when the value does not increase further the correct α is found. Beta and gamma are found using the SLOPE and INTERCEPT function (between Q and W) in Excel. The Weibull distribution is given by the following parameters:

Table 55 - Weibull distribution storm surge

Distribution	Alpha	Beta	Gamma
Weibull (2010)	0.84	0.209	0.467

The found distribution is validated with the use of the 1/10 000 exceedance probability which should be in the order of 4.6m +NAP, in this analysis 4.48m +NAP [58]. The exceedance probabilities are transferred to Weibull reduced variables with the formula shown, in which P is the exceedance probability and n is the number of surges per year.

$$\mathrm{W} = \ln\left(\frac{P}{n}\right)^{1/\alpha}$$

The number of surges in a year is calculated using:

$$n = \frac{total\ number\ of\ measures}{number\ of\ years} = 16$$

The circles on the axis present the exceedance probabilities of 1/10 up to 1/10 000.

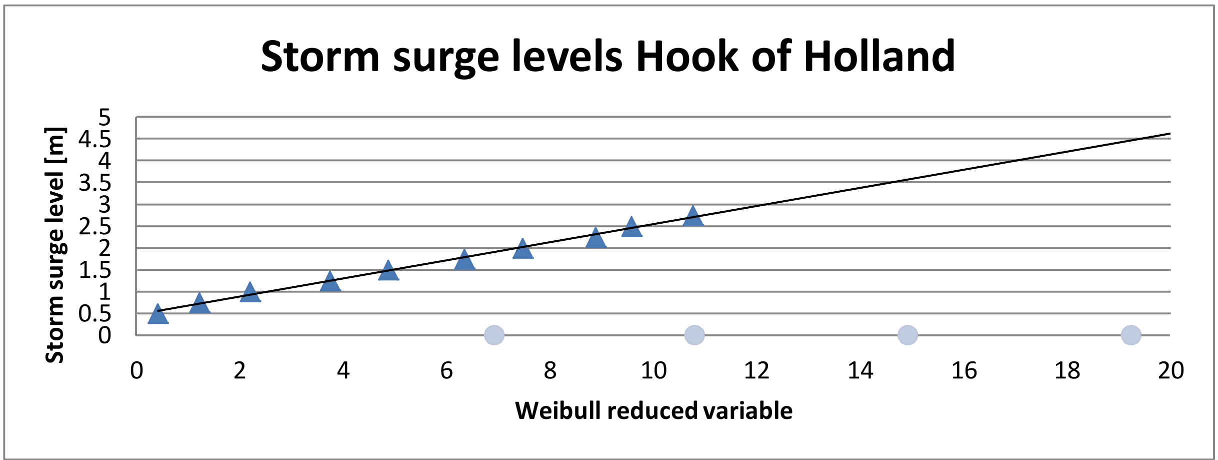


Figure 88 - Storm surge levels Hook of Holland

Appendix F.5 Water levels at the mouth of the Hollandsche IJssel

Appendix F.5.1 Equal level curves

The method of the equal level curves is described in the lecture notes of Probabilistic Design in Hydraulic Engineering (CIE5310) and in a paper were the water levels near Rotterdam were calculated [58, 36]. The combination of discharge and storm surge level for which the water levels stay the same is an equal level curve (shown in Figure 89). The equal level curve of +2.00 m NAP is equal to a water level of 2.00 m NAP at sea (storm surge and tide) when the discharge is 0 m^3/s, when the discharge increases to 16 000 m^3/s the water level at sea becomes +1.00 m NAP to maintain the equal level curve.

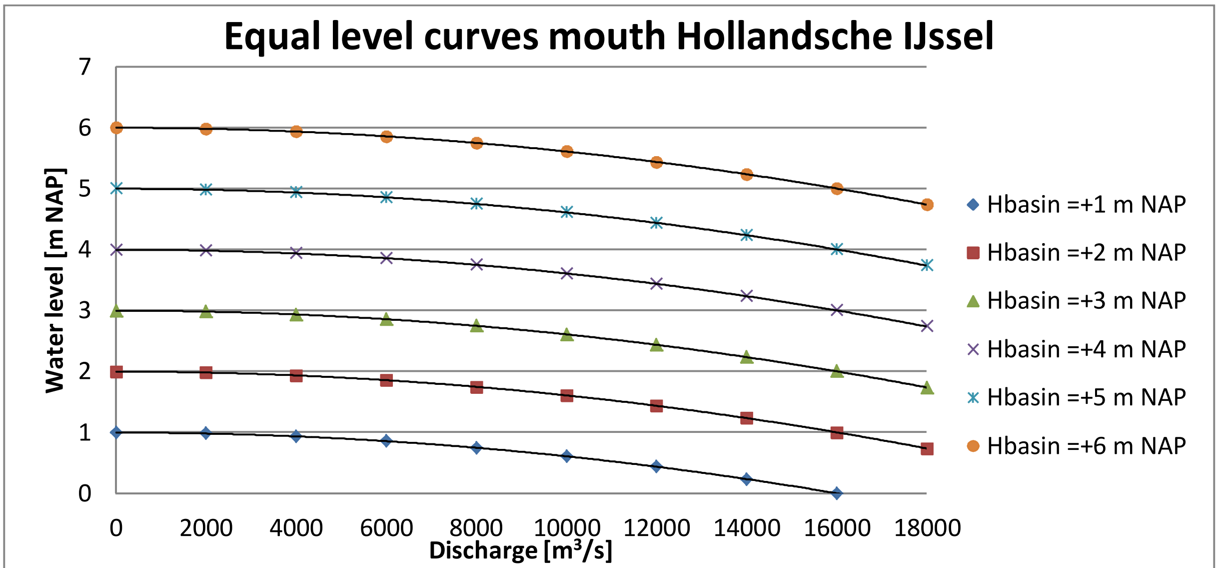


Figure 89 - Equal level curves mouth Hollandsche IJssel

The equal level curves for Rotterdam and the mouth of the Hollandsche IJssel are nearly the same. A comparison between the water levels of these two locations shows that there is a time lag of 20 minutes and a water level difference with a maximum of 10 centimeters. These two phenomena are caused by inertia and friction of the tide and storm surge. It is assumed that these effects are the same. The formula used for the equal level curves is given by:

$$h_{basin} = h_{sea} + \left(\frac{Q}{\mu A}\right)^2 * \frac{1}{2g} + h_{tide}$$

The discharge (Q) is measured at Lobith the flow area is the cross-section ($\mu A = 3620$ at Rotterdam) [58]. The discharge near Lobith is not the same as the discharge through the New Meuse. The flow area is however obtained due to the comparison of the discharge near Lobith and the water levels near Rotterdam. Due to this relation the discharge near Lobith can be used to calculate the effect of the discharge. The water levels near Rotterdam are expected to be the same as in front of the Hollandsche IJssel storm surge barrier.

Appendix F.5.2 Monte Carlo simulation

The water levels near the mouth of the Hollandsche IJssel depend on the independent discharge and storm surge distributions. The joint probability density function is used to calculate the water levels at the mouth of the Hollandsche IJssel:

$$\Pr\left(X_1, \ldots, X_N \in D\right) = \int_D f_{X_1,\ldots,X_N}(x_1, \ldots, x_N)\, dx_1 \cdots dx_N .$$

Integration of the distributions is difficult therefore the Monte Carlo simulation is used to estimate the joint probability density function. The Monte Carlo simulation is a simulation that makes use of random sampling to obtain numerical results. Computer software like MATLAB can be used to generate random samples from distributions. In MATLAB a script is written to draw random numbers from the two Weibull distributions. The simulation uses 1 000 000 random samples because the reliability in the tail depends on the number of values that are in the tail. The reliability is generally good enough when there are 100 values in the tail. Therefore the number of random samples is 100 multiplied with the inverse of the exceedance probability that should be obtained (1/10 000 in this calculation). The MATLAB script used is shown in Figure 90.

```
clf;
%Monte Carlo simulation water levels mouth Hollandsche IJssel
g = 9.81;
uA = 3620;
n = 1000000;
i=1;
Hbasin=[];
Q=[];
Hsea=[];
for i1 = 1:n
    q = wblrnd(1482.5,1.02)+2797.4; %discharge
    Hs = wblrnd(0.2086,0.84)+1.3171; %water level at sea
    Hb = Hs +(q/uA)^2/(2*g); %water level in basin
    Hbasin =[Hb Hbasin];
    Q=[q Q];
    Hsea= [Hs Hsea];
    i1 = i1+1;
end
    hist(Hbasin,100)
    plot(Hsea,Q)
```

Figure 90 - MATLAB script 2010, Monte Carlo simulation

The MATLAB script is executed for the situation in which there is no sea level rise, the sea level rise and tidal difference are added as normal values because there are not extreme distribution of the tide (which has a cyclic behavior) and the sea level rise (which only increases the water levels). In appendix F.5.3 the tide and sea level rise are added to the obtained water levels. The results of the script are shown in Figure 91; the data shown on the x-axis is the water level in the basin, the probability shown on the probability that the water level is lower than the given value. The exceedance probability is 1 minus the probability that a certain water level is not reached.

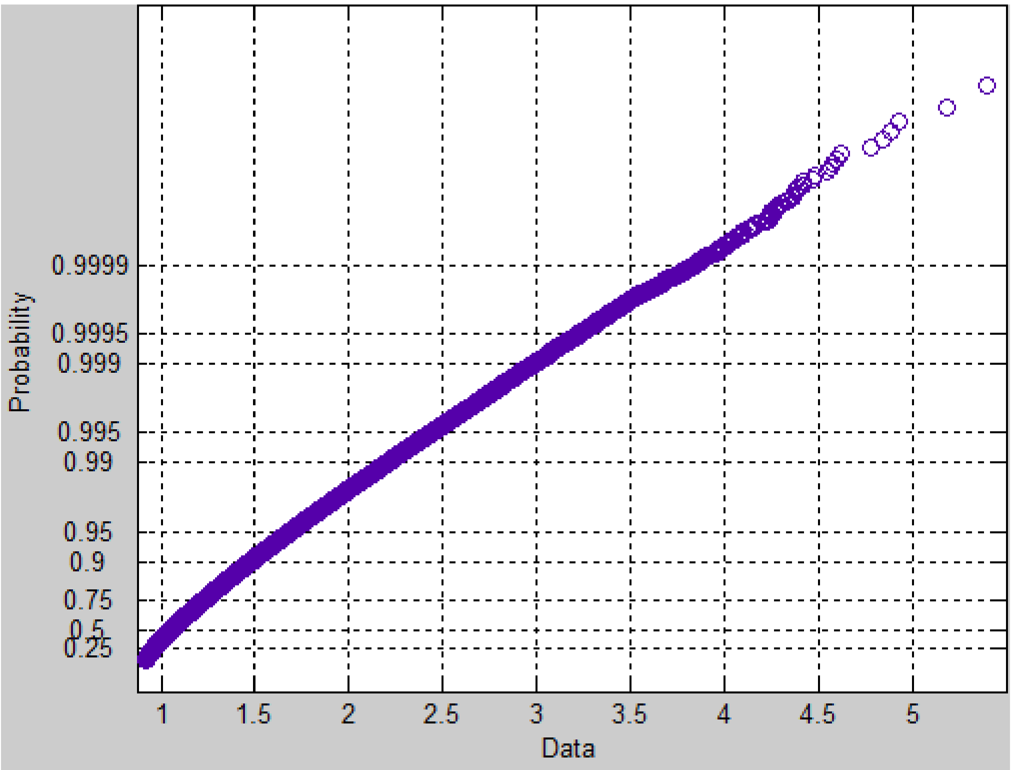


Figure 91 - Results MATLAB script SLR 0.00 m [data= meter, probability = -]

Appendix F.5.3 Water levels Hollandsche IJssel distribution calculation

The water levels obtained with the use of the Monte Carlo situation present the water levels at the mouth of the Hollandsche IJssel without the effect of the tide and sea level rise. The average tide from Table 50 is used (1.24 m NAP) because the extreme distributions (storm surge and discharge) used to calculate the high water levels at the mouth predict water levels with an exceedance probability of 1/10 000 per years, when the spring tide (occurs twice a month) is used this would result in a lower exceedance probability shown in Figure 92.

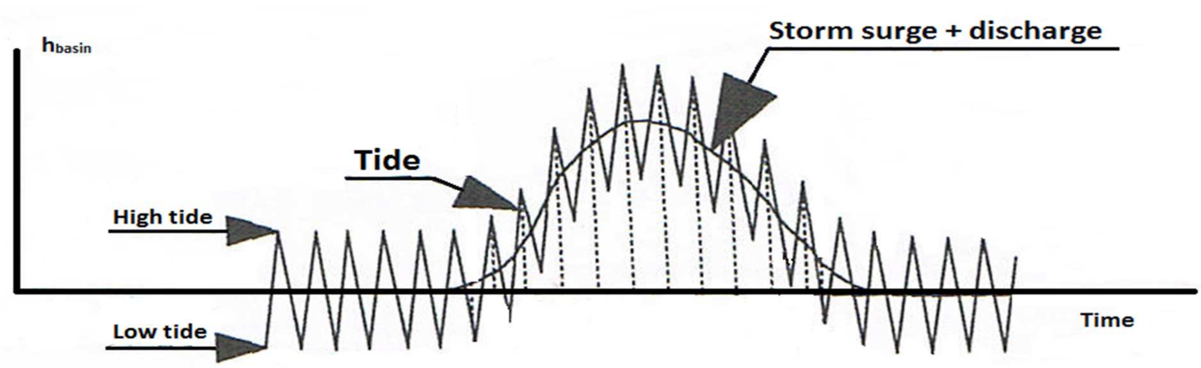


Figure 92 - Combination of tide, discharge and storm surge, source; lecture notes CT5310

There are three factors that ensure that the obtained values of the MATLAB script need to be increased, these factors are;

- The MATLAB script uses an average water level in Hook of Holland equal to 0.00 NAP, in reality there is however another average water level. The difference between the level used in Hook of Holland (0.00 m NAP and the average water level in the Hollandsche IJssel (+0.29 m NAP near the barrier) is 0.29 m.

- The tidal elevation above the average water level is used because the tidal wave has an up and down ward motion, only the upward motion is of interest for the high water levels. The difference between the average tidal elevation (+1.24 m NAP) and the average water level (+0.29 m NAP) in the Hollandsche IJssel is 0.95 m.
- The sea level rise that occurs needs to be taken into account.

$$Water\ level = MATLAB + \ average\ tidal\ elevation + SLR$$

The values that need to be obtained from the MATLAB file are the exceedance probabilities shown in the first column of Table 56 and shown in Figure 93.

Table 56 - Water levels mouth Hollandsche IJssel, open storm surge barriers

Exceedance probability [-]	Storm surge [m NAP]	Average tidal elevation [m]	SLR 0.00 m [m NAP]	SLR 0.20 m [m NAP]	SLR 0.35 m [m NAP]	SLR 0.85 m [m NAP]	SLR 1.20 m [m NAP]
1	1	1.24	2.24	2.44	2.59	3.09	3.44
0.995	1.05	1.24	2.29	2.49	2.64	3.14	3.49
0.90	1.1	1.24	2.34	2.54	2.69	3.19	3.54
0.75	1.2	1.24	2.44	2.64	2.79	3.29	3.64
0.50	1.3	1.24	2.54	2.74	2.89	3.39	3.74
0.25	1.5	1.24	2.74	2.94	3.09	3.59	3.94
0.1	1.7	1.24	2.94	3.14	3.29	3.79	4.14
0.05	1.9	1.24	3.14	3.34	3.49	3.99	4.34
1/100	2.25	1.24	3.49	3.69	3.84	4.34	4.69
1/200	2.4	1.24	3.79	3.99	4.14	4.64	4.99
1/1 000	2.9	1.24	4.14	4.34	4.49	4.99	5.34
1/10 000	3.7	1.24	4.94	5.14	5.29	5.79	6.14

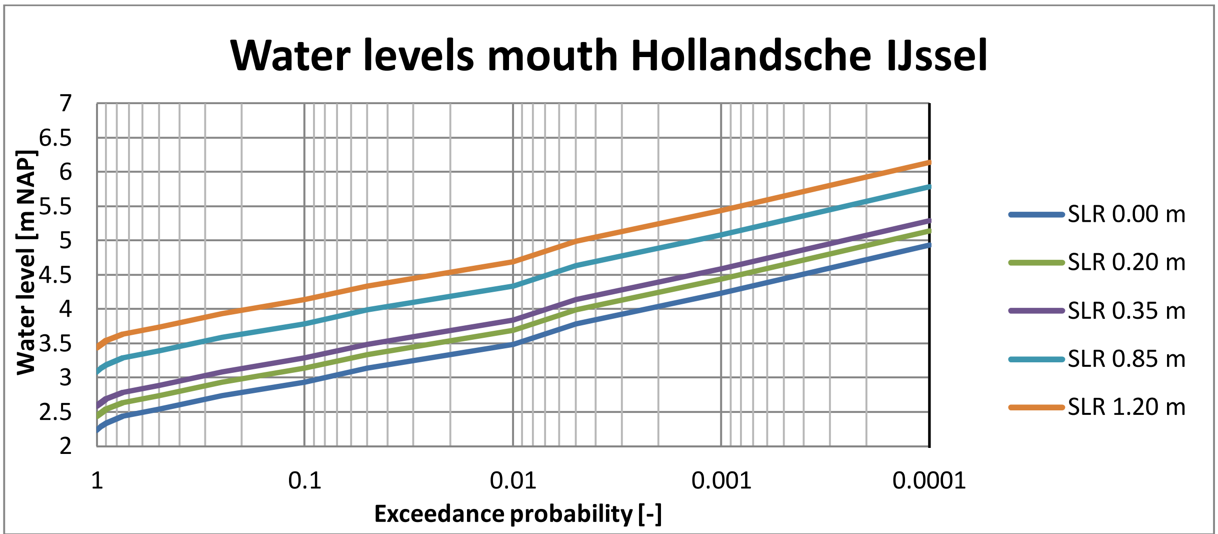


Figure 93 - Water levels mouth Hollandsche IJssel, open storm surge barriers

Appendix F.6 Approximate calculation of the water levels at Hook of Holland

The crude calculation of the water levels is conducted to analyze the closure levels of the Maeslant barrier. This calculation is crude because only Figure 94 is used to calculate Figure 95. The occurs of closure of the Maeslant barrier is analyzed because sea level rise ensures that number of closures of the Maeslant storm surge barrier increases.

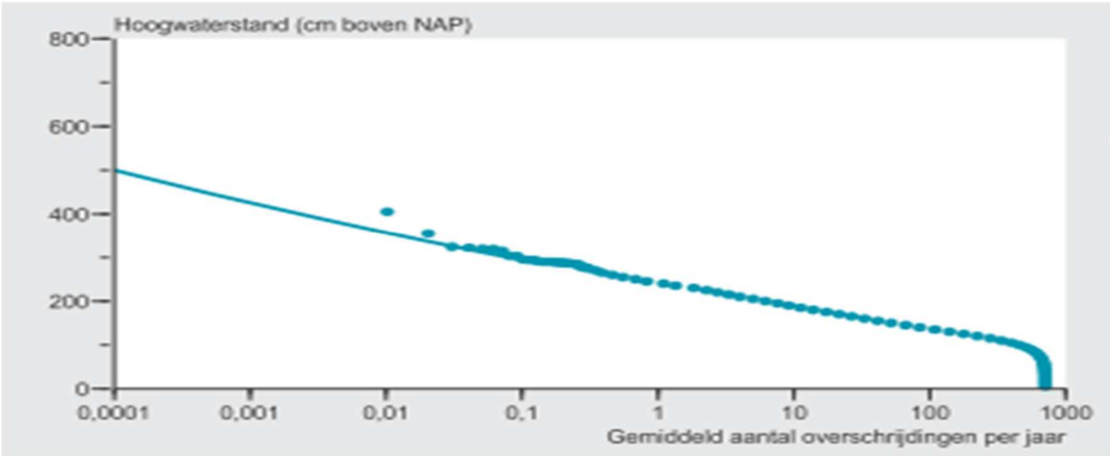


Figure 94 - Current exceedance curve, source; RIKZ

The exceedance probability and the (occurrence) are linked to each other using the following estimation;

$$Exceedance\ probabiliy = \frac{1}{Occurence}$$

In Figure 95 the occurrence of the water levels in used and not the exceedance probability because a probability cannot be higher than one, while a water level can be exceeded multiple times per year.

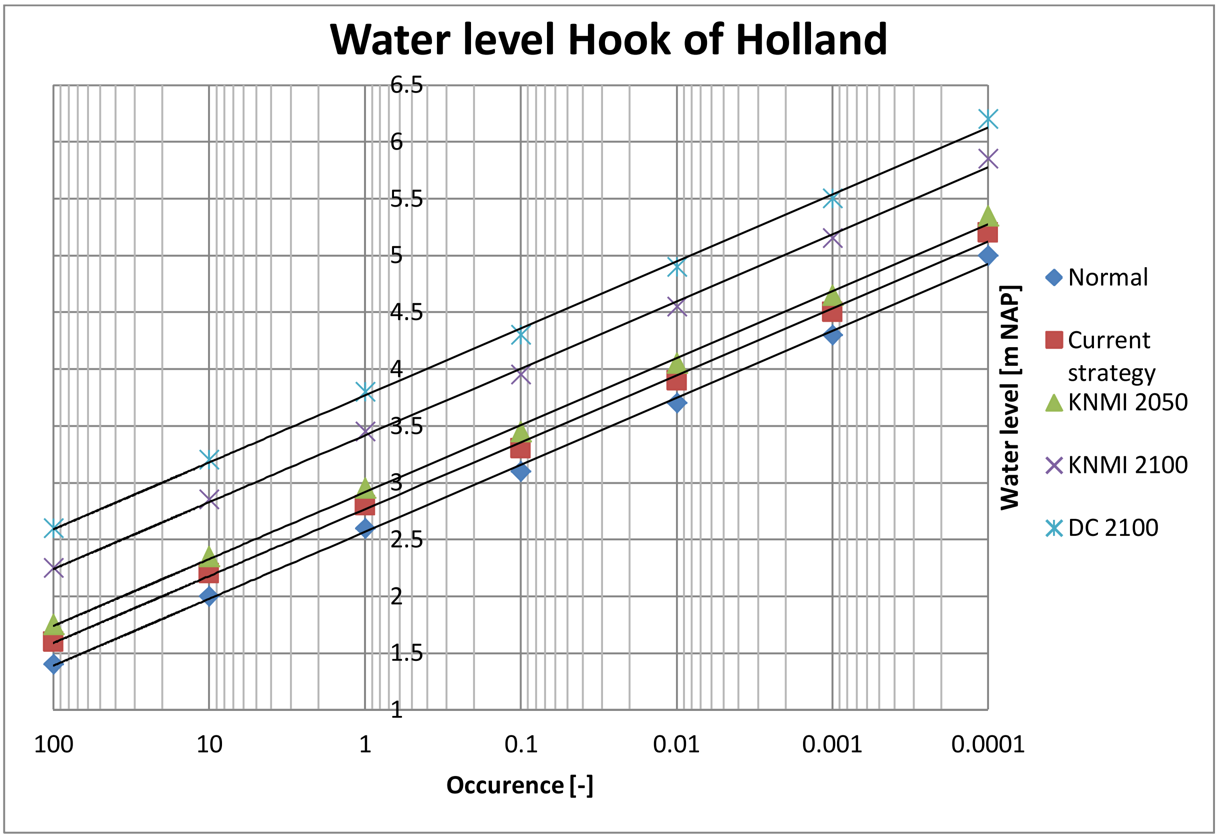


Figure 95 - Crude calculation water levels Hook of Holland

The diamond line in Figure 95 shows the exceedance curve for the normal situation in 2010. The other lines present the different studies that have been conducted for certain sea level rises. These lines are created when the sea level rise is added to the line of the current situation. Table 57 shows the closure levels needed to maintain the occurrence of once a year.

Table 57 - Closure level Maeslant barrier

Sea level rise [m]	Study	Closure level Maeslant for an exceedance probability of 1
0.00	Current situation	+2.60 m NAP
0.20	Normal increase 2100	+2.80 m NAP
0.35	KNMI W+ 2050	+2.95 m NAP
0.85	KNMI W+ 2100	+3.40 m NAP
1.20	IPCC 2100	+3.80 m NAP

Appendix F.7 Governing water levels on the New Meuse

When the storm surge barrier is closed the water levels behind the barrier are described by the discharge from the Rhine. Water coming from the Rhine accumulates behind the Maeslant storm surge barrier, resulting in an increase of the water levels.

The governing discharge is maximal when the storm surge barrier just closes due to a storm surge. The barrier just closes when a storm surge of +3.00 m NAP is predicted. The exceedance probability when the storm surge barrier just closes and the exceedance probability for the discharge together should create the 1/10 000 situation, this is the norm for the Hollandsche IJssel storm surge barrier.

In Figure 95 the exceedance probabilities for which +3.00 m NAP (closure level Maeslant storm surge barrier) is reached are shown for different sea level rises, sea level rises higher than 0.35 meter are not treated because the closure level of the storm surge barrier should change when this rise is reached (shown in Table 57). The exceedance probabilities of the closure level shown in Table 58 are comparable to the values shown in Figure 93 (executed Monte Carlo script in MATLAB).

The norm is however not directly calculated from the different exceedance probabilities because the duration of the two phenomena is different. When the barrier closes due to a storm surge of +3.00 m NAP the barrier will be closed for one tidal cycle (12 hours), the top of a high water wave lasts approximately 4 days. The governing exceedance probability of the discharge is therefore calculated using the probability that the storm surge occurs on a random day. The probability that a closure happens on a random day is equal to the probability that the barrier closes divided by the number of days in a year. The probability that the peak of a flood wave coincides with the closure is equal to the probability of occurring multiplied with the number of days that the peak of the flood wave lasts.

$$P_{norm} = \frac{P_{closure}}{365\ days} * P_{discharge;peak} * 4\ days$$

The governing discharges are presented in Table 58 and obtained from the Weibull distribution (appendix F.3).

Table 58 - Governing situation on the New Meuse

Sea level rise*	P_{norm}	Exceedance probability of the closure level	Exceedance probability of the governing discharge ($P_{discharge}$)	Governing discharge Rhine [m^3/s]
0.00	1/ 10 000	1/ 9	1/ 12	8 100
0.10	1/ 10 000	1/ 7	1/ 16	8 500
0.20	1/ 10 000	1/ 5	1/ 22	9 000
0.35	1/ 10 000	1/ 3	1/ 37	9 700

*sea level rise higher than 0.35 is not treated because the closure level of +3.00 m NAP should change for that

The total discharge near the Maeslant storm surge barrier is however not equal to the governing discharge of the Rhine near Lobith, the River IJssel transfers 1/9 of the discharge measured near Lobith to Lake IJssel. The Meuse also discharges water in the North Sea via the New Waterway and Haringvliet, it is assumed that the discharge of the Meuse is 2 500 m^3/s, which is equal to an exceedance probability of 1/10.

$$Q_{tot} = {}^{8}/_{9}\, Q_{rhine} + Q_{meuse}$$
$$Q_{0.00} = 9\,600;\ Q_{0.10} = 10\,100;\ Q_{0.20} = 10\,500;\ Q_{0.35} = 11\,100\ m^3/s$$

When discharges are higher than 6 000 m^3/s the Maeslant storm surge barrier closes during the ebb slack (described in section 2.3). The governing discharges are higher than 6 000 m^3/s therefore the water level during the ebb slack period is given as;

$$Closure\ level - Average\ tidal\ difference\ HoH = 3.00 - 1.74 = +1.26\ m\ NAP$$

Figure 96 shows the water levels behind the storm surge barrier when the Maeslant barrier is closed. Closure of the Maeslant barrier lasts 2.5 hours. During closure and opening of the Maeslant barrier the outflow of water through the New Waterway is already hampered, therefore it is assumed that the effective closure time increases with half of both the closure and opening. The closure time of the barrier becomes 12+2.5 = 14.5 hours.

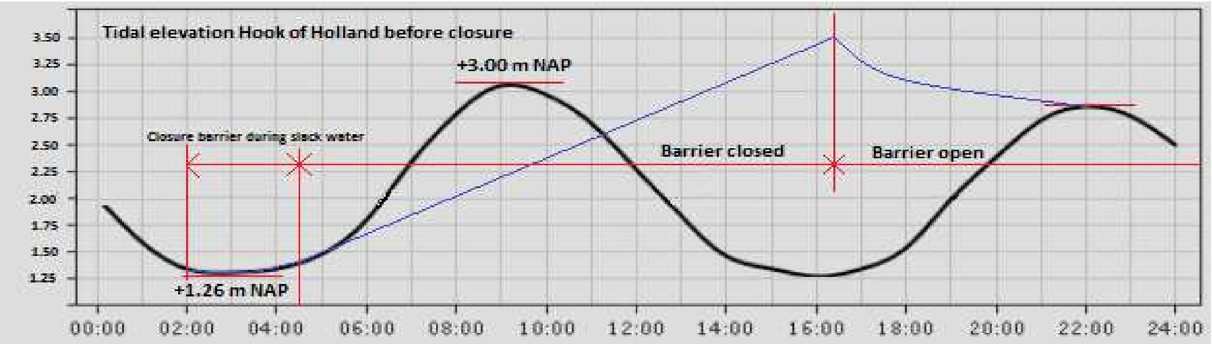


Figure 96 - Water level behind the Maeslant barrier, source; Rijkswaterstaat

The governing water level behind the storm surge barrier is calculated using the formula given below. The two parts of the formula are the water level just after closure and the effect due to the accumulation of water behind the barrier. The surface of the basin (Rijnmond, Haringvliet and Hollands Diep) is approximately 250 km^2 according to the data from Rijkswaterstaat [59].

$$h_{design\ level} = h_{closed} + \frac{Q_{tot}}{A_{basin}} * t_{storm}$$

Table 59 - Water level behind the closed Maeslant barrier

	Value	Unit
Total discharge (Q_{tot})	-	-
Duration design storm (t_{storm})	52 200 (14.5)	s (h)
Basin (A_{basin})	$2.50*10^8$	m^2
Water level when the barrier is closed (h_{closed})	+1.50	m NAP
Design water level SLR 0.00 m	+3.50	m NAP
Design water level SLR 0.10 m	+3.61	m NAP
Design water level SLR 0.20 m	+3.69	m NAP
Design water level SLR 0.35 m	+3.82	m NAP

Figure 97 and Figure 98 show the governing water levels at the mouth of the Hollandsche IJssel for the sea level rise of 0.0 and 0.35 m. The figures show that the exceedance probability, for which closure (green line) occurs, increases when the sea level rises. This is expected because the sea level rise ensures that the higher water levels occur more often. After closure the governing water levels on the Hollandsche IJssel are determined by the discharge that occurs. When the Maeslant barrier fails the governing water levels without barrier are introduced in the system. The governing water levels presented in Table 59 are reached just before failure of the Maeslant barrier, shown in Figure 97.

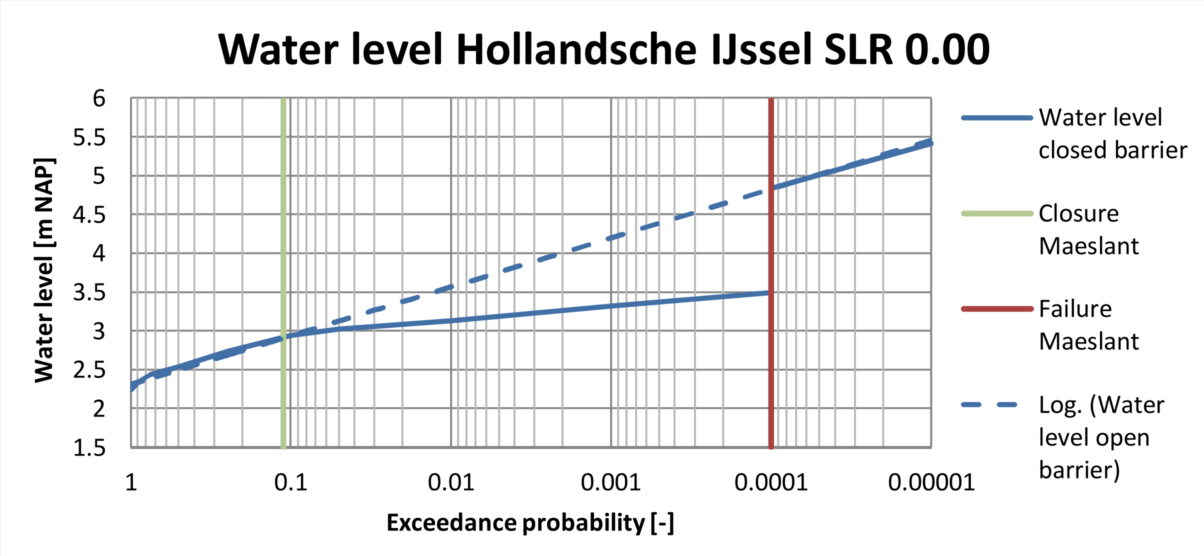


Figure 97 - Governing water levels at the mouth of the Hollandsche IJssel with 0.0 m

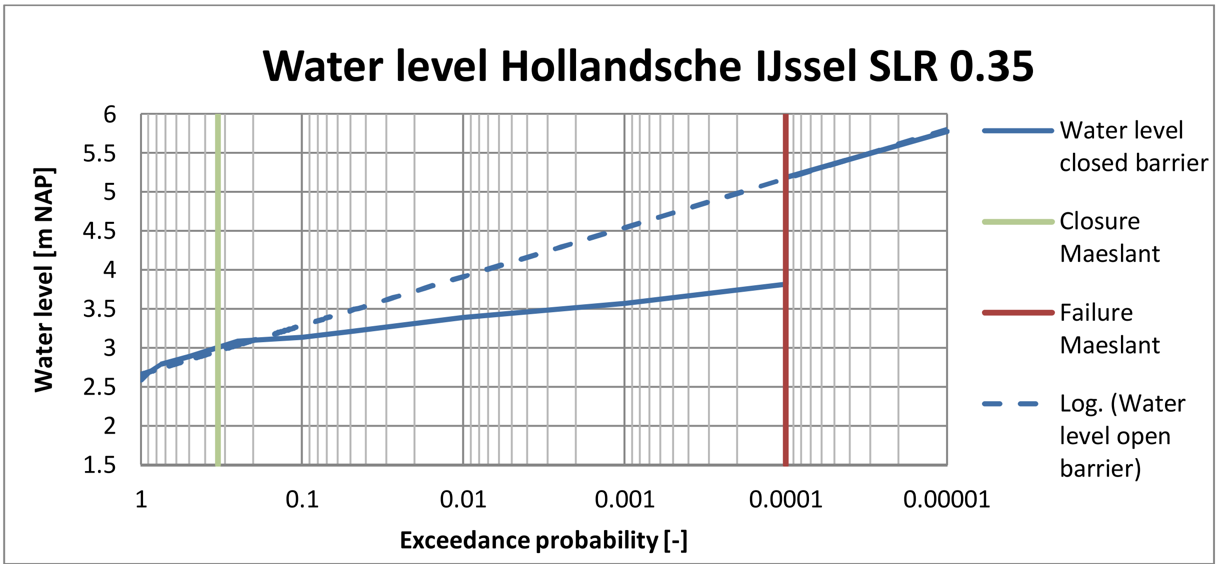


Figure 98 - Governing water levels at the mouth of the Hollandsche IJssel with 0.35 m

Appendix F.8 Non-closure probability

Appendix F.8.1 Theory non-closure probability

The non-closure probability is an important aspect which is treated in the VTV regulations published in 2007 [12, 60]. The check “reliability closure” is conducted for connecting defences (b-defences) with a movable part. The levees in the hinterland of the storm surge barrier are c-defences. The regulations for the safety assessment state that the closure reliability of is good when;

- The non-closure probability is lower than 1/10 times the exceedance probability of the storm surge barrier.
- The non-closure probability is higher than 1/10 times the exceedance probability of the storm surge barrier, but it can be guaranteed that;
    - There is no development in the growth of the breach.
    - The storm surge barrier does not fail due to the open situation.
    - The levees behind the storm surge barrier are higher than the water level in the open situation.

When those criteria are not fulfilled the effect of the non-closure probability should be added to the governing water levels on the system behind the closed barrier. In the third nationwide safety assessment the Hollandsche IJssel storm surge barrier was not up to the standard for the track “reliability closure”. The non-closure probability of the flood defences is 1/30 per event while it should at least be 1/100 000 per event for the levees of dike ring 14. In the hydrological boundary conditions of 2006 the effect of the non-closure probability was not added to the governing water levels in the hinterland [18].

Appendix F.8.2 Calculation of the effect of the non-closure probability behind the SSB

The effect of the non-closure probability should be calculated for the design conditions just behind the storm surge barrier (SSB) and for the water levels on the Hollandsche IJssel. The water level on the Hollandsche IJssel has to be increased with the effect of the high water level when the barrier is not closed.

$$h_{governing} = n * P_{ncl} * h_{open} + (1 - n * P_{ncl}) * h_{closed}$$

The formula exists of two parts, the part 1- $n*P_{ncl}$ which is the closed part and the n*P part which is the non-closure part. Together these parts should be equal to 1. The probability that the storm surge barrier does not close is the non-closure probability per event multiplied with the number of closures in a year ($n*P_{ncl}$). When the storm surge barrier is closed the water level is h_{closed}, when the barrier is open the water level is h_{open}.

In the calculation of the governing water levels the following assumptions are made:

- The Hartel barrier has no influence on the non-closure probability and increase of the water levels.
- The Maeslant barrier is closed during governing conditions; during closure discharge from the Rhine is governing for water levels on the New Meuse.

In Table 60 an example calculation is conducted for a situation where the storm surge closure should close once a year. This means that the non-closure probability per event is the probability that h_{open} will occur.

Table 60 - Example calculation governing water level

	Non-closure probability (P_{ncl})	Water level open barrier (h_{open})	Water level closed barrier (h_{closed})	Governing water level ($h_{governing}$)
Example 1	1/10	+5.0 m NAP	+2.0 m NAP	+2.3 m NAP

$$h_{governing} = 0.1 * 5.0 + (1 - 0.1) * 2.0 = +2.3\ m\ NAP$$

The water levels in the closed condition are equal to the closure level of the HIJ storm surge barrier (which is +1.75 m NAP). The water levels in the open condition are equal to the water levels shown in Figure 97 and Figure 98, the wind set-up which is important for the governing water levels along the Hollandsche IJssel is not important because the wind set-up is minimal at the mouth and maximal near Gouda (shown in Figure 101). The difference between $h_{governing}$ and h_{closed} is the effect of the non-closure probability given different exceedance probabilties. The number of closures is expected to increase to 10 according to the Delta program; therefore this value is used for n [11]. The effect of the non-closure probability and sea level rise (SLR) are shown in Figure 99 and Figure 100. The effect of the non-closure probability increases because the water levels outside increase due to the sea level rise.

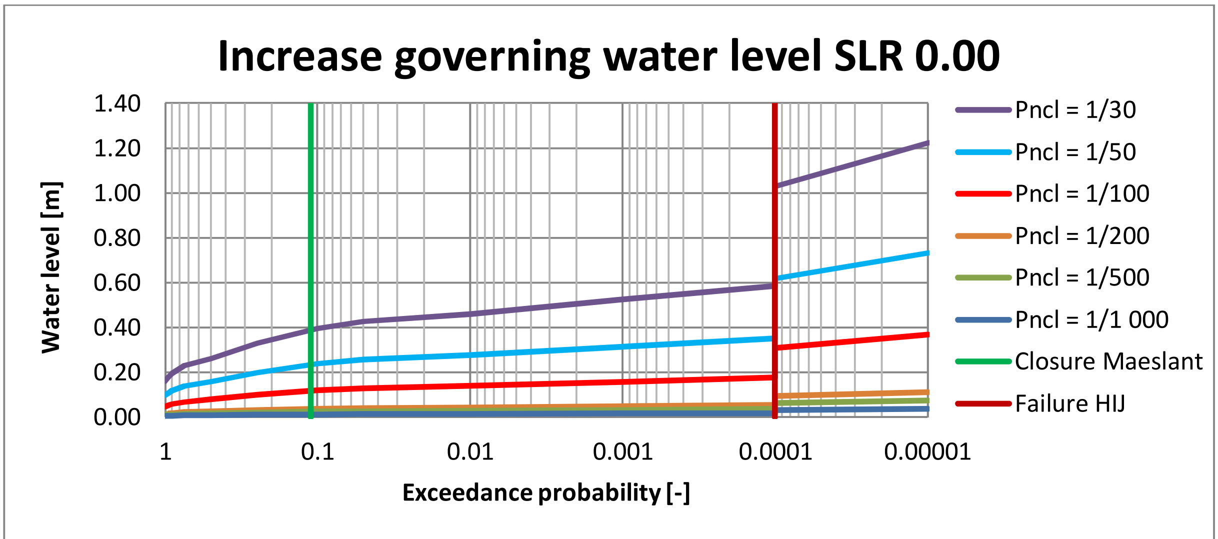


Figure 99 - Increase of the water levels due to non-closure, closure level 1.75 m NAP and SLR 0.00 m

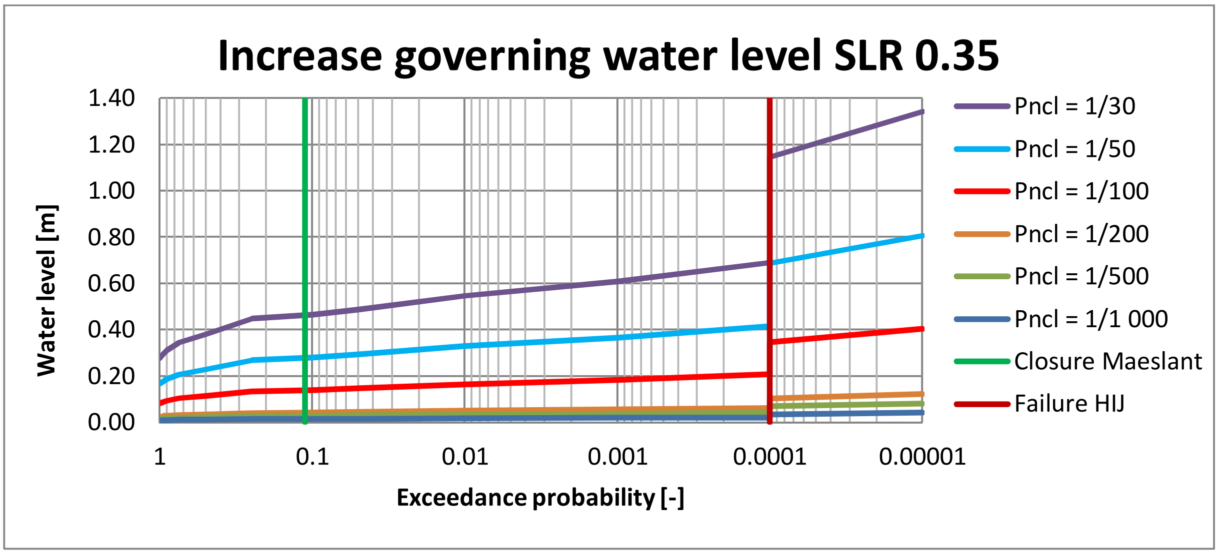


Figure 100 - Increase of the water levels due to non-closure, closure level 1.75 m NAP and SLR 0.35 m

Appendix F.8.3 Calculation of the effect of the non-closure probability along the HIJ

When the non-closure probability is zero the barrier closes always and h_{closed} is governing. When the non-closure probability is one there is no barrier and h_{open} is governing. For the effect of the non-closure probability the water levels h_{open} and h_{closed} in the Hollandsche IJssel should be known. During closure of the Hollandsche IJssel (HIJ) storm surge barrier the water level at closure and the possible wind set-up are governing (schematized in Figure 101).

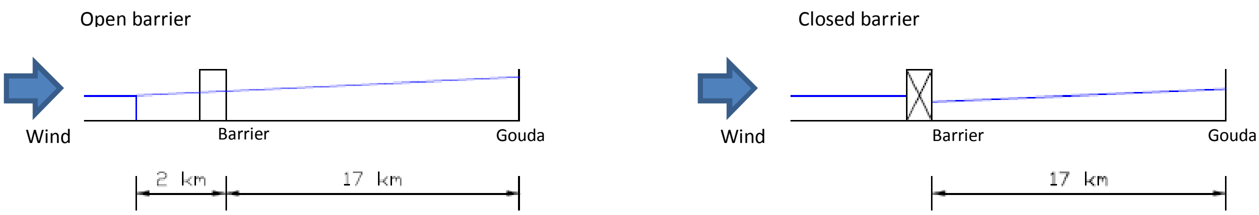


Figure 101 - Schematization wind set-up

The wind set-up is calculated using the formula obtained from the lecture notes of CIE3330 [61]. This formula uses the wind speed and friction of the water to estimate the set-up. For this calculation the Hollandsche IJssel is assumed to be a rectangular box in the direction of the wind. This is a reasonable assumption because the width of the river does not decrease that much and the general lay-out of the river is in the governing wind direction.

$$h_{wind\_max} = \frac{dS}{dt} * L_{open} = C_2 * \frac{U^2}{g * d} * L_{open}$$

Table 61 - Calculation wind set-up different situations

Parameter	Unit	Value	Unit
Wind speed 1/ 10 000 (appendix I.1)	$U_{10\,000}$	33.5	m/s
Wind speed 1/ 2 000 (appendix I.1)	$U_{2\,000}$	30.5	m/s
Wind speed HRC 2006	$U_{HRC2006}$	20.5	m/s
Friction coefficient	C_2	$3.50*10^{-6}$	-
Gravity (g)	g	9.81	m/s^2
Average water depth (d)	d	5.5	m
Length Hollandsche IJssel	L_{open}	19 000	m

Length Hollandsche IJssel	L_{closed}	17 000	m
HRC 2006			
Maximum wind set-up closed	H_{wind_max}	0.52	m
Wind set-up at the barrier closed	$H_{barrier}$	-0.26	m
Wind set-up at Gouda closed	H_{gouda}	0.26	m
Exceedance probability 1/ 10 000			
Maximum wind set-up open barrier	H_{wind_max}	1.38	m
Wind set-up at the barrier open barrier	$H_{barrier}$	0.14	m
Wind set-up at Gouda open barrier	h_{gouda}	1.38	m
Maximum wind set-up closed barrier	h_{wind_max}	1.24	m
Wind set-up at the barrier closed barrier	$h_{barrier}$	-0.62	m
Wind set-up at Gouda closed barrier	h_{gouda}	0.62	m
Exceedance probability 1/ 2 000			
Maximum wind set-up open	h_{wind_max}	1.15	m
Wind set-up at the barrier open	$H_{barrier}$	0.12	m
Wind set-up at Gouda open	H_{gouda}	1.15	m
Maximum wind set-up closed	h_{wind_max}	1.02	m
Wind set-up at the barrier closed	$h_{barrier}$	-0.51	m
Wind set-up at Gouda closed	h_{gouda}	0.51	m

When the storm surge barrier is open the water levels on the Hollandsche IJssel (h_{open}) are described with the use of the governing water levels on the New Meuse ($h_{governing;\ New\ Meuse}$) and the wind set-up ($h_{wind\_max}$) that is created over the Hollandsche IJssel.

$$h_{open} = h_{governing;New\ Meuse} + h_{wind\_max}$$

The water levels in the closed situation are based on the existing governing water levels on the Hollandsche IJssel, the HRC2006 (described in section 2.5.1) calculated the governing water levels in the Hollandsche IJssel for the nationwide assessment. These calculations did not account for the non-closure probability and used a lower wind speed. The water level h_{closed} is given as;

$$h_{closed} = h_{HRC2006} - \Delta_{cloure\ level} + \Delta_{wind\ speed}$$

The parameter $\Delta_{closure\ level}$ calculated the difference between the two closure levels, which is +2.25 m NAP minus +1.75 m NAP is 0.5 meter. The $\Delta_{wind\ speed}$ gives the difference between the wind speeds used in the HRC2006 and the governing wind speeds that are used in this calculations. The calculation of the wind set-up is conducted in Table 61 for the exceedance probabilities of 1/ 10 000 (dike ring 14) and 1/ 2 000 (dike ring 15). The calculation of the open and closed water level for the 1/ 10 000 and 1/ 2 000 exceedance probabilities is presented in Table 62 and Table 63 and shown in Figure 102 and Figure 103.

Table 62 - Calculation open and closed water level for the 1/10 000 situation

	km	HRC2006	Reduction ($\Delta_{closure\ level}$)	Difference wind speeds ($\Delta_{wind\ speed}$)	Closed (h_{closed})	Open (h_{open})
	0	-	-	-	-	3.50
	1	-	-	-	-	3.57
Barrier	2	2.49	0.50	-0.39	1.76	3.65
	3	2.49	0.50	-0.34	1.79	3.72
	4	2.5	0.50	-0.30	1.82	3.79
	5	2.5	0.50	-0.25	1.85	3.86
	6	2.52	0.50	-0.20	1.90	3.94
	7	2.54	0.50	-0.16	1.94	4.01
	8	2.57	0.50	-0.11	2.00	4.08
	9	2.6	0.50	-0.07	2.06	4.16

	10	2.63	0.50	-0.02	-6.88	4.23
	11	2.68	0.50	0.02	-0.41	4.30
	12	2.73	0.50	0.07	2.27	4.37
	13	2.8	0.50	0.11	2.37	4.45
	14	2.89	0.50	0.16	2.49	4.52
	15	2.98	0.50	0.20	2.60	4.59
	16	3.07	0.50	0.25	2.72	4.66
	17	3.17	0.50	0.30	2.85	4.74
	18	3.29	0.50	0.34	2.99	4.81
Gouda	19	3.4	0.50	0.39	3.13	4.88

Table 63 - Calculation open and closed water level for the 1/2 000 situation

	km	HRC2006	Reduction ($\Delta_{closure\ level}$)	Difference wind speeds ($\Delta_{wind\ speed}$)	Closed (h_{closed})	Open (h_{open})
	0	-	-	-	-	3.40
	1	-	-	-	-	3.46
Barrier	2	2.49	0.50	-0.28	1.71	3.52
	3	2.49	0.50	-0.25	1.74	3.58
	4	2.5	0.50	-0.22	1.78	3.64
	5	2.5	0.50	-0.18	1.82	3.70
	6	2.52	0.50	-0.15	1.87	3.76
	7	2.54	0.50	-0.12	1.92	3.82
	8	2.57	0.50	-0.08	1.99	3.88
	9	2.6	0.50	-0.05	2.05	3.94
	10	2.63	0.50	-0.02	2.11	4.00
	11	2.68	0.50	0.02	2.20	4.06
	12	2.73	0.50	0.05	2.28	4.12
	13	2.8	0.50	0.08	2.38	4.18
	14	2.89	0.50	0.12	2.51	4.24
	15	2.98	0.50	0.15	2.63	4.31
	16	3.07	0.50	0.18	2.75	4.37
	17	3.17	0.50	0.22	2.89	4.43
	18	3.29	0.50	0.25	3.04	4,49
Gouda	19	3.40	0.50	0.28	3.08	4.55

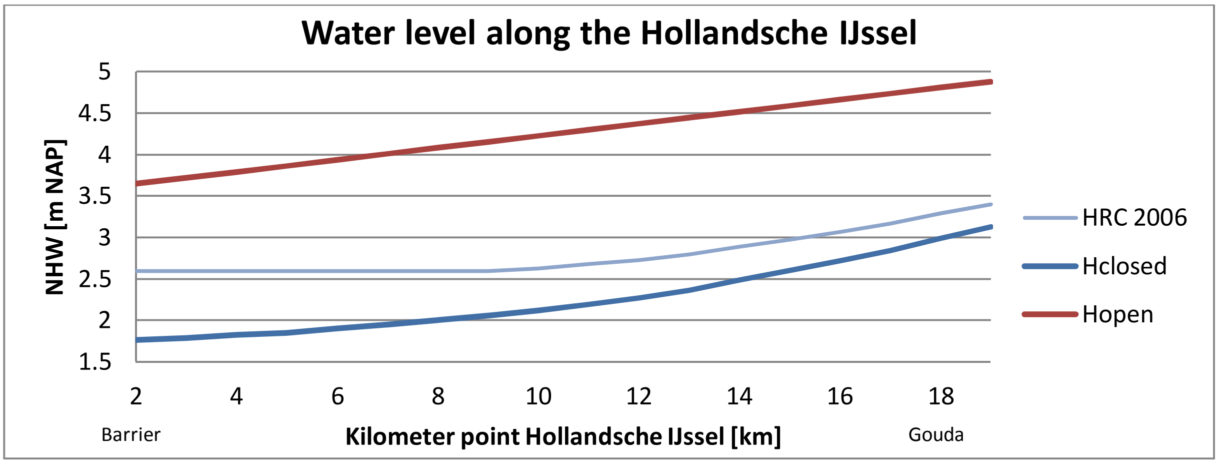


Figure 102 - Open and closed water level for the 1/10 000 situation

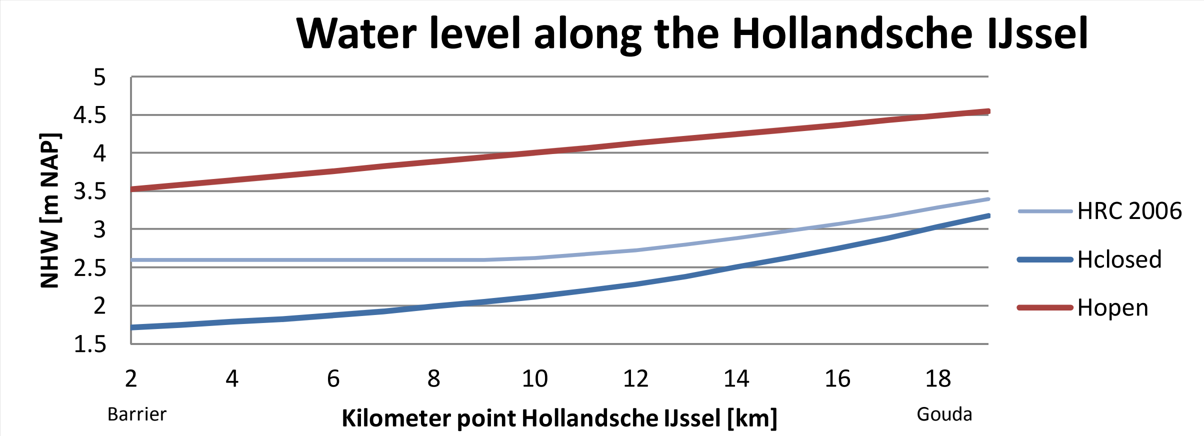


Figure 103 - Open and closed water level for the 1/2 000 situation

When the open and closed water levels in the Hollandsche IJssel are known the effect of the non-closure probability can be calculated using the same formula as in appendix F.8.2. The number of closures (n) of the Hollandsche IJssel barrier is 10. The results are shown in Figure 104 and Figure 105; the numerical results are presented in Table 64.

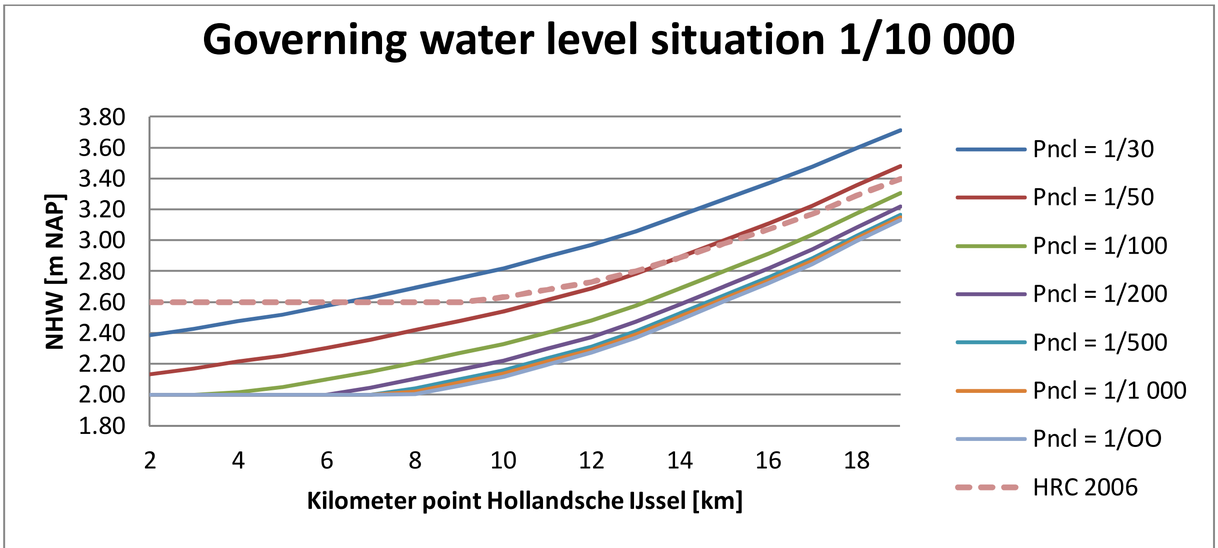


Figure 104 - Governing water levels along the Hollandsche IJssel with exceedance probability 1/10 000

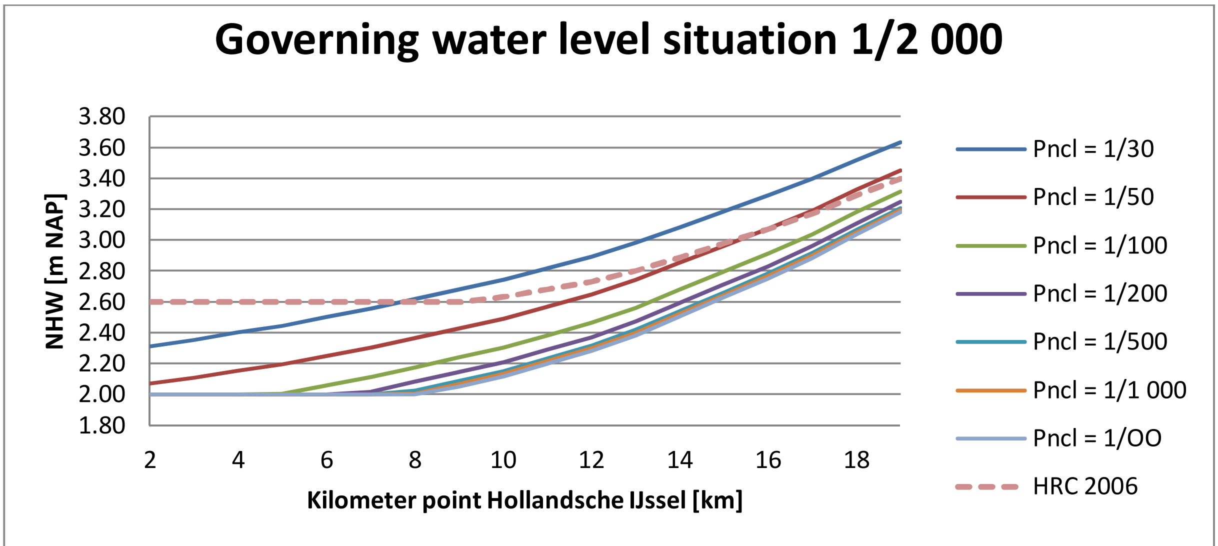


Figure 105 - Governing water levels along the Hollandsche IJssel with exceedance probability 1/2 000

n=	10	$1-nP_{ncl}$=	0.67	0.8	0.9	0.95	0.98	0.99	0.999
1/10 000	Situation	nP_{ncl}=	0.33	0.2	0.1	0.05	0.02	0.01	0.001
	Closed (h_{closed})	Open (h_{open})	1/30	1/50	1/100	1/200	1/500	1/1 000	1/10 000
2	1.76	3.65	2.39	2.14	1.95	1.85	1.80	1.78	1.76
3	1.79	3.72	2.43	2.17	1.98	1.88	1.82	1.80	1.79
4	1.82	3.79	2.48	2.22	2.02	1.92	1.86	1.84	1.82
5	1.85	3.86	2.52	2.25	2.05	1.95	1.89	1.87	1.85
6	1.90	3.94	2.58	2.31	2.10	2.00	1.94	1.92	1.90
7	1.94	4.01	2.63	2.36	2.15	2.05	1.99	1.97	1.95
8	2.00	4.08	2.70	2.42	2.21	2.11	2.04	2.02	2.00
9	2.06	4.16	2.76	2.48	2.27	2.16	2.10	2.08	2.06
10	2.12	4.23	2.82	2.54	2.33	2.22	2.16	2.14	2.12
11	2.19	4.30	2.90	2.62	2.40	2.30	2.24	2.21	2.20
12	2.27	4.37	2.97	2.69	2.48	2.38	2.31	2.29	2.27
13	2.37	4.45	3.06	2.78	2.58	2.47	2.41	2.39	2.37
14	2.49	4.52	3.16	2.89	2.69	2.59	2.53	2.51	2.49
15	2.60	4.59	3.27	3.00	2.80	2.70	2.64	2.62	2.60
16	2.72	4.66	3.37	3.11	2.91	2.82	2.76	2.74	2.72
17	2.85	4.74	3.48	3.23	3.04	2.94	2.89	2.87	2.85
18	2.99	4.81	3.60	3.36	3.18	3.09	3.03	3.01	3.00
19	3.13	4.88	3.72	3.48	3.31	3.22	3.17	3.15	3.13

Table 64 - Calculation water levels Hollandsche IJssel for the 1/10 000 situation (water levels are in m NAP)

The average increase of the water level compared to the closes water level is presented in Table 65 for the two different situations.

Table 65 - Average increase governing water levels behind the storm surge barrier

Non-closure probability	1/30	1/50	1/100	1/200	1/500	1/1 000	1/10 000
Average P_{exc} = 1/10 000	0.66 m	0.40 m	0.20 m	0.10 m	0.04 m	0.02 m	0.00 m
Average P_{exc} = 1/ 2 000	0.59 m	0.35 m	0.18 m	0.09 m	0.04 m	0.02	0.00 m

Appendix G Salt intrusion

This appendix describes the aspects related to salt intrusion. The first part of this appendix studies the change of the average discharge; the other parts describe different solutions and problems related to salt intrusion.

Appendix G.1 Average monthly discharge

The change of the monthly discharge is investigated in the KNMI studies [7]. Figure 106 shows the monthly discharge of the Rhine 2100 for the KNMI studies.

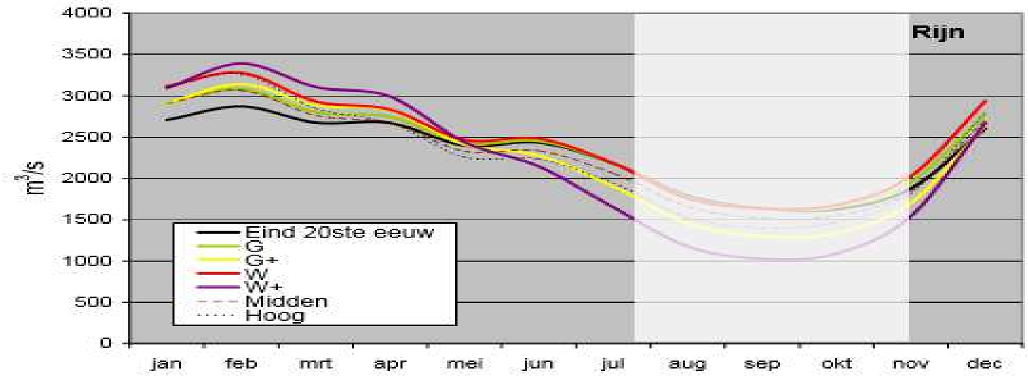


Figure 106 - Average discharge 2100 KNMI W+

Appendix G.2 Ecological Main Structure (EHS)

The nature in the ecological main structure is categorized in different nature types. According to the map of the province of South Holland there are a few important categories of nature in the Hollandsche IJssel [43]. The Hollandsche IJssel as a whole is part of category N02.01.

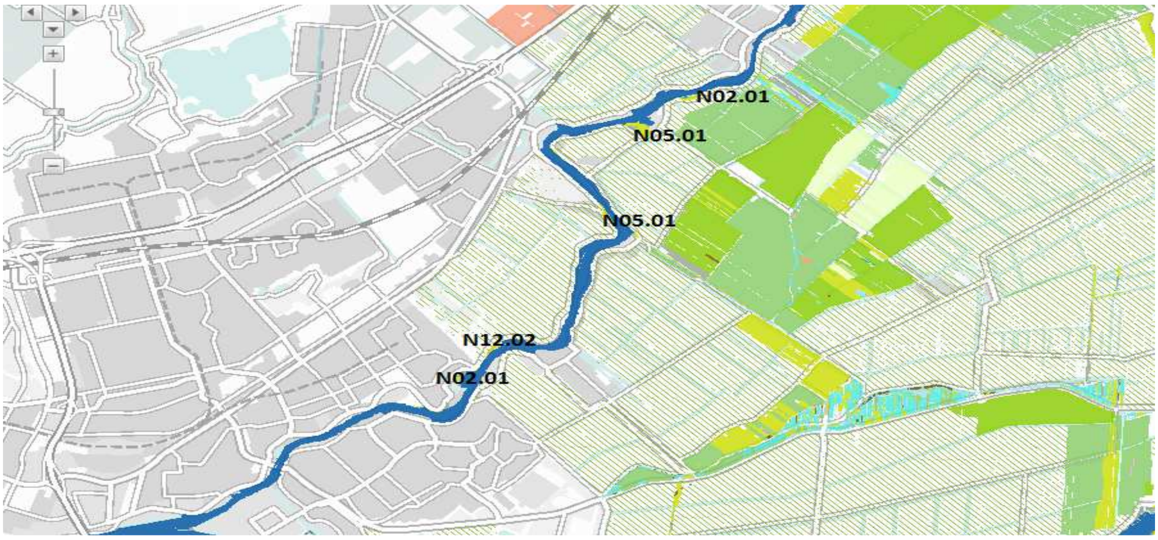


Figure 107 - Nature management map, source; province of South Holland

Category N02.01 River – includes all major rivers and canals with flowing water.
Category N05.01 Swamp – marshy area the water levels can change in this type.
Category N12.02 Herb heavy grassland – grassland which is above the tide line

Appendix G.3 Explanation salt stair and bubble screens

Appendix G.3.1 Salt stair

The salt stair (trapjeslijn shown in Figure 108) was created in 1968 it consists of a series of different bottom levels that slow the salt intrusion. Salt intrusion is slowed because the salt water that flows at the bottom of the New Meuse (because salt water is heavier than fresh water) needs to move upwards to pass a level of the salt stair, due to this upward motion turbulence occurs that mixes the salt and fresh water.

During the decades that the salt stair has been active it slowly eroded, measurement conducted in 2009 showed that the salt stair has nearly vanished. In research the effect of the renovated salt stair is studied [41].

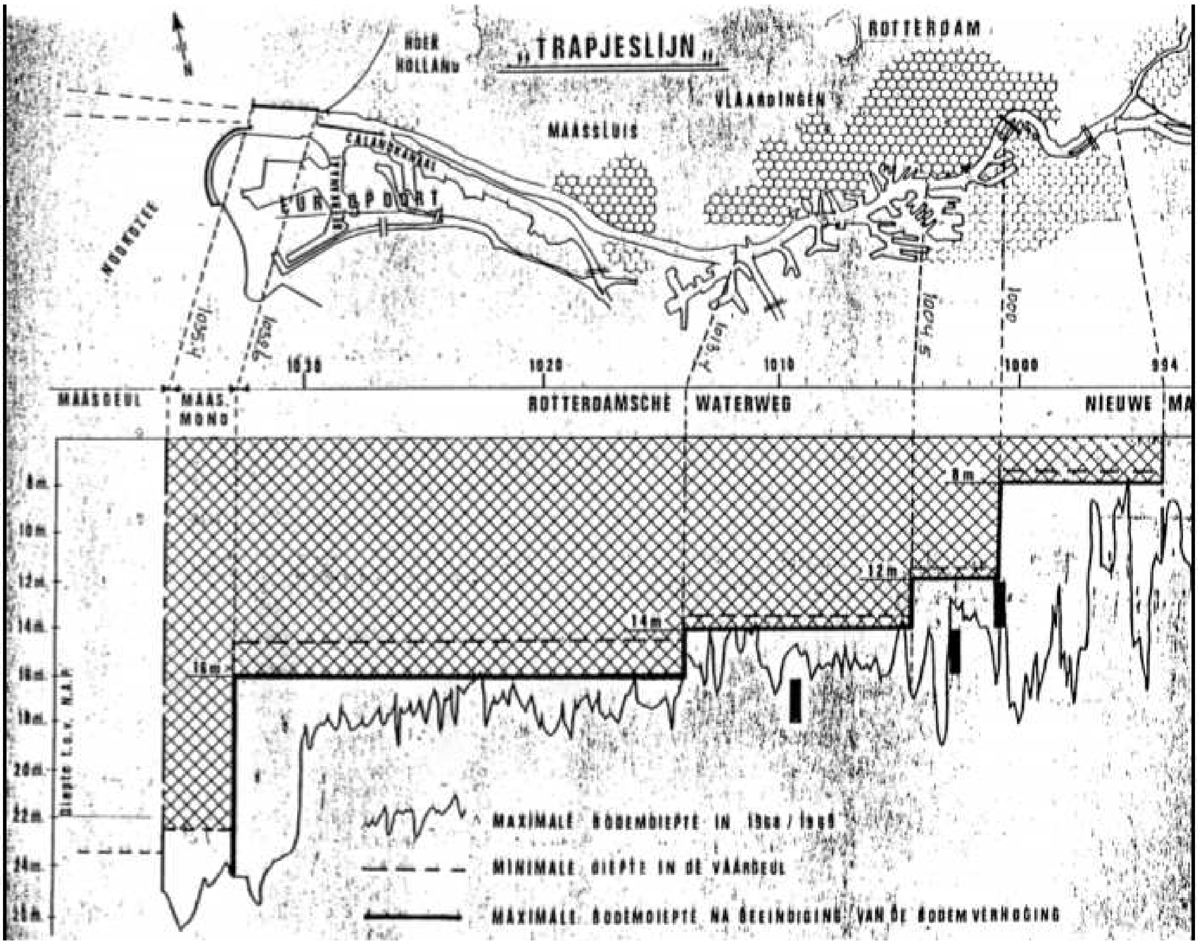


Figure 108 - Overview salt stair when executed in 1968, source; Deltares

Appendix G.3.2 Bubble screen

Bubble screens are screens of bubbles that fill up the entire cross-section, due to these bubbles the salt and fresh water mix and the salt tongue does not develop any further (shown in Figure 109). The bubbles are created on the bottom with the use of air that blows through vents laid on the bottom of the river. These screens are especially used to prevent salt intrusion during lock cycles. Important problems of bubble screens are the;

- High maintenance costs due to clogging of the vents
- Low effectiveness, 50% when designed properly
- High energy costs

Installation of bubble screen can be permanent and mobile. For the situation in the New Meuse the installation of a mobile bubble screen has its benefits because the return period of these events is rather high.

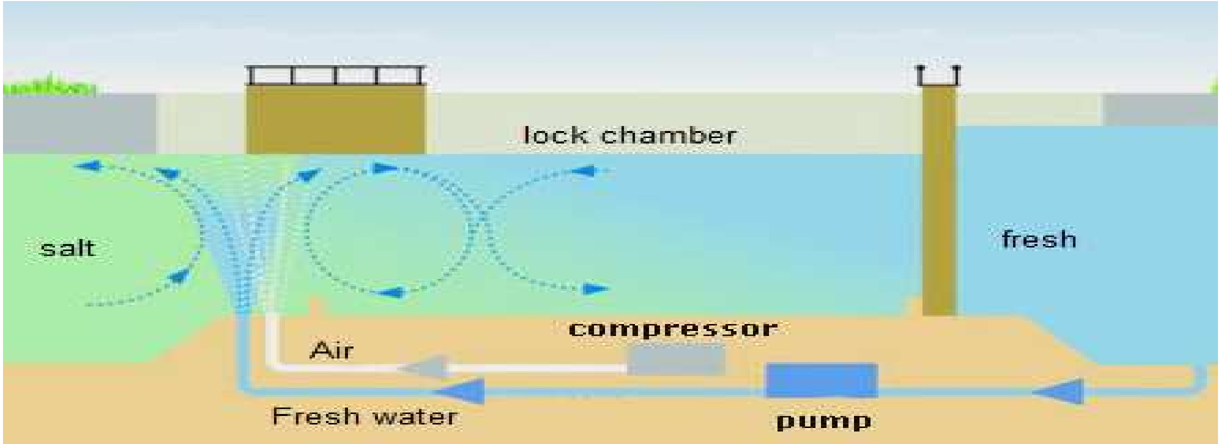


Figure 109 - Overview bubble screen used in lock, source; Deltares

Appendix G.4 Calculations salt intrusion and inlet Gouda

The water that can be used for the inlet of water is the difference between the water level at which the storm surge barrier closed during a salt intrusion closure and the water depth that is needed to maintain shipping in the Hollandsche IJssel.

The minimum water depth for which ships can still use the Hollandsche IJssel is -0.50 m NAP. This water level is chosen because the sill of the existing storm surge barrier is calculated using this water level. When there is a closure due to salt intrusion the low water level will be very close to this water level. The high water level is therefore -0.50 + the average tidal difference of 1.51= +1.01 m NAP. The water that can be used is the tidal difference multiplied by the surface area of the Hollandsche IJssel.

$$V = 1.51 * 135 * 19\,000 = 3.87 * 10^6\ m^3$$

	Inlaat (miljoen m³, benodigd tijdens het groeiseizoen)	Inlaat (m³/sec, benodigd tijdens het groeiseizoen)	Cl⁻ van het inlaatwater bij Gouda	Landbouwschade totaal (k€)	Aalsmeer	Boskoop	Bollenstreek	Duingebied	Haarlemmermeer	Nieuwkoopse Plassen	Noordplas, M-Tempelpolder e.a.	Overige Polders	Zuidelijke Veenpolders
				Landbouwschade (k€)									
Referentiesituatie	114	7,2	200	25,0	65	6465	4740	2	1528	0	2358	6743	3148
Zoutgehalte alle aandachtsgebieden +100 mg/l	85	5,4	200	30,2	62	9213	5541	2	1716	0	2358	8014	3312
Zoutgehalte alle aandachtsgebieden +300 mg/l	65	4,1	200	32,5									
Zoutgehalte alle aandachtsgebieden +300 mg/l	65	4,1	300	42,9									
Zoutgehalte alle aandachtsgebieden +300 mg/l; inlaat 500 mg/l	70	4,5	500	59,1	151	25867	8875	2	1929	9	2358	9885	9994
Zoutgehalte alle aandachtsgebieden -100 mg/l	147	9,3	200		64	7702	4733	2	1260	0	1834	5471	3081
Bij Gouda water inlaten 300 mg/l	127	8,1	300	36,6	99	13898	6613	2	1528	3	2358	6743	5368
Bij Gouda water inlaten 150 mg/l	113	7,2	150	21,4	44	5140	3561	2	1528	0	2358	6743	2027
Minimum landbouwschade bij 200mg/l inlaat	195	12,4	200	19,1									
Minimum landbouwschade bij 300mg/l inlaat	250	15,9	200	32,5									
Minimum landbouwschade bij 300mg/l inlaat	114	7,2	300	37,3									

Figure 110 - Values inlet Gouda, source; Alterra [16]

The inlet near Gouda needs a minimum discharge of 16 m^3/s (Q_i) to keep flushing of the canals possible. The need of water for Rijnland will increase to 24 m^3/s according to the master thesis of F. Bulsink; given a drought period that occurs once every 35 years [13]. With this discharge the storage of water on the Hollandsche IJssel can be used for approximately 2 days.

$$Inlet\ duration = \frac{V}{Q_i * 60 * 60 * 24} \approx 2\ days$$

Figure 111 - Source water canalized Hollandsche IJssel (green)

The KWA can maintain a discharge of 10 m^3/s; this means that the canalized Hollandsche IJssel has to be optimized to maintain a discharge of 14 m^3/s. The water from the canalized Hollandsche IJssel (green) comes eventually from the Lek (dark blue) and Kromme Rhine (light blue). The other waterways are the Gouwe Canal (orange) into the Rijnland system and the tidal Hollandsche IJssel (turquoise).

Appendix G.5 Occurrence salt intrusion

The KNMI studied the return period for certain years in which there were long periods of salt intrusion. Result of this study is shown in Table 66. The design situation is chosen to be the situation which occurs in 1990 and has a return period of 32 years now and 18 in 2050 [62]. This situation is chosen because no other information is known; when the situation is solved for 1990 it is also solved for the other reference years. This period lasted 60 days (salt days, salinity higher than 250 mg/l) which is equal to 9 weeks [41].

Table 66 - Return period (RP) salt intrusion, source; KNMI

Reference year	Scenario	Current RP (2012)	Future RP (2050)
1990	Very salt	32.1	17.6
2003	Salt	11.1	6.95
1996	Average salt	3.33	2.51
1994	Brackish	1.64	1.43
2002	Moderate brackish	1.19	1.12

The renovated salt stair will bring the number of salt days back from 60 to 30 days [41]. The closure of the Hollandsche IJssel storm surge barrier can last a month which happens in the very salt scenario. With the current configuration this means that salt intrusion is stopped. Other options (mentioned in appendix G.3) can be used to prevent salt intrusion when the number of salt days increases even further.

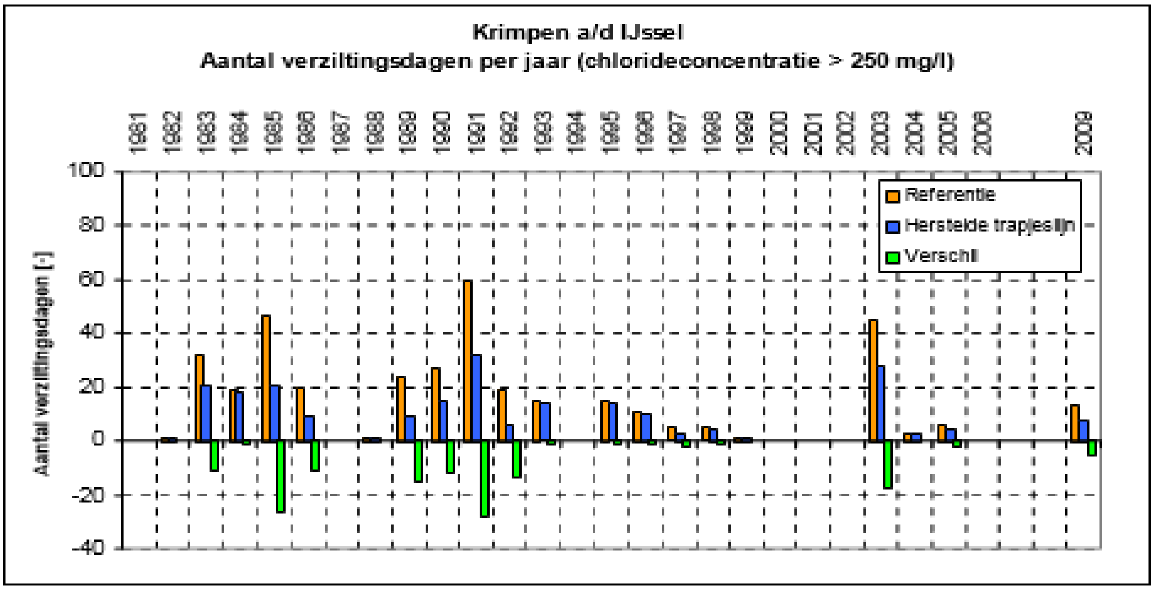


Figure 112 - Salt days for years with long period of low discharges, source; Deltares

Appendix G.6 Low water level for which salt intrusion becomes a problem

The lowest water levels during a salt intrusion period are shown in Table 67 for different salt intrusion periods. The water level for which salt intrusion becomes a problem is not known but happens with discharges lower than 1250 m^3/s. This water level is important for the determination of the governing water levels in appendix F.7.

Table 67 - Low water level near the storm surge barrier during salt intrusion; source Rijkswaterstaat [37]

Salt intrusion year	Water level
2009	-0.94 m NAP
2003	-0.85 m NAP
1996	-1.40 m NAP
1990	-0.96 m NAP
1976	-1.07 m NAP

When the water levels become lower than -0.85 meters salt intrusion becomes a problem (lowest value in the table).

Appendix H Water balance Hollandsche IJssel

This appendix focuses on the different aspects related to the water balance of the Hollandsche IJssel. The first part describes precipitation and overtopping, the second part treats the pumping stations in the system.

Appendix H.1 Extreme precipitation

The extreme precipitation during the design storm expected in 2050 is given in Figure 113 [7, 63, 45]. The data in this figure is used to create an extrapolation of the expected extreme precipitation with a return period of 10 000 years. The results of this extrapolation are shown in Figure 114. Result of this extrapolation is that the 10 000 year W+ extreme precipitation is 140 mm/day. The extreme precipitation for twelve hours is therefore 70 mm.

Extremes	1 hour					1 day					10 days				
Return period	Current	G	G+	W	W+	Current	G	G+	W	W+	Current	G	G+	W	W+
1 year	14	15	-	17	-	33	36	35	39	36	80	85	81	89	82
10 year	27	30	-	33	-	54	60	57	66	60	114	122	116	130	119
100 year	43	48	-	53	-	79	88	84	98	88	143	154	146	164	150

Figure 113 - Extreme precipitation W+ scenario, source; KNMI [63]

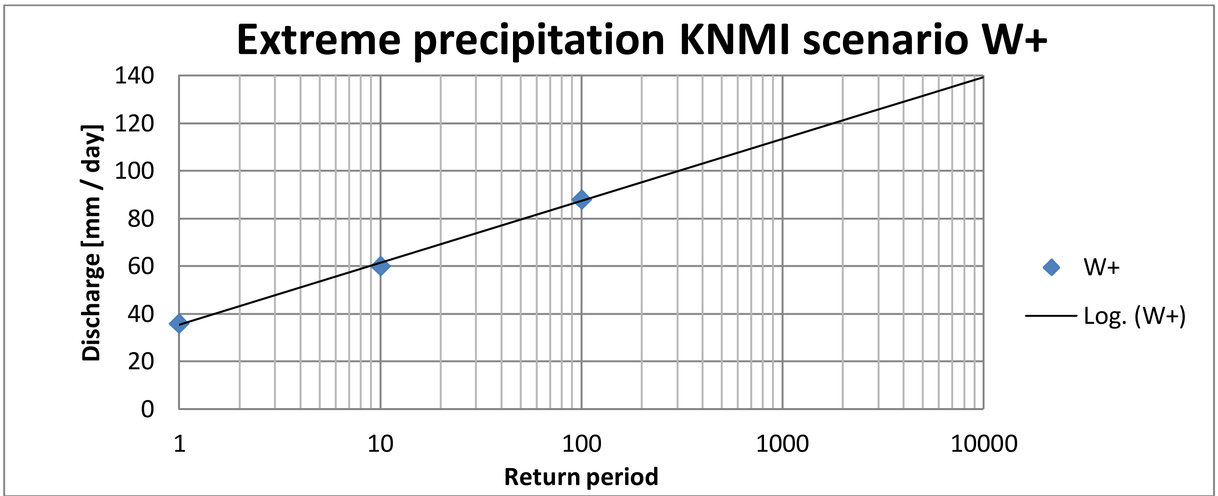


Figure 114 - Extrapolation extreme precipitation for W+ scenario, 1 day

Appendix H.2 Overtopping

Overtopping is calculated using formula 10.8 from the lecture notes of Breakwater and closure dams [64]. The formula reads:

$$q = 0.04 * e^{-1.8 * \frac{R_c}{H_{m0}}} * \sqrt{g * {H_s}^3}$$

$$z = \frac{q * B * D_s}{A_{basin}}$$

This formula can be used when the waves are non-impulsive (non-breaking). The waves are non-breaking because there is no depth-induced or steepness-induced breaking. Depth induced breaking happens when the significant wave height is approximately half of the depth which is not the case. Steepness induces breaking happens when the steepness is bigger than 0.14 which is not the case. The results and used values are shown in Table 68.

Table 68 - Overtopping calculation

Parameter	Value
Significant wave height (H_s)	1.18 m
Freeboard (R_c)	3.00 m
Check 0.1 < Rc/Hs < 3.5	1.72
Discharge (q)	0.01 m^3/m/s (10 l/m/s)
Storm duration (D_s)	12 h
Length gate (B)	80 m
Surface Hollandsche IJssel (A_{holl})	19 000 * 135 m^2
Water level rise Hollandsche IJssel (z)	0.01 m

Appendix H.3 Pumping stations

The maximum discharge of the pumping stations discharging water on the Hollandsche IJssel is given in this appendix; this information is obtained from the department of Public Works [18][46].

Table 69 - Information pumping station capacity and hinterland

Pumping station	Hinterland area	Max capacity
Abraham Kroes	7 242 ha	972 m^3/min
Middel Watering	609 ha	140 m^3/min
Johannes Veurink	2 483 ha	300 m^3/min
Kromme en Geer	1 140 ha	80 m^3/min
Hitland	602 ha	60 m^3/min
De Nesse	545 ha	40 m^3/min
Verdoold*	4 942 ha	450 m^3/min
Gouda Mallegat Hanepraai	-	150 m^3/min
Mr Pijnacker Hordijk**	19 000 ha	2 400 m^3/min
*Canalized Hollandsche IJssel****	-	*1 290 m^3/min (21.5 m/s)*
Total pumping stations	*36 013 ha*	*4 592 m^3/min*
Total Hollandsche IJssel	*257 ha*	-

* Capacity increase after renovation
** Capacity increase expected
*** Part of the river

Total discharge capacity:	Discharge can. HIJ + capacity pumping stations
	1 290 + 4 592 = 5 882 m^3/min * 60= 352 920 m^3/h
Total surface Hollandsche IJssel:	135 *19 000 = 2 565 000 m^2
Increase of the water level is:	0.14 m/h

Appendix I Hydrological boundary conditions

In this appendix the hydrological boundary conditions for the adapted storm surge barrier are described. The first part studies the different hydrological conditions; the second part treats the (wave) pressure on the gate.

Appendix I.1 Wind speed

The KNMI has several weather stations throughout the country; some of them also measure the wind speed. There are two locations in close proximity to the project site; Rotterdam Geulhaven (GH) and Zestienhoven (ZH). The two arrows highlighted in Figure 115 show the two weather stations. GH is located just above Schiedam, ZH is located near Vlaardingen. The weather station GH is located near the river and as such has the same general lay-out. Therefore this station is chosen as the governing station with representative wind speeds for the Hollandsche IJssel.

Figure 115 - Location weather stations KNMI, source; Google Maps

The data used to calculate the normative wind speed is found on the website of the KNMI (shown in Figure 116). Data given on the site is given per weather station that meaures the wind speeds. For the situation in the Hollandsche IJssel the weather station GH is thougth to be normative. The directions that are analysed are 230-250 and 050-070 (shown in Figure 117). These directions are paralel to the Hollandsche IJssel resulting in the largest waves.

Frequency table of potential wind speed - Distributive absolute
Location: 343 R'dam Geulhaven, Period Year, Evaluated from the years 1981-2000

Wind direction (x 10 degrees) — Distributive in hours per year

Wind speed (m/s)	Calm / Variable	35-01	02-04	05-07	08-10	11-13	14-16	17-19	20-22	23-25	26-28	29-31	32-34	Cumulative
0.0 - 0.9	7.8	3.1	2.3	0.9	1.1	1.5	1.7	2.3	1.8	1.3	2.1	2.7	3.6	32.2
1.0 - 1.9	23.9	41.1	27.3	16.9	9.9	15.8	28.5	29.3	20.2	15.8	17.9	30.9	47.5	325.0
2.0 - 2.9	7.0	111.1	86.1	50.6	19.0	34.4	80.6	71.5	49.4	42.2	37.3	57.9	117.2	764.3
3.0 - 3.9	1.2	103.9	129.9	99.7	37.7	57.4	163.8	112.0	70.9	75.9	58.3	81.9	112.1	1104.7
4.0 - 4.9	0.3	84.4	123.5	127.0	60.1	62.3	144.2	179.0	108.9	115.9	88.4	92.9	107.5	1294.7
5.0 - 5.9	-	62.0	75.6	89.1	55.7	63.8	107.1	169.5	137.7	130.4	102.8	86.8	97.4	1177.9
6.0 - 6.9	-	49.9	66.9	90.6	79.6	56.9	63.0	131.1	144.7	139.3	108.4	76.4	79.3	1086.0
7.0 - 7.9	-	35.1	51.2	66.4	66.4	48.5	35.1	112.5	146.5	141.5	107.3	60.3	61.3	932.1
8.0 - 8.9	-	19.2	32.2	39.2	46.6	31.9	15.4	89.7	130.3	123.9	88.6	47.8	42.4	707.0
9.0 - 9.9	-	12.7	14.8	24.9	29.3	14.0	5.9	63.2	76.1	93.7	57.1	38.1	29.2	459.0
10.0 - 10.9	-	8.4	11.8	11.5	13.9	5.7	3.8	35.6	65.6	77.0	51.7	26.7	17.0	328.8
11.0 - 11.9	-	3.8	5.7	5.7	5.9	2.5	1.0	21.9	47.9	61.7	40.6	22.2	11.5	230.4
12.0 - 12.9	-	2.7	1.9	1.9	2.9	0.6	0.8	11.2	28.3	39.9	27.6	15.0	8.0	140.7
13.0 - 13.9	-	0.8	0.5	1.4	1.2	0.2	0.1	5.3	12.7	25.1	15.1	8.0	5.0	75.3
14.0 - 14.9	-	1.0	-	0.4	0.5	-	0.1	2.5	5.9	17.4	9.2	5.0	2.3	44.2
15.0 - 15.9	-	0.3	-	0.2	0.1	-	0.1	2.0	4.1	11.1	6.1	3.3	0.9	28.1
16.0 - 16.9	-	0.1	-	-	-	-	-	1.1	2.6	5.8	3.8	2.0	0.5	15.9
17.0 - 17.9	-	0.1	-	-	-	-	-	0.3	1.5	3.4	2.3	1.6	0.4	9.6
18.0 - 18.9	-	-	-	-	-	-	-	0.1	0.6	1.8	1.2	0.8	0.1	4.5
19.0 - 19.9	-	-	-	-	-	-	-	-	0.4	1.0	0.5	0.3	0.1	2.2
20.0 - 20.9	-	0.1	-	-	-	-	-	-	0.3	0.4	0.2	0.2	0.1	1.1
21.0 - 21.9	-	0.1	-	-	-	-	-	-	0.2	0.2	0.7	0.1	-	1.2
22.0 - 22.9	-	-	-	-	-	-	-	-	-	-	0.3	-	-	0.3
23.0 - 23.9	-	-	-	-	-	-	-	-	0.1	0.2	0.1	0.1	-	0.4
24.0 - 24.9	-	-	-	-	-	-	-	-	-	0.1	-	0.1	-	0.2
25.0 - 25.9	-	-	-	-	-	-	-	-	0.1	0.2	0.1	-	-	0.3
26.0 - 26.9	-	-	-	-	-	-	-	-	-	-	-	-	-	-
27.0 - 27.9	-	-	-	-	-	-	-	-	-	-	-	-	-	-
28.0 and higher	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Cumulative	40.2	539.7	629.5	626.2	429.6	395.4	651.3	1040.2	1056.5	1125.0	827.8	660.9	743.5	8766.0

Figure 116 - Frequency table of potential wind speed, source; KNMI

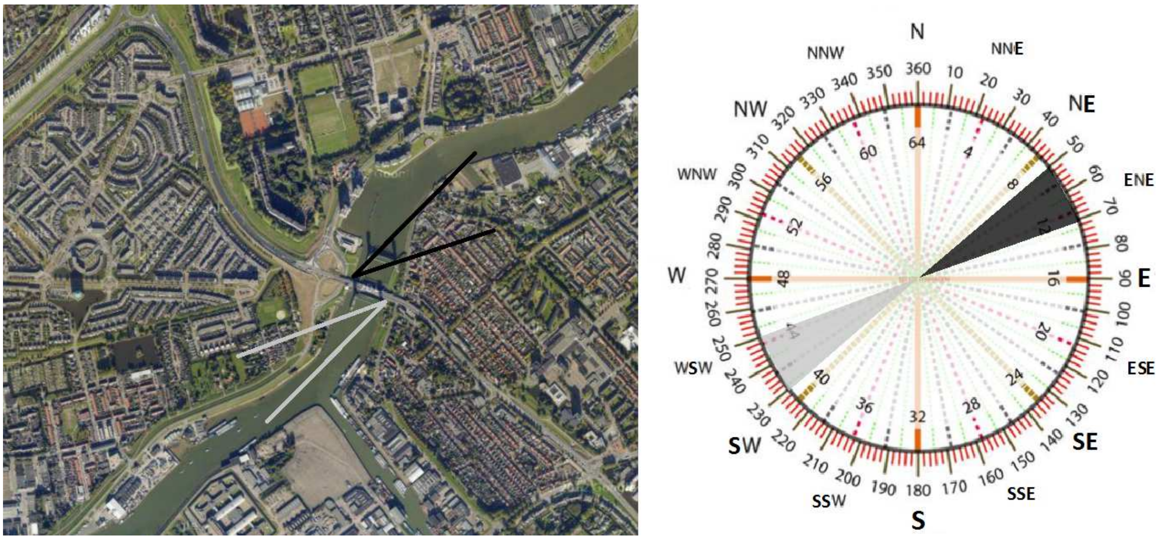


Figure 117 - Wind direction 050-070 and 230-250

The extreme wind speeds are calculated using a Gumbel distribution. The Gumbel distribution is generally used to calculate extreme wind speeds [65]. The Gumbel distribution is given as:

$$\mathrm{CDF} = \exp(-\exp^{-\frac{x-\mu}{\beta}})$$

$$PDF = \frac{1}{\beta} * e^{-z-e^{-z}}, where\ z = \frac{x-\mu}{\beta}$$

The calculation of the Gumbel Reduced Variable (G) is calculated from P which is the probability the wind speed (U) occurs. With the use of linear regression the values of G can be plotted against the wind speed. With the use of a Excel the best fit and fitting parameter (R^2) can be given.

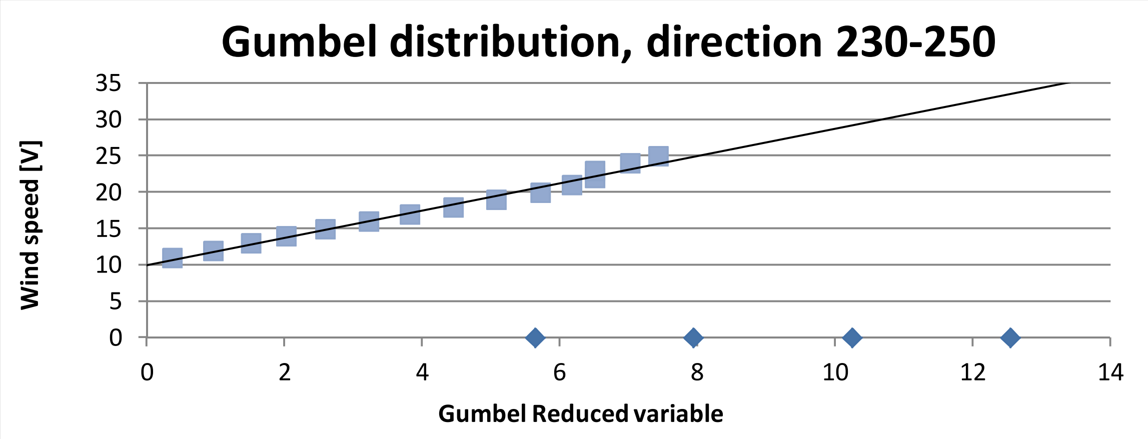
$$G = -\ln(\ln\left(\frac{1}{P}\right))$$

$$linear\ regression\ G = A * U + B$$

The windspeeds are dependent because in the same storm multiple windspeeds close to each other occur. This is prevented when the storms are statisitically analyzed. The normal storm duration in the Netherlands is approximatly 12 hours. Not every windspeed is a storm however. The KNMI advises to use 9 m/s as threshold for weatherstation Geulhaven. This threshold is chosen to prevent large sensitivity due to small wind speeds [66]. The number of storm in a year is therefore given as the hours above the threshold divided through the storm duration. The results of the Gumbel analysis are presented in the table and figures below. The points on the axis represent the exceedanace probabilities 1/10, 1/100, 1/1 000 and 1/10 000. These points are calculated using:

$$G = -\ln(\ln(\frac{Ns}{Ns - Qs})$$

In which Ns is the number of storms in a year and Q_s is the exceedance probability. This method is used in the lecture notes of Breakwater and closure dams [64].



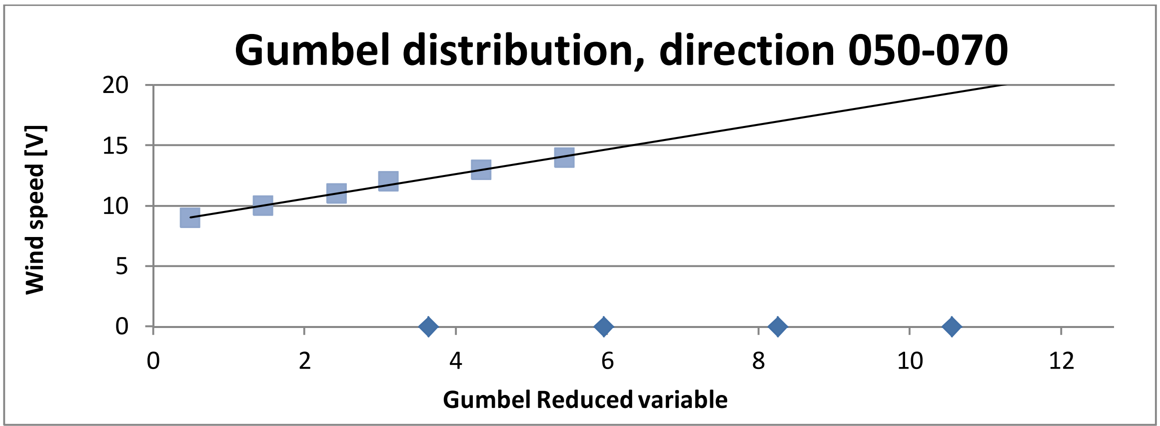


Figure 118 - Gumbel distributed extreme wind speeds

Table 70 - Normative wind speeds

Direction	Wind speed 1/10 000, threshold 9 m/s.	Wind speed 1/2 000, threshold 9 m/s.	Wind speed 1/1 000, threshold 9 m/s.
SWW (230-250)	33.5 m/s	30.5 m/s	29.2 m/s
NNO (050-070)	19.4 m/s	17.7 m/s	17.0 m/s

Appendix I.2 Wave conditions

The formula of Bretschneider can be used to calculate the significant wave height (H_s) and peak period (T_p) in front of the storm surge barrier. There are three variables that influence H_s and T_p, the wind speed, the water depth and the fetch. The fetch is the undisturbed length the wind can blow over the water. The wave height for the direction SWW and NNO is calculated for the 1/10 000 situation.

The fetch is estimated with the use of Google Maps and the normative wind direction that is given in the calculations of the normative wind speeds (shown in Figure 117). The SWW fetch does not take the structures on the West end of Stormpolder into account, it is expected that they could be removed or just flooded. The NNO fetch is limited due to the bends in the Hollandsche IJssel. The waterdepth that is based on the normative waterlevels for both situations and the bottom level of the Hollandsche IJssel needed for shipping during low water.

With these results H_s and T_p can be calculated with the program "Bretschneider Calculator", which is part of the software Hydra-B. The Bretschneider Calculator of Hydra-B uses the Bretschneider method which estimates the wave growth. The calculator is based on the two formulae given below.

$$\frac{g*H_s}{U^2} = 0.283 \tanh\left(0.53*\left(\frac{g*d}{U^2}\right)^{0.75}\right)*\tanh\left(\frac{0.0125\left(\frac{g*F}{U^2}\right)^{0.42}}{\tanh\left(0.53\left(\frac{g*d}{U^2}\right)^{0.75}\right)}\right)$$

$$\frac{g*T_p}{U} = 7.54\tanh\left(0.833*\left(\frac{g*d}{U^2}\right)^{0.375}\right)*\tanh\left(\frac{0.0077\left(\frac{g*F}{U^2}\right)^{0.25}}{\tanh\left(0.833\left(\frac{g*d}{U^2}\right)^{0.375}\right)}\right)$$

Table 71 - Significant wave height and peak period

Direction	Fetch	Waterlevel	Bottom level	Waterdepth
SWW (230-250)	1 700 m	+3.46 m NAP	-6.5 m NAP	10.0 m
NNO (050-070)	1 100 m	+2.00 m NAP	-5.0 m NAP	7.0 m

Direction	Significant waveheigth (H_s)	Peak period (T_p)
SWW (230-250)	1.18 m	3.92 s
NNO (050-070)	0.53 m	2.69 s

Appendix I.3 Governing situation storm surge and salt intrusion

The governing water levels are important for the adaptation of the storm surge barrier. The governing water levels are calculated for design conditions during storm surge and salt intrusion.

Storm surge

There are two scenarios that are important for the governing water levels in front of the storm surge barrier. First design condition is the normative situation during a storm surge. Second design condition is the water level that is reached during a salt intrusion closure. There are two extreme scenarios during a storm surge that could determine the governing water levels in frond and directly behind the storm surge barrier.

1. Extreme discharge + storm surge + no wind set-up + no discharge pumping stations

In this scenario there is extreme discharg and a storm surge but there is no storm in the surrounding that can create wind set-up and precipitation. After closure the water levels on the Hollandsche IJssel do not change because there is no wind and no precipitation. The results of the calculation are shown in Table 72

$$Design\ level\ in\ front = governing\ water\ level\ New\ Meuse$$
$$Design\ level\ behind = water\ level\ just\ after\ closure = closure\ level - average\ tidal\ elevation$$
$$Hydraulic\ head = Design\ level\ in\ front - Design\ level\ behind$$

2. Extreme discharge + storm surge + wind set up + discharge pumping stations

In this scenario there is extreme discharge, a storm surge and a local storm. Due to the local storm there will be wind set-up and precipitation. Within the time that the set up needs to develop the Hollandsche IJssel will be filled up due to the discharge of the pumping station. Result of this phenomenon is that the governing water levels in the Hollandsche IJssel are reached at the end of the closure period. This water level is equal to the pump stop level minus the developed wind set-up (the wind set-up calculation is also conducted in appendix F.8). The results of the calculation are shown in Table 72.

$$Design\ level\ in\ front = governing\ water\ level\ New\ Meuse + wind\ set\ up$$
$$Design\ level\ behind = pump\ stop\ level + wind\ set\ up$$

Table 72 - Calculation governing water level design condition 1 and 2

Parameters	Design condition 1	Design condition 2
Wind speed (U_{max})	0 m/s	33.5 m/s
Max wind set-up (h_{wind_max})	0 m	1.24 m
Wind set-up in front ($H_{In\ front}$)	0 m	0.14 m
Wind set-up behind ($H_{Just\ behind}$)	0 m	-0.62 m
Design level in front($H_{design\ in\ front}$)	3.50 m NAP	3.60 m NAP
Design level behind ($H_{design\ behind}$)	0.24 m NAP	1.38 m NAP
Hydraulic head	3.26 m	2.22 m

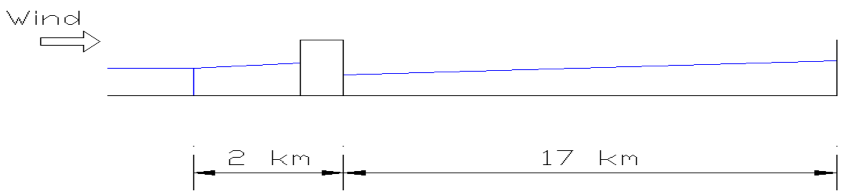


Figure 119 - Schematization design condition 2

The largest negative head is reached when inside the pump stop level is reached and outside the Maeslant barrier just closed. The Hollandsche IJssel barrier closes for lower water levels before the Maeslant barrier. It is therefore possible that the Hollandsche IJssel barrier reached the pump stop level and that the storm surge still needs to occur.

$$Water\ level\ in\ front = water\ level\ on\ New\ Meuse\ just\ after\ closure\ Maeslant\ barrier$$
$$Water\ level\ behind = pump\ stop\ level$$

Salt intrusion

The largest positive head is reached when the inlet stop level is reached and a spring tide occurs. The spring tide (1.61 m) is used because the barrier is closed for a long time (up to a month) compared to the storm surge (12 hours), in other cases the average tidal elevatios is used. The water level reached during salt intrusion is determined in appendix G.4.

$$Water\ level\ in\ front = water\ level\ reached\ during\ salt\ intrusion + spring\ tidal\ elevation$$
$$Water\ level\ behind = inlet\ stop\ level$$

The largest negative head is reached a few hours after closure. In front of the storm surge barrier the water levels are at the lowest point, behind the barrier the water levels are high due to the closure during the preceding flood slack period. The water levels in the Hollandsche IJssel are high because the storm surge barrier closes in the flood slack period before salt intrusion becomes a problem. Water levels in the Hollandsche IJssel are therefore determined by the low water level plus the average tide. For the low water level before salt intrusion -0.5 m NAP is used.

$$Water\ level\ in\ front = lowest\ water\ level\ reached\ during\ periods\ of\ salt\ intrusion$$
$$Water\ level\ behind = low\ water\ level\ in\ tide\ before\ closure + average\ tidal\ elevation$$

Table 73 - Design conditions storm surge and salt intrusion

Design condition	Head	In front	Behind	Hydraulic head
Storm surge	Max pos. head	+3.50 m NAP	+0.24 m NAP	3.26 m
	Max neg. head	+1.26 m NAP	+2.00 m NAP	-0.74 m
Salt intrusion	Max pos. head	+1.11 m NAP	-0.50 m NAP	1.61 m
	Max neg. head	-0.85 m NAP	+1.01 m NAP	-1.86 m

Appendix I.4 Pressure distribution gate

The pressure distribution on the gate is created with the use of the governing water levels for the design conditions storm surge and salt intrusion. There are two aspects that create the pressure distribution on the gate, the hydrostatic pressure and the wave pressure. The hydrostatic pressure at each depth is calculated using the formula given below.

$$F_{hydrostatic} = \rho * g * h$$

The wave pressure is calculated using the approximation given by Sainflou. This method can be used when the waves are non-breaking. The waves are non-breaking because there is no depth-induced or steepness-induced breaking.

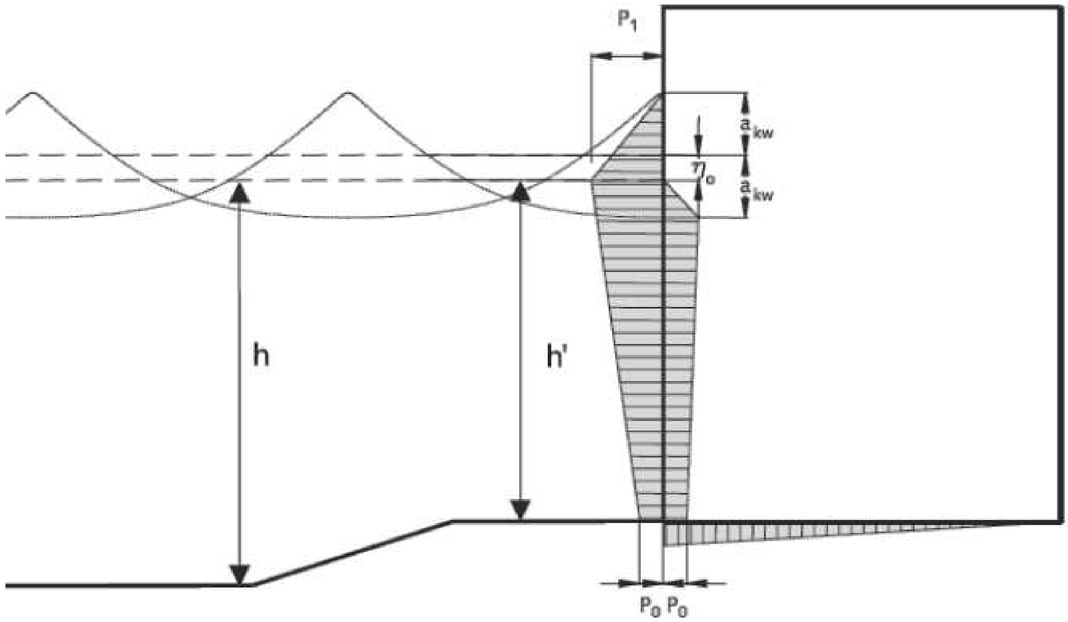


Figure 120 - Schematization according to the method Sainflou, source; Manual Hydraulic structure CT3330

$$\eta_0 = 0.5 * k * H_s * \coth(k * h')$$

$$H_d = 2 * H_i$$

$$k = \frac{2\pi}{1.56 * T^2}$$

$$p_1 = \rho * g * (H_i + \eta_0)$$

$$p_0 = \frac{\rho * g * H_i}{\cosh(k * h')}$$

With this formula the pressure distributions on both sides of the gate can be calculated. The resulting force casting on the gate is the difference between the pressures. The used parameters are shown in Table 74.

$$\rho_{gate} = \rho_{outside} + \rho_{wave} - \rho_{inside}$$

Table 74 - Overview used parameters Sainflou

Parameter	Storm surge	Salt intrusion	Unit
$H_{design\ outer}$	3.50	-0.85	m NAP
$H_{min\ inner}$	0.24	1.01	m NAP
H_{sill}	-6.50	-6.50	m NAP
H_i	1.18	0.60	m
T_p	3.92	2.69	s
k_{wave}	0.26	0.56	1/m
p1	11.58	5.20	kN
p0	1.69	0.45	kN
$\eta 0$	0.18	0.08	m
H_{design} +h_0	3.64	1.09	m

The resulting pressure distributions are shown in Figure 121 and Figure 122.

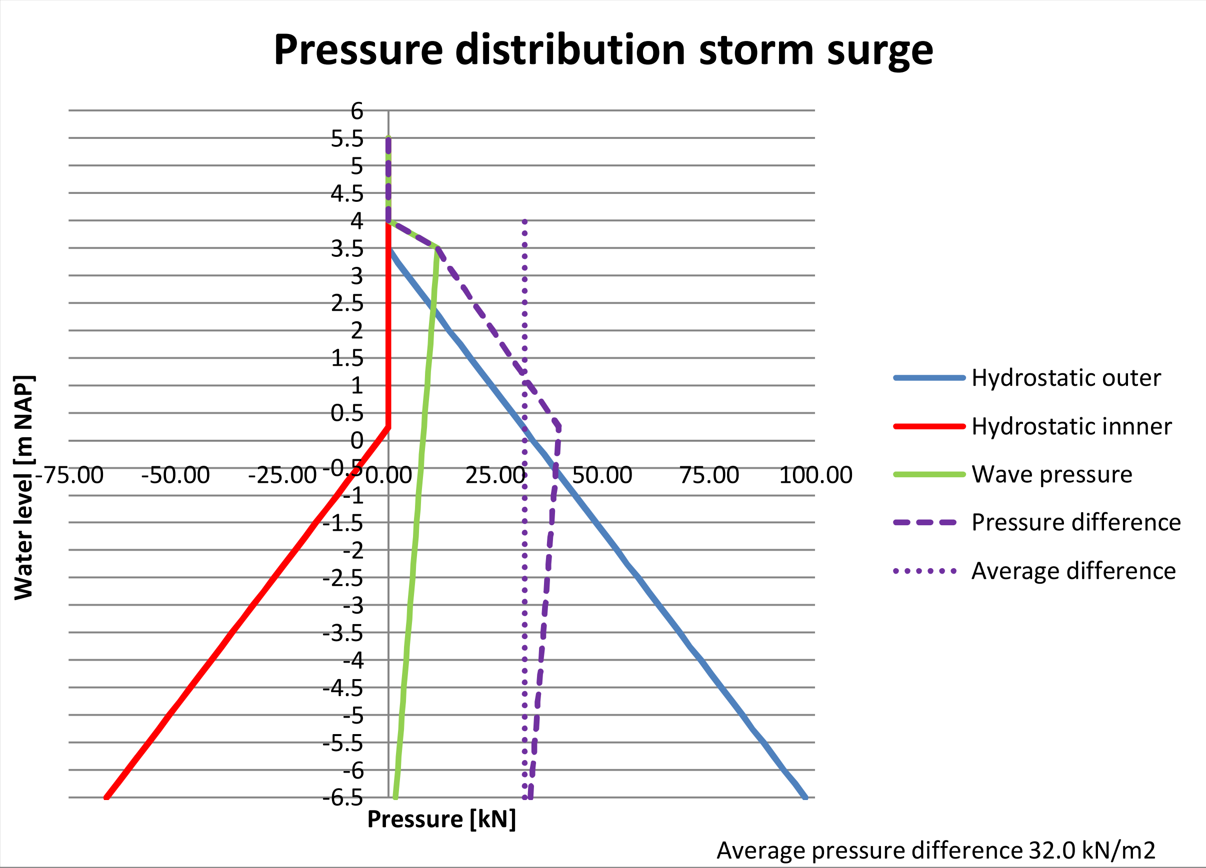


Figure 121 - Water pressure storm surge

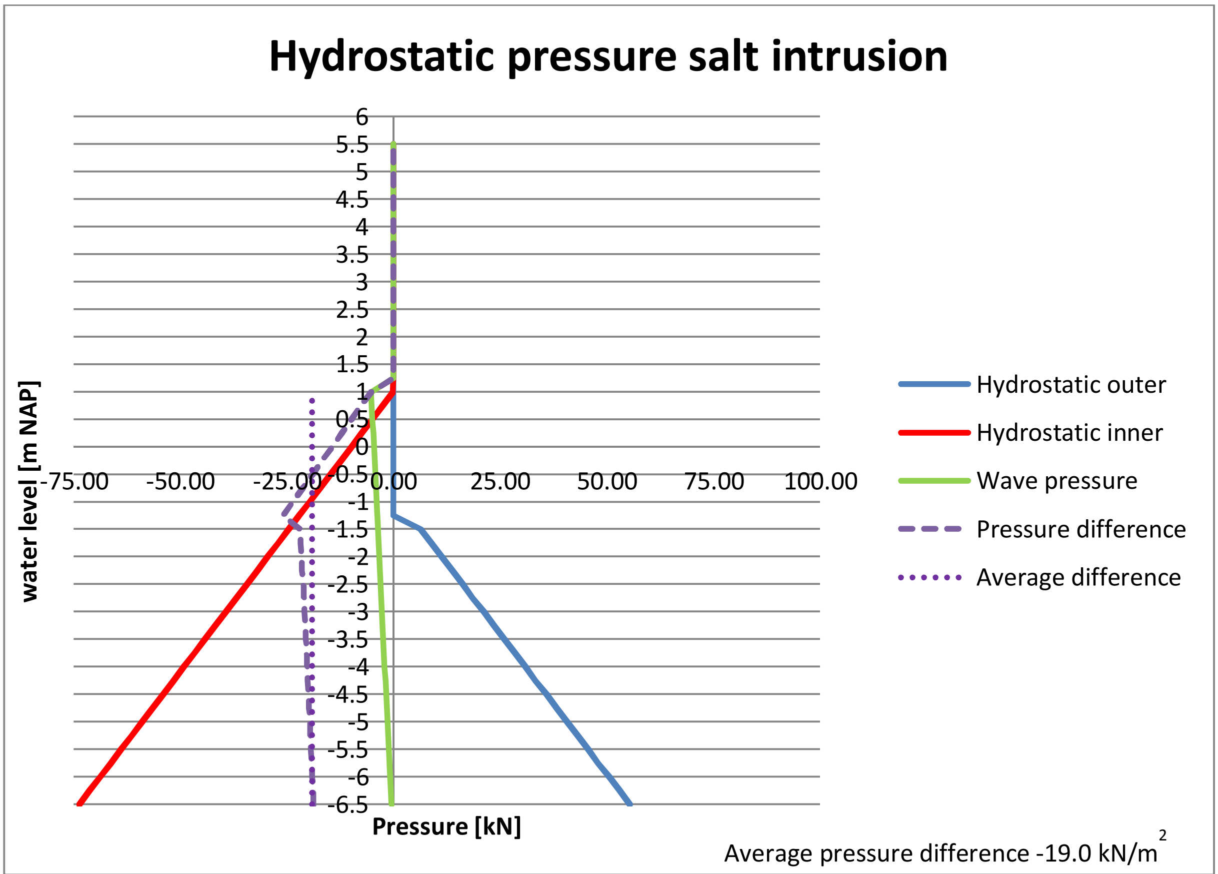


Figure 122 - Water pressure salt intrusion

Appendix J Lay-out and history storm surge barrier

In this appendix the lay-out and nationwide safety assessment are described. The first part describes the system and lay-out of the storm surge barrier; the second part focuses on the nationwide safety assessment conducted between 2006 and 2011.

Appendix J.1 Lay-out storm surge barrier Hollandsche IJssel

In the third nationwide safety assessment the Hollandsche IJssel storm surge barrier was assessed by Witteveen+Bos [47]. The overview of the storm surge barrier is shown in Figure 123; the different elements are described in Table 75. The different elements of the storm surge barrier are used in the assessment of the storm surge barrier and are laid down in the ledger of the barrier. The ledger describes the regulations and the different zones of the flood defence. All elements that are part of the flood defence are part of the core zone; the protection zone limits some activities in this area. This zone is not part of the flood defence but activities in this region could affect the safety. The stability of the flood defence is for example threatened if a hole is dug in this area.

Figure 123 - Overview elements Hollandsche IJssel barrier, source; Rijkswaterstaat

Table 75 - Elements of the Hollandsche IJssel barrier

Name	Section	Description	Length [m]
First barrier	A1	Connection to dike ring 14, earthen dam	100
	B1	Hollandsche IJssel lock	70
	C1	Storm surge barrier Hollandsche IJssel	80
	D1	Connection to dike ring 15, earthen dam	100
Second barrier	A2	Connection to dike ring 14, earthen dam	100
	B2	Hollandsche IJssel lock	60
	C2	Storm surge barrier Hollandsche IJssel	80
	D2	Connection to dike ring 15, earthen dam + concrete wall	100

There is not much information available on the storm surge barrier. Due to merges and bankruptcy a lot of information is lost. The calculations and models are described using technical drawings obtained from the archives of Rijkswaterstaat Utrecht. Drawings and models are shown in appendix O, P and Q.

Appendix J.2 History storm surge barrier

The Hollandsche IJssel storm surge barrier was built after the flood disaster of 1953 and completed in 1958. The constructed barrier consisted of a lock and one lift gate of the barrier. Because of the limited budget only one of the two gates was completed in 1958.

In 1976 the gate got stuck and needed to be renovated. After this the department of Public Works and the water boards decided to construct the second gate that was originally planned. This gate should increase the reliability of the system. The gate that was originally designed in 1958 was optimized on some points. The rivets were replaced with preloaded bolts and welds.

In 1998 a ship collided with the gate on the inner side. Both gates were replaced in 2000, only the end supports of the gates could be reused in the new gates. Each of the gates weighs approximately 400 tons.

In the last safety assessment the Hollandsche IJssel storm surge barrier was assessed. The results of this assessment are shown in Figure 124. The failure mechanisms that are assessed are; height (HT), piping (STPH) slope stability (STBI, STVL, STCG and STBU), micro instability (STMI), stability cover (STBK), non-water retaining objects (NWO), strength structure (STCO) and non-closure (BS). After the safety assessment, the problems concerning the grass cover (STBK) were solved, only the non-closure probability was too high.

Sectie	HT	STPH	STBI	STBU	STMI	STBK	STVL	NWO begroeiing	NWO bebouwing	Toetsoordeel derde toetsronde
A1	goed goed	goed goed	goed goed	goed goed	goed goed	onvoldoende onvoldoende	goed goed			onvoldoende onvoldoende
D1	goed goed	goed goed	goed goed	goed goed	goed goed	onvoldoende onvoldoende	goed goed	goed goed	goed goed	onvoldoende onvoldoende
A2	goed goed	goed goed	voldoende voldoende	onvoldoende voldoende	goed goed	onvoldoende onvoldoende	goed goed		goed goed	onvoldoende onvoldoende
D2	goed goed	goed goed	goed goed	goed goed	goed goed	onvoldoende onvoldoende	goed goed			onvoldoende onvoldoende
Eindscore										onvoldoende

Sectie	HT	STPH	STVL	STCG	STCO	BS	Toetsoordeel derde toetsronde
B1	goed goed	goed goed	goed goed	voldoende voldoende	voldoende voldoende	onvoldoende onvoldoende	onvoldoende onvoldoende
C1	goed goed	voldoende voldoende	goed goed	voldoende voldoende	voldoende voldoende	onvoldoende onvoldoende	onvoldoende onvoldoende
B2	goed goed	goed goed	goed goed	voldoende voldoende	voldoende voldoende	onvoldoende onvoldoende	onvoldoende onvoldoende
C2	goed goed	voldoende voldoende	goed goed	voldoende voldoende	voldoende voldoende	onvoldoende onvoldoende	onvoldoende onvoldoende
Eindscore							onvoldoende

Figure 124 - Results third nationwide safety assessment, source; Rijkswaterstaat

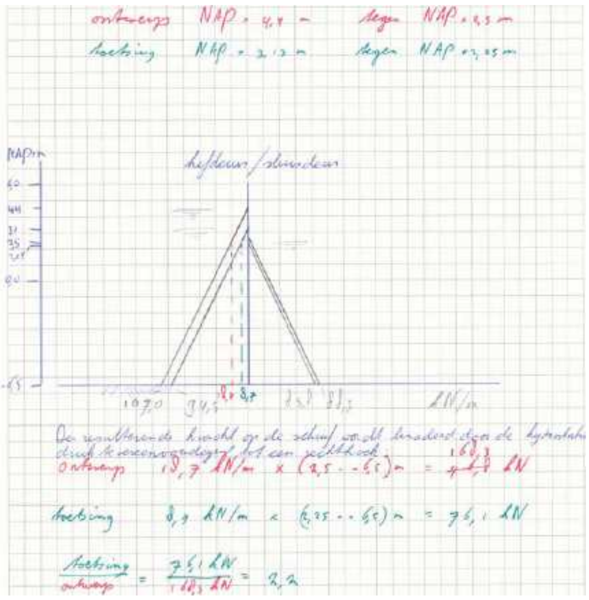


Figure 125 - Assessment STCO, source; Rijkswaterstaat

The assessment of the storm surge barrier does not focus on the exact assessment of all the failure mechanisms. It is important to make it plausible that the strength of the structure is up to the standards. The strength of the structure is for example assessed with the use of the old and new design levels (shown in Figure 125).

The assessment of piping (STPH) is conducted using the head difference (outside and inside water level). Piping is not a problem because the sheet piles are connected to deep clay layers which closes off the aquifers.

Appendix K Structural assessment storm surge barrier

In this appendix the structural analysis of the storm surge barrier is described. The first part describes the assessment of the gate; the second part treats the assessment of the tower and sill, the last part describes the total integrity of the storm surge barrier.

Appendix K.1 First assessment gate (proven strength)

The nationwide safety analysis assessed the strength of the gates with a comparison of the hydrostatic pressures [47]. With this simple comparison it can be shown if the gate can be used to withstand the storm surge. The hydrostatic pressures of the design are compared to the pressures of the new water levels, only the governing water levels are used (no waves).

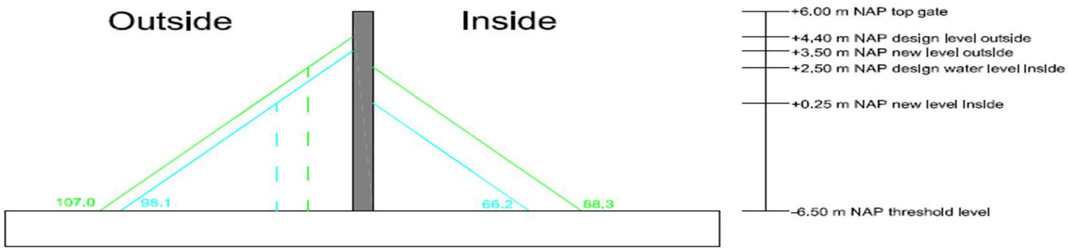


Figure 126 - Schematic overview assessment existing gate, design level (green), new level (cyan)

The resulting hydrostatic pressure is obtained when the water pressures of both sides are subtracted from each other.

$$\Delta design = outside - inside = 107.0 - 88.3 = 18.7\ kN/m$$
$$\Delta new \ = outside - inside = 98.1 - 66.7 = 31.4\ kN\ /m$$

This load can be simplified to a rectangle and a triangle, shown as the bracketed line in Figure 126. The total water pressure is given as:

$$F\ = \Delta * inside\ governing\ level + 0.5 * \Delta * difference\ water\ levels$$
$$Fdesign = 18.7 * 9.0 + 0.5 * 1.9 * 18.7 = 186\ kN$$
$$Fnew = 31.4 * 6.75 + 0.5 * 3.25 * 31.4 = 263\ kN$$

Result of this calculation is that the new load is larger than the design force. The first crude calculation does not show that the gate can withstand the forces.

Appendix K.2 Tower and sill

There is not much information available about the structural integrity of the storm surge barrier (towers and sill). During design conditions the towers should be able to transfer the forces out of the gates. The towers are made of concrete; concrete is especially used for the transfer of pressure forces. This part therefore focuses on the transfer of the forces (into the tower) using pressure. Transfer of the forces through the storm towers is not considered.

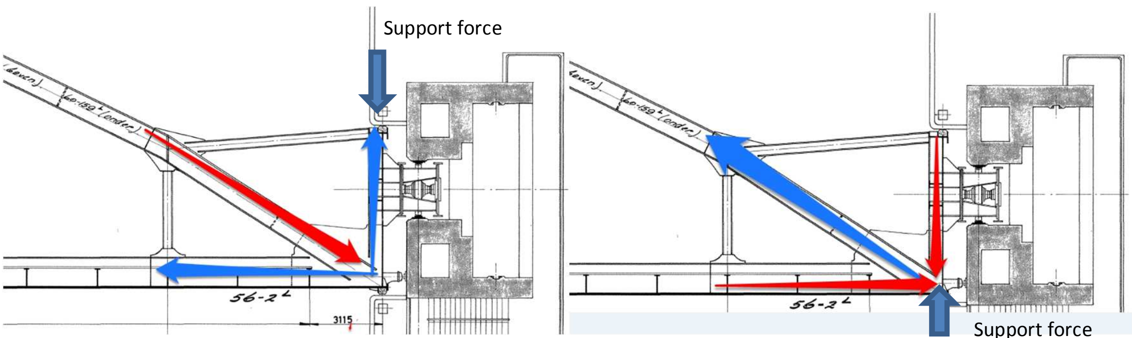


Figure 127 - Forces near the supports; storm surge (left), salt intrusion (right); compressive (blue), tensile (red).

The tensile forces resulting from the applied loads are not transferred to the tower. During a storm surge the gate wall is pressed against the back of the tower, during salt intrusion the plate wall is pressed against the front of the tower. In both cases there is a compressive support force from the tower, the transfer of the compressive forces through the concrete should not be a problem.

Appendix K.3 Increase forces storm surge barrier Hollandsche IJssel

The stability of the barrier is not checked with the use of the normal design checks (turning, bearing capacity and sliding) because the structure is founded on piles. These design checks are used for a shallow foundation. Therefore the relative increase of the forces is calculated. When the increase is between the safety margins it is expected that the storm surge barrier is safe. For forces that change sign the situation should be analyzed. The forces that are working on the storm surge barrier are described in Table 76 and schematized in Figure 128.

Table 76 - Description forces overall stability

Force	Description	Formula
F1	Hydrostatic pressure outside	$0.5*g*\rho*H_o\text{^}2 * L$
F2	Hydrostatic pressure inside	$0.5*g*\rho*H_i\text{^}2 * L$
F3	Water load on sill outside	$g*\rho*0.5*B*(H_o\text{-}c) * L$
F4	Water load on sill inside	$g*\rho *0.5*B*(H_i\text{-}c) * L$
F5	Upward water pressure rectangle (sill + tower)	$(\rho* g*H_i * B *L) + (\rho*g*Hi * B * 2*Bt)$
F6	Upward water pressure triangle (sill + tower)	$(0.5* g* \rho * (H_o\text{-}H_i) * W* L) + (0.5* g* \rho * (Ho\text{-}Hi) * W *Bt)$
F7	Wave pressure	$0.5* (\rho*g*H_s) * (H_o\text{-}c) *L$
F8	Wave pressure	$0.5* (\rho*g*H_s) * H_s *L$
F9	Dead weight towers	$V_t* \rho_b$
F10	Dead weight sill	$c * L* B* \rho$
F11	Dead weight gate	$m_g* g$

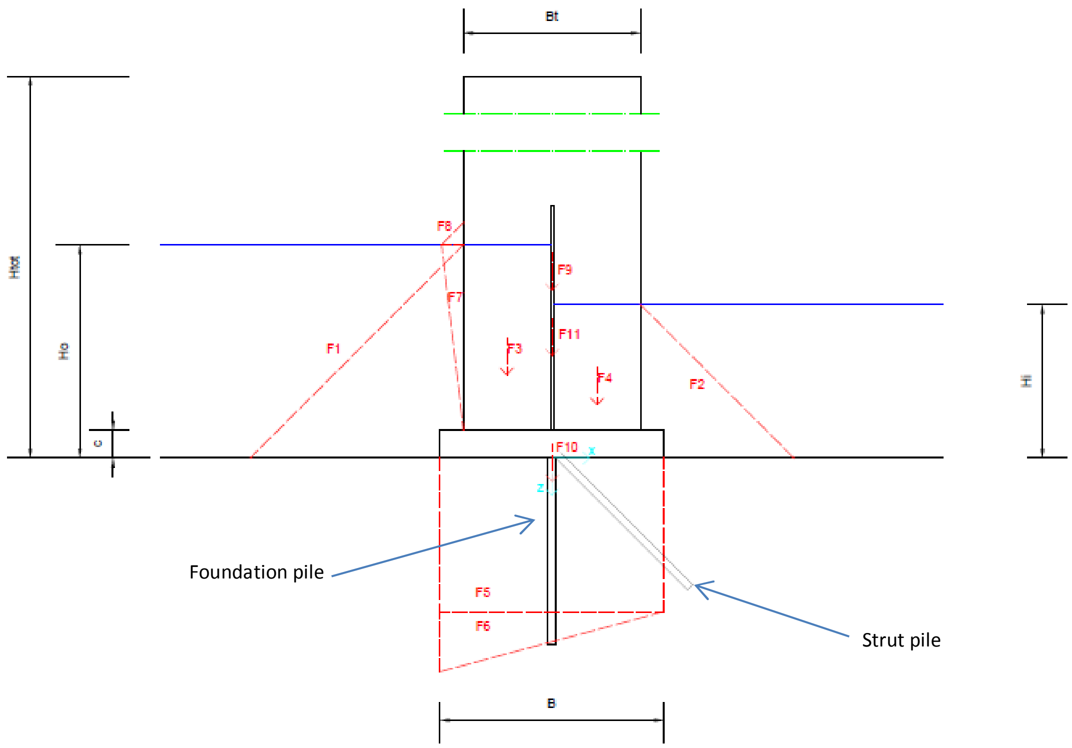


Figure 128 - Schematic overview forces and barrier

Table 77 - Parameters forces overall stability

Parameter	Symbol	Value	Unit
Gravity	g	9.81	m^3/s
Density	ρ	1 000 (water) and 2 500 (concrete)	kg/m^3
Governing water level outside	$H_{outside}$	4.4 (old design), 3.50 (storm), -0.85 (salt)	m NAP
Governing water level inside	H_{insde}	2.0 (old design), 0.24 (storm), 1.01 (salt)	m NAP
Sill level	H_{sill}	-8.0	m NAP
Height tower	H_{tower}	45.0	m NAP
Wave height	H_s	1.18 (storm) and 0.53 (salt intrusion)	m
Width sill	B	12.0	m
Width towers	B_t	9.5	m
Length sill	L	80.0	m
Length towers	W	6.5	m
Thickness sill	c	1.5	m
Weight gate	m_g	400	ton
Concrete volume towers	V_t	2 708	m^3

$$H_{tot} = H_{tower} - H_{threshold}$$
$$H_o = H_{outside} - H_{threshold}$$
$$H_i = H_{inside} - H_{threshold}$$

Table 78 - Forces for the two load combinations

Parameter	Old design situation	Storm surge	Salt intrusion	Unit
Horizontal force (H)	26 700	30 000	-10 200	kN
Vertical force (V)	68 500	71 500	75 500	kN
Bending moment (M)	164 000	160 000	-35 000	kNm
Relative increase H	100	112	-38	%
Relative increase V	100	104	110	%
Relative increase M	100	98	-21	%

Given the parameters shown in Table 77 the total forces acting on the structure can be calculated for the old and new design conditions. The calculation of the forces during a storm surge uses the governing water levels “storm”; the calculation of the forces during salt intrusion uses the governing water levels “salt” (shown in Table 78).

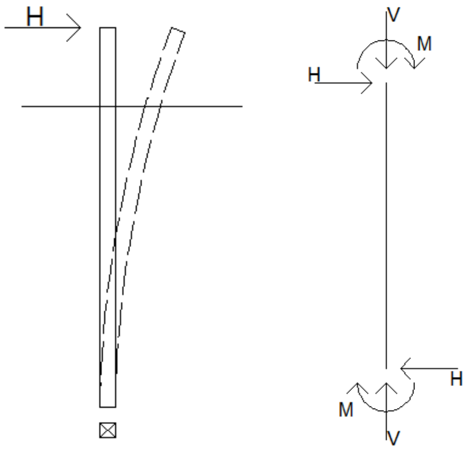
The difference between the forces can be used to check the stability during the new governing conditions. All forces act on the zero axis of the schematic overview, middle underside sill.

The pile foundation will transfer the forces to the soil. The horizontal load will be transferred through the strut piles (piles under an angle); the vertical force will be transferred through both the horizontal and strut piles. In general pile foundations have a robust design because there are a lot of uncertainties in the soil. After 50 years the soil around the storm surge barrier has settled and compacted, therefore it can be expected that the soil can bear more forces. If the increase (or decrease) of the forces is limited and positive no problems should be expected. The soil will probably be able to transfer the forces with the highest increase (which is 110%), because safety factors for loads are between 120% and 150% which is higher than the increase of the loads.

The negative forces due to the load combination can however cause problems, there are no strut piles schematized in the other direction. A horizontal pile can however transfer horizontal loads through momentum (bending of the foundation pile) [61].

Table 79 - Parameters horizontal pile force

Parameter	Symbol	Value	Unit
Horizontal force	H_{max}	100	kN
Pile length (h+t)	L	10 000	mm
Height pile above ground	h	0	mm
Height pile beneath ground	t	10 000	mm
Pile deflection (demand 1/100*L)	δ	100	mm
Bending stiffness $E*1/6*b*h^3$	EI	$8.53 * 10^{13}$	Nmm^2
Elasticity modulus concrete	E	20 000	N/mm^2
Thickness foundation pile	b	400	mm



$$H = \frac{(3 * \delta * EI)}{(h + 0.65t)^3}$$

Figure 129 - Schematization pile foundation

The formulae to calculate the horizontal force is given above, the parameters and result are given in Table 79. The horizontal force that needs to be transferred in the horizontal component of the moment and the horizontal force due to the water pressure on the gates. These forces are shown in Table 78. The total horizontal force is given as;

$$H_{tot} = H + \frac{M}{L} = -10\ 200 + \frac{-35\ 000}{10} = -13\ 700\ kN$$

The number of piles that would be needed to transfer the horizontal force and momentum is then given as;

$$n_{piles} = \frac{H_{tot}}{H_{max}} = \frac{13700}{100} \approx 140\ piles$$

The number of piles used in the storm surge barrier is more then 350, therefore there are no problems concerning the negative forces. The structural analysis of the tower shows that the tower is able to transfer the forces during the design conditions storm surge and salt intrusion.

Appendix K.4 Piping

The design checks concerning piping can be addressed whether a shallow or pile foundation is used. Piping is a process were water creates holes under the structure and threatens the stability when sand erodes under the structure. The increase of the governing water levels does not result in piping problems because the sheet piles under the storm surge barrier are connected into the second clay layer [4].

Appendix L Detailed structural assessment steel gate

The simple assessment (proven strength) of the gate conducted in appendix K.1 shows that the gate does not fulfill the requirements; therefore a detailed assessment of the steel gate is necessary. In the first part the loads and steel gate are schematized, after that the critical elements of the steel gate are assessed.

Appendix L.1 Loads and schematization steel gate

The first part of this section describes the loads on the gate; the second part of this section describes the model created in RSTAB.

Appendix L.1.1 Loads gate

The arch type gate is a gate that uses horizontal struts to transfer the forces from the plate wall to an arch that is loaded with a tensile or compressive force. The idea behind this type of structure is the reduction of the moments in the structure. Nearly all forces are transferred using either tension or compression. The gate of the storm surge barrier is loaded in two directions. During the storm surge the force distribution in the gate is given according to upper part of Figure 130. There is a tensile force in the arch, the struts and plate wall are under compression. During salt intrusion the water levels inside are higher than outside therefore the force distribution in the gate changes according to the lower part of Figure 130. There is a tensile force in the struts and plate wall, the arch is under compression. During salt intrusion the gate is closed for a longer period and there are changes in the hydraulic head due to the tide, therefore fatigue might be a problem.

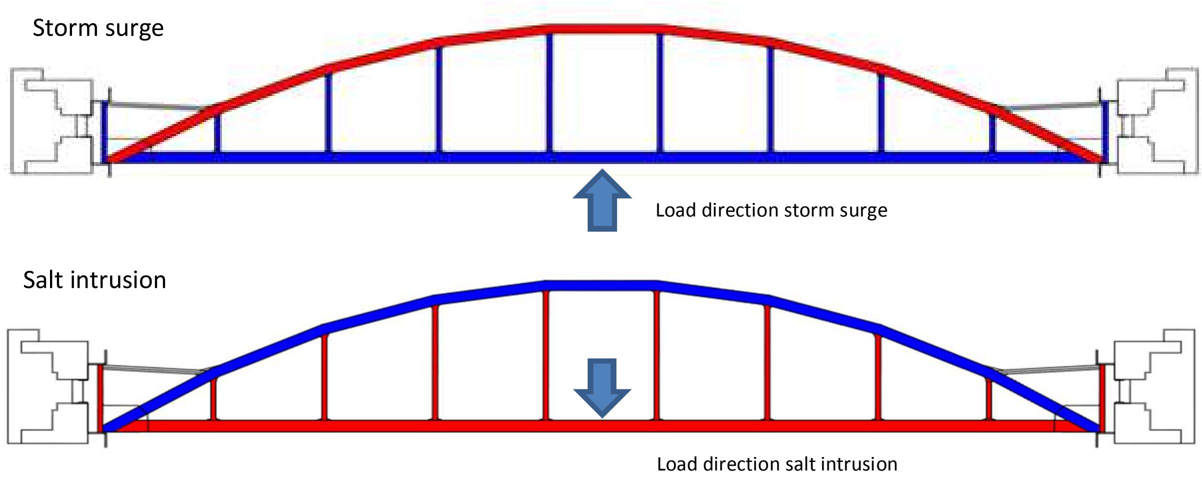


Figure 130 - Forces in the gate during; tensile (red) and compressive (blue)

In appendix I.3 and I.4 the pressure distribution and load combinations are studied. The important load combinations that change due to the adaptation of the storm surge barrier are storm surge and salt intrusion. The load combination storm surge changes because the governing water levels on the New Meuse increase. The load combination salt intrusion is new because the barrier has not been used during salt intrusion. The average difference between the pressure distributions, calculated in appendix I.4, is multiplied with the safety factor for permanent loads (shown in Figure 131). The water pressure is a quasi-permanent load during design conditions which does not vary a lot, therefore the permanent partial factor (γ_p) is used and not the variable load factor. The distributed load is applied on the part of the gate which is loaded due to the waves and hydrostatic pressure, this height ($h_{applied}$) is derived from Figure 121 and Figure 122.

$$q_{d;storm\ surge} = \gamma_p * p_{rep;storm\ surge} = 1.2 * 32 = 38.4\ kN/m$$
$$h_{applied;storm\ surge} = H_{top\ wave} - H_{underside} = 4 - -6.5 = 10.5\ m$$
$$q_{d;salt\ intrusion} = \gamma_p * p_{rep;salt\ intrusion} = 1.2 * -19 = -22.8\ kN/m$$
$$h_{applied;salt\ intrusion} = H_{top\ wave} + H_{underside} = 1.5 - -6.5 = 8\ m$$

The forces in the gate are calculated using simple calculations in combination with Matrix Frame and Dlubal RSTAB 8.01. RSTAB and Matrix Frame are software programs that can be used to calculate the forces in the gate. The plate wall (with transverse and longitudinal stiffeners) is schematized in Matrix Frame according to

Figure 131. The support reactions shown in this figure represent the distributed load (per running meter) acting on the girder (straight arch) directly connected to the plate wall (girder in same plane as the curved arch).

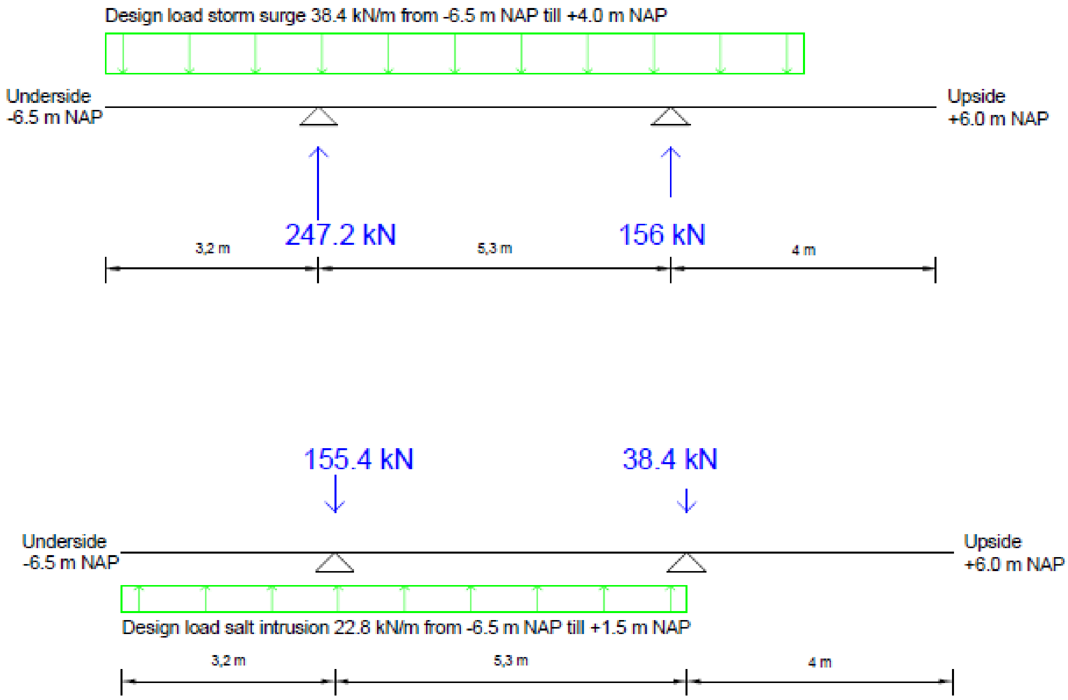


Figure 131 - Schematization design loads storm surge and salt intrusion on one meter plate wall

Appendix L.1.2 Schematization steel gate

The gate is schematized according to the technical drawings obtained from the archives of Public Works. The technical drawing is shown in Figure 157 and Figure 158; the gate is modeled in RSTAB and shown in Figure 132. The support forces of Matrix Frame (shown in Figure 131) are used as input for the model created in RSTAB.

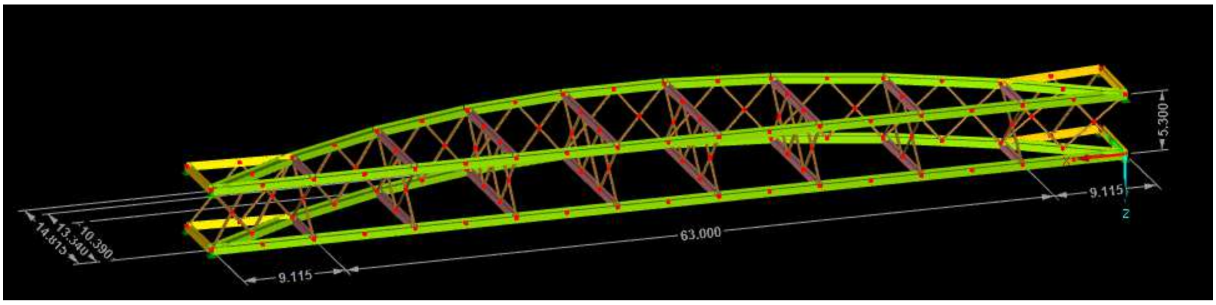


Figure 132 - Schematization steel gate RSTAB (dimensions in meters)

Figure 133 shows the normal forces when the load combination storm surge is occurs. This load combination occurs when the barrier is closed and the governing water level on the New Meuse has been reached. The distributed load is applied to the straight arch girder (green straight beam) that is directly connected to the plate wall (which schematized as part of the straight arch girder in this model). The struts (and x-bracing connected to the struts) transfer the compressive forces to the curved arch. The compressive force in the struts pushes the curved arch outward and creates a tensile force in the arch which is in equilibrium at the supports with a compressive force in the plate wall. The x-bracing in the gate is predominantly used for the redistribution of forces and stiffness of the gate in general.

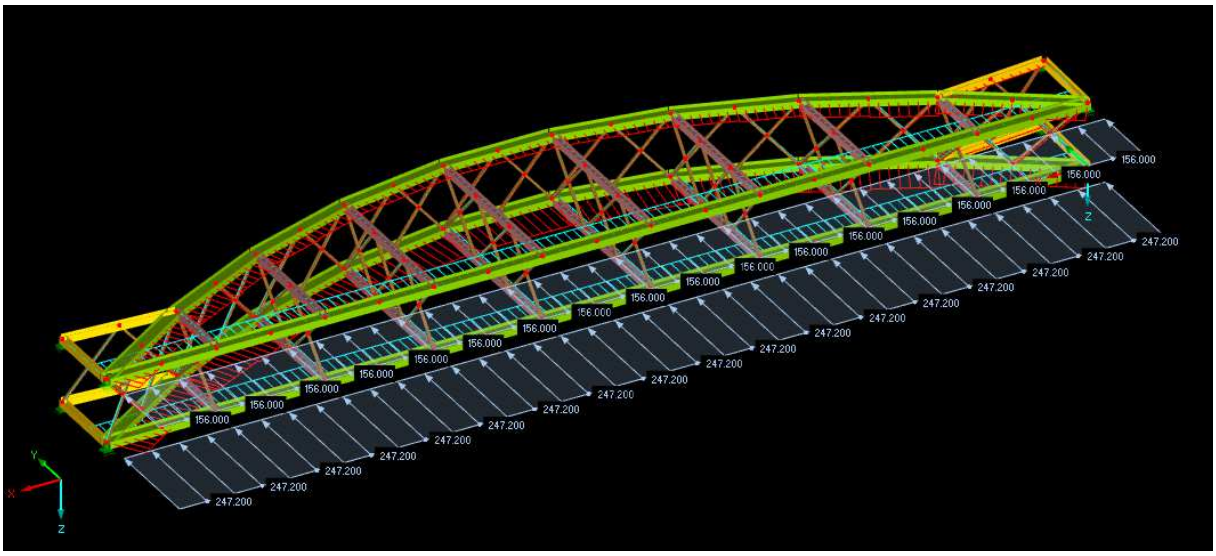


Figure 133 - Schematization storm surge RSTAB; tensile (red) and compressive (blue)

Figure 134 shows the normal forces when the load combination salt intrusion occurs. This load combination occurs when the barrier is closed during salt intrusion and there is an ebb tide outside. The distributed load is applied to the same straight arch which uses struts to transfer the loads to the curved arch. Because of the negative distributed load the parts which were under compression are now in tension and vice versa.

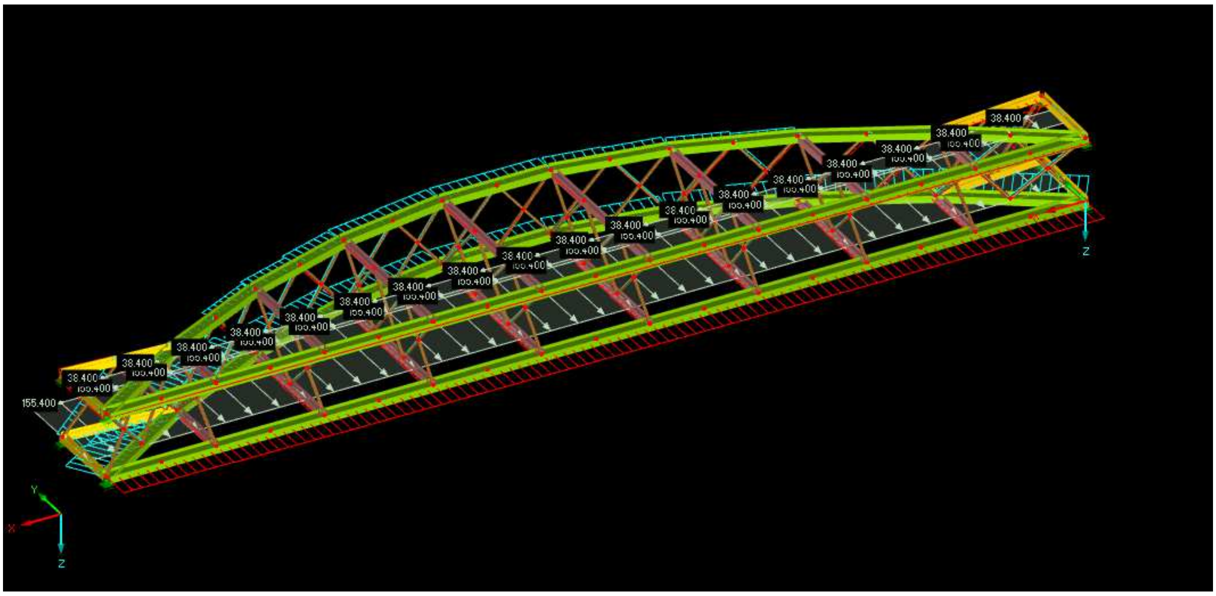


Figure 134 - Schematization salt intrusion RSTAB; tensile (red) and compressive (blue)

The supports that are connected to the arch and plate wall (shown in Figure 135) are schematized as roll supports because the force that is transferred through the arch should balance with a force in the schematized plate wall. When the support is schematized as a fixed support al forces will be transferred through these supports and will not result in a force in the schematized plate wall. The supports at the back of the steel gate are also roll supports but turned sideways to create a static equilibrium.

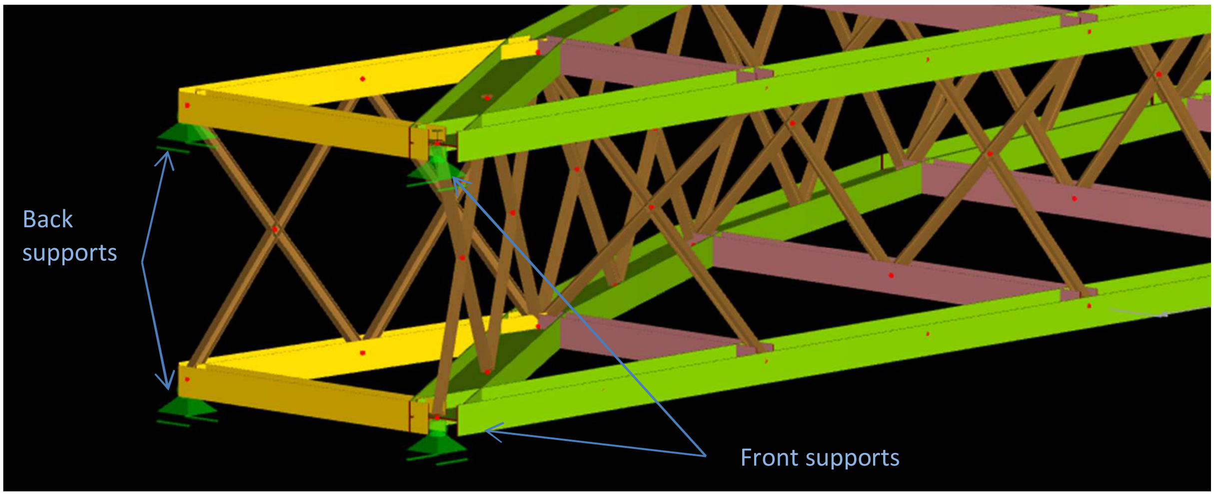


Figure 135 - Schematization supports

Appendix L.2 Assessment structural elements

In this part of the appendix the strut and arch are assessed for the load combinations storm surge and salt intrusion.

Appendix L.2.1 Assessment strut

The force in the horizontal strut is equal to the part of the distributed load that needs to be transferred to the arch. Each strut is located at 9 meter center to center. This means that the maximum force in the strut is given as;

$$F_{strut} = q_{max} * l_{ctc}$$
$$F_{strut;storm\ surge} = q_{max;storm\ surge} * l_{ctc} = 247.2 * 9 = -2\ 225\ kN$$
$$F_{strut;salt\ intrusion} = q_{max;salt\ intrusion} * l_{ctc} = 155.4 * 9 = 1\ 243\ kN$$

The model created in RSTAB shows similar results for the maximum forces in the struts.

$$F_{strut;storm\ surge} = -2\ 240\ kN$$
$$F_{struts;salt\ intrusion} = 1\ 301\ kN$$

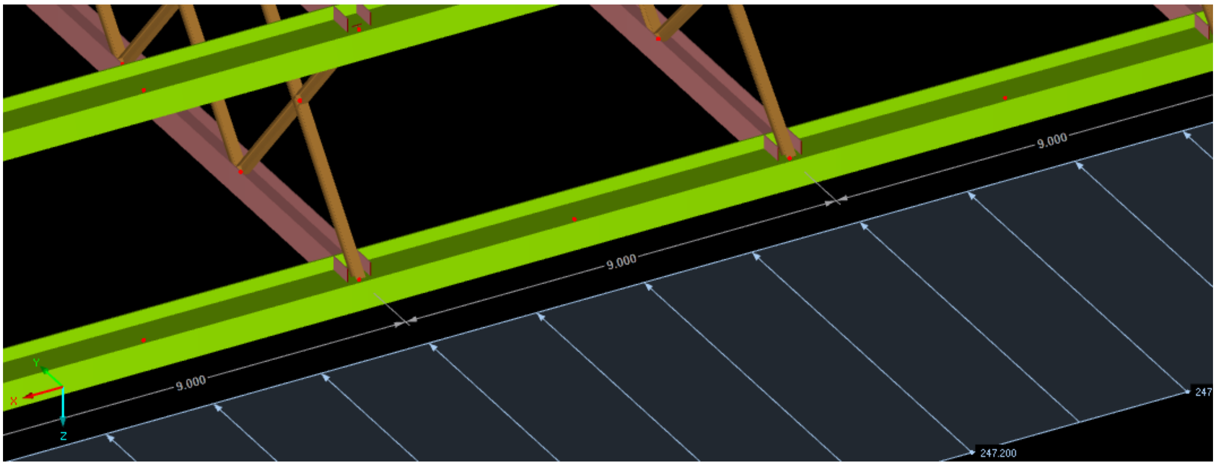


Figure 136 - Schematization strut (red) and arch (green) in RSTAB

The resulting compressive force during the load combination storm surge is larger than the tensile force during the load combination salt intrusion (other forces in the strut are negligible); therefore the strut should be assessed for a compressive force and consequently buckling.

Buckling

Buckling normally occurs in the direction of the weak axis. The weak axis is the axis that has the smallest section modulus; the strong axis is the axis that has the highest section modulus. The struts that transfer the largest forces are however supported in the weak axis by x-bracing (brown elements shown in Figure 136). Therefore the strong axis of the whole element is assessed and only the unsupported length of the weak axis is assessed. The governing situation depends on the slenderness of both parts. The specific slenderness is calculated using;

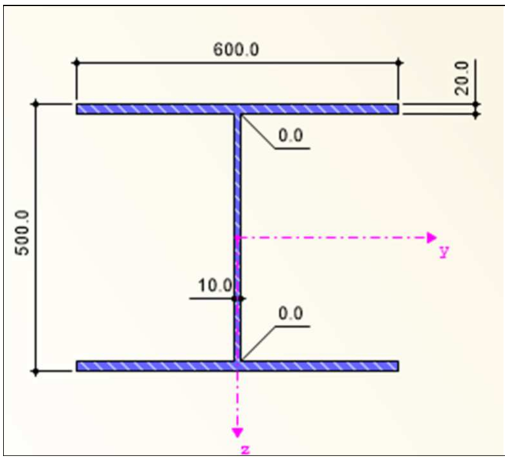
$$\lambda_1 = \pi * \sqrt{\frac{E}{f_y}} = \pi * \sqrt{\frac{210\,000}{235}} = 93.91$$

The relative slenderness of the steel element is calculated using the formula given below;

$$\bar{\lambda} = \frac{\frac{L_{cr}}{i}}{\lambda_1} =$$

In this formula the critical length L_{cr} is equal to the unsupported length of the element; the radius of gyration (i) is the parameter that is used to describe the distribution of the material around the axis. The radius of gyration is larger when more material is located at a larger distance. The unsupported length in the strong direction is the full profile length which is 14.815 m the unsupported length in the weak direction is 4.94 m.

$$i = \sqrt{I/A}$$



Profile information	
Cross-sectional area A	28600 mm^2
Moment of inertia I_y	$1.46*10^9$ mm^4
Moment of inertia I_z	$7.2*10^8$ mm^4
Section modulus W_y	$5.9*10^6$ mm^3
Section modulus W_z	$2.4*10^6$ mm^3
Radius of gyration i_y	226 mm
Radius of gyration i_z	159 mm
Buckling around y axis	curve b
Buckling around z axis	curve c

Figure 137 - Welded steel section strut, steel grade S235 and dimensions in mm

Table 80 - Parameters buckling

Parameters buckling calculation	Symbol	Value	Unit
Specific slenderness	λ_1	93.91	-
Elasticity modulus steel	E	210 000	N/mm^2
Yield strength S235	f_y	235	N/mm^2
Buckling length	L_{cr}	4.94 and 14.815	m
Relative slenderness	$\bar{\lambda}$	0.33 and 0.70	-
Model factor	γ_{m1}	1.1	-

$$\overline{\lambda_{weak}} = \frac{\frac{4\,940}{159}}{93.91} = 0.33$$

$$\overline{\lambda_{strong}} = \frac{\frac{14\,815}{226}}{93.91} = 0.70$$

The calculation of the relative slenderness shows that the strut is weakest around the unsupported strong axis. The reduction factor χ can be obtained from the buckling curves shown in Figure 138; buckling curve b should be used for the strong axis of the profile. The buckling reduction factor χ is 0.7.

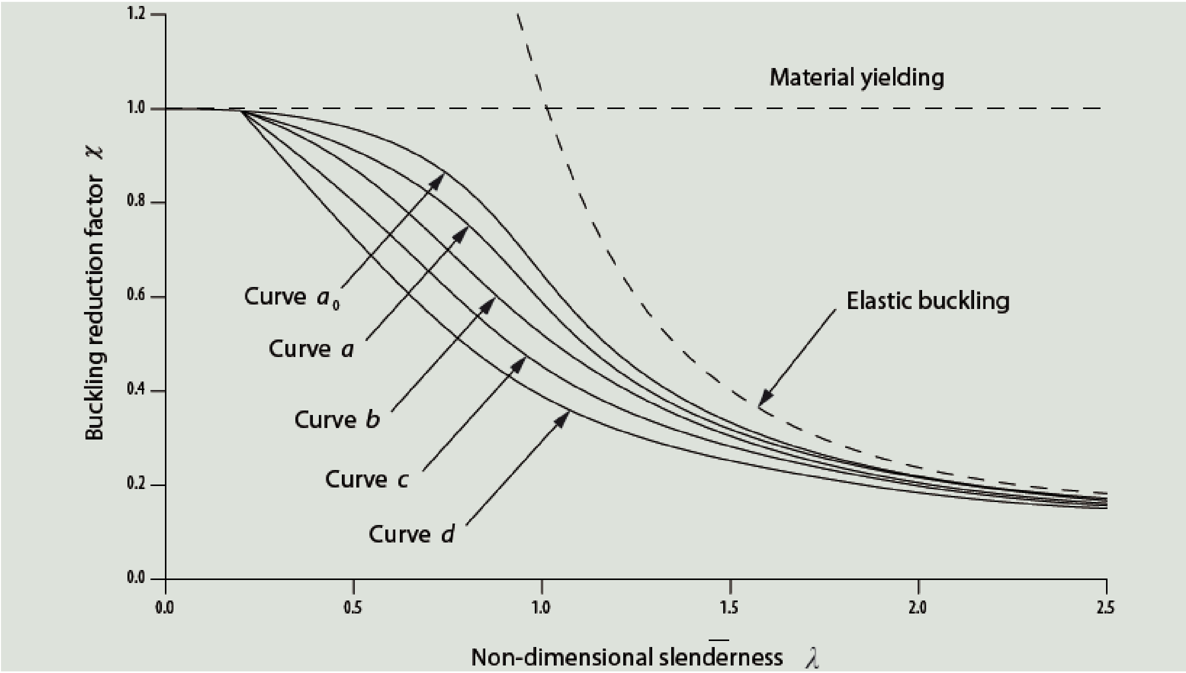


Figure 138 - Buckling curves, source; NEN EN 1993-1-1

The design buckling resistance for a compressive force is given as;

$$N_{b;R;d} = \frac{\chi * A * f_y}{\gamma_{m1}} = \frac{0.7 * 28\,600 * 235}{1.1} = 4\,277.0\ kN$$

The assessment of the strut compares the design value with the resistance of the element.

$$\frac{N_{strut}}{N_{b;R;d}} = \frac{2\,240}{4\,277} = 0.52 \leq 1\ OK$$

The assessment used in codes compares the resistance of the element with the design load, when the design load divided by the resistance is less than one the element is safe.

Appendix L.2.2 Assessment curved arch

The idea behind an arch is the transfer of the forces without the use of moments. The model created in RSTAB shows some moments due to the dead weight and introduction of the forces from the strut into the arch. The important force in the arch is however the normal force that transfers the forces applied to the plate wall to the supports.

Figure 130 shows that the arch transfers a compressive force during the load combination salt intrusion and a tension force during the load combination storm surge. The curved arch should be checked for two situations;

- High tensile force in combination with a moment due to the x-bracing and struts (load combination storm surge).
- Lower compression force in combination with buckling and a moment due to x-bracing and struts (load combination salt intrusion).

The arch should be checked for both combinations because buckling due to a compressive force lowers the actual strength of the profile. The moments generated near the supports are not used to assess the profiles because these moments are transferred using the stiffness of the end of the steel gate (combination of arch, plate wall and support beams shown in Figure 132). The moments in the z-direction are due to the dead weight of the structure, these moments are small (approximately 100 kNm) and do not increase the stress in the arch considerably.

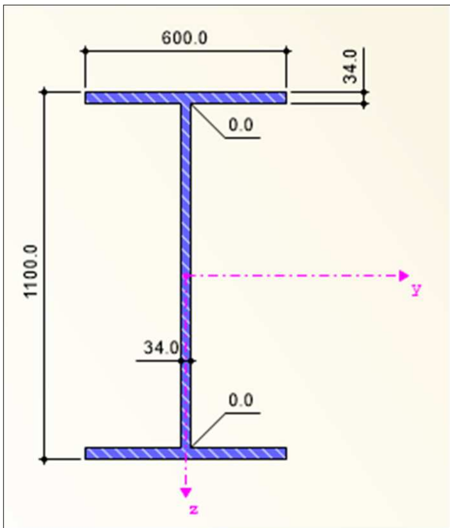
The resulting forces in the arch are obtained from the model created in RSTAB and presented in Table 81. The force in the arch can also be approximated with a manual calculation, the results are shown in the formulae below and comparable to the results obtained from the model. The arch should transfer the forces in the struts therefore the total force transferred through the arch should be of the same order as the force obtained from the model. X-bracing redistributes the forces in the steel gate therefore the forces are not exactly the same.

$$N_{arch;storm\ surge} = F_{strut} * n_{strut} = 2\ 225 * 8 = 17\ 800\ kN$$
$$N_{arch;salt\ intrusion} = F_{strut} * n_{strut} = 1\ 243 * 8 = 9\ 944\ kN$$

Table 81 - Resulting forces curved arch obtained from model

Load combination	Normal force N_{arch}	Moment in y direction $M_{y\text{-}arch}$
1 Storm surge	17 200 kN	1 370 kNm
2 Salt intrusion	-8 505 kN	960 kNm

<u>Buckling</u>
The methods used for the calculation of the buckling reduction factor can be repeated for the curved arch. The profile used for the arch is a welded I profile shown in Figure 139.



Profile information	
Cross-sectional area A	75 888 mm^2
Moment of inertia I_y	$1.47*10^{10}$ mm^4
Moment of inertia I_z	$1.23*10^9$ mm^4
Section modulus W_y	$2.7*10^7$ mm^3
Section modulus W_z	$4.1*10^6$ mm^3
Radius of gyration i_y	440 mm
Radius of gyration i_z	127 mm
Buckling around y axis	curve b
Buckling around z axis	curve c

Figure 139 - Welded steel section arch (steel grade S235 and dimensions in mm)

Figure 140 shows that the profile used for the arch is supported in the weak axis and unsupported in the strong axis. The critical length (L_{cr}) of the arch becomes 5.01 and 10.02 meter.

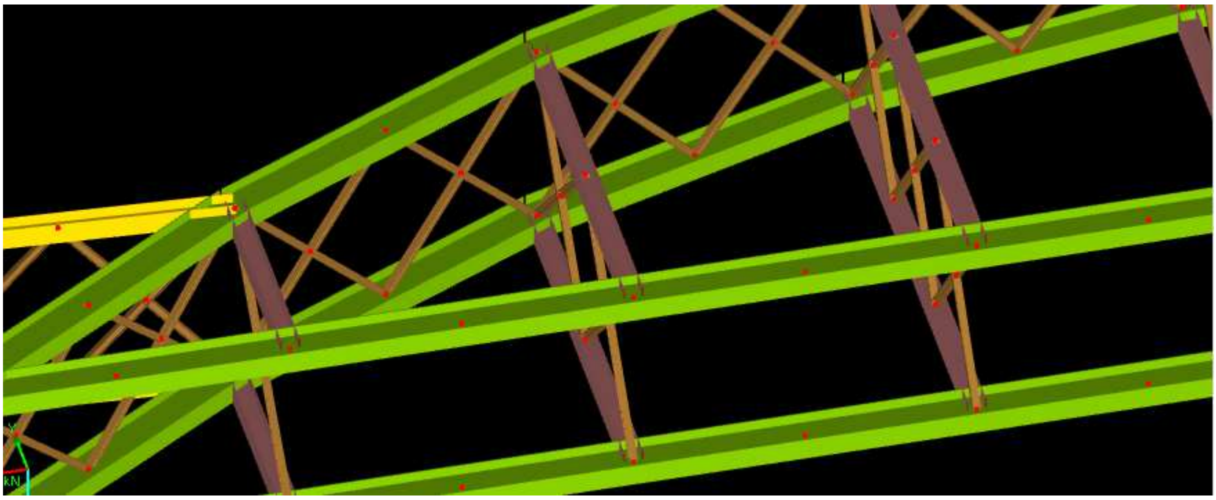


Figure 140 - Schematization strut (red), x-bracing (brown) and arch (green) in RSTAB

$$\overline{\lambda_{weak}} = \frac{\frac{5\,010}{127}}{93.91} = 0.42$$

$$\overline{\lambda_{strong}} = \frac{\frac{10\,020}{440}}{93.91} = 0.24$$

The buckling reduction factor is obtained from Figure 138 with the use of the relative slenderness. The buckling reduction factor χ is 0.8. The resistance of the profiles is calculated using the formulae given below. The model factor γ_{m1} for buckling is 1.1; the model factor for tensile forces is 1.0.

$$(tensile)\ N_{R;d} = \frac{A * f_y}{\gamma_{m1}} = \frac{75\,888 * 235}{1.0} = 17\,833.7\ kN$$

$$(compressive)\ N_{b;R;d} = \frac{\chi * A * f_y}{\gamma_{m1}} = \frac{0.8 * 75\,888 * 235}{1.1} = 12\,970\ kN$$

$$M_{M;R;d} = f_y * W_{strong} = 235 * 2.7 * 10^7 = 6\,345\ kNm$$

The assessment of the arch compares the design value with the resistance of the element, when the result of the formula is less than one the element can transfer the forces.

$$(tensile)\ UC_1 = \frac{N_{storm\ surge}}{N_{R;d}} + \frac{M_{storm\ surge}}{M_{R;d}} = \frac{17\,200}{17\,833.7} + \frac{1\,370}{6\,345} = 1.18 \leq 1\ NOT\ OK$$

$$F_E = \frac{\pi^2 * E * I}{L^2} = \frac{\pi^2 * 210\,000 * 1.47 * 10^{10}}{10\,020^2} = 303\,000\ kN$$

$$n = \frac{F_E}{F} = \frac{303\,000}{8\,505} = 36$$

$$(compressive)\ UC_2 = \frac{N_{sslt\ intrusion}}{N_{b;R;d}} + \frac{n}{n-1}\frac{M_{salt\ intrusion}}{M_{R;d}} = \frac{8\,505}{12\,970} + \frac{36}{36-1} * \frac{960}{6\,345} = 0.81 \leq 1\ OK$$

The assessment shows that the arch is safe concerning the new load combination salt intrusion and not safe concerning the increased load combination storm surge.

Appendix L.3 Assessment connection

The connections in the steel gate are designed using preloaded bolts and gusset plates. The connections between the arches and struts predominantly transfer compressive forces because the strut transfers compressive forces when the storm surge barrier is closed during a storm surge. These forces are transferred using the contact surface between the two profiles.

When the steel gate is closed due to salt intrusion the struts and connections transfers a tensile force, the connections are not designed to transfer these forces therefore the connection should be assessed. The tensile force that should be transferred is equal to the tensile force in the strut, which is 1 301 kN.

The connection shown in Figure 141 is modeled using the technical drawings found in the archives of Rijkswaterstaat, the pictures are shown in appendix P.

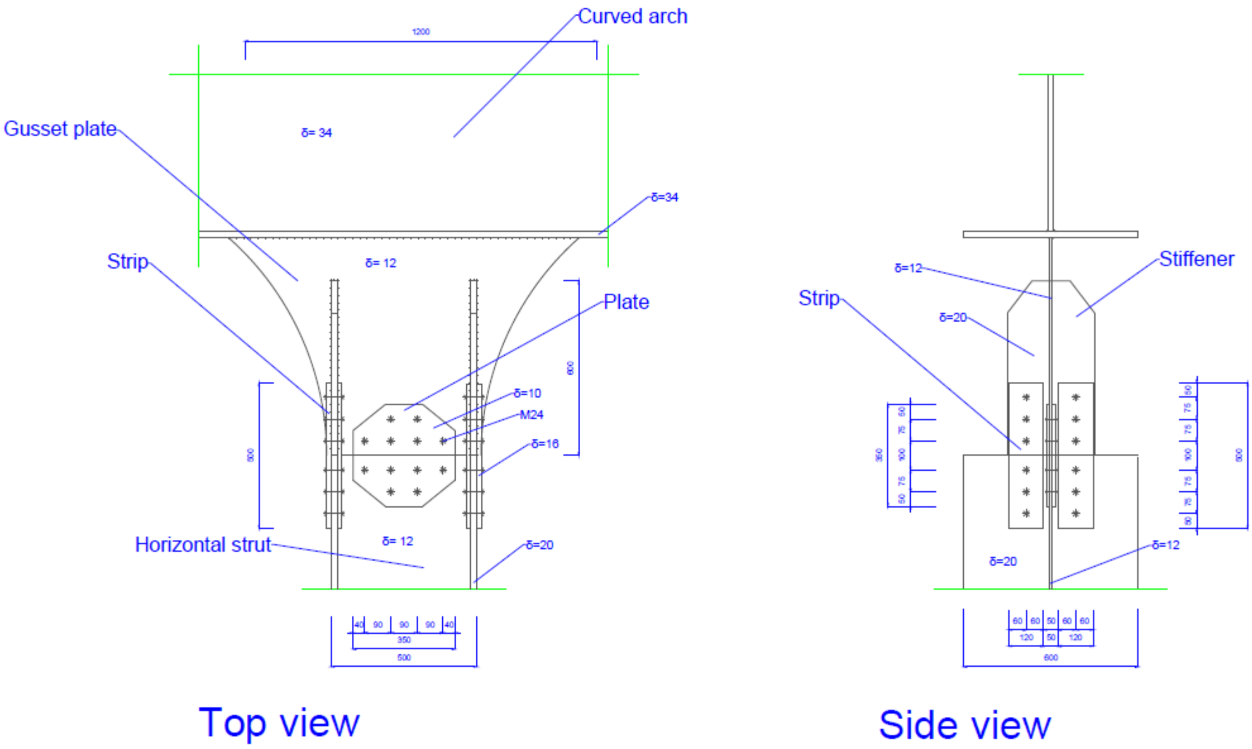


Figure 141 - Modeled connection strut-arch (steel grade S235 dimension in mm)

When the connection needs to transfer a tensile force the two profiles are pulled apart. The contact surface between the two profiles is therefore not used to transfer the forces. The forces should be transferred using the bolted and welded connection. The bolted connection consists of M24 preloaded injection bolts. There are three parts of the connection that transfer the forces from the horizontal strut to the gusset plate;

- The strips connected to the flange of the strut and to the stiffener of the gusset plate,
- The plates connected to the web of the strut and to the gusset plate,
- Weld connecting the gusset plate to the arch.

Appendix L.3.1 Capacity connection strut-gusset plate

The strips should be assessed for slip of the preloaded bolts and failure of the welds. The plate should be assessed on slip and shear of the bolts. The forces of the two parts together result in the tensile capacity of the connection. Due to the higher governing compressive force during a storm surge other parts are not assessed. The assessments of the connections are conducted using lecture notes and college sheets of CIE4115 Steel structures 2 [67].

Capacity strip

Bolt

The capacity of the preloaded bolted connection can be calculated using the design formulae of the design slip resistance. This formula is given as;

$$F_{g;u;d} = \frac{k_s * m * n * \mu}{\gamma_{m1}} * F_p$$
$$F_p = k_p * f_{t;b;rep} * A_{b;s}$$

Table 82 - Parameters buckling

Parameters preloaded bolts	Symbol	Value	Unit
Model factor for size holes	k_s	1 (holes with normal clearance)	-
Model factor for method preloading	k_p	0.7 (turn of the nut)	-
Model factor	γ_m	1.25 (ultimate limit state)	-
Number of cuts	m	2 (cuts both strips one time)	-
Number of preloaded bolts	n	6 (six bolts per side)	-
Friction coefficient	μ	0.3 (only brushed)	
Preload force	F_p	246.4	kN
Strength of the bolt	$f_{t;b;rep}$	1 000 (strength class 10*9)	N/mm^2
Surface bolt	$A_{b;s}$	352 (M24)	mm^2

$$F_p = 0.7 * 1\,000 * 352 = 246.4\ kN$$
$$F_{g;u;d} = \frac{1 * 2 * 6 * 0.3}{1.25} * 246.4 = 710\ kN$$

There is however an extra assessment for long connections, due to the long connection the force distribution in the strip may not be even and one bolt could transfer all the forces. The assessment is that the total distance between the first and last bolt is less than 15 times the bolt diameter ($d_{b;nom}$).

$$L_{fb;lb} \leq 15 * d$$
$$150 \leq 360\ OK$$

Weld

The capacity of the strips also depends on the stiffener which is welded to the gusset plate. There are three different possibilities in which the weld could fail; the gusset plate could tear apart (1), the weld could tear apart (2) and the stiffener could tear apart. The capacity of the stiffener (3) is governing because the forces need to be transferred using shear stress which is unfavorable compared to the normal stress. The strength of these welds is calculated using;

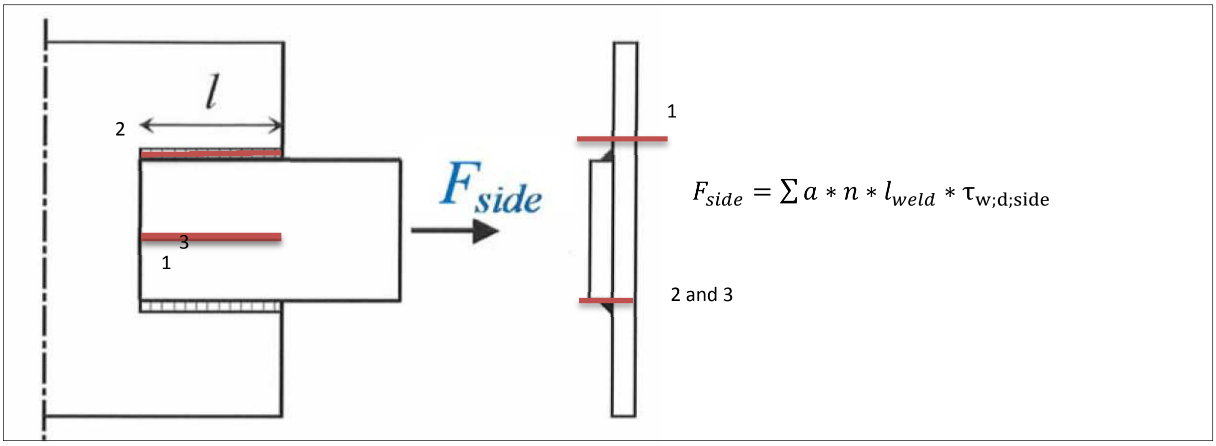


Figure 142 - Side fillet welds, source; CIE 4115

Table 83 - Parameters buckling

Parameters preloaded bolts	Symbol	Value	Unit

Throat of the weld	a	5	mm
Sides of the weld	n	2 (both sides of the stiffener are welded)	-
Length of the weld	L_{weld}	600 (shown in Figure 141)	mm
Yield strength of the	$\tau_{w;d;side}$	$f_{yd}/\sqrt{3} = 136$	N/mm^2
Capacity weld	F_{side}	-	kN

$$F_{side} = 2 * (5 * 2 * 600 * 136) = 1\,632\ kN$$

The capacity of the preloaded bolts is governing because the capacity of the weld is much larger.

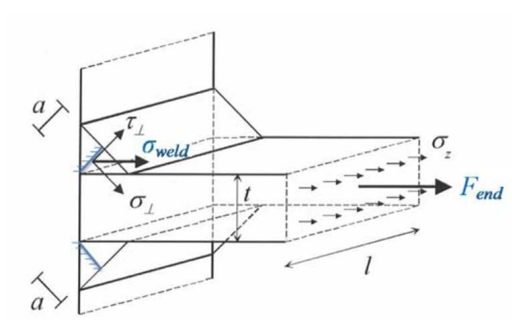
Capacity plate
The capacity of the preloaded bolts in the plate is calculated using the same formulae as the preloaded bolted connection in the strip. The number and kind of bolts that are used are the same. The capacity of this connection is therefore the same as the capacity of the strip, which is 710 kN.

Conclusion
The total capacity of the strut-gusset plate connection is the sum of the strip and plate capacity because the capacity of the weld is larger than the capacity of the preloaded bolts.

$$F_{strut-gussetplate} = F_{strip} + F_{plate} = 710 + 710 = 1\,420\ kN$$

Appendix L.3.2 Capacity connection gusset plate-arch

The gusset plate and arch are connected using a double sided fillet weld. The assessment of the weld is described using Figure 143 and formula 18.19 from the lecture notes of steel structures 2.



$$\frac{t}{2 * a} * \sqrt{2\sigma_z{}^2 + 3\tau_{zy}{}^2} \leq \frac{f_t}{\beta * \gamma_m}$$

Figure 143 - End fillet weld, source; CIE 4115

The connection only transfers a normal force (σ_z); there is no transfer of a shear force (τ_{zy}). This means that the formula reduces because $\tau_{zy}=0$

$$\frac{\sigma_z}{f_t} * \frac{\beta * \gamma_m}{\sqrt{2}} * t \leq a$$

$$\sigma_z = \frac{F_d}{t * l_{weld}}$$

Table 84 - Parameters preloaded bolts

Parameters preloaded bolts	Symbol	Value	Unit
Throat of the weld	a	4 (shown in Figure 141)	mm
Tensile strength	f_t	360 (S235)	N/mm^2
Normal stress in the gusset plate	σ_z	-	N/mm^2
Design tensile force	F_d	1301	kN
Thickness gusset plate	t	12 (shown in Figure 141)	mm
Length of the weld	l_{weld}	1300 (shown in Figure 141)	mm
Model factor	β	0.80 (S235)	-
Model factor	γ_m	1.25 (ultimate limit state)	-
Length of the weld	l_{weld}	1 300	mm

$$\sigma_z = \frac{1\,301 * 10^3}{12 * 1\,300} = 83.4\,N/mm^2$$

$$\frac{83.4}{360} * \frac{0.8 * 1.25}{\sqrt{2}} * 12 \leq 4 = 2\,mm \leq 4\,mm\,OK$$

The calculated throat is smaller than the throat of the weld, therefore the weld is safe.

Appendix L.3.3 Assessment total capacity connection

The total capacity of the connection is at least 1 420 kN ($F_{R;d}$) the design tensile force is 1 301 kN ($F_{E;d}$), therefore the connection does not fail during the load combination salt intrusion.

$$\frac{F_{E;d}}{F_{R;d}} \leq 1 = \frac{1\,301}{1\,420} = 0.92 \ \leq 1\,OK$$

Appendix L.4 Assessment plate wall

The plate wall consists of multiple elements in different directions that together create a wall that should withstand the distributed load applied by the water pressure. The different elements of the plate wall are:

- Two straight horizontal girders that are part of the framework that transfer the forces to the curved arch.
- Transverse girders (stiffeners) that are supported by horizontal girders and welded to the plates.
- Longitudinal stiffeners that are welded to the plates and transverse girders.
- Steel plates that cover the frame of transverse girders and longitudinal stiffeners.

The plate wall is schematized in Figure 144 using the technical drawings obtained from the archives of Rijkswaterstaat. The distributed load applied to the plates is transferred to the transverse girders using the longitudinal stiffeners, the transverse girders transfer the loads to the arch and struts connected to the plate wall. The normal force applied to the side of the plate wall is created due to the equilibrium needed at the end supports.

The framework of the arch and strut created in RSTAB is used to calculate the forces that are applied to the sides. The local forces due to the distributed load are manually calculated. The plate wall is schematized to make the assessment of the plate wall possible; when the original plate wall is used finite element programs are needed.

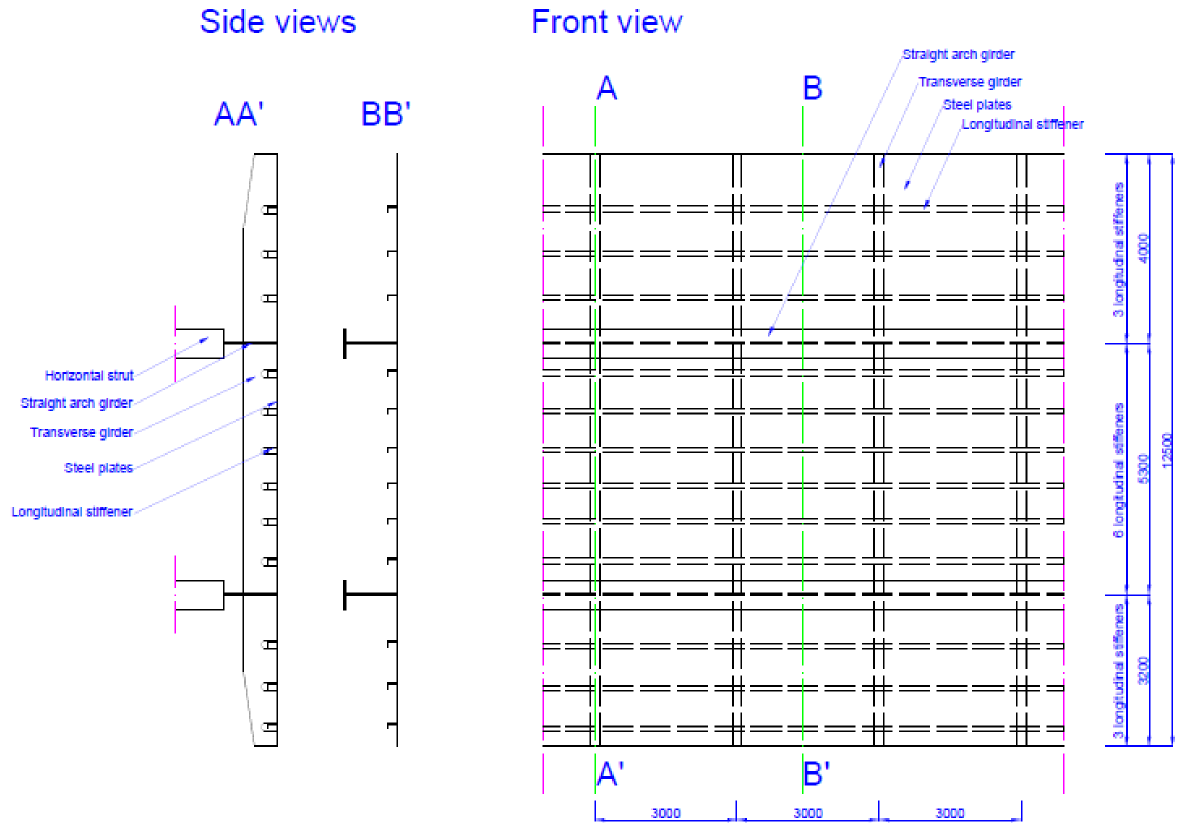


Figure 144 - Schematization plate wall (dimensions mm)

Normal force in the plate wall

The normal force applied to the side of the plate wall is generated because the resulting force from the curved arch needs to be balanced. During the load combination storm surge there is a compressive force in the plate wall, during the load combination salt intrusion there is a tensile force in the plate wall (shown in Figure 130). The values of these normal forces are obtained from the framework model created in RSTAB. The total force $N_{plate\ wall}$ in the plate wall is the sum of the force N_{upper} in the upper straight arch girder and the force N_{lower} in the lower straight arch girder.

Table 85 - Normal forces plate wall

Element	Storm surge	Salt intrusion	Unit
N_{upper}	9 700	1 700	kN
N_{lower}	10 800	6 000	kN
$N_{plate\ wall}$	20 500	7 700	kN

The normal forces obtained from the model are validated when the equilibrium of the end supports is analyzed. This equilibrium is shown in Figure 145 and calculated in Table 86. The normal force in the curved arch is obtained with the use of the model created in RSTAB. The sum of the maximum normal force in the lower arch and the normal force in the upper arch at the same location give the total normal force in the arches. The maximum force in the upper arch is not used because the x-bracing transfers forces from the lower to the upper arch, part of the force in the lower arch therefore also occurs in the upper arch.

The horizontal support reaction due to the applied load is distributed over the two supports. Half of the applied load is transferred to the left and half of the applied load is transferred to the right side. The horizontal reaction force per side is therefore given as;

$$R_{h;storm\ surge} = \frac{q_d * l}{2} * h_{applied} = \frac{38.4 * 81}{2} * 10.5 = 16\ 300\ kN$$

$$R_{h;salt\ intrusion} = \frac{q_d * l}{2} * h_{applied} = \frac{22.4 * 81}{2} * 8 = 7\ 400\ kN$$

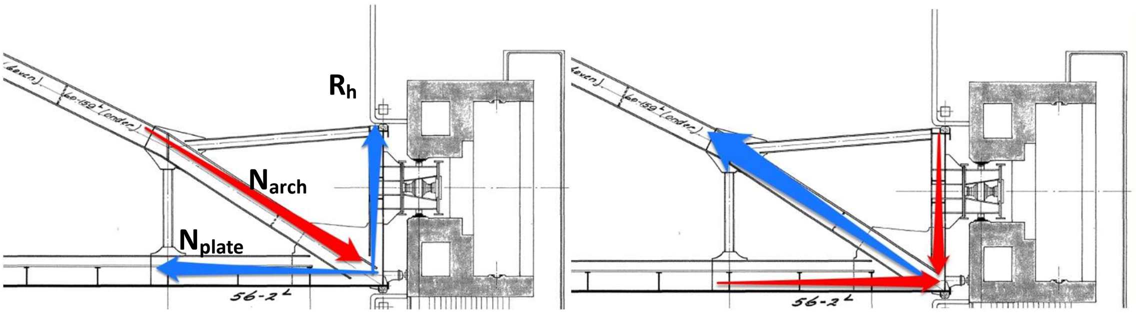


Figure 145 - Forces near the supports; storm surge (left), salt intrusion (right); compressive (blue), tensile (red).

The force in the plate wall is validated using the equilibrium of the total support (upper and lower arch). This equilibrium is calculated using the rule of Pythagoras.

$$N_{arch}{}^2 - R_h{}^2 = N_{plate}{}^2$$

Table 86 - Equilibrium of forces in the supports

Element	Storm surge	Salt intrusion	Unit
Normal force curved arch N_{arch}	17 200 + 8 300= 25 500	8 500 + 1800= 10 300	kN
Horizontal support reaction R_h	16 300	7 400	kN
Pythagoras calculation N_{plate}	19 600	7 200	kN

The normal compressive force in the plate wall during storm surge is 20 500 kN which is comparable to 19 600 kN, obtained from the equilibrium calculation Table 86. The normal tensile force in the plate wall during salt intrusion is 7 400 kN which is comparable to 7200 kN, obtained from the equilibrium calculation in Table 86.
The normal forces obtained from the model can therefore be used in the assessment of the elements in the plate wall.

Distributed load on the plate wall
The distributed load applied to the plate wall due to waves and hydrostatic compressive is the same as the distributed load calculated in appendix L.1.

Appendix L.4.1 Assessment transverse girder (moment)

The transverse girder shown in Figure 144 is used to transfer the loads from the plates and longitudinal stiffeners to the straight arch and struts connected to the arch. Each transverse girder transfers the distributed load of 3 meter plate; the transverse girders are located at 3 meter center to center. The resulting distributed load applied to part (from $H_{underside}$ to $H_{underside} + h_{applied}$) of the transverse girder therefore becomes;

$$q_{d;ss;transverse\ girder} = q_{d;storm\ surge} * l_{ctc} = 38.4 * 3 = 115.2\ kN/m$$
$$q_{d;si;transverse\ girder} = q_{d;salt\ intrusion} * l_{ctc} = -22.8 * 3 = -68.4\ kN/m$$

The resulting moment distribution in the transverse girder is given in Figure 146. The moments are given in kNm the distributed load is given in kN/m.

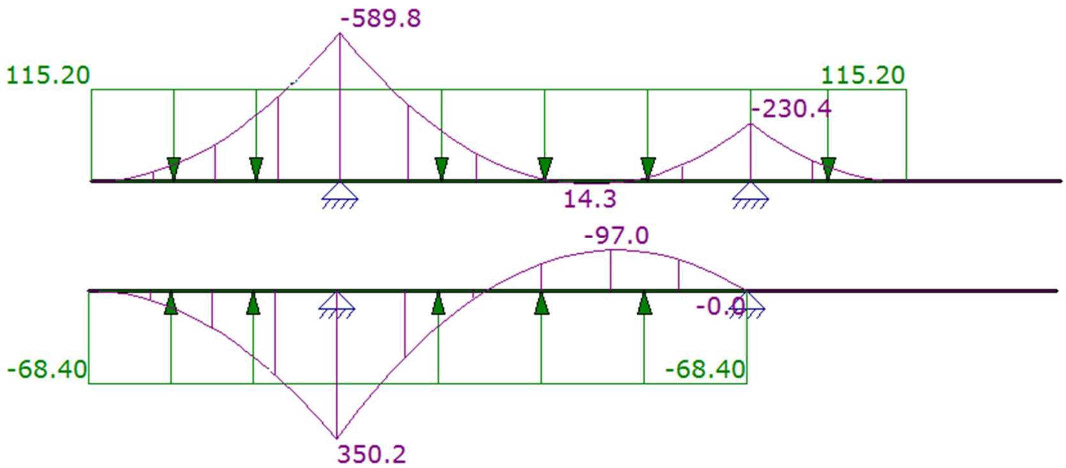


Figure 146 - Moment distribution transverse girder; storm surge (upper) and salt intrusion (lower)

The highest moment in the transverse girder is the design moment $M_{E;d}$, in this case 589.8 kNm.

Moment capacity

The moment capacity of the transverse girder is calculated using the formula;

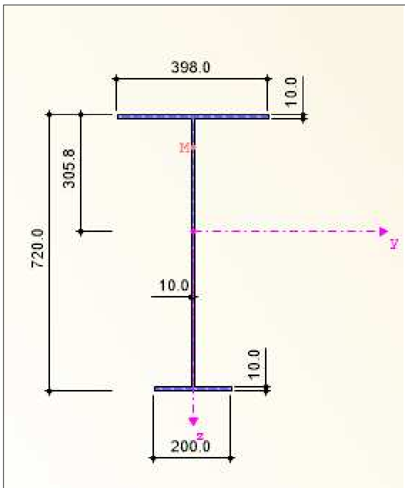
$$M_{R;d} = W_{y;eff} * f_{y;d}$$

The steel grade used for the transverse girder is S235 this means that the yield stress $f_{y;d}$ is 235 N/mm^2. The section modulus $W_{y;eff}$ depends on the profiles that is used for the transverse girder. The profile of the transverse girder is composed of multiple steel plates that together form an I-section. The steel plates forms one of the flanges of the profiles, the effective width of this part is calculated with the use of;

$$b_e = 1.33 * t * \sqrt{\frac{E}{f_{y;d}}} = 1.33 * 10 * \sqrt{\frac{210\,000}{235}} = 398\,mm$$

Table 87 - Parameters effective width

Parameters effective width	Symbol	Value	Unit
Effective width flange	b_e	398	mm
Thickness flange	t	10 (steel plate)	mm
Elasticity modulus	E	210 000	N/mm^2
Yield stress	$F_{y;d}$	235 (S235)	N/mm^2



Profile information

Cross-sectional area A	12 980 mm^2
Moment of inertia I_y	1.00*10^9 mm^4
Moment of inertia I_z	5.93*10^7 mm^4
Eccentricity e	414.2 mm
Section modulus W_y*	2.41*10^6 mm^3

*The section modulus is obtained when the moment of inertia is divided through the eccentricity.

Figure 147 - Welded steel section transverse girder, steel grade S235 and dimensions in mm

$$M_{R;d} = W_{y;eff} * f_{y;d} = 2.41 * 10^6 * 235 = 566\ kNm$$

Conclusion
The assessment for a girder loaded with a bending moment is given as;

$$\frac{M_{E;d}}{M_{R;d}} = \frac{589.8}{566} = 1.04\ NOT\ OK$$

The transverse girder fails because the bending moment in the girder is too high.

Appendix L.4.2 Assessment longitudinal stiffener

The longitudinal stiffeners are used to increase the stiffness of the steel plates and prevent the use of thick steel plates. The longitudinal stiffeners do however attract forces and therefore the longitudinal stiffeners should be assessed. There are two important requirements that should be assessed according to Eurocode EN 1993-1-5 and NEN 6771 [68, 69]. The torsional buckling of the stiffener should be assessed and the first order elastic bending and buckling should be assessed.

The two load combinations that should be assessed are storm surge and salt intrusion, each of these combinations results in a different situation;

- During the load combination storm surge there is a compressive force active in the plate wall and the distributed load is applied to the outside of the plate wall resulting in a local bending moment.
- During the load combination salt intrusion there is a tensile force in the plate wall and the distributed load is applied to the inside of the plate wall resulting in a local bending moment.

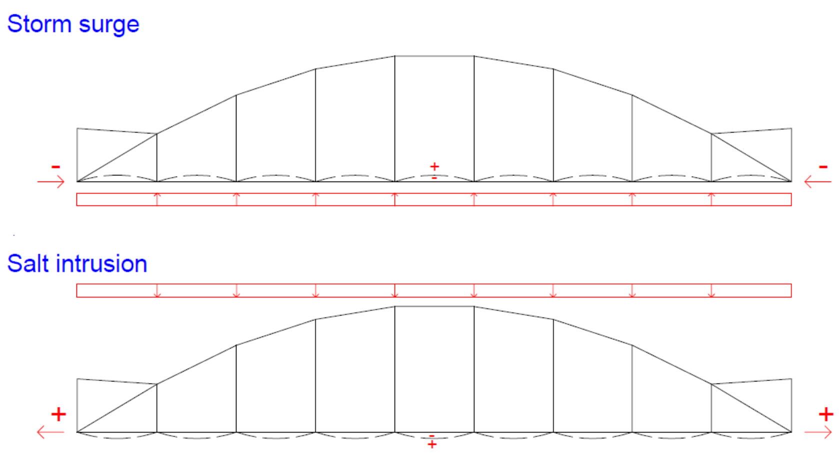


Figure 148 - Schematization load combinations, tensile (+) and compressive (-)

Governing loads
The normal force shown in Figure 148 is applied to the whole side of the plate wall. This means that the cross-section are of both the longitudinal stiffeners and straight arch girders is used for the transfer of this forces. It is assumed that the plates do not transfer a part of the normal force because the steel plates are predominantly used for the transfer of the distributed load.

Table 88 - Cross-sectional area elements

Elements	Number [-]	Cross-sectional area [mm^2]	Total cross-sectional area [mm^2]
Straight arch girder	2	45 904	91 808
Longitudinal stiffener (A_{stiff})	12	6 880	82 560
Plate wall ($A_{plate\ wall}$)	-	-	174 368

The average stress due to the applied normal force is calculated using the cross sectional area obtained from Table 88 and the total normal force applied to the plate wall.

$$\sigma_{d;ss;N} = \frac{F_{storm\ surge}}{A_{plate\ wall}} = \frac{-20\ 500 * 10^3}{174\ 368} = -118\ N/mm^2$$

$$\sigma_{d;si;N} = \frac{F_{salt\ intrusion}}{A_{plate\ wall}} = \frac{7\ 700 * 10^3}{174\ 368} = 44\ N/mm^2$$

The force transferred through one longitudinal stiffener is equal to the stress in the stiffener multiplied with the cross-sectional area of one stiffener.

$$N_{d;ss} = A_{stiff} * \sigma_{d;ss;N} = 6\ 880 * -118 = -811.8\ kN$$
$$N_{d;si} = A_{stiff} * \sigma_{d;si;N} = 6\ 880 * 44 = 302.7\ kN$$

The distributed load applied to the steel plate results in a bending moment in the longitudinal stiffener. The length between the transverse girders is 3 meters; 0.9 meter steel plate transfers the distributed load to the stiffener (total height of the gate divided through the number of arches and stiffeners).

$$q_{d;ss;stiffener} = 0.9 * q_{d;storm\ surge} = 0.9 * 38.4 = 34.6\ kN/m$$
$$q_{d;si;stiffener} = 0.9 * q_{d;salt\ intrusion} = 0.9 * -22.8 = -20.5\ kN/m$$

The web of the longitudinal stiffener is welded to the web of the transverse girder at every intersection, therefore it is expected that the moments generated around the support do not results in failure. The moment generated in the unsupported midfield (shown in Figure 144) of the longitudinal stiffener is governing. The governing moment distribution is shown in Figure 149 the moments are shown in kNm the distributed load is shown in kN/m.

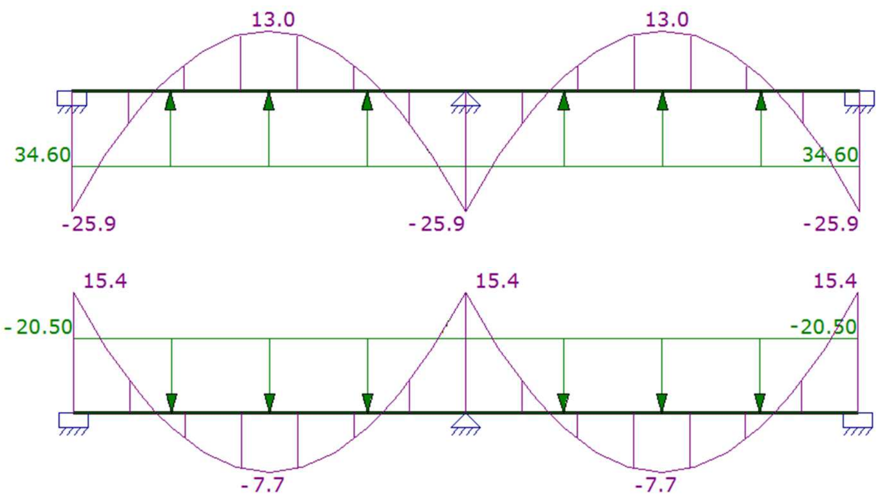


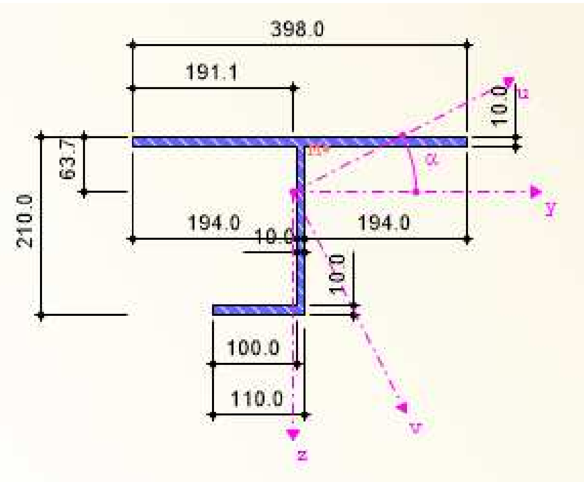
Figure 149 - Moment distribution longitudinal stiffener, storm surge (upper) and salt intrusion (lower)

The governing forces in the longitudinal stiffener are;

$$load\ combination\ storm\ surge; N_{d;ss} = -811.8\ kN\ and\ M_{d;ss} = 13\ kNm$$
$$load\ combination\ salt\ intrusion; N_{d;si} = 302.7\ kN\ and\ M_{d;si} = -7.7\ kNm$$

The stresses in the longitudinal stiffener are calculated using the formula give below. The moment of inertia (I), cross-sectional area (A) and distance to the outer fibre (z) are presented in the profile information of the longitudinal stiffener. The effective width of the longitudinal stiffener is the same as the effective width of the transverse girder.

$$\sigma = \frac{N}{A} + \frac{M * z}{I}$$



Profile information

Cross-sectional area A	6 880 mm^2
Moment of inertia I_y	$4.52*10^7$ mm^4
Moment of inertia I_z	$5.60*10^7$ mm^4
Outer fibre plate z_p	63.7 mm
Outer fibre flange z_f	146.3 mm
Section modulus $W_{y;eff}$	$3.09*10^5$

Figure 150 - Welded steel section longitudinal stiffener, steel grade S235 and dimensions in mm

The wide upper flange of the profiles from Figure 150 is created because a part of the plates connected to the longitudinal stiffener may be used as part of the stiffener (effective width). The effective width of the plates is calculated using [70];

$$b_e = 1.33 * t * \sqrt{\frac{E_d}{f_{y;d}}} \le b$$

Table 89 - Parameters effective with

Parameters torsional capacity	Symbol	Value	Unit
Effective with	b_e	398	mm
Thickness plate	t	10	mm
Elasticity modulus steel	E_d	210 000	N/mm^2
Yield stress steel	$f_{y;d}$	235	N/mm^2
Width plate	B	900	Mm

$$\sigma_{d;ss;plate} = -\frac{N_{d;ss}}{A} - \frac{M_{d;ss} * z_p}{I_y} = \frac{-811.8 * 10^3}{6\,880} - \frac{13 * 10^6 * 63.7}{4.52 * 10^7} = -136.3\ N/mm^2$$

$$\sigma_{d;ss;flange} = -\frac{N_{d;ss}}{A} + \frac{M_{d;ss} * z_f}{I_y} = \frac{-811.8 * 10^3}{6\,880} + \frac{13 * 10^6 * 146.3}{4.52 * 10^7} = -75.9\ N/mm^2$$

$$\sigma_{d;si;plate} = \frac{N_{d;si}}{A} + \frac{M_{d;si} * z_p}{I_y} = \frac{302.7 * 10^3}{6\,880} + \frac{7.7 * 10^6 * 63.7}{4.52 * 10^7} = 54.8\ N/mm^2$$

$$\sigma_{d;si;flange} = \frac{N_{d;si}}{A} - \frac{M_{d;si} * z_f}{I_y} = \frac{302.7 * 10^3}{6\,880} - \frac{7.7 * 10^6 * 146.3}{4.52 * 10^7} = 19.1\ N/mm^2$$

In both load combinations the whole cross-section is loaded in tensile or in compressive (shown in Figure 151).

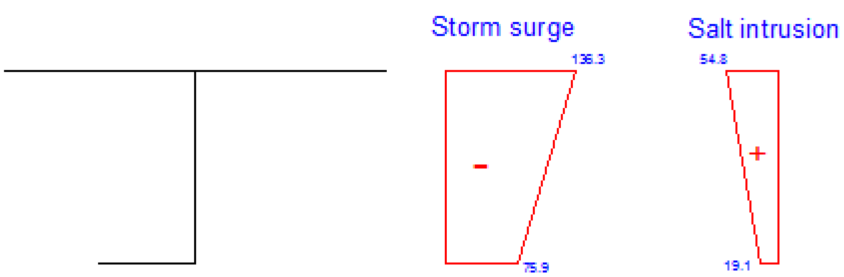


Figure 151 - Stresses (N/mm^2) in the longitudinal stiffener

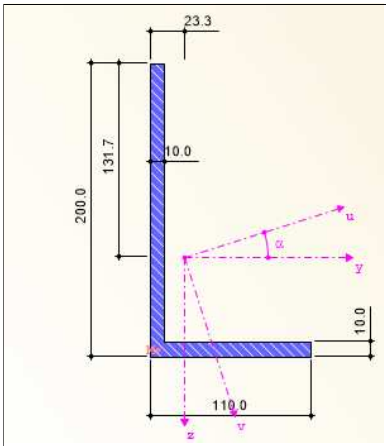
Assessment torsional capacity

The Eurocode 3 Part 1-5 formula 9.3 describes a simple criterion that should be satisfied concerning torsional buckling of the open cross-section [68].

$$\frac{I_t}{I_p} \geq 5.3 * \frac{f_y}{E_d}$$
$$I_p = I_y + I_z + A_{stiff} * {h_z}^2$$
$$I_T = \frac{1}{3} * ({t_f}^3 * b + {t_w}^3 * h)$$

Table 90 - Parameters torsion Eurocode

Parameters torsional capacity	Symbol	Value	Unit
Polar second moment	I_p	6.76*10^7 (around the edge fixed to the plate)	mm^4
St. Venants torsional constant	I_t	1.03*10^5 (only the stiffener)	mm^4
Elasticity modulus steel	E_d	210 000	N/mm^2



Profile information	
Cross-sectional area A_{stiff}	3000 mm^2
Moment of inertia I_y	1.27*10^7 mm^4
Moment of inertia I_z	2.87*10^6 mm^4
Thickness flange t_f	10 mm
Thickness web t_w	10 mm
Height profile h	200 mm
Width profile	110 mm
Distance to rotation h_z	131.7 mm

Figure 152 - Welded steel section longitudinal stiffener, steel grade S235 and dimensions in mm

With all the parameters known the assessment can be conducted;

$$\frac{1.03 * 10^5}{6.76 * 10^7} \geq 5.3 * \frac{235}{210\,000}$$

$$1.52 * 10^{-3} \geq 5.93 * 10^{-3} \; NOT\ OK$$

The simple criterion is not satisfied therefore an advanced method needs to be used. The Dutch code NEN 6771 section 13.8.3 describes the assessment of torsional capacity for open cross-sections. In this assessment it should be assessed whether;

$$\frac{F_{lv;s;d}}{F_{tk;d}} \leq 1$$

$$F_{tk;d} = A * \sigma_{tk;d}$$

$$\sigma_{tk;d} = f_{y;d} * \frac{1.593 - \lambda_s}{1.41} \; if\ 0.183 < \lambda_s < 1$$
$$\lambda_s = \sqrt{\frac{f_{y;d}}{E_d}} * \eta$$

Table 91 - Parameters torsion NEN Norm

Parameters torsional capacity	Symbol	Value	Unit
Normal compressive force	$F_{lv;s;d}$	-811.8 ($N_{ss;d}$)	kN
Torsional capacity force	$F_{tk;d}$	-1058.4	kN
Torsional capacity stress	$\sigma_{tk;d}$	153.8	N/mm^2
Specific torsional slenderness	λ_s	0.67	-
Factor specific for type stiffener	η	20	-

The factor η specific for the type of stiffener depends on a number of parameters. These parameters are;

$$\lambda_y; \ \frac{I_{p:N}}{I_{p;z}}; \ \frac{I_{p;z} * l^2}{I_z * h^2 + I_{wa}}; \ \sqrt{\frac{I_{p:N}}{I_t}}; \ \frac{k_\varphi * l^2}{E_d * I_{p;z}}$$

The exact formulae of the parameters that specify the factor η are found in NEN 6771 page 88 [69]. The values of the different parameters are;

$$\lambda_y \approx 40$$
$$\frac{I_{p:N}}{I_{p;z}} = 1.03$$
$$\frac{I_{p;z} * l^2}{I_z * h^2 + I_{wa}} = 5\,000$$
$$\sqrt{\frac{I_{p:N}}{I_t}} = 25$$
$$\frac{k_\varphi * l^2}{E_d * I_{p;z}} = 0.07 \ in\ which\ n = 1\ and\ k_\varphi = 42\,700$$

With these pararmeters the factor η can be obtained from the graph shown in Figure 153. The factor η in the graph is approximatetly 18. The graph is usable for the conditions given below, these conditions are satisfied.

$$\lambda_y = 40; \ \frac{I_{p:N}}{I_{p;z}} \leq 1,15; \ \frac{I_{p;z}\, l^2}{I_z h^2 + I_{wa}} \leq 10^5$$

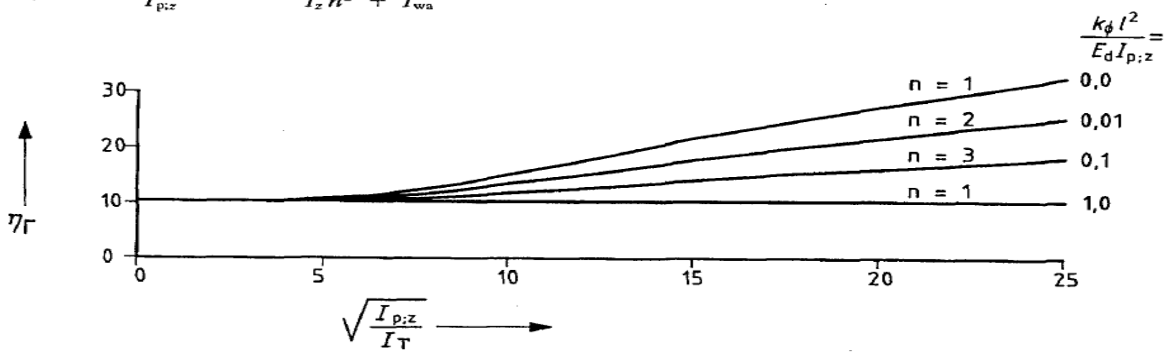


Figure 153 - Graph factor η, source; NEN 6771

The assessment of the torsional capacity becomes;

$$\frac{F_{lv;s;d}}{F_{tk;d}} \leq 1$$
$$\frac{811.8}{1058.4} \leq 0.77\ OK$$

The assessment shows that the torsional capacity is higher than the applied load and therefore safe.

<u>Assessment compression and uniaxial bending</u>

Stiffeners which are loaded with a compression force and bending moment should be assessed for the combination of both. This assessment is similar to the assessment of a column loaded with a compression force and a bending moment. In the Eurocode 1993-1-5 this assessment is presented as;

$$\frac{N_{E;d}}{\frac{f_{y;d} * A_{eff}}{\gamma_{M0}}} + \frac{M_{E;d} + N_{E;d} * e_N}{\frac{f_{y;d} * W_{eff}}{\gamma_{M0}}} \leq 1$$

The values A_{eff} and W_{eff} depend on the part of the profile that is in compression. In the important load combination the bending moment and normal force do not result in a tensile stress in the stiffener (shown in

Figure 151). Therefore A_{eff} and W_{eff} are equal to the design values of the total stiffener (effective width included). The partial factor γ_{M0} is 1.0 according to the Eurocode. The eccentricity e_N due to local imperfections is described in Eurocode 1993-1-1 Table 5:1. For buckling curve b the eccentricity divided by the total unsupported length is equal to 1/250. This means that e_N = 3000/250 which is 12 mm.

$$\frac{811.8 * 10^3}{\frac{235 * 6880}{1.0}} + \frac{13 * 10^6 + 811.8 * 10^3 * 12}{\frac{235 * 3.09 * 10^5}{1.0}} \leq 1$$

$$0.51 + 0.19 + 0.13 = 0.83 \leq 1 \; OK$$

Conclusion

The assessment of the longitudinal stiffeners shows that the capacity of the stiffeners is high enough.

Appendix L.5 Assessment fatigue

Fatigue is a problem that occurs when an element is loaded by a frequently changing load. Failure of the element can occur for stresses far below the yield stress of steel. Fatigue could be a problem for the Hollandsche IJssel barrier because the HIJ barrier closes off the Hollandsche IJssel for a longer period during salt intrusion. The tidal elevation in the New Meuse remains however because the ML barrier does not close.

Welded connections are sensitive to fatigue because the welding processes leave discontinuities and create local stress concentrations. The sensitive parts in the steel gate are the connections that only consist of a welding. The connection between the curved arch and the gusset plate (shown in Figure 141) is such a connection. The capacity (number of cycles N to failure) should be compared to the number of cycles that are expected to happen in the design lifetime of the gate. The capacity N is expressed as;

$$\log(N) = \log(d) - m * \log(\Delta\sigma_r)$$

In which m is a constant that is 3 for most welded structures, d is a value for the strength of the weld and $\Delta\sigma_r$ is the difference between the occurring stresses. Fatigue tests conducted on simple welds showed that the strength d of the weld is linked to the stress range [71]. The stress range in the weld depends on the force in the struts and therefore the changing distributed load that is applied to the steel gate. Figure 154 shows the schematizations of the water levels.

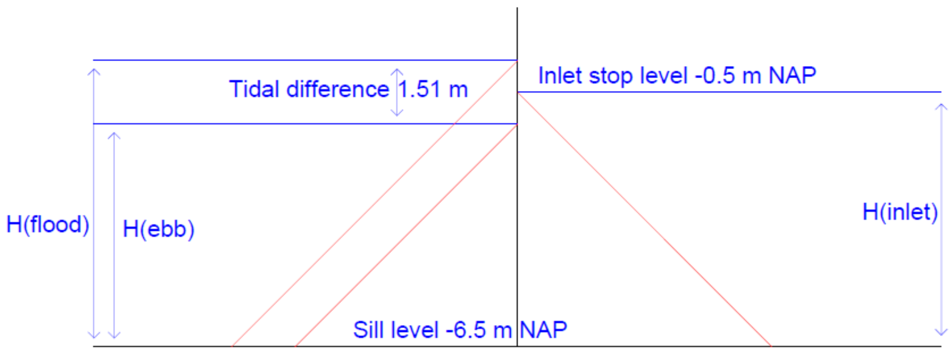


Figure 154 - Schematization water levels fatigue

The difference between the two distributed loads (ebb and flood) acting on the steel gate is calculated using;

$$q_{gate} = q_{flood} - q_{ebb} = \rho * g * (H_{flood} - H_{inlet}) - \rho * g * (H_{ebb} - H_{inlet})$$

$$q_{gate;change} = 1\,000 * 10 * (6.75 - 6.00) - 1\,000 * 10 * (5.25 - 6.00) = 15 \; kN/m^2$$

The resulting difference between the force applied to each strut and therewith the stress in the weld is calculated using the area op the steel plates which transfer the distributed load to the strut. The width is equal to the center to center distance between the strut. The height is equal to the average height between the flood and ebb level which is equal to the inlet stop level. It is assumed that only the struts in the lower arch transfer forces, the struts in the upper arch do not transfer any force because the upper arch is located at 2.5 m NAP which is 2 meter above the flood water level.

$$A_{transfer} = H_{flood} * l_{ctc} = 6.00 * 9 = 54\ m^2$$

The force in the strut is equal to;

$$F_{strut} = A_{transfer} * q_{gate;change} = 54 * 15 = 810\ kN$$

The stress in the weld is calculated using the formula to assess the capacity of the weld in appendix L.3.2;

$$\Delta\sigma_r = \frac{F_d}{t * l_{weld}} = \frac{810 * 10^3}{12 * 1\ 300} = 51.9\ N/mm^2$$

The value for the strength of the gate is then equal to $0.356*10^{12}$ according to figure 7 of the lecture notes on fatigue [71]. The calculation of N results in;

$$\log(N) = \log(0.356 * 10^{12}) - 3 * \log(51.9)$$
$$N = 2.5 * 10^6\ load\ cycles$$

The number of load cycles during the remaining life time of the barrier is predominantly determined by salt intrusion because the barrier is closed for a longer time. There is a tidal elevation on the New Meuse because the Maeslant barrier is not closed during salt intrusion. It is assumed that the barrier closes every year because of salt intrusion. This means that the barrier is closed 50 months during the remaining lifetime of the barrier (50 years) multiplied with the maximum closure of one month. The number of load cycles is then equal to 50 months multiplied with the number of days and the number of tides per day.

$$N_{lifetime} = number\ of\ months * days * number\ of\ tides\ per\ day$$
$$N_{lifetime} = 50 * 30 * 2 = 3\ 000\ load\ cycles$$
$$N \gg N_{lifetime}$$

Fatigue is no problem, because the number of cycles to fatigue is much larger than the number of cycles that will occur during the remaining lifetime. Even when the storm surge barrier is always closed fatigue does not become a problem, the number cycles with a permanent closure = 365 years *50 years *2 tides a day = $3.65*10^4$ cycles.

Appendix L.6 Increased forces due to the sea level rise

The forces in the arch and transverse girder increase because the design load q_d increases, this load is obtained from the models (described in Figure 121) that were used to estimate the representative load. The load on the arch is calculated using the same method as described in Figure 131. The increased load in the arch is calculated using the formula shown below.

$$F_{strut} = q_{arch} * l_{ctc}$$

The design loads for the curved arch are obtained from the model in RSTAB in which the distributed load q_{arch} is put on the underside of the arch. The bending moment reduces because the curved arch attracts more normal forces therefore the bending moment are redistributed elsewhere in the structure. The design moment in the transverse girder is calculated in MATLAB using the same procedure as shown in Figure 146. The obtained design load can be used as input into the model created in RSTAB to calculate the increase of the loads in the curved arch and transverse girder. The increased forces are shown in Table 93.

Table 92 - Increase forces due to sea level rise

Sea level rise*	$h_{governing}$ [m NAP]	Increase q_d [kN/m^2]	Load q_{arch} [kN/m]	Design load strut [kN]	Design load curved arch [kN, kNm]	Design moment transverse girder [kNm]
0.00	+3.50	38.4	247.2	2240	17 200, 1 370	589.8
0.10	+3.58	39.8	256.3	2310	17 800, 1 340	611.3
0.20	+3.65	40.8	262.7	2360	18 100, 1 320	627.6
0.35	+3.82	42.8	275.6	2480	19 000, 1 240	657.4
0.50	+3.92	43.4	279.4	2510	19 300, 1 220	666.6
1.00	+4.42	47.6	306.5	2760	21 200, 1 180	731.1

The increased loads are used to compare the increased loads to the capacity of the element; the formulae that are used for this calculation are given below, the results are given in Table 93.

$$UC_{strut} = \frac{N_{strut}}{N_{b;R;d}} \leq 1$$

$$UC_{arch} = \frac{N_{storm\ surge}}{N_{R;d}} + \frac{M_{storm\ surge}}{M_{R;d}} \leq 1$$

$$UC_{transverse} = \frac{M_{E;d}}{M_{R;d}} \leq 1$$

Table 93 - Summary unity checks detailed assessment

Element	Unity check salt intrusion	Unity check storm surge					
		SLR 0.0	SLR 0.10	SLR 0.20	SLR 0.35	SLR 0.50	SLR 1.00
Strut	-	0.52	0.54	0.55	0.58	0.59	0.64
Curved arch	0.74	1.18	1.21	1.22	1.26	1.27	1.37
Connection	0.92	-	-	-	-	-	-
Transverse girder	-	1.06	1.08	1.11	1.16	1.18	1.29
Longitudinal stiffener	-	0.83	0.85	0.88	0.93	0.95	1.06
Fatigue	0.00	-	-	-	-	-	-

Appendix L.7 Adaptation elements

The elements that need to be adapted are the curved arch located in the underside of the gate and the transverse girder that is located 3 meters center to center. The result of chapter 4 was that an increase of the sea level with 0.35 meter is possible. Therefore the elements are adapted to withstand at least 0.35 meter.

The difference in strength needed to reach 0.5 meter sea level rise is however not that much because the governing discharge does not increase only the closure level of the Maeslant barrier increases. The capacity of the curved arch and stiffener is increased to withstand the loads generated by a governing situation and 0.50 meter sea level rise. Other elements within the steel gate do not fail.

Appendix L.7.1 Capacity transverse stiffener

The current capacity of the curved arch is 566 kNm (described in appendix L.4.1) the governing bending moment is 666.6 kNm according to Table 92. The capacity should therefore be increased with;

$$Increase\ capacity\ \Delta M = (Governing\ bending\ moment - Current\ capacity) * safety\ factor$$
$$\Delta N = (666.6 - 566) * 1.2 = 120\ kNm$$

The resistance of the element is given as;

$$M_{R;d} = f_y * W$$

Therefore the section modulus (W) of the profile needs to be increases with;

$$\Delta W = \frac{\Delta M}{f_y} = \frac{120 * 10^6}{235} = 51.4 * 10^4\ mm^3$$

The capacity of the section modulus is increased when flange plates are welded to the flanges of the profile. One flange is part of the wall plates; therefore the free flange is used. The increase of the section modulus is predominantly affected by the distance to the center of the profile, therefore only the influence of the rule of Steiner is used. The moment of inertia (I) is increased by the effect of elements which have a different center of gravity. The rule of Steiner calculated the increase of the moment of inertia given the distance to the center of gravity (z) and the cross-sectional area of the element within the profile.

$$I = z^2 * A\ (Steiner)$$
$$W = \frac{I}{z} = z * A$$

When a flange plate with a thickness of 10 mm and a width of 250 mm is added to the profile the increase of the section modulus becomes 75*10^4 which is larger than the capacity needed.

$$\Delta W = z * A = 300 * (250 * 10) = 75 * 10^4\ mm^3$$
$$UC = \frac{M_{storm\ surge}}{M_{R;d} + \Delta M_{R;d}} = \frac{666.6}{566 + 176.25} = 0.89 \leq 1\ OK$$

Appendix L.7.2 Capacity increase curved arch

The current capacity of the curved arch is 17 833 kN (described in appendix L.2.2) the governing normal force is 19 300 kN according to Table 92. The safety factor used to increase the capacity is 1.2. Only the cross-sectional are for the normal force needs to be increased because the bending moment reduce as the tensile force increases. The resistance of the element is given as;

$$N_{R;d} = A * f_y$$

The capacity of the cross-sectional area is increased when flange and web plates are welded to the profile. The flanges are used for the connection to the strut therefore only web plates are used. Predominantly the cross-sectional are increases because of web plates, the section modulus increases slightly. Two web plates each 12*1 000 mm (b*h) are welded to the web of the profile.

$$\Delta W = \frac{1}{6} * b * h^2 = \frac{1}{6} * (2 * 12) * 1000^2 = 400 * 10^4 mm^3$$
$$\Delta A = A_{web} + A_{flange} = 2 * 12 * 1\ 000 = 24\ 000\ mm^2$$

$$\Delta M = \Delta W * f_y = 400 * 10^4 * 235 = 940\ kNm$$
$$\Delta N = \Delta A * f_y = 24\ 000 * 235 = 5\ 640\ kN$$

$$UC = \frac{N_{storm\ surge}}{N_{R;d} + \Delta N_{R;d}} + \frac{M_{storm\ surge}}{M_{R;d} + \Delta M_{R;d}} = \frac{19\ 300}{17\ 833.7 + 5640} + \frac{1\ 220}{6\ 345 + 940} = 0.99 \leq 1\ OK$$

Appendix L.7.3 Costs adaptation steel profiles

The costs that are needed for the adaptation of the steel gate needed to weld the steel plates to the profiles and the costs that are needed to lift the steel gate from the towers and transfer it to Hollandia. The costs for the welding and material are 5 euro per kilogram [72]. The specific weight of steel is 7 700 kg/m^3.

Table 94 - Costs adaptation steel gate

Profile	Number [-]	Length [m]	Cross-sectional area [m^2]	Volume [m^3]	Weight [kg]
Curved arch	1	87	0.024	2.09	16 000
Transverse girders	27	12.5	0.0025	0.84	6 500

The costs for the welding of the steel profiles is (16 000 + 6 500)*5= 112 500 euros.

Appendix M Preliminary analysis non-closure tree

The effects of the non-closure probability have been calculated in section 4.1.3, 4.3.1 and appendix F.8, the non-closure probability in itself is treated in this section. Different reports (third nationwide assessment, HKV report on governing water levels and different reports within the department of Public Works) already studied the non-closure probability of the existing barrier in detail, the non-closure probability of the storm surge barrier lies around the 1/30 per closure event [4, 47, 18]. An intensive study of the non-closure probability is therefore not useful.

A non-closure event occurs when the storm surge barrier does not close when there should have been a closure. The probability that this occurs is called the non-closure probability per event. These events are often analyzed using a fault tree, a fault tree is a diagram in which all the aspects that lead to the top event (in this case failure of the Hollandsche IJssel storm surge barrier) are presented in the form of a failure tree (shown in Figure 155). The different aspects are linked using two possible connections;

- AND-port, an AND-port is a port in which both aspects should occur before the element fails.
- OR-port, an OR-port is a port in which the element fails when one aspect fails.

In the third nationwide safety assessment the closure reliability of the storm surge barrier did not meet the standards. In this assessment the guideline on Hydraulic structures is used to analyze the closure reliability [60]. The fault tree of the Hollandsche IJssel storm surge barrier uses the TAW guideline Hydraulic structures as the basis, this fault tree is specified for the Hollandsche IJssel storm surge barrier (using amongst others information obtained from an interview with L. Hove one of the operators of the Hollandsche IJssel storm surge barrier working at the department of Public Works district New Waterway).

The fault tree in Figure 155 shows that there are two elements that describe failure of the storm surge barrier; governing situation and non-closure event. The non-closure event occurs when the closure level is exceeded and the storm surge barrier does not close, there are four aspects that influence the reliability of closure; high water alarm system fails, mobilization fails, control and technical failure.

The high water alarm system is a system that warns operators when a storm is expected/ predicted. After that the mobilization of the operators starts which ensures that there is always manpower at the barrier. When the barrier should close the closure (control) procedures will start and the driving mechanism starts. The last part of the closure is the technical part which lowers the gate into the river. A preliminary analysis of the technical part of the non-closure tree is shown in Figure 156. The other parts of the non-closure tree (1,2 and 3) are not described an overview of the different parts is found the guideline Leidraad Kunstwerken [60].

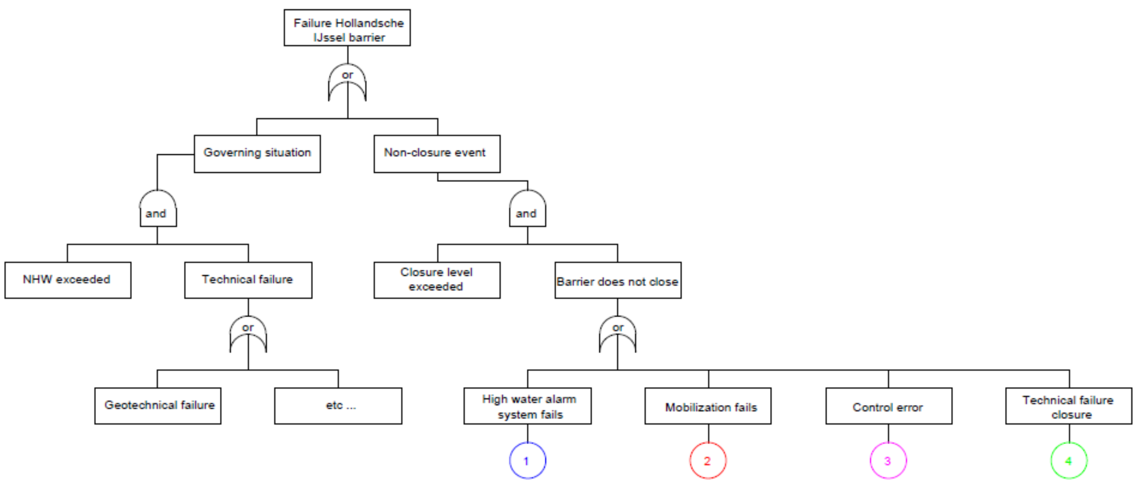


Figure 155 - Fault tree storm surge barrier Hollandsche IJssel, source; Leidraad Kunstwerken TAW

The closure of gate 2 in Figure 156 shows the same non-closure tree as the closure of gate 1. In Table 95 the different elements in the fault tree are described.

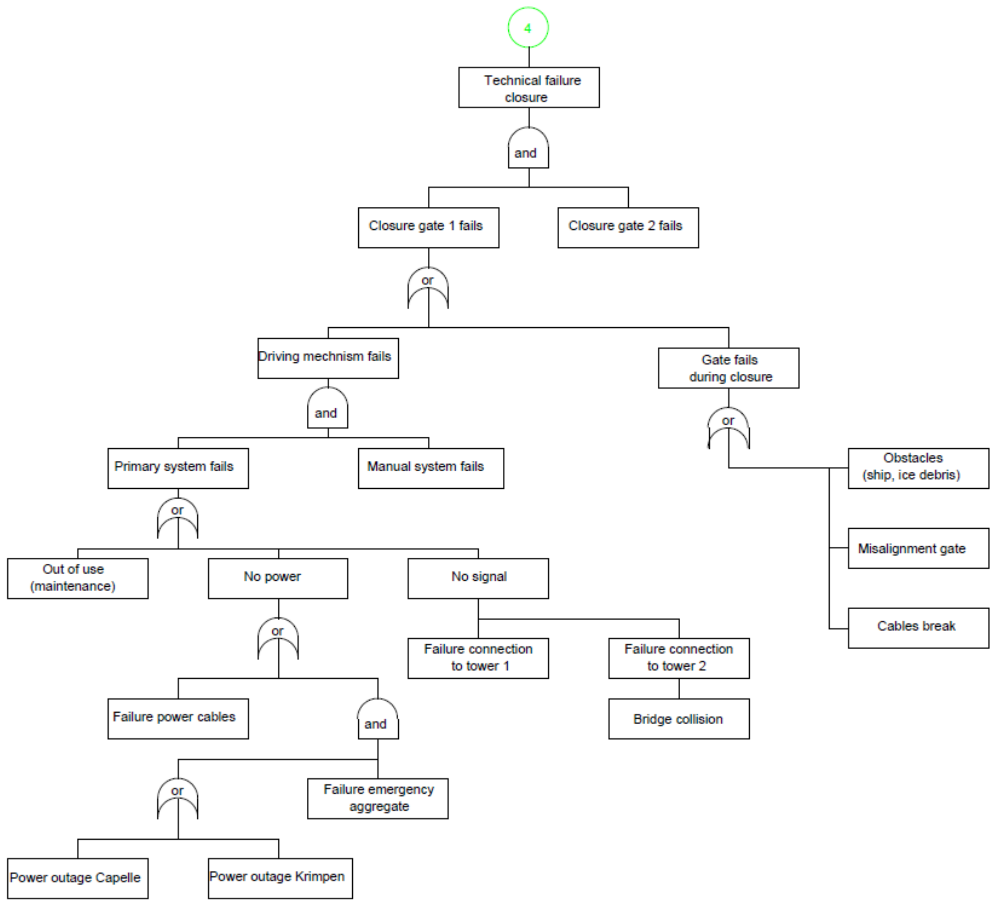


Figure 156 - Technical part non-closure tree, source; Leidraad Kunstwerken TAW

Table 95 - Elements branch 4, non-closure tree

Branch 4: Technical failure closure	Due to technical reasons the closure of the steel gate fails
Closure gate 1	The first gate of the barrier fails
• **Driving mechanism fails**	The mechanism that controls and activates the movement of the gat fails.
- **Primary system fails**	The driving mechanism that is normally used fails.
Out of use	Due to repair/maintenance the driving mechanism is not available.
No power	There is no power to start or continue the downward movement of the gate.
Failure cables	One of the power cables snaps.
Power outage Capelle	There is a power outage in the region Capelle.
Power outage Krimpen	There is a power outage in the region Krimpen.
Failure aggregate	The aggregate fails or does not start.
No signal	There is no signal to the driving mechanisms of both towers
Connection to tower 1	The connection to tower 1 (same side) fails.
Connection to tower 2	The connection to tower 2 (other side) fails.
Bridge collision	The cables connecting tower 2 run under the fixed bridge, when a ship collides with the bridge the connection fails.
- **Manual system fails**	The system to manually start the movement fails (upper floor tower).
• **Gate fails during closure**	The gate fails during closure.
- **Obstacles**	Obstacles in the water damage the gate or stop the movement of the gate.
- **Misalignment**	Due to an uneven lowering of the gate the gate got stuck.
- **Cable break**	One of the cables lowering the gate fails.
Closure gate 2	The second gate of the barrier fails when the first gate has already failed; the same non-closure tree is governing for the second gate.

Appendix N Flood slack closure, scour protection

The scour protection is needed because of the tidal flow in the system, the ships that navigate through the Hollandsche IJssel might cause temporary erosion but this is wiped out because of the sediment transports during every tide. When a scour protection is constructed it should be checked whether the scour protection is stable when a ships passes, because the scour protection should be stable during tidal flows and can therefore not wipe out the damage caused by ships.

Figure 19 shows the tidal form of the tide in the Hollandsche IJssel, the tidal rise of the tide on the Hollandsche IJssel is the fastest and therefore the most interesting. The duration of the tidal rise last 4.18 hours (15048 second) and the duration of the tidal fall lasts 8.23 hours. The velocity during a tidal rise is calculated using the storage of the Hollandsche IJssel;

$$Storage = Lenght * Width * Spring\ elevation$$
$$Storage = 19\ 000 * 135 * 1.61 = 4.130 * 10^6\ m^2$$

The discharge into the Hollandsche IJssel during the tidal rise is equal to;

$$Discharge(Q) = \frac{Storage}{tidal\ rise} = \frac{4.130 * 10^6}{15048} = 275\ m^3/s$$

The velocity at which the water enters the Hollandsche depends on the cross-section of the flow; the cross section when the barrier is nearly closed is equal to the formula given below. The length of the barrier is 80 meters, the governing height of the opening before closure is 1.00 m.

$$A_{flow} = L * \Delta h = 80 * 1.00 = 80\ m^2$$

The contraction coefficient is not used because the gate is supported on a concrete sill which does not erode, the speed of water is;

$$u = \frac{Q}{A_{flow}} = \frac{275}{80} = 4.43\ m/s$$

The scour protection should be stable during closure of the storm surge barrier. The formula to check the stone diameter that is needed for a stable scour protection is obtained from the book Introduction to Bed, bank and shore protection [49]. The formulae are;

$$d_{n50} = \frac{{u_c}^2}{\psi_c * \Delta * C^2}$$

$$C = 18 * \log((\frac{12 * R}{2 * d_{n50}})$$

$$R = \frac{A}{O}$$

Table 96 - Parameters stone diameter

Parameters torsional capacity	Symbol	Value	Unit
Nominal stone diameter	d_{n50}	0.14	m
Shields parameter	Ψ_c	0.03	-
Water depth	h	8.25	m
Cross sectional opening	Δh	1.0	m
Length of the barrier	L	80	m
Cross sectional surface	A	660	m^2
Wet circumference	O	96.5	m
Hydraulic diameter	R	6.84	m
Chezy coefficient	C	39	√m/s
Water velocity	$\overline{U}$	4.43	m/s

The nominal stone diameter that is needed is 0.14 meter. The scour holes that will develop need to be kept away from the storm surge barrier. When a scour hole becomes unstable a part of the scour protection will slide into the scour hole, the part that slides into the scour hole should be smaller than the scour protection. The length of the scour protection is calculated using the following formulae [49];

$$h_s(t) = \frac{(\alpha_c * \bar{u} - \overline{u_c})^{1.7} * {h_0}^{0.2} * t^{0.4}}{10 * \Delta^{0.7}}$$

$$\Delta = \frac{\rho_{stone} - \rho_{water}}{\rho_{water}}$$

$$\alpha_c = 1.5 + 5 * r_0 * f_c$$

$$r_0 = 1.2 * \sqrt{\frac{g}{C}}$$

$$u_c = \sqrt{\frac{d_{n50}}{\psi c * \Delta * C^2}}$$

$$L_{bed} = h_s(t) * 15$$

$$f_c = \frac{C}{40} (f_c = 1 \; when \; C \leq 40)$$

Table 97 - Parameters length bed protection

Parameters length bed protection	Symbol	Value	Unit
Scour depth	$h_s(t)$	11.1	m
Tidal rise	t	4.18	h
Gravity	g	9.81	m^2/s
Turbulence	r_0	0.60	-
Factor turbulence	f_c	1.0	m
Nominal sand diameter	d	0.0003	m
Density water	ρ_{water}	1 000	kg/m^3
Density stone	ρ_{stone}	2 650	kg/m^3
Factor densities	Δ	1.65	-
Turbulence factor	α_c	4.51	-
Critical water velocity	$\overline{U_c}$	0.15	m/s
Length bed protection	L_{bed}	170	m

Ships might permanently damage the scour protection when navigating through the Hollandsche IJssel storm surge barrier therefore the stone diameter that might be needed is checked. The propeller power of ships navigating to the Hollandsche IJssel is 1 500 kW. The speed of the water jet is calculated using;

$$u_0 = 1.15 * \left(\frac{P}{\rho_w * d^2}\right)^{\frac{1}{3}} = 13.2 \; m/s$$

$$u_{b-max} = 0.3 * u_0 * \frac{d}{z_b} = 1.98 \; m/s$$

$$d_{n50} = \frac{{u_{b-max}}^2}{\Delta * 2g} = 0.12 \text{ m}$$

Parameters damage ships	Symbol	Value	Unit
Ship power	P	1 500	kW
Density water	ρ_w	1 000	kg/m^3
Propellor diameter	d	1.0	m
Jet speed	u_0	13.2	m/s
Depth to the bottom	z_b	2	m
Water speed bottom	u_{b-max}	1.32	m/s
Gravity	g	9.81	m^2/s

The stone diameter is smaller than the nominal stone diameter of the bed protection and therefore the bed protection is not damaged.

Appendix O Technical drawings

In this appendix the technical drawings of the storm surge barrier are presented.

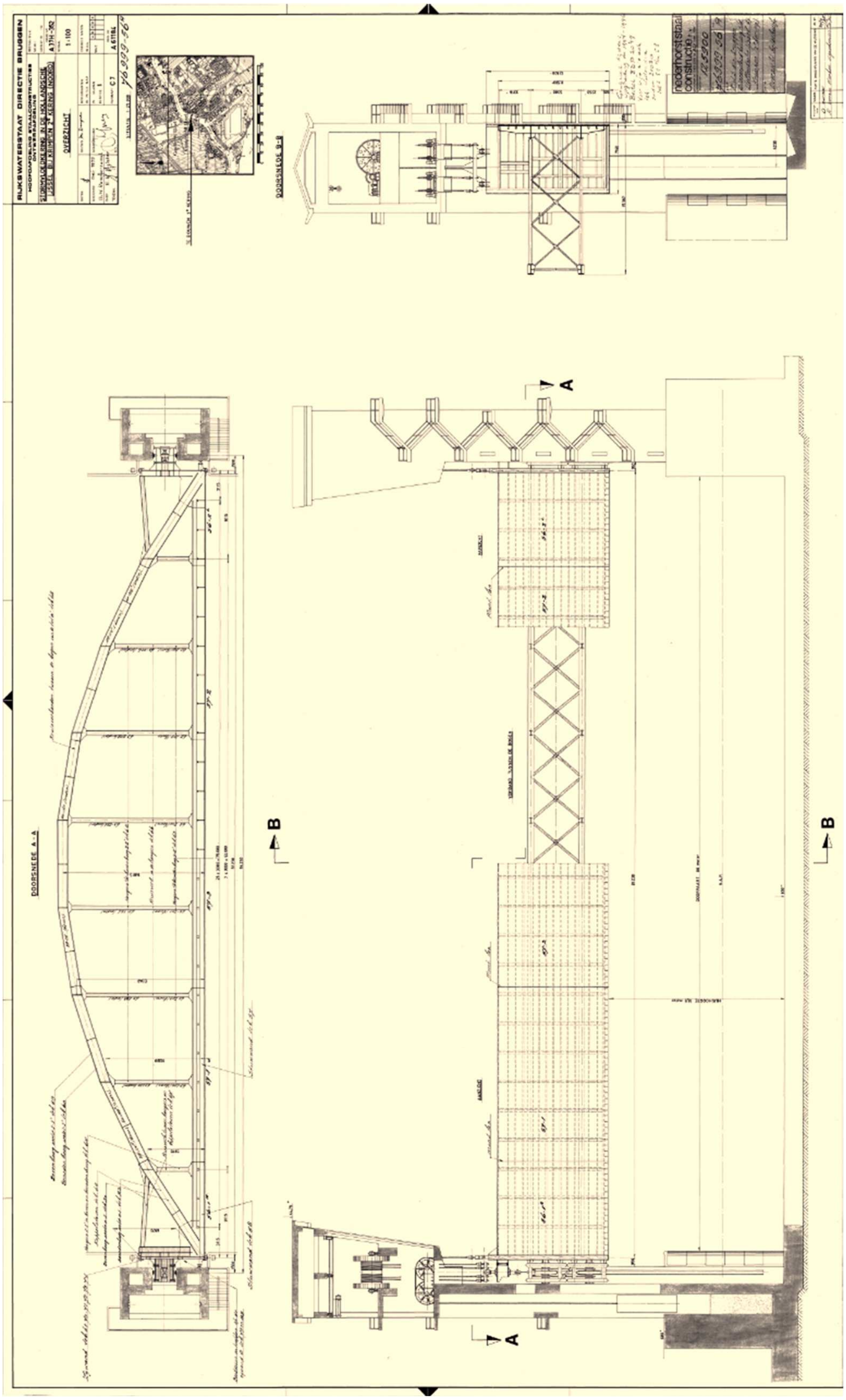


Figure 157 - Technical drawing storm surge barrier 1978, source; Rijkswaterstaat

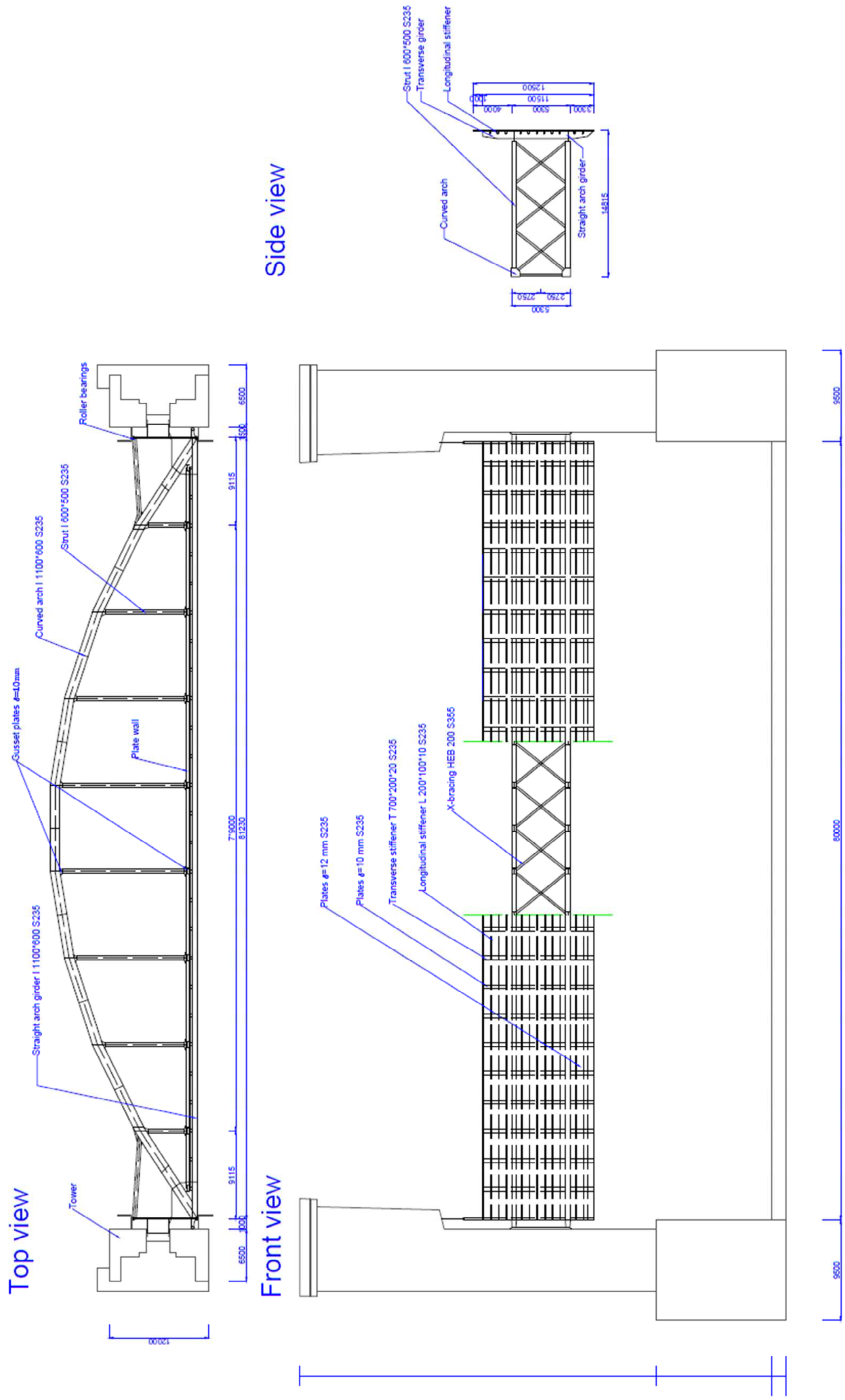


Figure 158 - Technical drawing storm surge barrier Hollandsche IJssel (dimension mm)

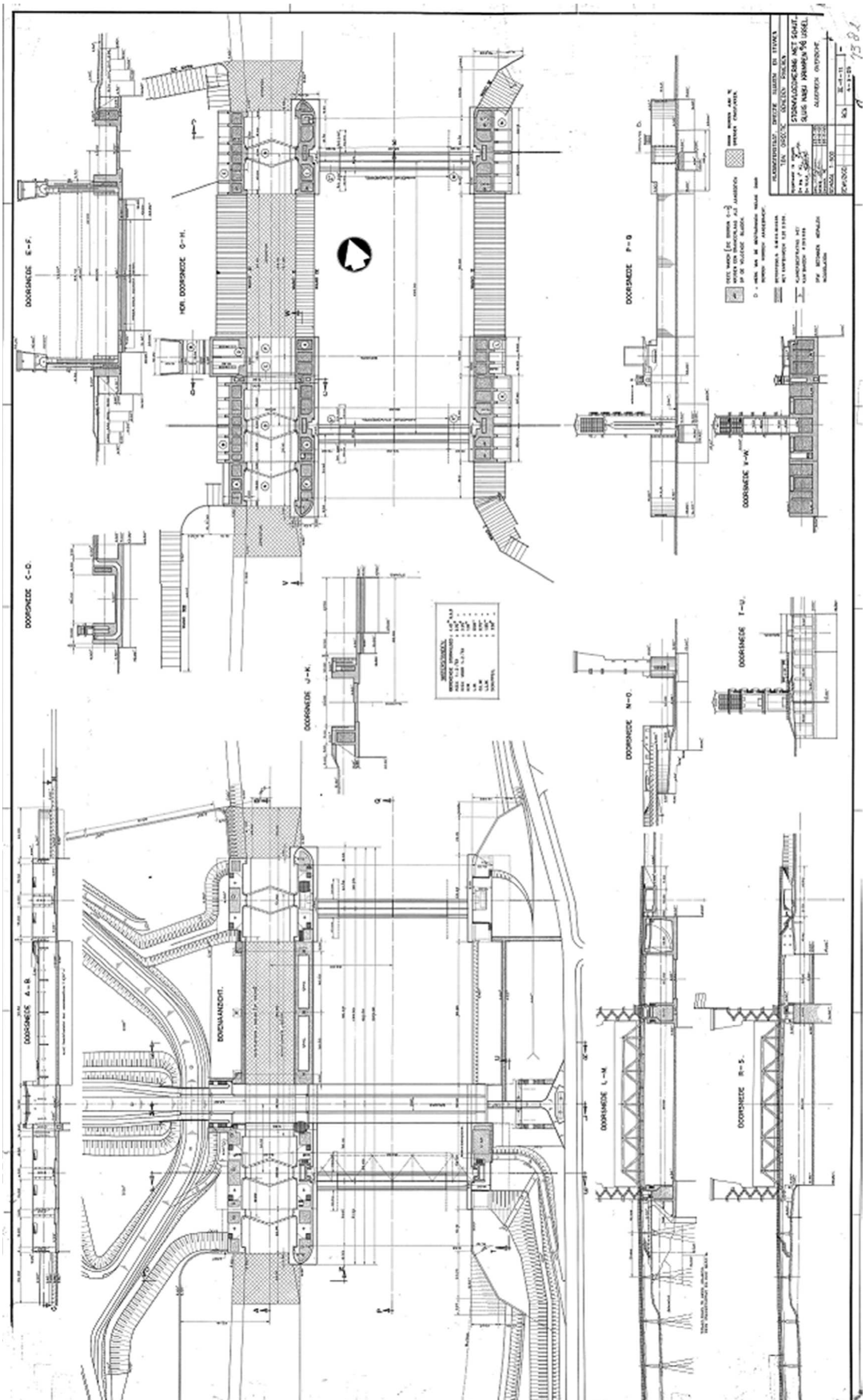


Figure 159 - Technical drawing overview storm surge barrier 1978, source; Rijkswaterstaat

Appendix P Pictures existing steel gates (technical drawings 1978)

In this appendix the pictures that are used to model and assess the steel gate are presented. The location of the picture is shown in Figure 160; the picture is presented in the figures of this appendix.

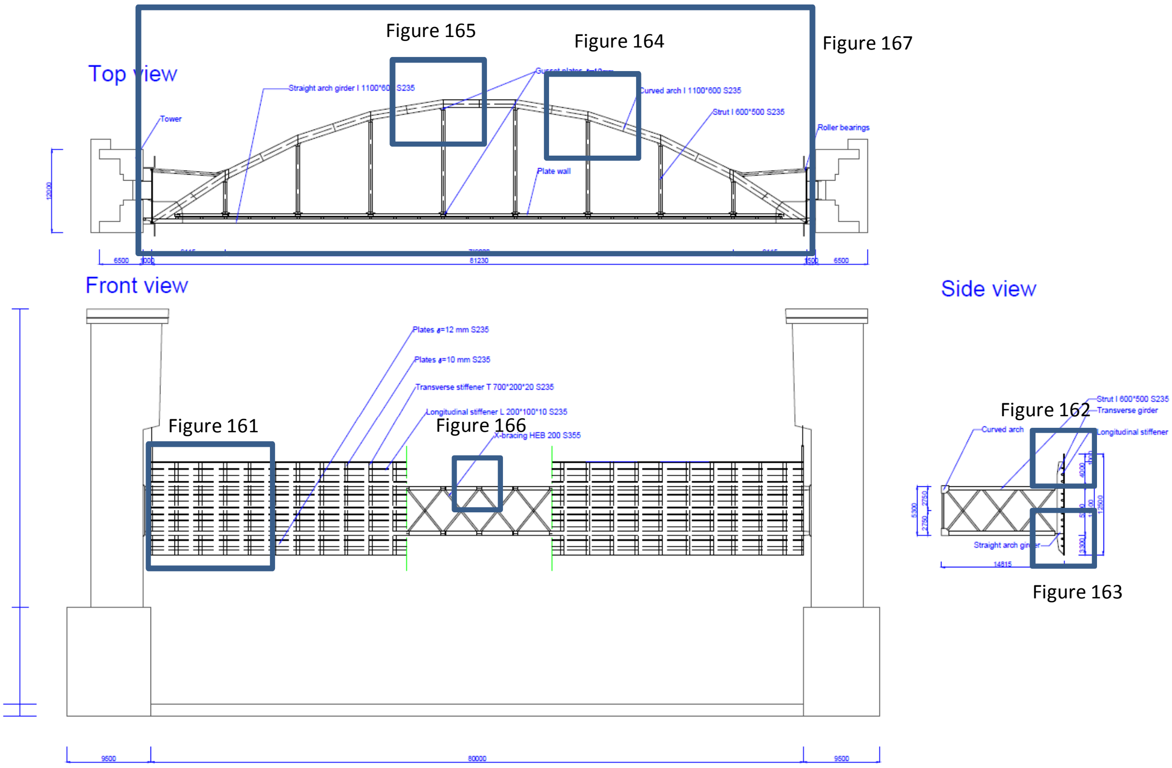


Figure 160 - Steel gate, location pictures

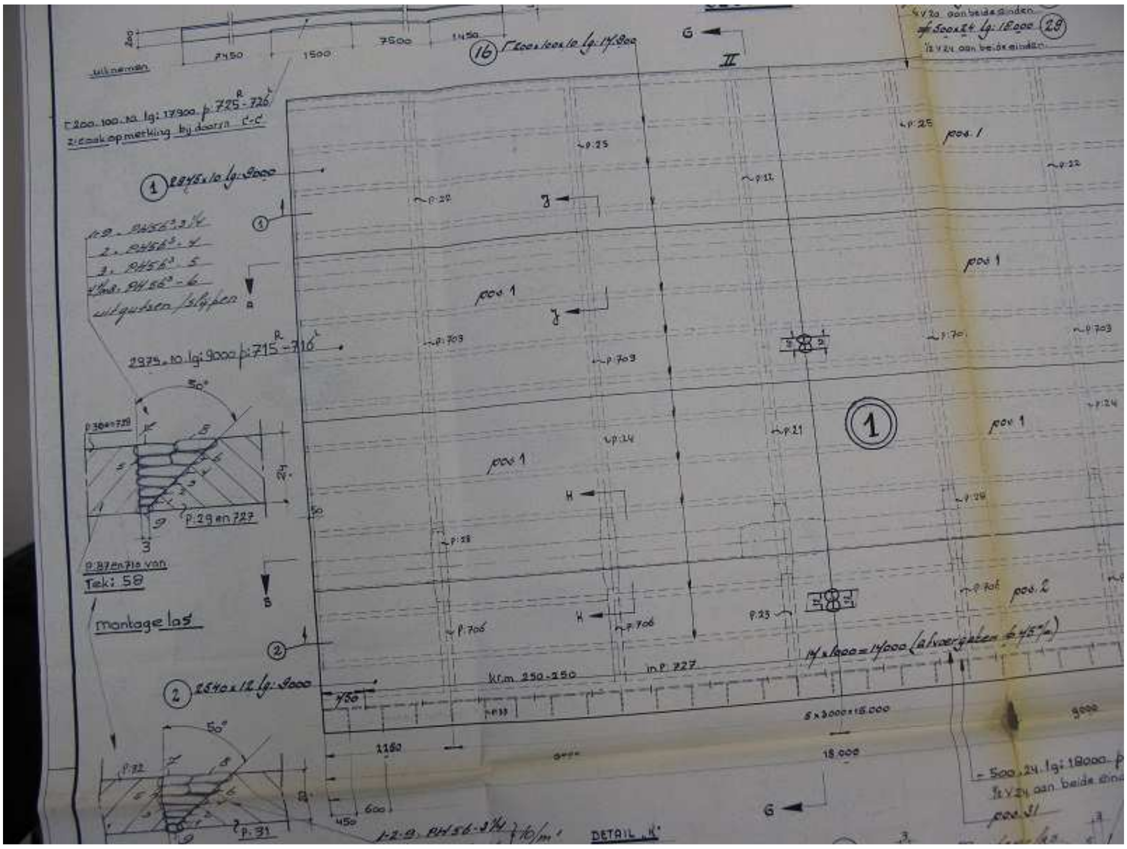


Figure 161 - Overview plate wall

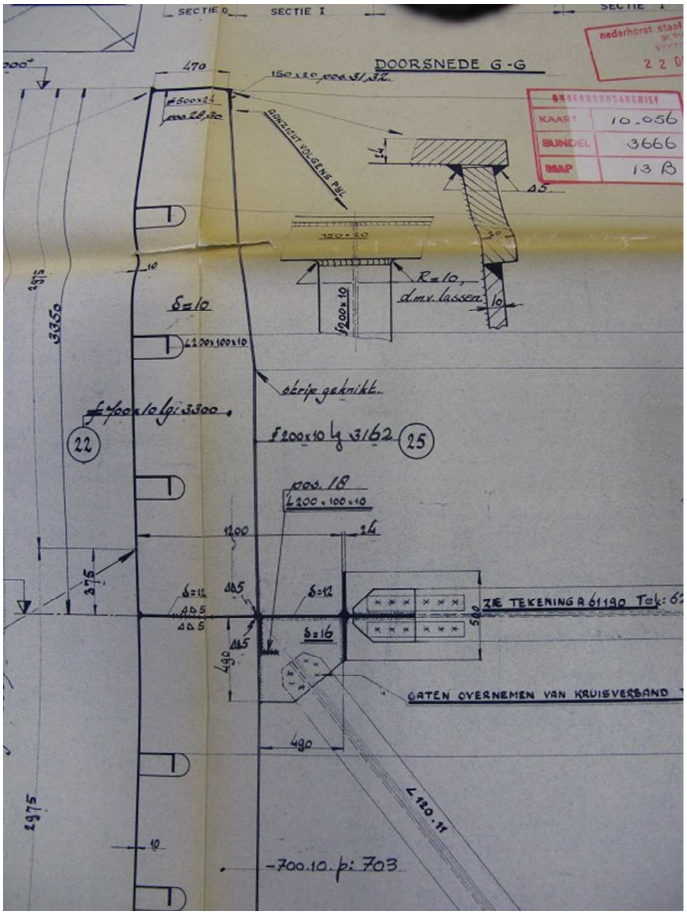


Figure 162 - Cross-section upper part plate wall

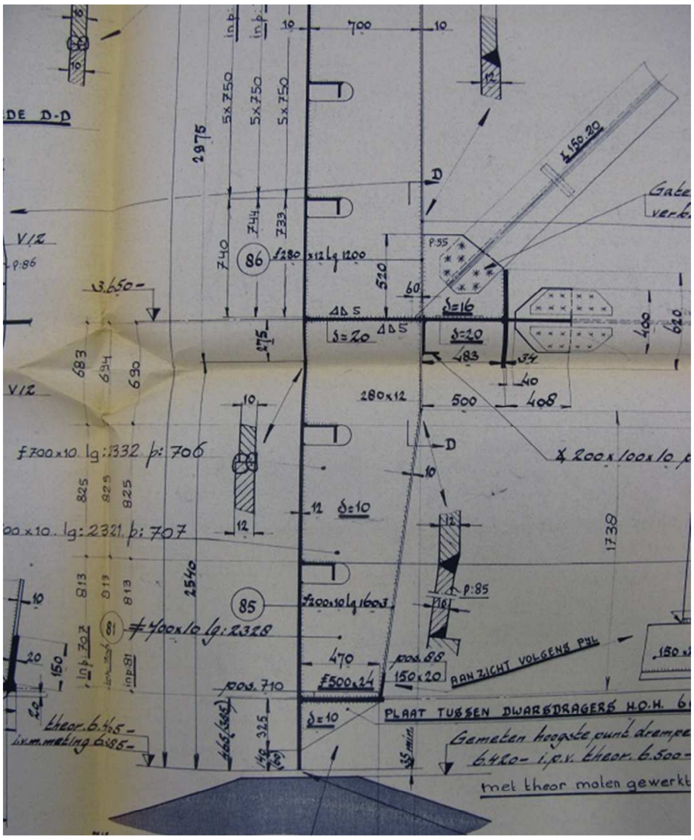


Figure 163 - Cross-section lower part plate wall

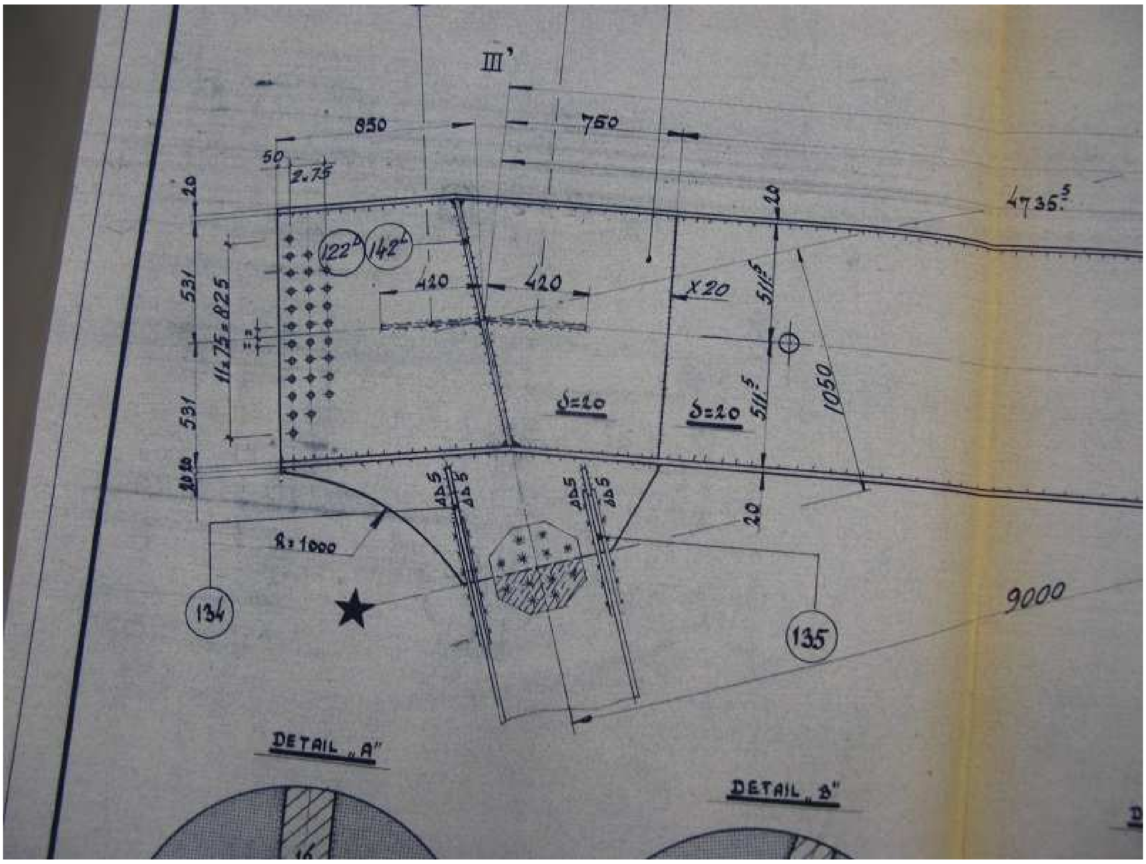


Figure 164 - Connection between strut and curved arch (1)

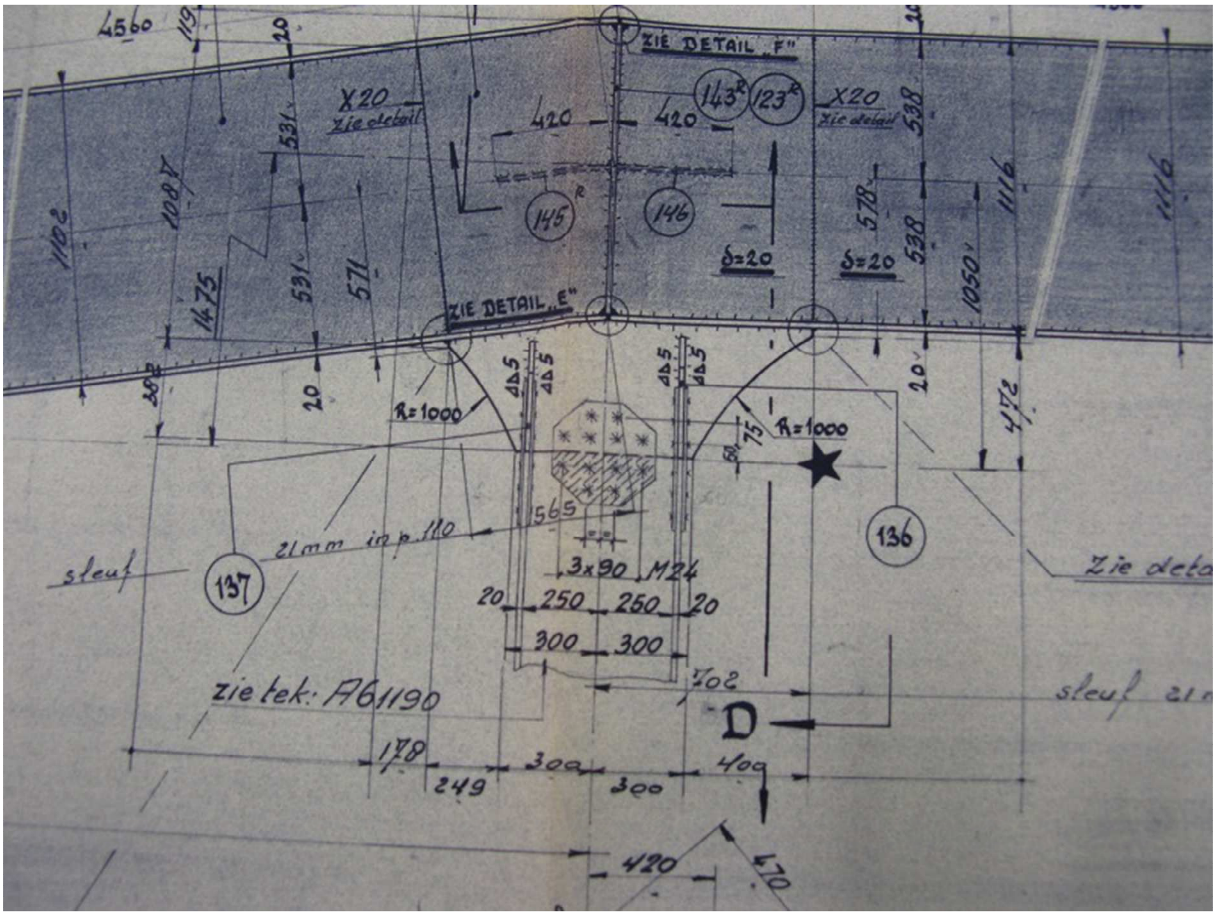


Figure 165 - Connection between strut and curved arch (2)

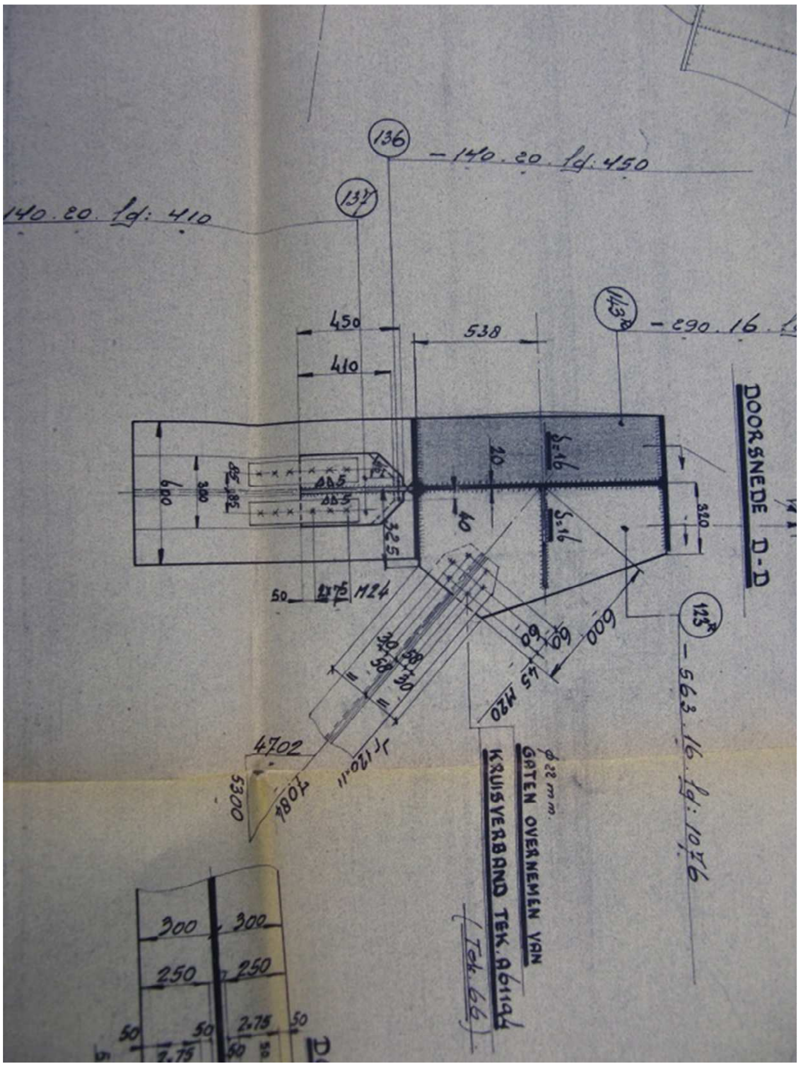


Figure 166 - Connection between strut and curved arch (3)

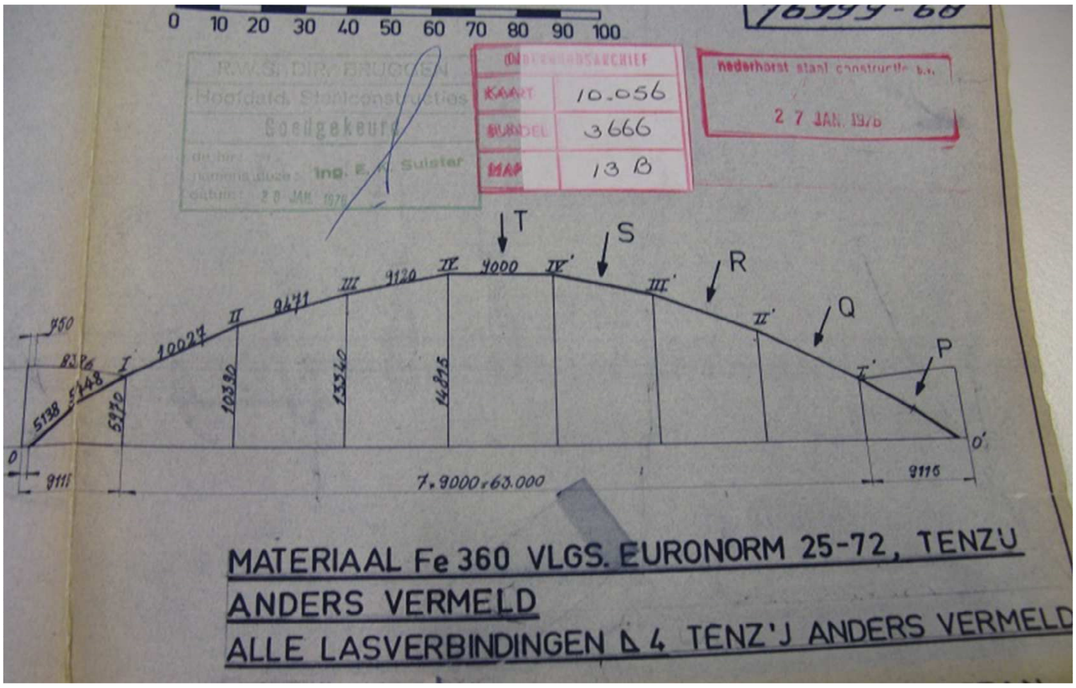


Figure 167 - Schematization steel gate

Appendix Q Results model RSTAB

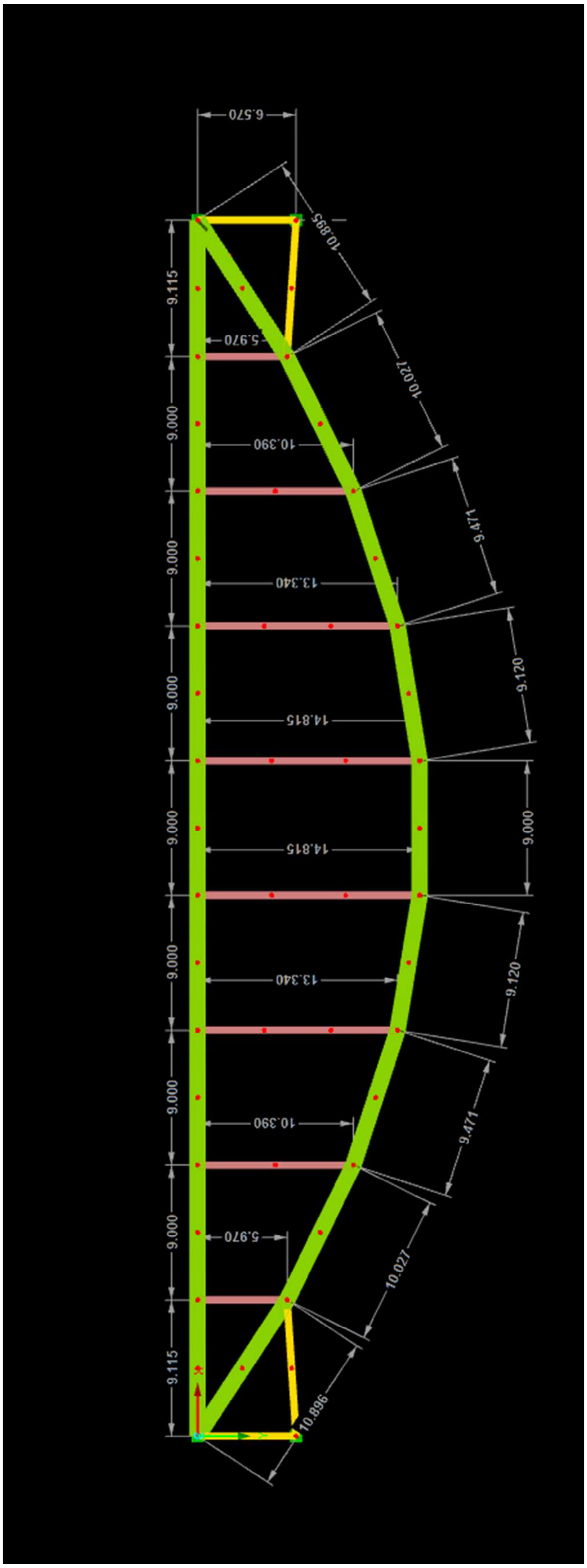


Figure 168 - Top view schematization gate RSTAB [measurements m]

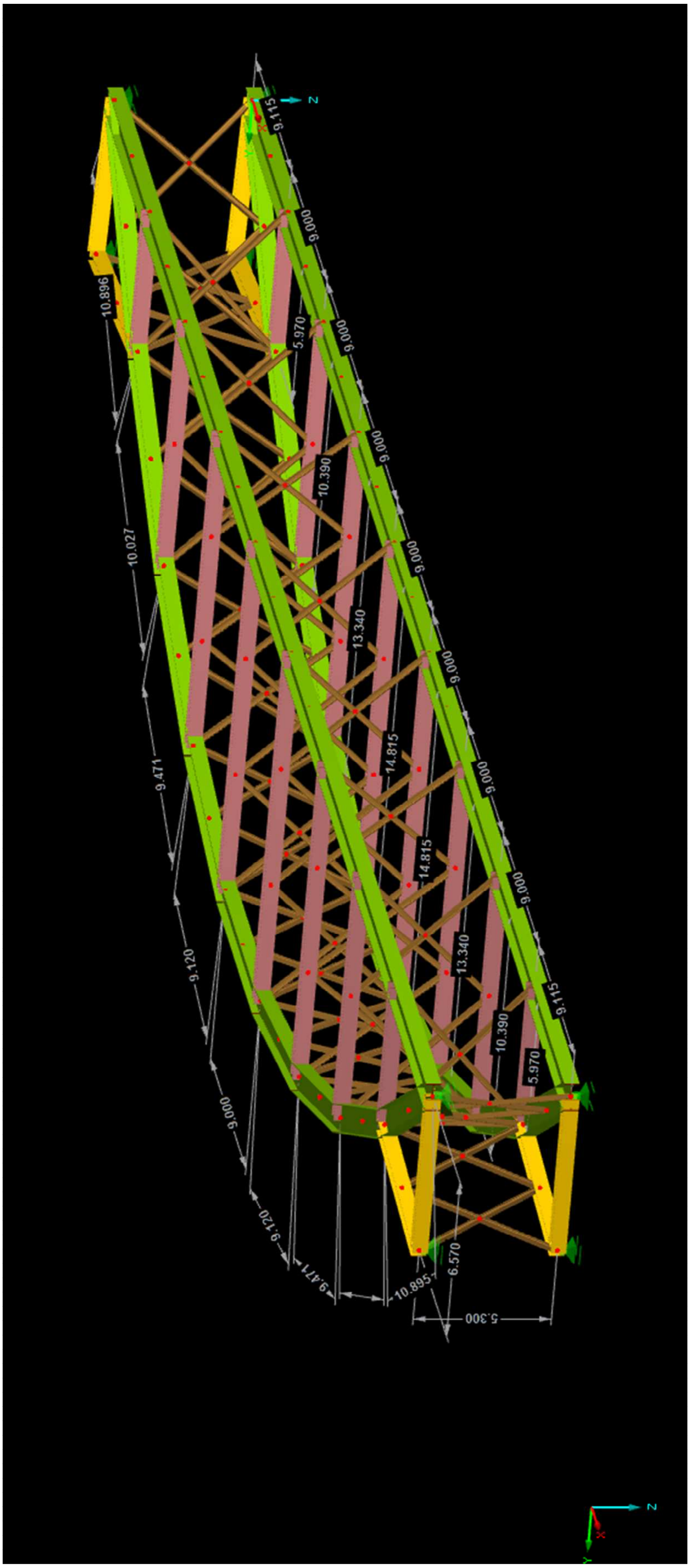


Figure 169 - 3D schematization gate RSTAB [measurements m]

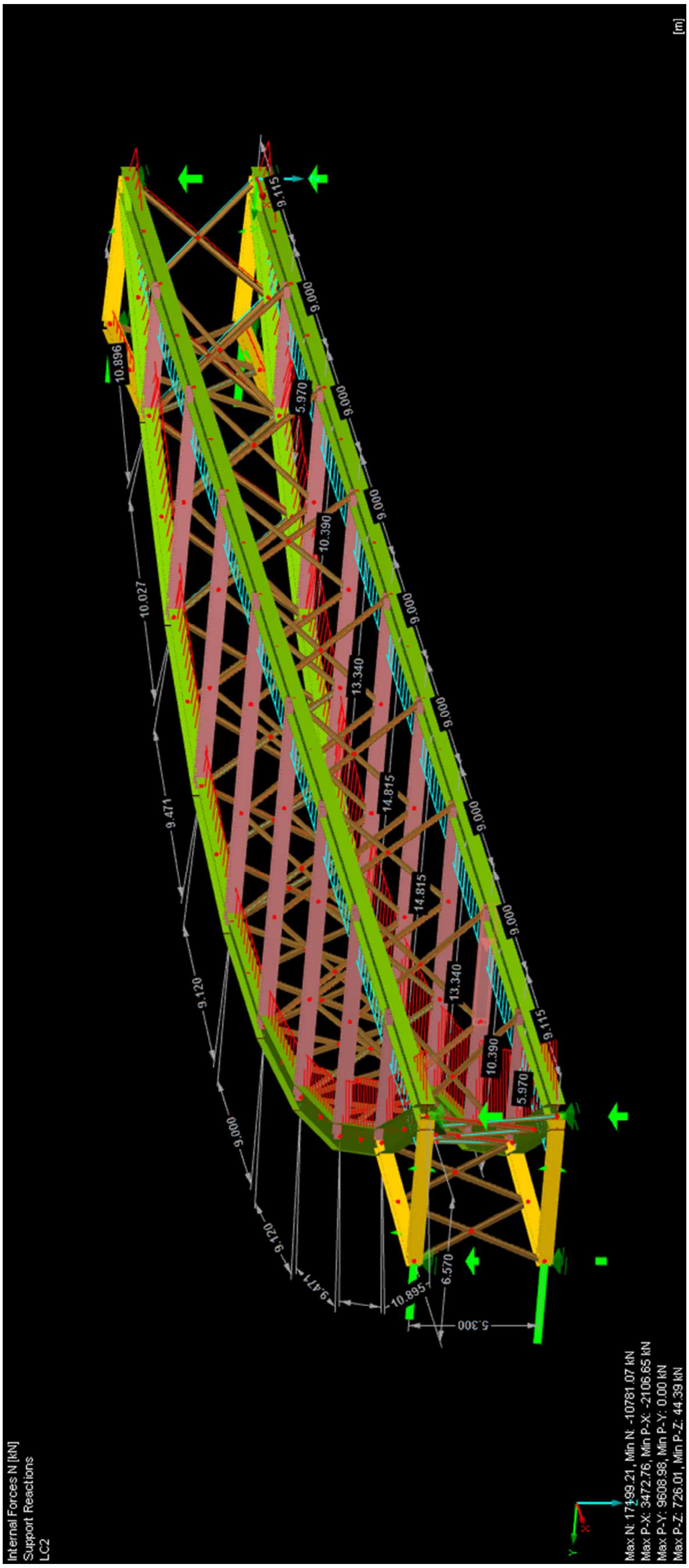


Figure 170 - 3D schematization gate RSTAB, Normal force storm surge [measurements m]

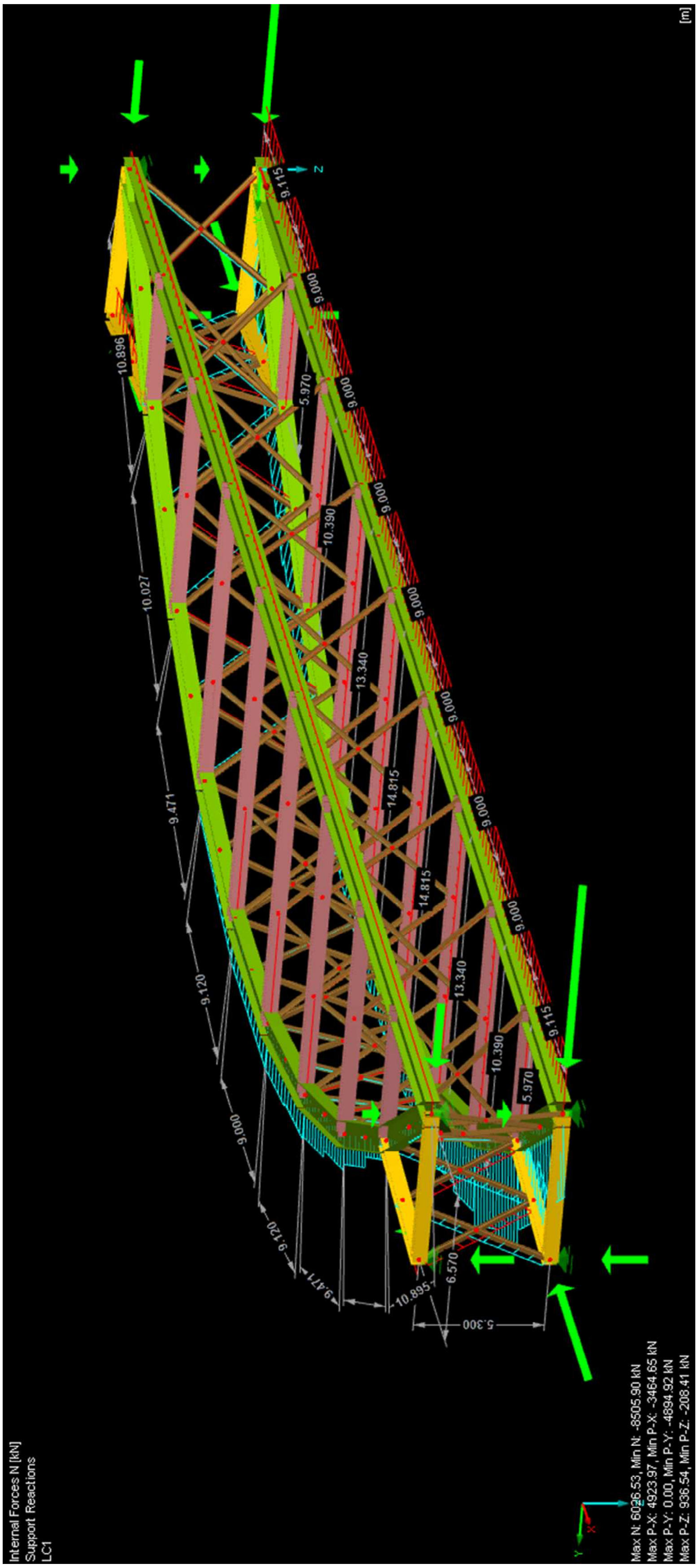


Figure 171 - 3D schematization gate RSTAB, Normal force salt intrusion [measurements m]

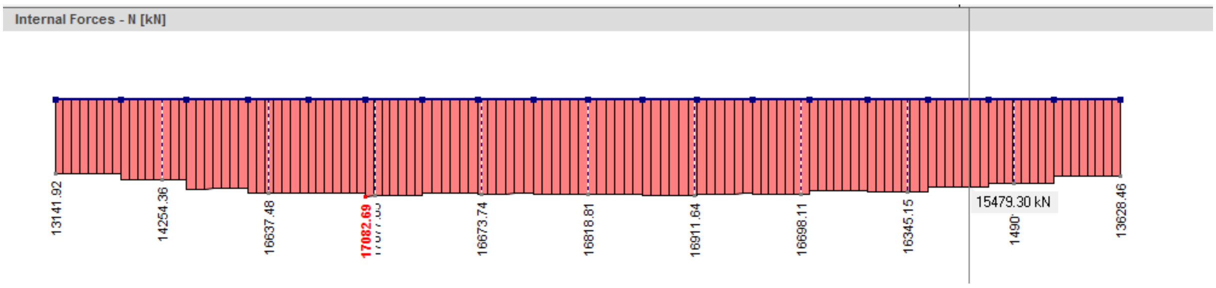


Figure 172 - Underside curved arch, load combination storm surge

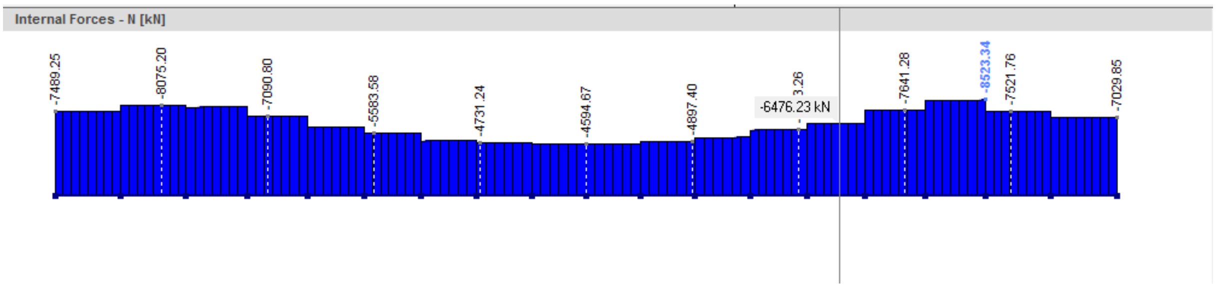


Figure 173 - Underside curved arch, load combination salt intrusion

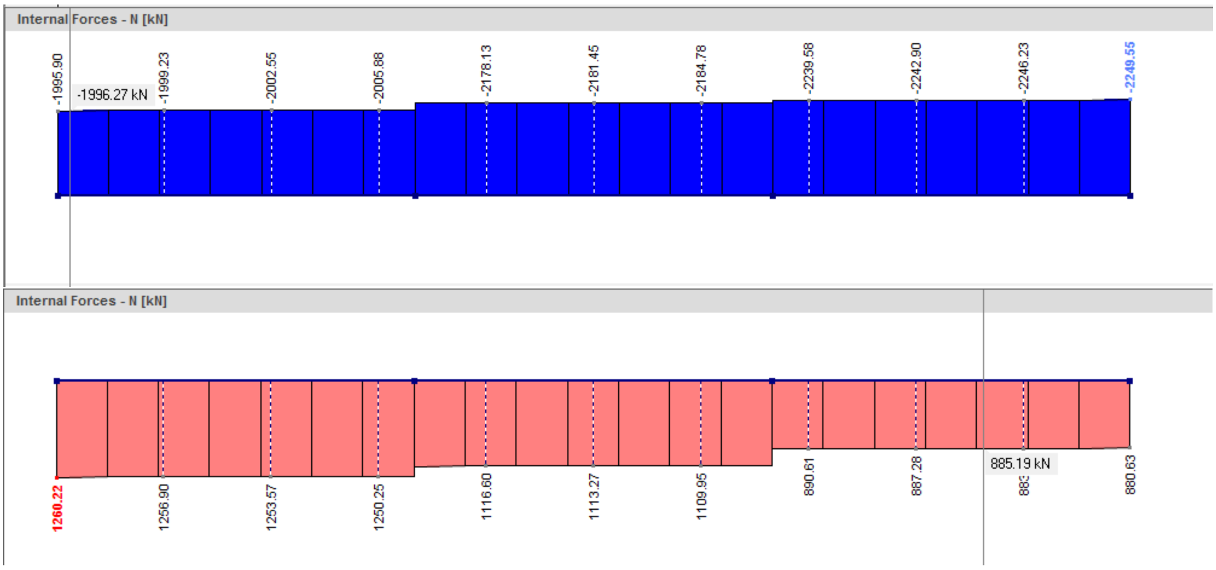


Figure 174 - Strut in the middle of the framework, storm surge (upper) and salt intrusion (lower)