THE IMPACT OF A BREACH IN THE AFSLUITDIJK ON THE PROBABILITY OF FAILURE OF IJSSELMEER DIKES

MSc thesis - Hydraulic Engineering

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Witteveen + Bos

Challenge the future

Cover image on the front page: Afsluitdijk, two days before closure in 1932. Rijkswaterstaat. Retrieved November 2014: https://beeldbank.rws.nl/MediaObject/Details/319940

MSc Thesis

The impact of a breach in the Afsluitdijk on the probability of failure of IJsselmeer dikes

by

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PREFACE

In this report the results of my research titled ”The impact of a breach in the Afsluitdijk on the probability of failure of IJsselmeer dikes” are given. The research was carried out as a MSc thesis and is the graduation work that concludes the study Civil Engineering at Delft, University of Technology.

I would like to show my gratitude to the members of my graduation committee, Prof. dr. ir. Matthijs Kok, Dr. ir. Oswaldo Morales Napoles, Dr. ir. Paul Visser, and ir. Philippe Schoonen, for their guidance and their supervision during my work on this thesis. I especially would like to thank them for their function as mentor during my research as I learned a lot about dike breaches, flood risks, and research on an academic level.

In addition to my graduation committee, I would like to thank a number of other people who helped me and contributed to the realisation of this thesis. First of all I would like to thank the people who helped me starting up my thesis. Coen Kuijper and Rina Clemens from Witteveen + Bos helped me finding the interesting topic of this report when the previous topic had to be abandoned. The change of the subject of the thesis required new motivation and I would like to thank Peter-Jules van Overloop from the TU Delft and Nienke Kramer from Deltares for their help and advice during the early stages of this research.

Writing this thesis and doing the research was mostly done at the company Witteveen + Bos in Deventer. I am very grateful for their support and would like to thank my colleagues from the department Water defences (both Deventer and Rotterdam) for the warm welcome. Especially I would like to thank my daily supervisor Philippe Schoonen and my roommate Joost Noordermeer for all the small talk and the daily ’fruit moments’ that kept me healthy and thinking sharp.

I also would like to thank Deon Slagter from Rijkswaterstaat, André Veenhuijsen from Deltares, Jes Kaihatu from Waterschap Zuiderzeeland, Peter van Tol from Witteveen + Bos, and especially Bastiaan Kuijper from HKV for their guidance and assistance with the PC-Ring model. Using and understanding this program required more time then I expected and therefore their help was very welcome.

For the final stages of my research I would also like to thank Willemijn Hoebert from NOS, Andrew Stepek from KNMI, Guus Kruitwagen from Witteveen + Bos, Gerbrant van Vledder from the TU Delft for helping me with their expertise on topics I am not very familiar with.

Finally I would like to thank the ones who supported me during my spare time. My family, friends, and roommates in both Delft and Deventer supported me whenever I found difficulties in my research. For this support I am very grateful.

Pim van den Akker
Deventer, December 2014

Summary

Introduction

Until the early years of the 20th century, floods around the Zuiderzee led to many deaths and high economic losses. During multiple storms, the dikes along the Zuiderzee failed to protect the hinterland against high water levels resulting in many casualties. To prevent these kind of floods, the Afsluitdijk was created and the Zuiderzee was turned into a lake: the IJsselmeer. Since its completion in 1932, the Afsluitdijk fulfilled its function of protecting the IJsselmeer and its hinterland properly.

During the statutory second and third assessment of primary water defences in the Netherlands (in 2006 and 2011) the Afsluitdijk was judged as 'not sufficient', which means it did not meet the governing standards. According to these assessments the Afsluitdijk is not able to resists the loads of an 1/10,000 per year storm. As an effect of these assessments it was decided to improve the strength of the Afsluitdijk. This is currently an ongoing project and the project aims to secure the requirement of providing enough strength to reduce the probability of failure to meet the formal safety standard. In this way, the Afsluitdijk can continue to protect the IJsselmeer and the surrounding cities and villages until at least 2050.

At the moment, for determining the safety of the dike ring areas around the IJsselmeer a possible failure of the Afsluitdijk is assumed to have no influence on loads occurring on the IJsselmeer dikes (see figure 1). As failure of the Afsluitdijk is always possible, the consequences of such a failure should be investigated. The forming of a breach in the Afsluitdijk will have impact on the IJsselmeer. The influence of a breach on the water level might be significant for the safety of the dike ring areas around the IJsselmeer (see figure 2). If this is the case, the risks of flooding of the IJsselmeer dikes are higher than currently is assumed.

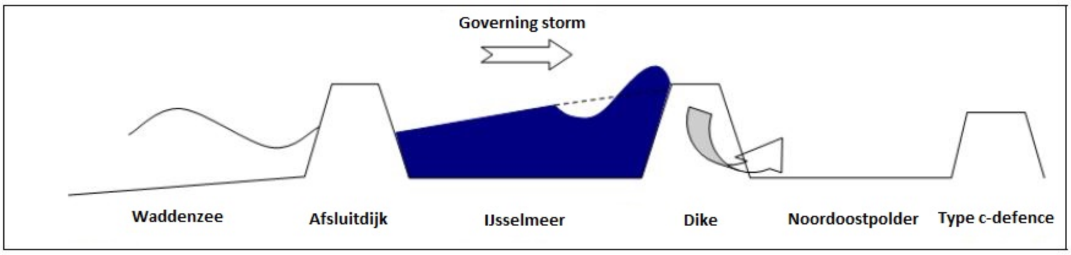

Figure 1: Current governing conditions for failure of the dikes of the Noordoostpolder, assuming the Afsluitdijk stays intact.

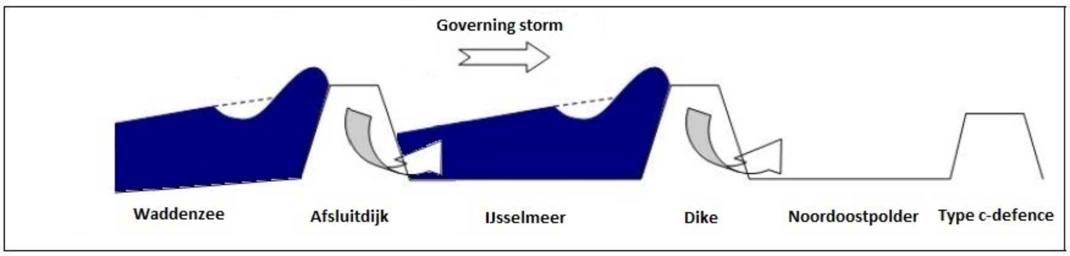

Figure 2: Governing conditions for failure of the dikes of the Noordoostpolder, assuming the Afsluitdijk also fails.

The objective of this thesis is to investigate the impact of a possible failure of the Afsluitdijk. What happens with the water level of the IJsselmeer if during extreme conditions the Afsluitdijk breaches? This objective is achieved by first defining different possible breach scenarios. Then, the effect of such a breach on the water level of the IJsselmeer is investigated. Subsequently, the consequences of this increase in water level on the probability of failure of the dikes of the Noordoostpolder is studied. Finally, a discussion and the conclusions of the results are presented.

POSSIBLE BREACH SCENARIOS

In previous studies it was found that the Afsluitdijk can either fail at the sluices or at the levee itself. It is found that both have a more or less same order of probability of failure. Failure of the (current) sluices is expected to be 1/250 per year, whereas the probability of failure of the levee is expected to be 1/140 per year. For both failure locations scenarios are constructed, one for a breach at the sluices (scenario 1) and two for a breach at the levee (scenario 2 and 3). It is also possible a breach takes place at both the sluices and at the levee. This possibility is assumed in another scenario (scenario 4).

This thesis shows that other important factors for determining the impact of a breach are the dimensions of the breach, the timing and the development of the breach during a storm, the duration of high water, and the properties of the ground layers in the Afsluitdijk. Inside the Afsluitdijk a boulder clay layer is present that might prevent the erosion process of a breach during failure. As the effect of this layer is very uncertain, two possible scenarios for a levee breach are used in this thesis. Both are extremes of the effect of boulder clay. At one scenario the boulder clay is assumed to not erode away at all (scenario 2), where at the other scenario the boulder clay layer is assumed to have the same properties as a sand layer (scenario 3).

For determining the depth and width of the possible breach, assumptions are done for each of the scenarios. These assumptions are based on literature. In case of failure of the sluices it is assumed all the sluices will blow out, creating a sudden breach width of more than 300 meters. In case of failure at the levee the breach dimensions depend on the effect of the boulder clay layer and on the intensity of the storm hitting the Afsluitdijk. Higher Waddenzee water levels cause bigger breach widths. Based on literature a relation was found between extreme water levels in the Waddenzee and the expected width of a levee breach in the Afsluitdijk.

INFLUENCE ON THE IJSSELMEER

To investigate the effects of the four breach scenarios on the water level of the IJsselmeer a Matlab [1] model is build. This model calculates the increase in average IJsselmeer water level. It first simulates possible Waddenzee water levels. Subsequently, the amount of discharge that flows through the breach is calculated. The amount of discharge is calculated for each of the four breach scenarios. Executing these models results in exceedance frequency curves for each scenario. These curves show the increase in average IJsselmeer water level due to an Afsluitdijk breach with its corresponding probability of exceedance.

INFLUENCE ON THE NOORDOOSTPOLDER

With the results of the average IJsselmeer water level increases, the local IJsselmeer water level can be found by adding the wind set up or subtracting the wind set down. These results are used as input for the PC-Ring model [2]. This model is used to calculate the total probability of failure of the whole dike ring area. In this thesis research PC-Ring is run twice. Once for the boundary conditions used in the current assessment and once given the breach is already present in the Afsluitdijk. For the second run the increased water levels due to a breach in the Afsluitdijk are added to the boundary conditions and the wind set up is adjusted to the timing of the highest local water levels.

Next to the change in probability of failure of the Noordoostpolder also the consequences of flooding change in case of a breach in the Afsluitdijk. If the Afsluitdijk fails, the water level of the IJsselmeer will rise to the average water level in the Waddenzee. This is a few decimeters higher than the current target level of the IJsselmeer water. A higher water level in the IJsselmeer will cause more water to flow through the breach in the dike of the Noordoostpolder leading to higher water levels in the Noordoostpolder.

CONCLUSIONS

For both cases, failure and no failure of the Afsluitdijk, the risks and consequences are combined to see the change in the risks of flooding of the Noordoostpolder. With the results from this thesis a couple of conclusions can be drawn:

- The current probability of failure of the Afsluitdijk is much higher than the current standard. Both the sluices and the levee itself are prone to failure. The Afsluitdijk can fail at either (or both) of these locations.

- Although the current probability of failure is low, the whole IJsselmeer system provides enough safety for the dikes of the Noordoostpolder during a storm at which the Afsluitdijk fails. It is found that the maximum local water levels are found at the peak and not in the hours after the peak of the storm at which the Afsluitdijk failed. The increase in water level of the IJsselmeer due to failure of the Afsluitdijk during an extreme storm, has no effect on the safety of the dikes around the IJsselmeer during that same storm.

- In the levee of the Afsluitdijk a boulder clay layer is present. If this boulder clay layer will not erode during the breach process, a levee breach in the Afsluitdijk will hardly have any effect on the IJsselmeer.

- If the boulder clay layer will erode at a levee breach or if the breach takes place at the sluices, the whole IJsselmeer system is able to deal with large quantities of discharge of water through a possible breach in the Afsluitdijk. This is because (1) the retention area of the IJsselmeer is relatively big compared to the dimensions of a possible breach in the Afsluitdijk, (2) in case of a levee breach it takes a long time for the breach to grow to significant dimensions, and (3) the large distance between the Afsluitdijk and the other side of the IJsselmeer that causes a delay in the local water level increase.

- In case multiple extreme storms occur during one winter half year, a breach in the Afsluitdijk does effect the probability of failure of the dike of the Noordoostpolder. However this is only the case if the first extreme storm is devastating enough to cause a breach in the Afsluitdijk. As the probability of a combination of such a storm together with a second storm is very low, the risks of flooding does not increase significantly if possible failure of the Afsluitdijk is taken into account.

- If this conditionality is not taken into account (e.g. ad hoc decision making) the risks do increase. Given that an 1/250 per year storm has already hit the Afsluitdijk and given that this storm caused a complete blow out of all the sluices in the Afsluitdijk, the risks of flooding of the Noordoostpolder increases with a factor of 2.5 (as long as this breach is not repaired).

RECOMMENDATIONS

Following this research it is recommended to improve the knowledge about the retardant effect the boulder clay layer has on the breaching process. Also it is important to research the effects of wind on the local water conditions. And it is strongly advised to investigate how the wind speed can best be modelled in order to compare the correct wind speeds with the correct water level increases, as the results of this thesis are estimations of the real increased risks.

It is also recommended to investigate the increased risks of failure of the other dike ring areas around the IJsselmeer, as these are also affected by a breach in the Afsluitdijk. Finally, it is advised to do research about the effect of a breach in the Afsluitdijk on the other functions of the Afsluitdijk (e.g. the impact on the fresh water storage function).

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List of symbols

Below a list is given of all the symbols presented in the main part of the report. Symbols used in the appendices are explained in their associated appendix. For all variables SI-units are used unless stated otherwise. Probabilities are expressed as probability per year throughout the report.

Symbol	Definition	Unit
a	Used as a constant	-
A_{lake}	Surface area of the IJsselmeer, $A_{lake} = 1100$	km^2
A_{polder}	Surface area of the Noordoostpolder, $A_{polder} = 494$	km^2
a_w	Parameter to describe the directional wind statistics	s^2/m^2
B	The width of the breach	m
b	Used as a constant	-
b	Vertical gate opening	m
b_w	Parameter to describe the directional wind statistics	s/m
c	Used as a constant	-
c	Constant in the wind set up formula, $c = 3.4 * 10^{-4}$	-
c	Travelling speed of a disturbance in shallow water	m/s
C_c	Contraction coefficient for underflow, $C_c = 0.61$	-
C_d	Discharge coefficient, $C_d = 0.81$	-
c_w	Parameter to describe the directional wind statistics	-
$F(x,\mu,\sigma)$	Cumulative distribution function with parameters x, μ, and σ	-
g	Acceleration due to gravity, $g = 10$	m/s^2
h	Depth of the IJsselmeer	m
$\bar{H}_{IJsselmeer}$	Average water level in the IJsselmeer	m + NAP
Δh	Increase of water level	m
ΔH	Water set up	m
$\Delta H_{average\ water\ level}$	Change in average water level due to a breach in the Afsluitdijk	m
ΔH_{local}	Change in local water level	m
ΔH_{wind}	Change in water level due to wind set up	m
H_{IJ}	Water level in the IJsselmeer	m + NAP
$H_{IJsselmeer,local}$	Local water level in the IJsselmeer	m + NAP
h_{polder}	Average height of the ground level in the polder	m + NAP
H_w	Water level in the Waddenzee	m + NAP
i	Number of runs in the Monte Carlo analysis	-
$Kr(u)$	Function to fit the wind statistics, $Kr(u) = au^2 + bu + c$	-
L	Fetch length	m
m	Flow coefficient for overflow, $m = 0.88$	-
m_{final}	Average water level in the final situation	m + NAP
m_{start}	Average IJsselmeer water level at the beginning of the storm	m + NAP
P	Probability of occurrence	$year^{-1}$
$P(F_A)$	Probability of failure of the Afsluitdijk	$year^{-1}$
$P(F_N \cap F_A)$	Probability that both the dikes of the Noordoostpolder and the Afsluitdijk fail	$year^{-1}$

Symbol	Definition	
$P(F_N\|F_A)$	Probability that the dikes of the Noordoostpolder fail given that the Afsluitdijk has failed	$year^{-1}$
$P(X \leq x)$	Probability that X is smaller or equal to x	-
P_f	Probability of failure	$year^{-1}$
Q	Discharge through the breach	m^3/s
q	Exceedance frequency	-
t	Timing after the beginning of the storm	h
u	Wind speed	m/s
W	Width of a gate	m
x	Water level	m
X	Random variable water level	m
y_1	Upstream water level	m
y_2	Water level at the lowest point in a hydraulic jump	m
y_3	Downstream water level	m
α	Shape factor in the cumulative distribution function, directly related to ξ	-
μ	Location parameter in the cumulative distribution function	m
σ	Scale parameter in the cumulative distribution function	m
ξ	Shape factor in the cumulative distribution function, directly related to α	-

I

Part I: Introduction

1

Chapter 1: Introduction to the thesis

1.1. Introduction

Until the early years of the 20th century, floods around the Zuiderzee led to many deaths and high economic losses. For instance the severe winter storm on 13 and 14 January 1916. It coincided with high discharges of the rivers leading to extreme water levels in the Zuiderzee. The dikes along the Zuiderzee failed to resist this high water level resulting in many casualties. This (among others) resulted in the realisation of the Zuiderzee Act. One of the goals of this act was to protect the central part of the Netherlands from flooding by closing of the Zuiderzee from the North Sea by creating the Afsluitdijk and turning the sea (Zuiderzee) into a lake (IJsselmeer). From its completion in 1932 until today the Afsluitdijk fulfilled its function of protecting the IJsselmeer and it surroundings properly.

During the statutory second and third assessment of primary water defences in the Netherlands the Afsluitdijk got the judgement 'not sufficient' which means it did not meet the governing standards [3]. According to these assessments the Afsluitdijk is not able to retain the loads of an once in 10,000 year storm which is the current standard for the Afsluitdijk (see appendix C for an explanation of the term safety standard). This is because of two reasons. First, the sluices in the Afsluitdijk at Den Oever and Kornwerderzand are not able to cope with water levels that correspond with an once in 10,000 year storm. This means it is likely the sluices fail at less extreme storms. Second, the levee itself is not resilient enough to deal with the failure mechanism overtopping (see appendix A for the definition of overtopping). An average height of NAP +7.75 m [1] [4] is not enough to prevent the occurrence of overtopping. The grass layers at the crest and the inner slope of the dam are probably not able to deal with the amount of overtopping that will occur during these extreme conditions. The crest and the slope of the Afsluitdijk will be washed away resulting in instability of the dam and a possible breach.

As an effect of these assessments it was decided to improve the strength of the Afsluitdijk [5]. This is currently an ongoing project and aims to secure the 1/10,000 per year requirement. In this way, the Afsluitdijk can continue to protect the IJsselmeer and the cities and villages lying around its boundaries until at least 2050 [6].

1.2. Problem formulation

In this section the problem is formulated, the goal of the thesis is set, and the method to reach this goal is described.

1.2.1. Problem

During every assessment the Afsluitdijk is tested to make sure the probability of failure lies lower than the set norms. This is done to secure the safety of the area behind the Afsluitdijk. However, at the moment, for determining the safety of the dike ring areas around the IJsselmeer a possible failure of the Afsluitdijk is not

[1] NAP is an abbreviation of Normaal Amsterdams Peil (English: Amsterdam Ordnance Datum). It is used as a vertical reference point in large parts of Western Europe

taken into account.

Both the Afsluitdijk and the IJsselmeer dikes are assessed but their dependence is not taken into account. With this simplification an error can be made, as the Afsluitdijk is assumed to retain 'only' storms as extreme as an once in 10,000 year occurrence and not more extreme storms. This means there is a possibility that the Afsluitdijk fails. The error at the moment is even bigger as the Afsluitdijk currently does not fulfil this requirement. The forming of a breach in the Afsluitdijk has consequences for the IJsselmeer. However, these consequences are not quantified. The influence of a breach on the water level might be significant for the safety of the dike ring areas around the IJsselmeer.

1.2.2. OBJECTIVE

Goal of this thesis is to investigate the consequences of a possible failure of the Afsluitdijk. What happens with the IJsselmeer if during extreme conditions the Afsluitdijk breaches? And what happens to the probability of failure of the water defences around the IJsselmeer? Goal is to quantify these consequences.

1.2.3. MAIN QUESTION

To reach the goal of this thesis the following main question has to be answered:
What are the consequences of failure of the Afsluitdijk on the IJsselmeer and on the safety of its surrounding dike ring areas?

1.2.4. RESEARCH QUESTIONS

To get to the answer on the main question, the main question is split up into multiple sub questions:

1. What are the main characteristics of the current IJsselmeer system? (Chapter 2)

2. What possible scenarios of breaching of the Afsluitdijk can be expected? (Chapter 3)

3. What is the influence of a breach in the Afsluitdijk on the IJsselmeer and what kind of water level increases can be expected? (Chapter 4)

4. What is the effect of this increase of the IJsselmeer water level on the safety of the IJsselmeer dikes? (Chapter 5)

Note on sub question 3: At the start of this thesis research, this sub question was stated differently. First, it was attempted to find a new probability density distribution of the water level in the IJsselmeer. Later, it was found that expressing the influence of a breach on the IJsselmeer as water level increases was more useful for implementing the water level in the PC-Ring model (in sub question 4). Also, the impact of a breach on the IJsselmeer water level is now presented easier.

1.3. RESEARCH METHODOLOGY

Questions numbers 1 and 2 of the research questions is answered by doing a **literature study**. In this study a system analysis takes place to get an idea about the project and to answer research question 1. Question 2 is answered by consulting previous studies and selecting possible scenarios of a breach in the Afsluitdijk. It gives input parameters that are used for answering the other research questions.

For research question 3 a **numerical model** is used. A simplified scheme of the Waddenzee - IJsselmeer interaction is used as input in Matlab. Different models will be used for (1) the increase in volume of water in the IJsselmeer and (2) the wind set up. In this way the influence of a breach on the system can be calculated.

Research question 4 will is solved by using the **model PC-Ring** (see appendix B for a description of PC-Ring). In this way new probabilities of failure are determined. To also deal with the consequences, use is made of already investigated reports (Dutch: dijkringrapporten). Different consequences for different failure probabilities are assumed. The probabilities and the consequences combined give risks which can then be compared (see also appendix C for the definition of risks). Also the effect of a dike breach on other functions of the IJsselmeer will be briefly investigated (chapter 6).

1.4. READERS GUIDE

The four research questions are treated in four different chapters in this report. As chapter 1 is reserved for the introduction, the number of the research questions does not correspond one on one with its chapter number (e.g. research question 3 is answered in chapter 4).

The first two questions are coupled in part I. Part I, together with this chapter, forms the introduction of the research. The first chapter will give an introduction to the subject and to the thesis report. Chapter 2 is also part of the introduction. Chapter 2 answers the first research question and gives an overview of the IJsselmeer system. Chapter 3 is coupled to the introductions as this chapter is also part of the literature study. In this chapter the possible breach scenarios (in both volume and time) are addressed together with its probability. End result of this chapter are possible scenarios of breaching of the Afsluitdijk that will be used as input in chapter 4.

In part II the last two research questions are answered. These two chapters focus on the simulations and the results of these simulations. Chapter 4 uses a model to calculate the increases in water level of the IJsselmeer. These increases are used in chapter 5 to estimate the increased probability of failure of the dikes of the Noordoostpolder. Chapter 5 also looks at the increase in consequences and risks of a breach in the Afsluitdijk.

In part III the results are analysed in a reflection. This is done in chapter 6. Here are some final remarks made. In chapter 7 the conclusions and recommendations will be given.

An overview of the readers guide is schematized in figure 1.1.

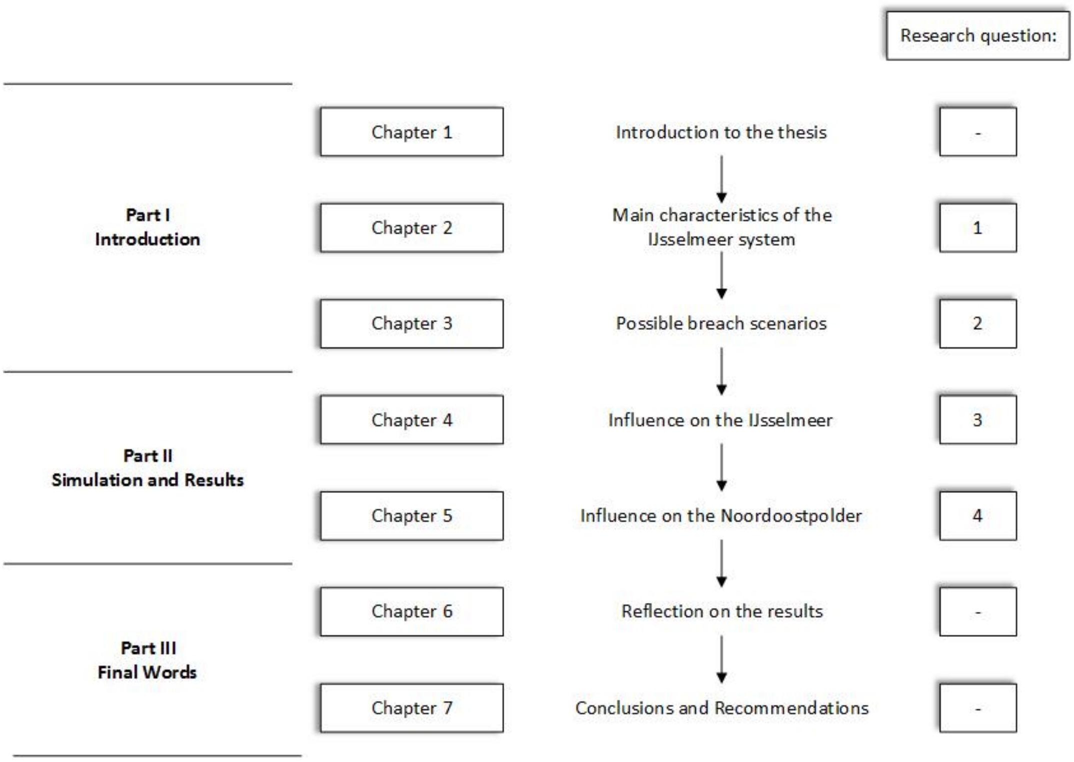


Figure 1.1: Readers guide of this thesis

2

Chapter 2: Main characteristics of the IJsselmeer system

In this chapter the first research question will be answered: *What are the main characteristics of the current IJsselmeer system?* As many studies about the Afsluitdijk and the IJsselmeer have already been done the question of this chapter is answered by doing a literature study.

First, in section 2.1, an overview is given about the history of the flood defences in the Netherlands. Section 2.2 continues with the history of the IJsselmeer and Afsluitdijk. It describes the location, lay out, and functions of the whole IJsselmeer system. Section 2.3 describes how the water in the IJsselmeer system works and how the Afsluitdijk protects the Noordoostpolder.

2.1. Flood defences in the Netherlands

Approximately two thirds of the Netherlands ($\approx 25000\ km^2$) is at risk of flooding. Of this, a large area lies below sea level (see figure 2.1). This floodplain area comprises very large, densely populated polders accommodating most of the Dutch population and economy. The existence of the Netherlands is dependent on reliable flood protection structures. Protection against flooding is thus an important national issue and is a political task embedded in the Dutch constitution. Failure of flood defence structures has devastating consequences and not only in the stricken area. As the economic hart of the country lies in the Randstad (one of the lower lying parts in the West of the Netherlands), the entire country will be disrupted. Today, due to the need for flood protection structures and its importance, a statutory safety level is taken into account when designing and assessing flood protections.

Before 1200 there were few dikes as we know them today. People adapted to nature instead of preventing floods. In coastal areas settlements were in the higher dune areas or on artificially made hills (Dutch: terpen). In the river areas people lived on the natural levees of the rivers. Of course this way of living had its short comings. Not only caused the flooding a lot of casualties, also land needed to be abandoned with the rising sea level and fields were destroyed by the intrusion of salt on the land.

Between 1200 and 1400 the main rivers of the Netherlands were all enclosed by real dikes, but they were not of the same level we are used to today. They were designed with a lack of knowledge leading to too steep or too low levees. But as wars, plagues, and other diseases (which were very common in those times) killed enormous amount of people, flood protection was not given high priority.

It was not until the early 1900's that things started to change. Fluid and soil mechanics, mathematical and statistical knowledge improved, leading to a more scientific approach in dike design. The hydraulic loads on water defences could be predicted with a higher accuracy and the strength of the water defences could be calculated. These techniques were not used immediately, disasters had to occur to convince policy makers this approach was necessary.

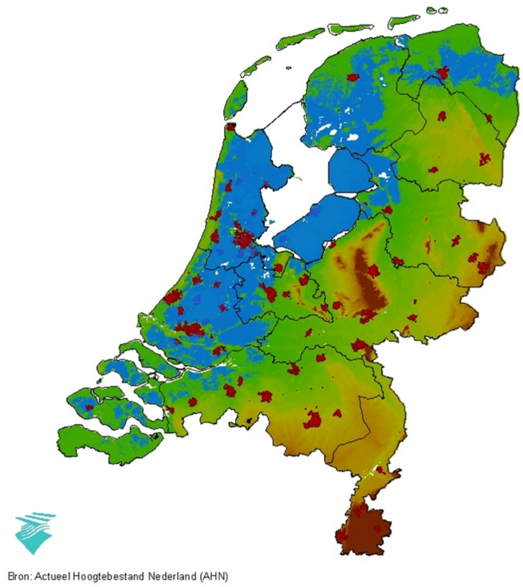


Figure 2.1: The Netherlands below sea level (in blue)

In 1912, serious dike breaching and flooding around the Zuiderzee occurred leading to the construction of the Afsluitdijk (see section 2.2.1). The disaster of 1953 gave the biggest shock. A combination of high spring tide, wind, and low atmospheric pressure led to extreme water levels causing 1835 deaths and enormous economic loss to in particular the province of Zeeland (see figure 2.2). To prevent these kind of disasters the 'Delta Committee' was formed. This committee proposed a new way of looking at the safety of the Netherlands against flooding. It was the beginning of the development of probabilistic techniques in hydraulic engineering. The use of statistical techniques in determining the hydraulic load and the realisation that the strength of the defence is a stochastic parameter changed the way of designing dikes. Next to this new approach, it was decided to close off the islands in the south west of the country from the sea. Several large dams and storm surge barriers were constructed of which the last one, the Maeslant Barrier, was completed in 1997.

Figure 2.2: Floodings of 1953 (Source: watersnoodmuseum.nl)

Since then, Dutch flood defence systems have improved over the years, but still serious dike problems with high water in the rivers occurred in the later years of previous century. With the expected rise of sea level, increases of discharge of the main rivers, increase of population, and increase of the value of property new challenges occur for the (near) future. By initiatives such as the Delta program, policy makers aim for a durable and safe water protection systems to prevent disastrous flooding events like the ones that occurred in the past.

Sources used in this section: [7] [8]

2.2. OVERVIEW OF THE IJSSELMEER SYSTEM

This chapter will give an overview of the current situation of the Afsluitdijk and the IJsselmeer. The section will start with the history of the Zuiderzee works (Dutch: Zuiderzeewerken). It will continue with a description of its location and the lay out. Last section is about the functions of the Afsluitdijk and the IJsselmeer.

2.2.1. ZUIDERZEE WORKS

As was described in previous section, in the 19th century and the early years of the 20th century multiple floods around the Zuiderzee occurred resulting in many casualties and economic damage. Together with the benefits of land reclamation this led to multiple studies to reclaim the Zuiderzee.

In 1886 de Zuiderzeevereniging was founded. Led by Cornelis Lely they investigated if land reclamation of the Zuiderzee was possible. This research led to plan Lely in 1891 and looked a lot like the current layout of the IJsselmeer.

However, as the costs of creating the Zuiderzee works were enormous for that time the plan was not started until 1918. During January 13 and 14 1916 a severe winter storm coincided with high discharges of the rivers leading to extreme water levels in the Zuiderzee. The dikes along the Zuiderzee failed to resist this high water level resulting in many casualties. Together with the food shortage in Europe as a result of World War I it was decided the original plan of Lely needed to be executed. It resulted in the Zuiderzee Act. This law led to the availability of financial funds to work out the plan for protecting the central Netherlands from flooding on the one hand and creating agricultural land on the other hand.

From 1920 till 1926 a committee led by Hendrik Antoon Lorentz executed detailed calculations, making minor adjustments to the original plan. In 1920 the Zuiderzee works started with the construction of the Amsteldiepdijk. This relatively small work gave a lot of new insights that were used in the construction of the Afsluitdijk which started in 1927. The Afsluitdijk was constructed from 4 geographical points. Den Oever and Zurich (the mainland) and two constructed working stations in the Afsluitdijk Breezand and Kornwerderzand.

After 5 years of work the afsluitdijk completed in 1932 (see the picture on the front cover of this thesis), turning the sea (Zuiderzee) into a lake (IJsselmeer and later also in Markermeer). One year later (1933) the road on the Afsluitdijk was finished resulting in an official opening to traffic. In the years after the completion the other parts of the IJsselmeer area were developed. First the Noordoostpolder was reclaimed followed up by the Eastern part of Flevoland and finally the Southern part which finished in 1968.

2.2.2. LOCATION

The location of the Afsluitdijk is chosen in such way that tidal velocities were as low as possible during the construction phase. This resulted in the connection Den Oever – Zurich instead of Den Oever – Piaam a little more to the south of Zurich (as was described in the original plan Lely) (see figure 2.3). Also a nod near Kornwerderzand was found to be favourable over a straight connection because of the alignment of the sluices at Kornwerderzand in respect to the bathymetry.

Since the construction of the Afsluitdijk not much has changed. Biggest change was made during the seventies. During those years the initial road over the Afsluitdijk was improved to a multiple lane highway allowing more cars on the Afsluitdijk.

2.2.3. LAY OUT

A description of the lay out of the IJsselmeer system can be split into two relevant parts. A description of the Afsluitdijk (useful for chapter 3) and a description of the IJsselmeer (useful for chapters 4 and 5).

THE AFSLUITDIJK

The total length of the Afsluitdijk is approximately 32 km and has an average width of 90 meters at the height of the mean sea level. On average, the crest height of the Afsluitdijk lies at NAP + 7.75 m [4].

Figure 2.3 gives an overview of the lay out of the Afsluitdijk. On the west and east side of the Afsluitdijk locks (for allowing ships to pass) and sluices (for the discharge of IJsselmeer water on the Waddenzee) are situated. The sluices in the east are located at the island Kornwerderzand and the sluices in the west are located very close to the village Den Oever. In the middle of the Afsluitdijk the Breezanddijk is situated. This is a former island that was used as a working station to assist with the construction of the other parts of the Afsluitdijk. This island is connected to the two lock complexes by the levee of the Afsluitdijk. This levee looks more or less the same along the full length of the 32 km long Afsluitdijk.

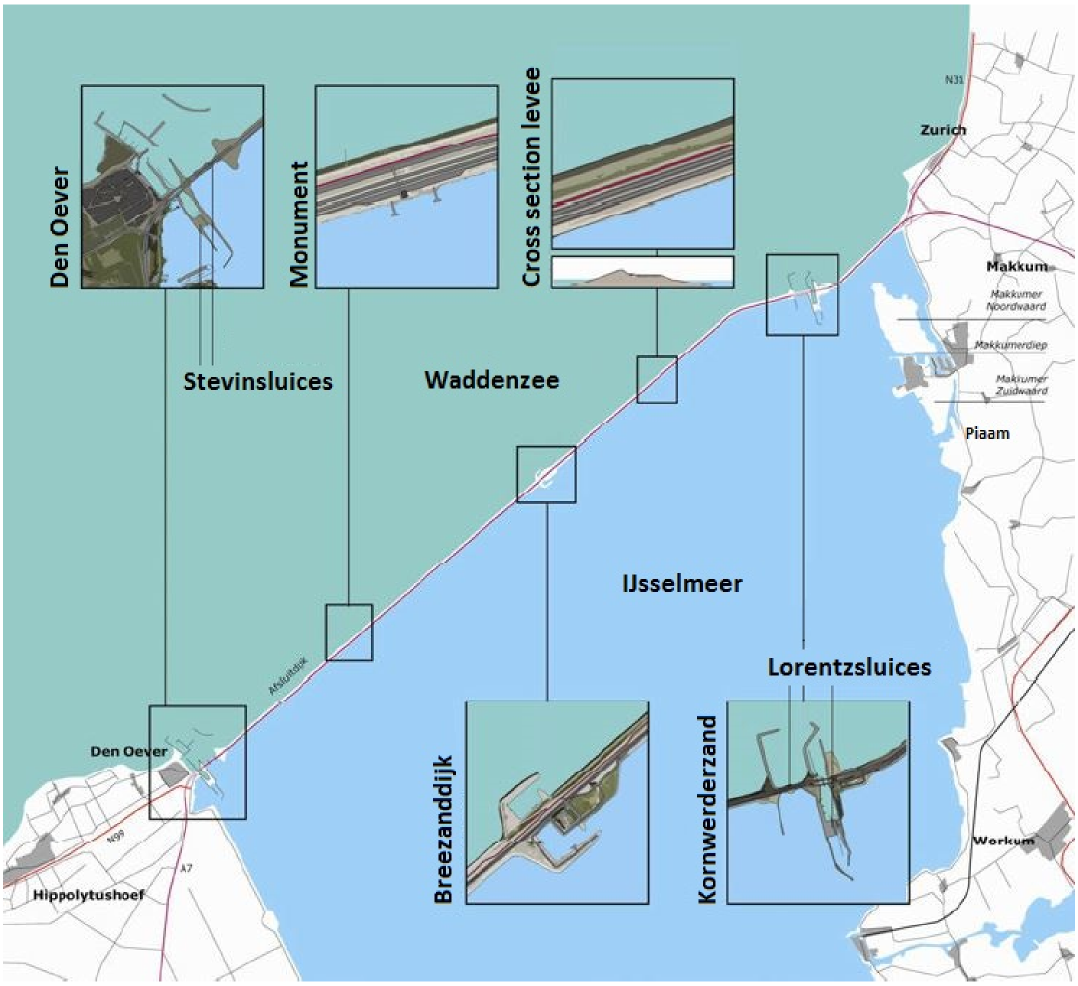


Figure 2.3: The current lay out of the Afsluitdijk (source: [9])

A typical cross section of the levee can be seen in figure 2.4. Important to notice is the boulder clay layer inside the Afsluitdijk. This layer is very relevant for determining the consequences of a breach in the levee, as will be shown later on in chapter 3.

THE IJSSELMEER

With the creation of the Afsluitdijk, and later the various land reclamations, the IJsselmeer-areas were founded (see figure 2.5). This area consists of the areas that were once part of the Zuiderzee. It not only consists of the water bodies, but also the land reclamations are part of this area. The main water bodies are the IJsselmeer and the Markermeer. The other water bodies are grouped as the Randmeren (English: side lakes). The Randmeren are enclosing the province of Flevoland and consists of (from west to north-east) IJmeer, Gooimeer, Eemmeer, Nijkernauw, Veluwerandmeren, Ketelmeer, and Randmeren North. Main functions of these lakes is storage of water for water management purposes of the polders and to provide an outlet for rivers (like the river IJssel).

The IJsselmeer and the Markermeer are divided by another dam called the Houtribdijk (again see figure

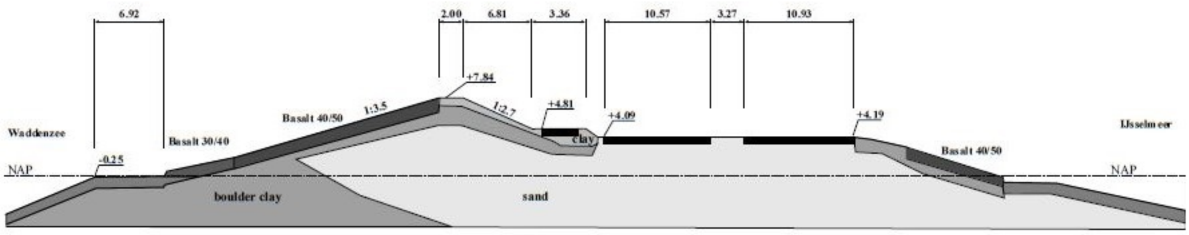


Figure 2.4: A typical cross section of the levee of the Afsluitdijk (source: [10])

2.5). This dam has a length of 26 km and connects Enkhuizen with Lelystad. Originally this dam was created to also reclaim the Markermeer. It never came that far, as in 2003 it was decided the storage of fresh water was a better function of the Markermeer than the possible functions new land would provide [11].

The IJsselmeer area is surrounded by 6 provinces (Friesland, Overijssel, Flevoland, Gelderland, Utrecht, Noord-Holland) and many water boards have their interest in the area of which Waterskip Fryslan, Hollands Noorderkwartier, and Zuiderzeeland are the most important. The IJsselmeer is also surrounded by multiple dike ring areas. The dike ring areas connected to the IJsselmeer are number 6, number 7, number 8, number 12, and number 13 (see also figure C.2 in the appendix). All but number 13 have at the moment a required safety standard of 1/4,000 per year. Number 13 has a stricter standard of 1/10,000 per year.

2.2.4. FUNCTIONS OF THE IJSSELMEER SYSTEM

The Afsluitdijk and the IJsselmeer have multiple functions. An overview is given in this section.

Primary function of the Afsluitdijk is to provide **safety against flooding** of the former Zuiderzee. This was the original thought behind building the dam and this function still is of high importance. It is seen as a primary defence system, protecting the hinterland from high water coming from the North Sea. How this system works is further explained in section 2.3.

Probably the most important secondary function of the Afsluitdijk is its **water retaining** function, separating the salt water of the Waddenzee from the fresh water from the IJsselmeer. This creates the biggest fresh water storage in the Netherlands. Providing drinking water and water for agricultural purposes for big parts of North-Holland, Friesland, Groningen, and Drenthe. In total about 30% of the Netherlands is directly or indirectly dependent on the water storage of the IJsselmeer area [12]. Next to providing enough fresh water, the discharge of rain and river water is also an important aspect of the water management function of the IJsselmeer area.

The Afsluitdijk has also a **mobility** function. It connects the province of Noord-Holland with Friesland. This way making it possible for cars to travel without the use of a ferry. The same goes for the Houtribdijk, connecting Noord-Holland with Flevoland. Navigational traffic is also possible at the sluices in both dams. This, together with traffic channels on the IJsselmeer and Markermeer, navigational traffic is possible from the river IJssel to the North sea or Amsterdam and vice versa.

This traffic function is not only for commercial navigation, but also for **recreational** purposes. Apart from recreational sailing, other forms of water sports are also active on the IJsselmeer. The Afsluitdijk is part of a walking and a bicycle route as are many dikes around the IJsselmeer. Finally the Zuiderzee works are seen as one of the most impressive hydraulic works all over the world and for this reason it attracts many tourists.

Many villages (and cities) around the IJsselmeer used to be very active in the **fishing industry**. With the transformation of the Zuiderzee into the IJsselmeer this changed. Since then, the fishing industry decreased over time. In 1970 the IJsselmeer fishing industry received another blow as a certain method of fishing was prohibited by the Dutch government. Nowadays only a few fishing companies are left.

Figure 2.5: The IJsselmeer area (source: wikipedia.org)

The IJsselmeer area also has an function for the **ecology**. The original plan was to reclaim the Markermeer. As stated before, in 2003 it was decided the Markermeer was not supposed to be reclaimed any more. Apart from its fresh water storage function the developed nature in the Markermeer was also an argument for this. Not only the Markermeer, but also other parts in the IJsselmeer area have an ecological value to be reckoned with.

This thesis only focusses on safety against flooding function of the Afsluitdijk. Failure of the Afsluitdijk can have very disastrous consequences for the other functions mentioned above. For the effect of a breach in the Afsluitdijk other studies can be found. In this thesis these functions are not taken into consideration (but it is treated briefly in section 6.1).

2.2.5. IJSSELMEER WATER CONTROL

The water level in the IJsselmeer is kept as close as possible to a certain target level. This is (mainly) done by operating the sluices in the Afsluitdijk. Two target levels are maintained, one for the summer months and one for the winter months. During summer (April - September), the IJsselmeer fulfills its very important function of providing enough fresh water for the Northern part of the Netherlands. As high evaporation rates are possible in these summer months, the target water level is relatively high: NAP -20 cm.

In the winter months (September - April), extreme winds and discharges are more likely to occur and thus the probability of flooding of the dikes around the IJsselmeer also increases. To reduce this probability of flooding the target water level in the IJsselmeer is reduced to NAP -40 cm.

These target levels are very difficult to be maintained as the amount of evaporation, percipitation, or discharge through the IJssel fluctuates more heavily than the possibility to discharge on the Waddenzee. Therefore, the actual average water level in the IJsselmeer can be much higher or lower than the target level. Figure 2.6 shows the record of the IJsselmeer water level for the years 1997-2006. On average the water level lies around NAP -19 cm in the summer months and around NAP -30 cm during the winter months. In the summer the water level is most of the time around the target level. However, during the winter, high standard deviations can be found.

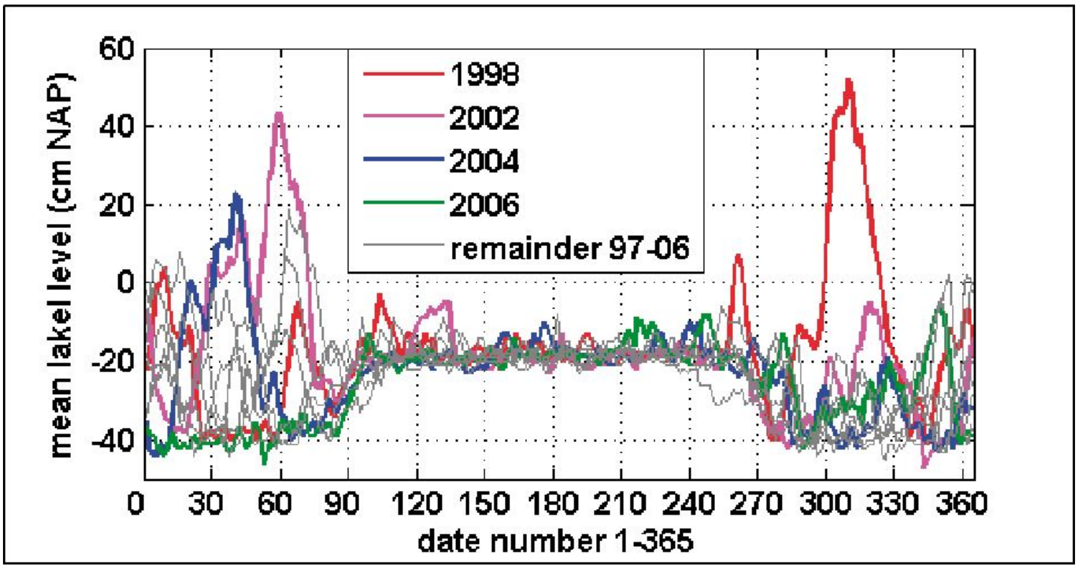


Figure 2.6: Average water level of the IJsselmeer as a function of the day number (1 is January 1th) (source: [13])

Later on, in section 4.1.2 and section 4.1.3, input caused by the river IJssel and discharge through the sluices of the Afsluitdijk is further discussed.

2.3. FLOODING OF THE NOORDOOSTPOLDER

The Afsluitdijk prevents that water from the Waddenzee can directly flow into the IJsselmeer. The other way around is allowed. Water from the IJsselmeer can flow into the Waddenzee through the sluices in the Afsluitdijk. In this way excesses of water can be discharged to the Waddenzee. These sluices open if the IJsselmeer water level is higher than the Waddenzee water level (at low tide) and close if the Waddenzee water level gets higher than the water level in the IJsselmeer. By doing this, the target average IJsselmeer water levels are tried to achieve (see previous section).

These average daily conditions in the IJsselmeer water system can be schematized as shown in figure 2.7. A balance is found between input (discharges through the river IJssel and Vecht, the surrounding water boards, precipitation, and the Markermeer) and the output (through the Afsluitdijk and evaporation).

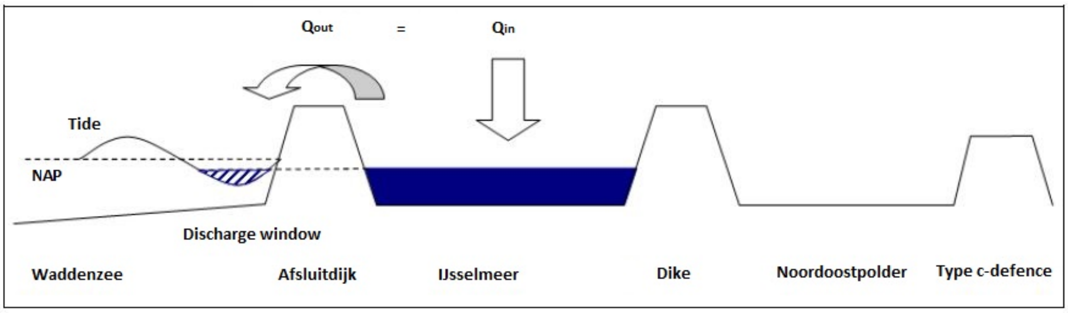


Figure 2.7: Schematistation of the IJsselmeer water system during daily conditions (source: [14])

If the discharge to the Waddenzee is temporarily impossible and discharges through the river IJssel are extremely high, the water level in the IJsselmeer can reach high values (see figure 2.6). This is schematized in figure 2.8

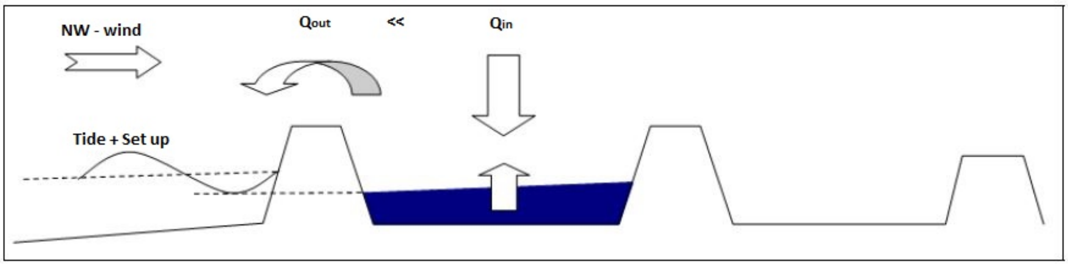


Figure 2.8: Schematistation of the IJsselmeer water system during no possible discharge through the Afsluitdijk (source: [14])

If a storm occurs that causes a big wind set up the Afsluitdijk can fail. If also the average IJsselmeer water level gets high enough during a storm the dikes around the IJsselmeer can fail too (see figure 2.9). Because these governing storms for both water defences depend also on tide (for the Afsluitdijk) and on the height of the average IJsselmeer water level (for the dikes around the IJsselmeer) these failures are not completely dependent on each other.

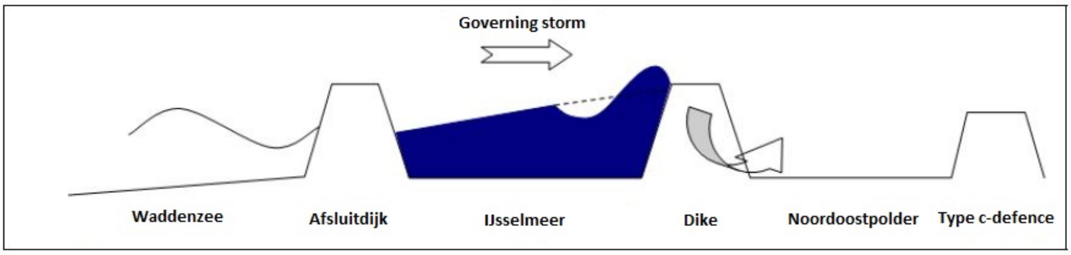


Figure 2.9: Schematistation of the IJsselmeer water system during a storm on the IJsselmeer dikes (source: [14])

However, as wind speeds (and direction) influence both possibilities of failure it can happen that failure of the Afsluitdijk also influences failure of the IJsselmeer dikes (see figure 2.10). This amount of influence is investigated in this thesis (see section 5.4).

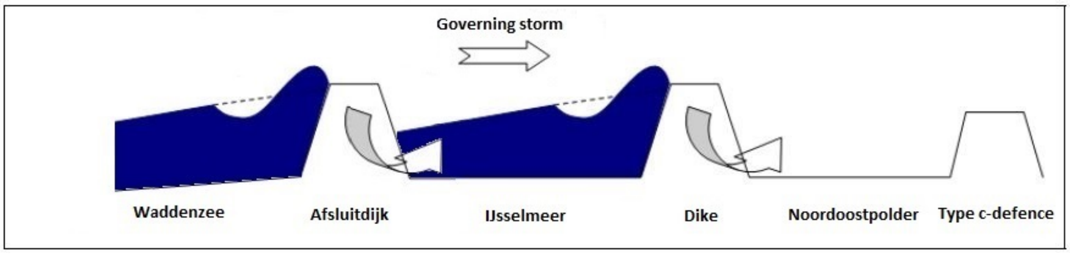


Figure 2.10: Schematistation of the IJsselmeer water system during a breach in the Afsluitdijk

If the dikes around the IJsselmeer fail, water will flow into the hinterland. In the case of a still intact Afsluitdijk the water level of the IJsselmeer will go down and water in the polder will rise to arrive at a new equilibrium (see figure 2.11).

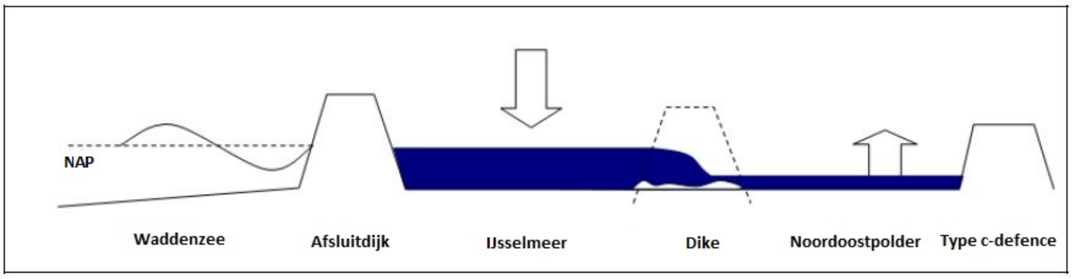


Figure 2.11: Schematistation of the IJsselmeer water system during a breach in the IJsselmeer dikes (source: [14])

In the case the Afsluitdijk has failed an equilibrium between the Waddenzee, the IJsselmeer, and the polder will be realised (see figure 2.12). As the Waddenzee has an open connection with the North Sea the IJsselmeer and the polder will take the equilibrium of the Waddenzee (which is approximately NAP + 0 m). This leads to bigger consequences of the flood than in the case of no breach in the Afsluitdijk (see section 5.5).

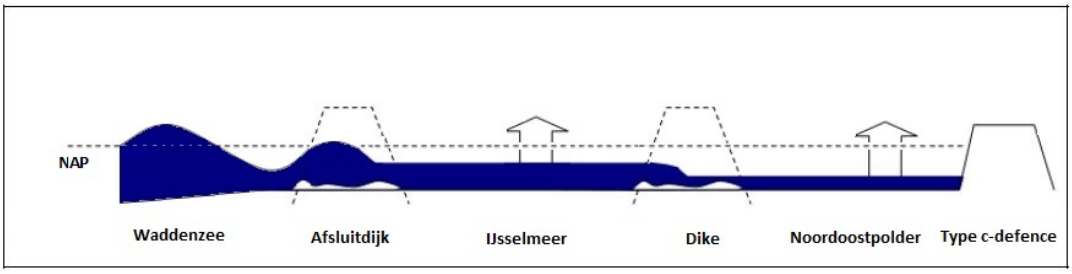


Figure 2.12: Schematistation of the IJsselmeer water system during a breach in the IJsselmeer dikes and in the Afsluitdijk

3

Chapter 3: Possible breach scenarios

In this chapter possible scenarios of breaches in the Afsluitdijk are given. Section 3.1 starts with an inventarisation of the most important factors that influence the consequences of failure of the Afsluitdijk. These factors are treated in different sections. In section 3.2 the probability of failure of the Afsluitdijk is defined. Sections 3.3 and 3.4 describe the dimensions of a breach at the sluices or in the levee. In section 3.5 an expression is given so dimensions can be calculated for storms with different magnitudes. Subsequently, section 3.6 treats the timing and the development of the breach and section 3.7 describes the duration of the storm.

The sections of this chapter show that the forming and development of a breach depend on many different factor and is a difficult process to predict. Some studies conflict with each other. Therefore the possibilities of a breach are expressed in 4 different scenarios. With help from already executed studies these scenarios are made as realistic as possible. These scenarios are presented in the final section 3.8.

3.1. Important factors for a breach in the Afsluitdijk

Once the rare event of failure of the Afsluitdijk occurs (see section 3.2) various factors are of importance. Factors influencing the consequences of the failure of both the levee and the hydraulic structures show much similarities. Therefore both are treated the same except for the parts where they differentiate significantly.

3.1.1. Load interdependencies

In W.J.Klerk's MSc thesis [15] the importance of system interaction (called load interdependencies in his thesis) in flood defences was addressed. A study was done to investigate the influence of failures of river dikes on the loads at other locations along the river. Despite the fact that the thesis focused on river systems and smaller areas within dike ring areas, it shows much relevance to the subject of the influence of a breach of the Afsluitdijk on the IJsselmeer dikes.

Klerk showed the (quite trivial) relation that non-exceedance probabilities of high discharges become lower if negative load interdependencies of polders are taken into account. Meaning high discharges happen to occur more likely if negative load interdependencies are also taken into account. For our case this means higher loads on the IJsselmeer dikes occur more likely if the negative load interdependency of a failure of the Afsluitdijk is taken into account.

3.1.2. Main factors effecting the discharge through the breach

In Klerk's report [15] three main factors were defined as the most influential on the effect of load interdependencies: polder retention volume, time of the breach, and polder side failures. Translated to the Afsluitdijk-IJsselmeer case the polder retention volume effect can be seen as the dimensions of a breach in the Afsluitdijk in respect to the retention volume of the IJsselmeer and the time of the breach can be seen as the point in time the Afsluitdijk starts to breach (before the storm, in the middle, at the end?). The third factor, polder side failures, was taken into account to express the (unusual) influence of water loads on the inside of polder dikes. For our case, this factor is not of importance as a failure of the Afsluitdijk does not attack the IJsselmeer dikes from behind. This leads us to two mayor factors determining the intensity of the load interdependency of the Afsluitdijk and the IJsselmeer dikes.

A third factor can be added to Klerk's findings. This factor is less relevant for river systems, but it is for high water at the coast. This factor is the duration of high water. High water in rivers can almost be seen as a stationary process as it lasts for multiple days (and thus will probably last longer than the time it takes to fill up the polder). High water at the coast is different as this lasts in the order of 2 to 4 astronomical tide cycles (i.e. 1-2 days). Therefore, the remaining duration of the storm has a huge influence on the amount of water that is discharged through the breach. The development of the outside water level is therefore the third important factor.

For investigating the load interdependencies of a failure of the Afsluitdijk on the IJsselmeer these three factors are of major importance:

1. Dimensions of a breach in the Afsluitdijk when a breach occurs (in respect to the dimensions of the IJsselmeer) (treated in section 3.3, section 3.4 and section 3.5)
2. Timing (and development over time) of the breach (treated in section 3.6)
3. Duration of high water (treated in section 3.7)

The next sections give possible scenarios in which these factors are included. Where the factor timing of the breach is discussed not only the start of breach is of importance but also the development of the breach over time. One can imagine that the dimensions (defined in section 3.4) of the breaches need time to develop.

3.2. CURRENT SAFETY OF THE AFSLUITDIJK

Before the expected dimensions and development of a possible breach is discussed, the probability of the event of a breach is investigated first.

3.2.1. FIRST AND SECOND NATIONAL ASSESSMENT

Around the time of the first recommendations of the Deltacommittee (1960) the probability of failure of the Afsluitdijk was estimated to be 1/1,430 per year ([16] p67). This probability of failure was seen as acceptable in that time. As such, for the first national assessment (1996-2001) this norm was taken as the governing standard [17]. However, this first assessment led to no outcome for the Afsluitdijk as it was found that defining the safety of the Afsluitdijk was not easily done.

During the second round of assessment (2001-2006) another attempt was made to define the safety of the Afsluitdijk. For this second assessment the Afsluitdijk was tested to resists an 1/10,000 per year storm [18]. This value was supported by the safety norm (also 1/10,000) for Dike ring 13 (North-Holland). According to the assessment a b-type defence system (see appendix D for the definition of a b-type defence water defence system) must be assessed on the hydraulic loads corresponding with the highest standard of the dike ring area it protects [16]. Partly because of this increase in safety standard (and partly because of the new insights in strength and loads) the second assessment could not classify the Afsluitdijk as sufficient.

This did not mean the Afsluitdijk was marked as not sufficient. According to the assessment a hinterland study must be carried out to see the influence of a disfunctionality of the Afsluitdijk on the dike ring areas behind the Afsluitdijk. The expected probability of failure was not mentioned in this assessment.

In the assessment it became clear that failure of the Afsluitdijk can be expected with regard to two different aspects of the dam [18]. The first is the dam itself (called the levee of the dam in this thesis). In the assessment, the dam showed not enough resistance to erosion protection at the crest and the inside of the dam. This means that over-topping waves will erode the levee to such extend a gap will occur leading to failure of the Afsluitdijk. Second aspect are the structures within the levee. All four hydraulic structures (2 sluices and 2 navigation locks) were also judged as not sufficient. All four do not show enough strength in the construction elements to resist the loads corresponding with an 1/10,000 year occurrence [4]. Also the reliability of closing the locks and the sluices was not assessed as sufficient. As with the first assessment, no probability of failure was given, thus it can only be assumed to be higher than 1/10,000. What also was concluded in this assessment is that failure of the Afsluitdijk can occur by failure of the levee or failure of the sluices. In this chapter both types of failure are investigated and described separately.

3.2.2. ECONOMIC OPTIMAL SAFETY BY CPB

The assessment gave no indications about the current safety of the Afsluitdijk. To assemble indications of the current level of safety of the Afsluitdijk multiple studies were consulted. In January 2014 the Dutch Bureau for Economic Policy Analysis (Dutch: CPB) presented a study for the economic optimal safety level of the IJsselmeer area [19] (bases the assumption on studies of Deltares [20]). It presents the optimal investment pattern for the dikes in the IJsselmeer area by using a newly developed model called Diqe-Opt. The results of this model were twofold: how much the dikes needed to be strengthened and the most ideal timing of this improvement. For the research a time span is chosen from present till 2050. It is assumed that after the year 2050 improvements are necessary and other studies need to take over to look at the year after 2050.

For the failure probability of the Afsluitdijk the study made a few assumptions. The most important is that the current failure probability of the Afsluitdijk is 1/250 per year. This includes both failure of the levee and failure of the sluices. As stated in the study, this assumption is based on 'personal communication' of the Dutch ministry of Infrastructure and Environment.

3.2.3. PROBABILITY OF FAILURE OF THE LEVEE

Another study about the safety of the Afsluitdijk was carried out by S. Veraart [21]. In his MSc thesis he investigated the current probability of failure of the Afsluitdijk. He used the model PC-Ring to calculate the failure probabilities of the most common failure mechanisms on the levee of the dam. He found that overtopping, followed up by instability of the outer slope, were the most important failure mechanisms. Veraart found a probability of failure of 1/140 per year [1]. This differs almost a factor two from the assumptions of the CPB. On top of the 1/140 per year the failure of the hydraulic structures must also be added as Veraart focused on the levee part only. As previously stated this is also a weak spot in the Afsluitdijk and must therefore not be neglected.

The assumption that a 1/140 year storm will cause failure of the Afsluitdijk is questionable. The storm in 1953 (see appendix E) caused water level that have return period of 1/250 per year near Hoek van Holland. During that same storm not even the slightest threat of failure occurred at the Afsluitdijk [22]. Although the storm 1953 was not as extreme at the Afsluitdijk as was at Hoek van Holland, a failure frequency of 1/140 per year must be placed in perspective.

3.2.4. PROBABILITY OF FAILURE OF THE HYDRAULIC STRUCTURES

Unfortunately, no specific numbers about the probability of failure of the sluices were found. It can be concluded from the second national assessment [18] that the probability of failure of the sluices are of the same order of magnitude as the probability of failure of the levee since both aspects are defined as weak spots.

The report of the CPB [19] continues by using the same assumption that was used in another study on the CPB about the future of the Afsluitdijk [23]. In these reports a failure of the hydraulic structures is taken as a representative breach for their further calculations. This means that the probability of failure of the sluices is ought to be even higher than the failure of the levee. This is also in line with the statement in 'Structuurvisie Afsluitdijk' by Rijkswaterstaat [9]. In this report by Rijkswaterstaat, the hydraulic structures are also seen as the weakest link.

In all of the above described literature no direct numbers are given for the probability of failure of the sluices in the Afsluitdijk. All studies agree that the probability of failure is higher than 1/10,000 per year and most studies add the comment that the contribution to the current probability of failure of the Afsluitdijk is dominated by failure of the sluices.

3.2.5. CONCLUSION ON CURRENT SAFETY

All studies agree that the current probability of failure is much higher than 1/10,000 per year. It can be concluded that both the hydraulic structures (the sluices) and the levee itself can be the cause of failure of the Afsluitdijk.

In the previous subsections it was found that the total probability of failure of the Afsluitdijk used in the Deltares report was 1/250 per year. Of this total probability of failure, failure of the sluices is seen as the dominant factor. Veraart investigated failure at the levee only and found a probability of failure of 1/140 per year. Although these results differ from each other, the values are more or less in the same order of magnitude. The expected current safety used in this thesis therefore lies in the range of 1/140 and 1/250 per year. Both outer bounds will be used in the scenarios later on (see section 3.8). It is assumed the Afsluitdijk will start forming a breach once these extreme conditions occur.

[1] During the writing of this thesis, Veraarts thesis was not finished yet. In the final version of Veraarts thesis this probability is corrected to 1/200 per year. Although this correction changes the outcome of this thesis (up to chapter 4), this correction is not adjusted in this thesis. From section 5.2 and onwards the breach is taken at the sluices and not at the levee, therefore adjusting this probability of failure to 1/200 does not influence the conclusions of chapter 5.

3.3. DIMENSIONS OF A BREACH AT THE SLUICES

There are 4 different hydraulic structures in the Afsluitdijk. According to the second assessment these are all evenly likely to fail during the event of an extreme storm [18].

At the Den Oever side of the Afsluitdijk the 'Stevinsluizen' are located (see also figure 2.3). This complex comprises of 15 gates each with a length of 12 meter. The navigation lock is of a smaller volume compared to the navigation lock at the Friesland-side. Here the lock has a width of 13 meter [24]. On the other side of the Afsluitdijk the 'Lorentzsluizen' can been found. Here, 10 gates are located, also with a length of 12 meter each. The navigation lock system consists of a small lock of 9 meter and a larger lock of 14 meter in width [25].

An overview of the width of the hydraulic structures in the Afsluitdijk is given in table 3.1.

		Width single gate	Number of gates	Total width
Stevin	Sluices	12 m	15	180 m
	Lock	13 m	1	13 m
Lorentz	Sluices	12 m	10	120 m
	Small lock	9 m	1	9 m
	Large lock	14 m	1	14 m
Total			28	336 m

Table 3.1: Dimensions hydraulic structures Afsluitdijk

If all of the structures (both sluices and locks) will fail, a total breach width of 336 meter will occur.

The report of Deltares [20] assumes that all the hydraulic structures will 'blow out' during extreme water head differences. In the CPB report [19] this assumption is explained as a breach with an ultimate width of 400 meter. Remarkably this is the same width as the dimensions of a breach in the levee used in the CPB report (see section 3.4). This, together with the fact that total width of all the hydraulic structures is only 336 meter, makes the assumption of a 400 meter breach in the structures questionable.

In the CPB report reference is made to Grevers and Zwaneveld [23]. This cost-effective analysis report assumes also a breach of 400 meter. However, in this report no indication is given whether this is the breach at the structures or in the levee. It simply assumes that if the Afsluitdijk fails a breach with a width of 400 meter will develop. No further reference is made here. Both studies refer to Deltares again [20]. In this report the width of a breach at the structures is found to be 'comparable' with the dimensions of a breach at the levee. The report states that the main difference is the depth of the breach. For the structures this is equal to the bottom level of the sluices: NAP -4.40 m.

In this thesis it is also assumed all the sluices will 'blow out' at once. A total width of 400 meter seems physically impossible, therefore a total width of 336 meter is taken as the maximum width of a breach at the sluices. Although the probability that all sluices blow out together is not likely, it is the worst case scenario and is therefore used in the following parts of this thesis (more about the uncertainty in this assumption in appendix Q).

3.4. DIMENSIONS OF THE LEVEE BREACH

Already in 1996 research was started to look at the influence of the Afsluitdijk on the safety level of the IJsselmeer dikes. In 1998 the first report was published [26]. This report formed an indication on the required safety of the Afsluitdijk. One of the conclusions of this report was that the dimensions of the breach are of great importance on the safety of the areas behind the Afsluitdijk. A breach resulting in eroding of the (boulder) clay layer inside the Afsluitdijk was seen as highly dangerous to the hinterland.

Whether or not this (boulder) clay layer will erode during extreme storm conditions was investigated in Futloo's 'Breach growth in clay dikes; evaluation of bres and breach models' [27]. However, as erosion in cohesive material is very complex, Futloo did not come to satisfying conclusions.

In 2002 another attempt was made. Dr.ir. Visser did research on the growth of a breach in the Afsluitdijk after failure of the dam [28]. The report focussed on the possible development of a breach in the Afsluitdijk after failure of the Afsluitdijk during the occurrence of the design water level (Dutch: MHW). It states that if the breach in the Afsluitdijk is limited in growth, the rise in water level will also be limited and the IJsselmeer dikes are not in danger. However, Visser also states that if the breach reaches a significant state the secondary IJsselmeer dikes are in trouble.

3.4.1. VISSER'S BRES MODEL

Visser uses a model called BRES [29] to simulate the development of the dike opening after a breach. In this model he schematized the Afsluitdijk without neglecting the different properties of the dam. The model included two elements of the Afsluitdijk that increase its resilience against breach growth significantly. These are the boulder clay layer and the outer berm structure. The boulder clay layer is the inner part of the Afsluitdijk (see also figure 2.4) and was originally used to close of the Afsluitdijk. At the stage of near closing the dam high flow velocities occurred (see the picture on the cover of this report). The boulder clay was used to prevent washing out the dam during execution. The second element is about the bank and toe protection. The protection on the outside of the Afsluitdijk is relatively strong when compared to dikes and dams without such a berm protection and this reduces the process of breach forming. Both of these elements are able to resist high flow velocities and have shown in history to prevent large breaches.

For input in the model BRES, Visser used three different scenarios.

- Scenario I: The most likely scenario. In this scenario a 10^{-4} storm is used resulting in a maximum water level of NAP + 5.00 m. At this level the breach will occur. The depth of the breach stays limited to NAP + 2.00 m because of the boulder clay layer at NAP + 3.00 m. This input results in a breach width of approximately 380 meter.
- Scenario II: Extreme scenario, two storms. Here the same 10^{-4} storm occurs with its corresponding NAP + 5.00 m water level and the breach till NAP + 2.00 m. After the 10^{-4} storm (1/10,000) a $2.5 * 10^{-4}$ storm (1/4,000) will hit the Afsluitdijk before any restoration measures are executed. This input in the model leads to a width of approximately 1340 meter.
- Scenario III: Very extreme scenario. Again the 10^{-4} storm is used in this scenario. Difference is the assumption that the whole Afsluitdijk consists of sand instead of clay and thus increasing the depth of the breach to the level of the outer berm structure at NAP -0.4 m. This input leads to a breach width of 1300 meter. As the depth is much larger than in previous situations the discharge on the IJsselmeer will be much more.

Conclusion of the report was that a breach of the Afsluitdijk leads to no big problems as the boulder clay layer provides much resilience against growth of the breach. Only scenario III showed a significant breach. However, this scenario was not seen as realistic as the assumption was made no clay layer was present in the dam.

The report of Deltares [20] also gave an expression of the expected dimensions of the breach. It shows most resemblance to Visser's Scenario I. The width of the possible breach in the levee is expected at 400 meter. However, as was already described in section 3.2, Deltares expects a breach will occur much sooner than the 10^{-4} storm. More on this will follow in the last part of this chapter (3.8). The report of CPB did not mention the expected depth of the breach and no mention about whether or not the boulder clay layer will effect the breach forming was made.

3.4.2. SHORTCOMINGS AND RECOMMENDATIONS

Visser's report [28] has a few recommendations and shortcomings for using the model BRES on the Afsluitdijk. First, the (negative) effect of waves and the (positive) effect of revetments at the toe were not included in the model. It was assumed these effects will cancel each other out. Second, and probably more important, the model that was used is designed to simulate breach growth in sand dikes. For taking the boulder clay in the Afsluitdijk into account average flow velocities were calculated and it was assumed boulder clay will only erode if this flow velocity will become larger than 5 m/s (which did not happen in any of the scenarios).

This assumption about the boulder clay raises some questions. According to Table 2.4 in the scour manual [30], rough estimates about the critical depth-averaged velocity for cohesive sediments are in the order of 0.4 m/s for loamy sand to 1.5 m/s for heavy loamy clay up to 1.9 m/s for hard clay. This questions the assumption that boulder clay will only erode at flow velocities higher than 5 m/s.

Because of this it is debatable if scenarios III is a very extreme unrealistic scenario. This does not mean the boulder clay layer can be neglected. Still much research needs to be done to investigate the importance of the boulder clay layer in the Afsluitdijk. Even if the resistance against scouring is overestimated by Visser, the boulder clay layer may provide (much) retardancy to the development of the breach.

One final remark on the effect of the boulder clay layer. Visser states that, taking the current knowledge into account, it can still be said the boulder clay layer will have a significant retardant effect on development of the breach (personal communication, Augusts 20th, 2014). New research has shown that the strength of boulder clay can differ considerably and that the strength of the boulder clay used in the Afsluitdijk is remarkably high. He made reference to a study in 2007 about the strengths of boulder clay [13]. This study showed that boulder clay is not weaker than normal clay and that the erosion of both materials is more or less the same. Conclusion of the study is that boulder clay influences the strength of a dike/dam significantly and must not be neglected.

3.4.3. CONCLUSION

Because of this ongoing debate and lack of further (quantitative) research of the effect of the boulder clay layer on the development of the breach, both dimensions described in scenario I and in scenario III are used simultaneously in this thesis (see section 3.8). Also a partly eroded boulder clay layer is assumed (scenario 4) [2].

[2] This is also assumed by Deltares [20]. This report used the assumption of a partly eroded boulder clay layer till NAP + 0 m. (Scenario I uses no erosion = NAP + 2 m and Scenario III uses complete erosion = NAP -0.4 m)

3.5. EVENT DEPENDENT DIMENSIONS

The presented dimensions of the breaches in the levee (see section 3.4) are the ultimate values for an 1/10,000 per year storm. These expected values are the (depth-averaged) width and the average depth of the breach when such a storm hits the Afsluitdijk. As was found in section 3.2, the Afsluitdijk is likely to fail at much less extreme storms (in the order of 1/140 to 1/250 per year). If such a storm occurs it will not cause the same damage as the effect of an 1/10,000 per year storm. On the other hand, even more extreme storms (e.g. 1/100,000) are also able to hit the Afsluitdijk. If such a storm occurs the dimensions described in previous sections will be surpassed. This section is used to investigate the expected relation between the width of a breach and the magnitude of a storm.

3.5.1. PROPORTIONALITY OF THE USED EROSION FORMULA

The dimensions of the breach are strongly dependent on the event that occurs (i.e. the water level standing on the outside of the breach). To find the less and more extreme dimensions of the breach at the 1/10,000 storm a closer look is taken at the model used to find the breach dimensions at the once in a 10,000 year storm. This is explained in detail in appendix F.

Visser used 5 different stages in his BRES-model [29] to find the breach dimensions. It is expected the effect of the outside water level on the final width of the breach is dominated by stage IV in the breach model (see appendix F.2 for the definition of the 5 stages). To find the relation between outside water level and final breach width the proportionality of the sediment transport formula in stage IV is needed. In stage IV the increase of the breach will mainly be in increasing the width of the breach (not the depth) and therefore it is dominated by erosion at the slide-slopes. Visser validated a selection of sediment transport formulae and found that the Van Rijn (1984) transport formula can best be used during stage IV (also in [29]).

By looking at the proportionality of the Van Rijn transport formula the following proportionality can be found. This is done in appendix F.3. It gives the proportionality between the width of a breach in the levee and the water level in the Waddenzee after a storm at which the Afsluitdijk fails.

$$B(H_w) = a + b * (H_w)^{0.2} \tag{3.1}$$

The values of a and b depend on the scenario and are found by using the following values for B and H_w:

Storm	H_w	Width scenario 2	Width scenario 3	Width scenario 4
1/140	NAP + 4.02m	0 m	0 m	-
1/250	NAP + 4.18m	-	-	0 m
1/10,000	NAP + 5.02m	380 m	1300 m	300 m

Table 3.2: Values used to solve equation (3.1)

A description of the different scenarios in explained in section 3.8. Scenario 1 is not included in the table as the breach is expected to be at the sluices in this scenario. Therefore it will not be described with this relation. At scenario 2 and 3 the Afsluitdijk fails at an 1/140 storm and at scenario 4 at an 1/250 storm. Therefore the data points used for fitting equation (3.1) are a width of 0 and a Waddenzee water level corresponding to the most frequent storm at which the Afsluitdijk fails [3].

3.5.2. CONCLUSION RELATION WATER LEVEL AND BREACH WIDTH

Solving equation (3.1) for all scenarios takes place in the Matlab model in chapter 4. To give an idea about the relation equation (3.1) is plotted in figure 3.1 for the value of scenario 2. To be able to fit the equation through the two known points (from table 3.2) high values of a and b are needed. This results in a relation that does not differ much from a linear relation ($(H_w)^1$ in equation (3.1)). This means the water level - breach width relation is not very sensitive to assumptions made in section 3.5. The values from table 3.2 are most

[3] The - marks in table 3.2 are used to indicate that those points are not relevant for fitting equation (3.1)

important for defining the ultimate width for different storms. If more accurate values are needed it is advised to improve the reliability of these points or to find additional points through which the curve can be fitted.

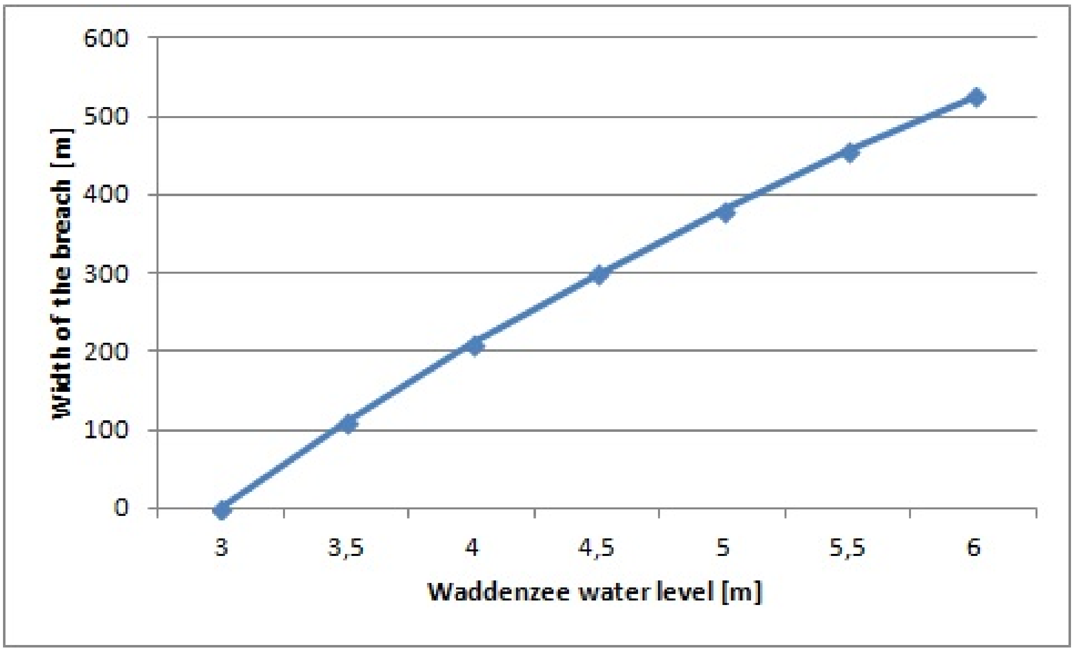


Figure 3.1: Relation ultimate width of the breach to the Waddenzee water level for scenario 2

3.6. TIMING AND DEVELOPMENT OF THE BREACH

In this section the development of a breach in the sluices and in the levee are discussed. Also the timing of the start of the breach is handled. Timing of the breach during a (winter) season is treated as well as this is important in case of a second storm occurs after the storm that causes the breach.

3.6.1. TIMING AND DEVELOPMENT OF A BREACH IN THE SLUICES

As was assumed in section 3.3, a breach in the hydraulic structures will act spontaneously and will immediately develop into its maximum width as the doors and gates will blow out. As the probability of failure of the doors and gates is assumed to be equal for each of the doors/gates, it is assumed either no breach is formed or all doors/gates will blow out together.

This means it is assumed there will be no breach if the water level on the outside does not exceed the limit state and a full breach of 336 meter (see section 3.3) will occur once this limit state is exceeded. Of course this is not conform reality. Probably, failure of a sluice gate will have effect on its neighbouring gates. Also, the structures are located at two different places (Stevin and Lorentz). Because of simplification this effect is not taken into account in this thesis.

3.6.2. TIMING AND DEVELOPMENT OF A BREACH IN THE LEVEE

The expected timing and the assumed development of a breach in the levee is discussed below.

TIMING

The timing of the breach in the Afsluitdijk is of great importance to the amount of inflow of water. One can imagine that a breach during the final hours of the storm has a much lower impact than a breach that is already present at the start of the storm.

It is assumed the Afsluitdijk will start to breach at the maximum occurring water level. Thus, when the tide and wind set up are together at its maximum point (see also appendix E). This is at the peak of the storm. (The effect of this assumption is discussed in appendix M.)

DEVELOPMENT

Not only did Visser gave the maximum expected breach dimensions in [28], he also states in his thesis [29] that at later stages in the breaching process the breach grows mainly laterally and the width of the breach grows linearly in time during a constant outside water level.

The Deltares report simplifies this assumption and assumes a breach in the Afsluitdijk will grow linearly to its ultimate dimensions (see section 3.4) regardless of the lowering of the outside water level after the peak of the storm. For this growth Deltares expects to need 12 hours of high water due to the storm. High water for at least 12 hours during one storm is seen as a reasonable assumption (see appendix E). Therefore, linear development of the breach width will also be used in this thesis.

3.6.3. TIMING DURING A WINTER PERIOD

Timing of the breach can also be seen in another perspective which can also be of importance and is not yet addressed before. This is the timing of the storm itself that results in a breach. As repairs of a possible breach can probably only be executed during a summer period (see section 6.2.1 for the explanation of this assumption), a breach in the Afsluitdijk can influence the IJsselmeer for months.

Grevers and Zwaneveld [23] assume the extreme storm that will cause a breach in the Afsluitdijk can only occur during a winter period (October to March). This seems a reasonable assumption when looking at historical records. Grevers and Zwaneveld also assume that the winter period last for half a year (=26 weeks). The probability of occurrence of the storm is assumed to be equal in every week. The timing is of importance as a breach in the first weeks gives a higher probability of a second storm than a breach in the last weeks of the winter period. These assumption are also used in this thesis (see section 5.4.4).

The models Hydra-M and Hydra-Zoet also use the statistics of a winter season only [31]. In these models the contribution of the water levels during summer half year on the extreme water level statistics is negligible.

These two assumptions (an extreme storm will occur only during the 6 winter months and that the probability of occurrence during that period is equal on every day/week) about the timing of extreme storm seem reasonable assumptions. Figure 3.2 shows the timing of severe storms on the Dutch coast in the past millennium. As can be seen in the figure one of the most severe storms 1570 (November 1th, more than 20,000 deaths [32]), 1906 (March 12th), and 1953 (January 31th/February 1st) are spread over the 6 winter months. Of course it is hard to draw hard conclusions on this sample, but it is an indication that the assumption used by Grevers and Zwaneveld and in the model Hydra-Zoet seems reasonable.

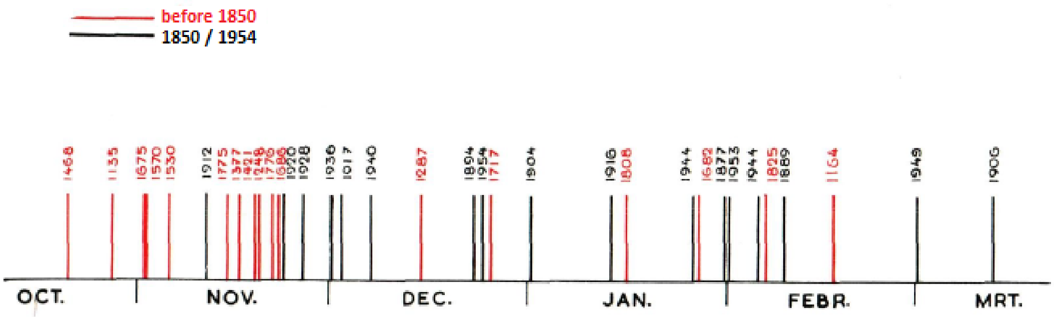


Figure 3.2: Previous occurences of severe storms in the history of the Netherlands (source [22])

3.7. DURATION OF THE STORM

For the development and the consequences of the duration of high water, the duration of the storm is of much importance. The longer the storm holds on, the longer the time water will flow over/through the dam, and thus the larger the dimensions of the breach will be after the storm. The final dimensions of the breach will mostly be formed due to the extreme storm and in less intend due to the 'normal' conditions in the days/weeks after the storm. Therefore the duration of the storm highly effects the final consequences.

For getting to the results of the three scenarios described in section 3.4, Visser [28] also modeled the duration of the high water. Visser assumed a design storm at which the Afsluitdijk will fail, will have the same characteristics as the storm that occurred in 1953. For each of the scenarios the same wind direction, tidal influence, etc. was used and these aspects were increased to the extreme water level of its probability corresponding with each scenario. This assumption seems reasonable as these extreme water levels will almost certainly occur in a combination of extremely high wind speeds coming from the North-West and the occurrence of spring tide. The used storm in this thesis will have the same characteristics of the storm in 1953.

Appendix E gives a detailed description of the storm of 1953. Figure 3.3 shows the water levels at different places along the Dutch coast during the storm of 1953. If the effects of tide are subtracted from the water levels, the storm effect can be seen (the bottom figure in figure 3.3). It can be noticed that for the different locations the water levels at the peak of the storm differs a lot, but in the beginning and at the last hours of the storm the water levels do not differ much. This storm is scaled up in this thesis to model different magnitudes of storms. The development to the peak is assumed to be roughly 20 hours and after the peak, the storm will decrease again during a period of 30 hours. For this rise and decrease a linear relation is assumed. The effect of this simplification of the description of the water level during the storm is discussed in appendix M. In this appendix, it is shown that the used simplifications are a conservative approach.

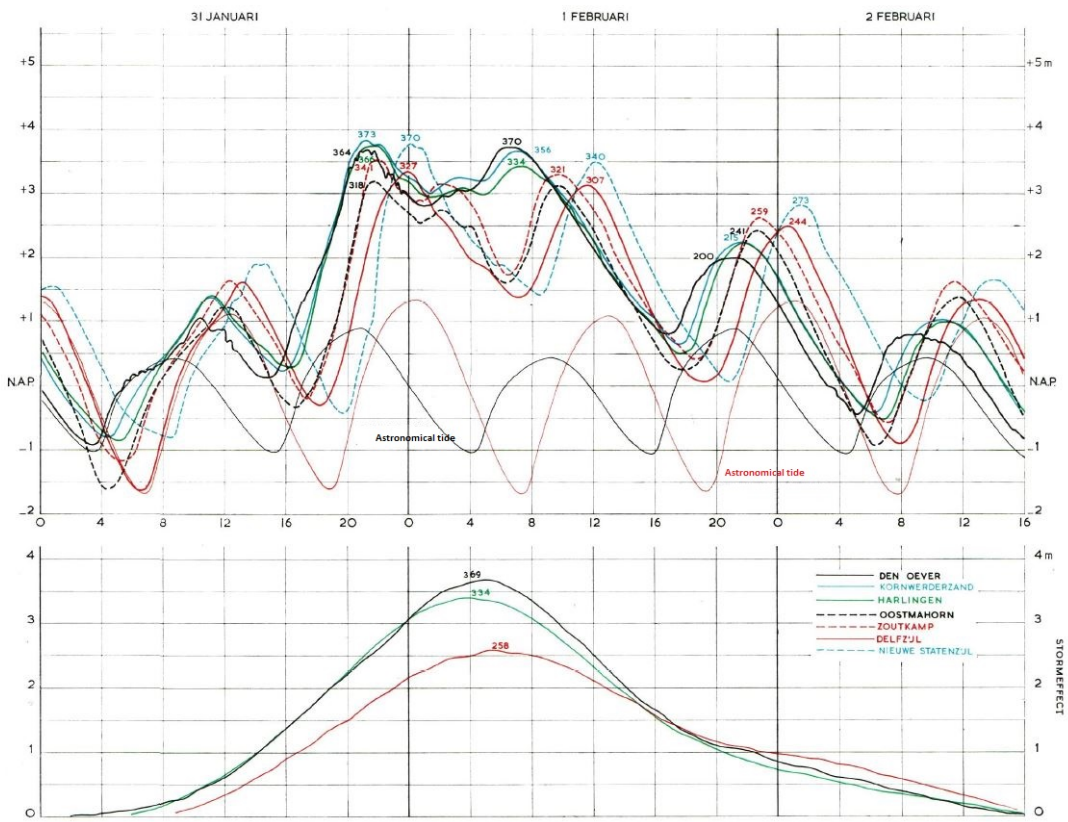


Figure 3.3: Water levels during the storm of 1953, contribution of tide and storm effect (source [22])

3.8. CONCLUSION: RESULTS PRESENTED IN 4 SCENARIOS

The studies analysed in this chapter show that the forming and development of a breach in the Afsluitdijk is not easily characterised. Because of uncertainties in the characteristics of a possible breach the results of the studies in this chapter are presented in 4 scenarios. These scenarios are used as a framework for answering the questions in the next chapters.

Three main uncertainties in the characteristics were found in this chapter. First, the current probability of failure of the breach is still a debate. Studies show that the probability of failure lies in the range of 1/140 and 1/250 per year. This is a big difference compared to the standard of the Afsluitdijk. Second, it is uncertain if the breach will occur in the levee or at the hydraulic structures (i.e. the sluices). To deal with this disagreement a division is made between two scenarios (Scenario 1 and scenario 2). Third, if the breach happens at the levee of the Afsluitdijk (as scenario 2 assumes), it is uncertain how the boulder clay layer will react. This uncertainty has a big effect on the dimensions of the breach as was shown in section 3.4. Therefore scenario 2 is split into two scenarios (scenario 2 and scenario 3). The first assumes the boulder clay does not erode. The second assumes it does. Of course, failure at the sluices does not mean no failure at the levee can occur and vice versa. Therefore also scenario 4 is used in this research. It combines scenario 1 with scenario 2 and 3.

3.8.1. THREE SCENARIOS

The first three scenarios differ in characteristics that have an impact on the influence of the breach on the IJsselmeer [4]. Together with a combination of these three scenarios (= scenario 4) research question 2 (*What possible scenarios of breaching of the Afsluitdijk can be expected?*) is answered.

- **Scenario 1** This scenario follows the assumptions used in the report of Deltares [20].
- **Scenario 2** This scenario uses the probability of failure of Veraart [5] [21] and the dimensions from scenario I from Visser [28] (clay stays intact).
- **Scenario 3** This scenario uses the probability of failure of Veraart [21] and uses scenario III from Visser [28] (clay gets eroded).

Table 3.3 gives a summary of the three scenarios and its corresponding characteristics defined in this chapter.

Characteristic	Scenario 1	Scenario 2	Scenario 3
Probability of failure at water level	1 / 250 per year	1 / 140 per year	1 / 140 per year
Location of failure:	Sluices	Levee (see Note)	Levee (see Note)
Width of the breach (at an 1/10,000 storm)	336 meter	380 meter	1300 meter
Depth of the breach	NAP -4.40 meter	NAP + 2.00 meter	NAP -0.40 meter
Timing of the breach	During the peak of the storm		
Duration storm	20 hours before the peak and 30 hours after peak		
Development breach (from 0 to full breach)	Immediately	Linear during 12 hours	

Table 3.3: Summary of scenarios 1, 2, and 3

Note: A note about the sluices can be made for scenario 2 and 3. Veraart and Visser did only investigate a possible breach in the levee and the sluices were left out of their research. For these scenarios it is assumed the sluices will fail at much lower probabilities than the levee and are therefore simulated as non-failable structures.

3.8.2. A 4TH SCENARIO

Up to now, failure of the sluices or levee is seen as two different phenomena of which only one can occur. This is off course is not conform reality. A breach can happen at both the sluices and the levee during the

[4] These three scenarios are not the same three as defined by Visser in section 3.4!

[5] The probability used here is found in a draft version of his report. In the final version this probability has changed. See also the footnote at page 20

same storm. Therefore a combination between scenario 1 and the other two scenarios needs also to be investigated. To limit the total number of scenarios a combination of scenario 2 and scenario 3 is found first before it is combined with scenario 1. This leads to scenario 4.

For the combination of scenario 2 and 3 (one can say this is an in between variant of the two) the Deltares report [20] is used. The report assumes a width of a breach in the levee of the same order as a breach at the sluices. Deltares also assumes the boulder clay in the levee will erode partly up to a depth equal to NAP.

Finally the probability of failure of both the sluices and the levee must be defined. For this the value that was used in the CPB report and provided by Rijkswaterstaat is used. The assumptions are summarized in table 3.4.

Characteristic	Scenario 4
Probability of failure at water level	1 / 250 per year
Location of failure:	Sluices **and** levee
Width of the breach (at an 1/10,000 storm)	336 meter at sluices and 300 meter at levee
Depth of the breach	NAP -4.40 meter for sluices and NAP + 0 meter for the levee
Timing of the breach	During the peak of the storm
Duration storm	30 more hours after breach
Development breach (from 0 to full breach)	Immediately at the sluices and Linear for 12 hours at the levee

Table 3.4: Summary of scenario 4

II

PART II: SIMULATION AND RESULTS

4

Chapter 4: Influence on the IJsselmeer

This chapter is dedicated to investigate the effects of a breach in the Afsluitdijk on the IJsselmeer. By making a simple model discharges through the breach are calculated for each scenario (see chapter 3) and for each storm. The model then calculates the increase in average water level. The first section (4.1) explains this procedure. In the other sections (4.2, 4.3, and 4.4) different elements of the model are described. The later sections gives the results of the model (4.5), a discussion about the results, and the final conclusion (4.6).

The research question to be answered in this chapter is: *What is the influence of a breach in the Afsluitdijk on the IJsselmeer and what kind of water level increases can be expected?*

This chapter will give the water level increase in the IJsselmeer at the end of the storm. At that moment the wind speeds will be normal (i.e. not extreme) again. This might not be the most governing situation for the safety of the IJsselmeer dikes. Section 5.1 treats other possible governing situations. This chapter will give the maximum increases in the average IJsselmeer water level and thus the maximum effect on the IJsselmeer (not on the IJsselmeer dikes).

4.1. Construction of the model

The increase in average IJsselmeer water level is found by using a Matlab model which will be explained below in 4.1.1. In this model two distributions of water levels are needed. (1) The current water level distribution of the IJsselmeer water level and (2) the current distribution of the Waddenzee water level. The current IJsselmeer water level distributions is gathered from the software model Hydra-M. This will be explained in section 4.2.

The model Hydra-K is used to gather the water level distribution of the Waddenzee. This process is described in section 4.3. The Matlab model assumes no correlation between the two distributions. This is argued in section 4.1.2 and section 4.1.3.

Also needed as input for the model are the possible breach scenarios. The derivation of these scenarios are already explained in chapter 3. In section 4.4 the implementation of these scenarios in the matlab script is explained. Section 4.4 is also used to describe the discharge formulas through the breaches.

To conclude, three aspects are needed as input for the model: the water level distribution of the IJsselmeer, the water level distribution of the Waddenzee, and the possible breach scenarios.

4.1.1. The matlab model

For each scenario a different Matlab script is made. In each run (so for each scenario) the model will be run multiple times, as a Monte Carlo approach is used. The explanation of the model is done for the first scenario only. The other three scenarios are similar to scenario 1, only the input of the scenario and the simulation of the discharge is different. Appendix G shows one of the Matlab scripts. In this script also the used characteristics of the IJsselmeer and other variables are presented (e.g. surface area of the IJsselmeer is assumed to be 1100 m^2, duration of the storm after the peak is 30 hours, etc.).

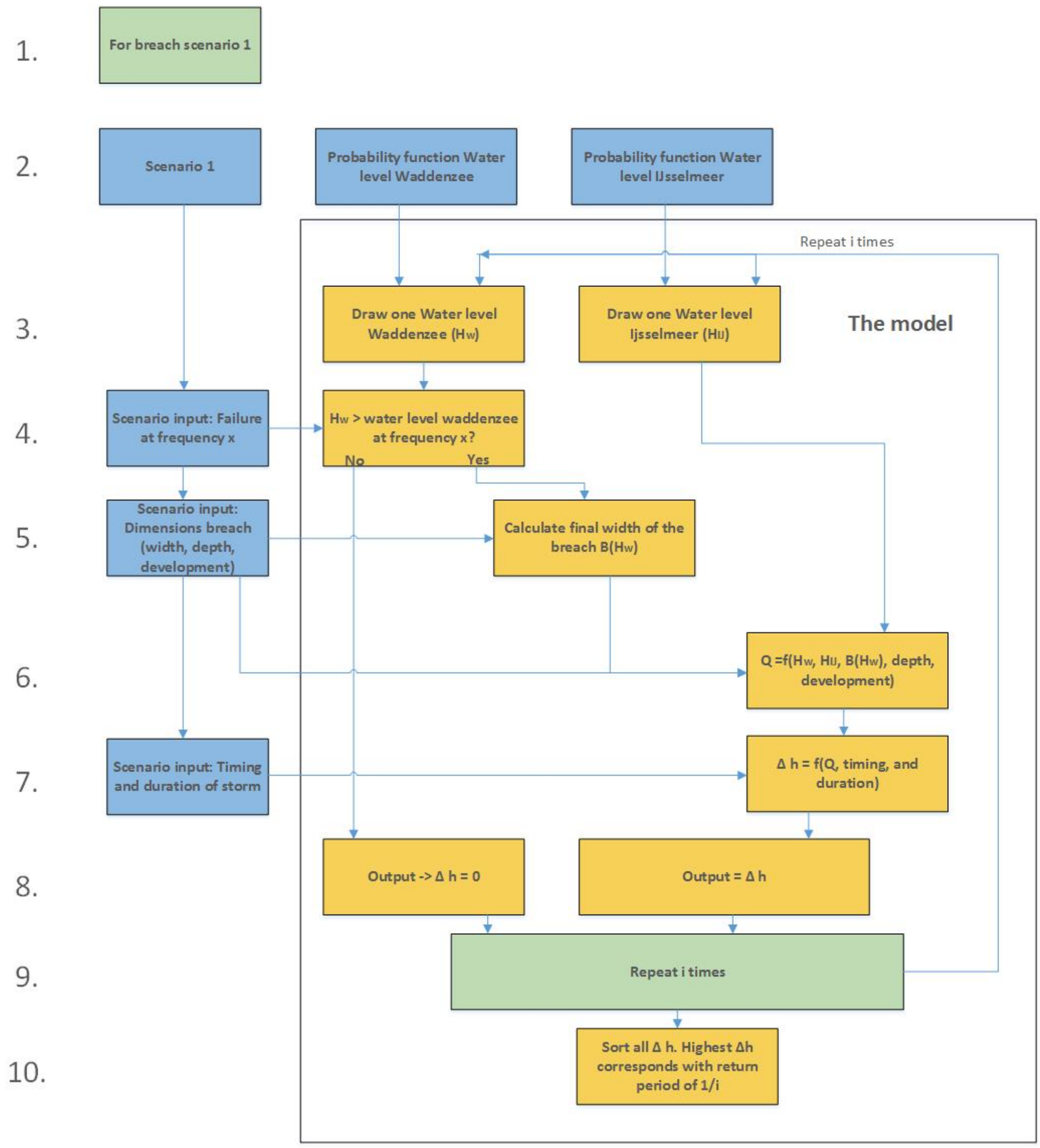


Figure 4.1: Set up of the matlab model

In figure 4.1 a schematic roadmap is given to explain the steps leading to the final result. The three squares in the second row give the three main input aspects that are used in the model. Input 'scenario 1' consists of three other input characteristics: Failure frequency, dimensions of the breach, and timing of the breach. These input factors are used along the way in the script and are visualised as three boxes in the first column in figure 4.1.

First step of the actual model is the third row in the figure. Here the Monte Carlo simulation starts with drawing one Waddenzee water level and one IJsselmeer water level out of the probability distribution from row 2 (see section 4.2 and section 4.3 for determining the probability distributions).

One step further (= row 4) the drawn water level of the Waddenzee is compared with the failure frequency of the Afsluitdijk. If the drawn water level Waddenzee is lower than the water level occurring at the failure frequency the Afsluitdijk will not breach. This means that for the output of this single run, no increase in water level occurs. For scenario 1 this means that if the drawn water level is lower than the water level occurring once in 250 years, the Afsluitdijk will fulfil its function and the Waddenzee water level has no influence on the water statistics of the IJsselmeer. If the Waddenzee water level is higher than the water level at the given frequency the Afsluitdijk will fail. This means water will flow in. For this step the assumption is made that the probability that the Afsluitdijk fails at an 1/250 storm is 1 and that the probability that the Afsluitdijk fails at an 1/249 storm is 0. Of course this sudden change will never happen in reality, the difference between an 1/250 storm and an 1/249 storm is too little to be certain it is the final step to exceed the load capacity (see appendix H for more information on probabilities). For the levee, this assumption does not cause a big error. Failure at the smallest storm causes only a small breach. For the sluices the assumption maybe cause big errors, as it is assumed all the sluices are blown out all at once. This problem is solved by the Monte Carlo approach. The Matlab model runs the whole script at least 200,000 times (8,000,000 runs is used for the results in section 4.5). Therefore many storms around the 1/250 probability will be simulated. The more storms around 1/250 the less effect this error has on the final results (the error gets averaged out).

In row 5 the final width of the breach is calculated. This is done as is explained in section 3.5. Width of the breach is 0 meters at the storm at which the Afsluitdijk fails and grows as the storms get more extreme. For the once in 10,000 year storm the dimensions of section 3.4 are used.

How much water will flow through the breach is calculated in row 6 (see section 4.4.3). In this row the discharge through the gap is calculated using an overflow (for a breach in the levee) or underflow (for a breach in the sluices) formula. Input for this step are both water levels (Waddenzee and IJsselmeer) and the dimensions of the breach corresponding with the scenario. The development of the levee breach during a storm is assumed to develop linearly to its maximum after 12 hours (see section 3.6.2).

This discharge leads to a rise in water level in the IJsselmeer (row 7). The rise in IJsselmeer water level is stored in the memory and the whole process is repeated i times. This leads to i increases of IJsselmeer water level, of which the most values are zero. Most of the increases in water level have the value zero as the breach will 'only' occur once in every 250 years (or once in 140 years for scenario 2 and 3).

Now, i data points are gathered. Each data point resembles the average water level increase through a breach in the Afsluitdijk for one year. The original Waddenzee water level is derived from a distribution that gives the probability per year. For this an Annual Maxima Series (AMS) is used (see appendix H.5.1 for its definition). The problem with using AMS is that multiple (independent) high water levels can occur during one year. In such a case, only the highest one gets recorded and the other(s) do(es) not. This results in a misrepresentation of the real phenomenon. Luckily, this is not a big problem in this thesis as only the higher return periods (>140 and >250) are of interest. The probability of two independent water levels in one year with a frequency of 1/140 per year is too low to disapprove the use of the AMS method.

If the all the runs are sorted (row 10), an exceedance probability curve can be drawn of the increase in average IJsselmeer water level. This means the highest value of Δh corresponds with a probability of 1/i per year. The second highest value corresponds with a probability of exceedance of 2/i per year, etc. The closer the water level increase to the highest water level increase, the less reliable its corresponding probability of exceedance. To solve this, the number of runs used for the calculation of the highest desirable return period is factor of 20-100 more.

To see specific numbers also a vector is computed which gives values for certain frequencies. The results are presented in section 4.5.

4.1.2. CORRELATION WADDENZEE - IJSSELMEER AVERAGE WATER LEVEL

The first step in the model (row 3 in figure 4.1) assumes that the probability function of the Waddenzee water level is independent of the probability function of the average water levels (Dutch: meerpeil) in the IJsselmeer.

This assumption is thought to be safe as the wind direction and wind speed hardly have any effect on the *average* water level in the IJsselmeer. (Note: not on the *local* water level in the IJsselmeer! See section 4.1.3 for their correlation.)

The water level in the Waddenzee is influenced by the water level in the North sea, the tide, the wind speed and the wind direction. The water level in the IJsselmeer is influenced by the input from the river IJssel, the discharge of the water boards around the IJsselmeer, interaction between the Markermeer and the IJsselmeer, and the discharge of the IJsselmeer on the Waddenzee through the Afsluitdijk.

The two most important factors influencing the average IJsselmeer water level are the input from the river IJssel and the output through the sluices in the Afsluitdijk. In 2009 Rijkswaterstaat did research on the influencing factors of the average IJsselmeer water level and the trends for the future [33].

Figure 4.2 shows the discharge through the river IJssel near the town Olst during October/November 1998. Also the average water level in the IJsselmeer during the same period is plotted in the figure. The figure clearly shows both lines are following the same fluctuation. This indicates that the average water level is highly influenced by the discharge of the IJssel.

An average IJsselmeer water level of NAP + 0.50 m is very exceptional as is a discharge of 1600 m^3/s through the river IJssel. When looking at less extreme situations the correlation is not as clear as in figure 4.2. Still, Rijkswaterstaat showed that a clear relation can be found between the discharge through the IJssel and the average water level in the IJsselmeer. On average, about 90% of the total IJsselmeer input comes from this river.

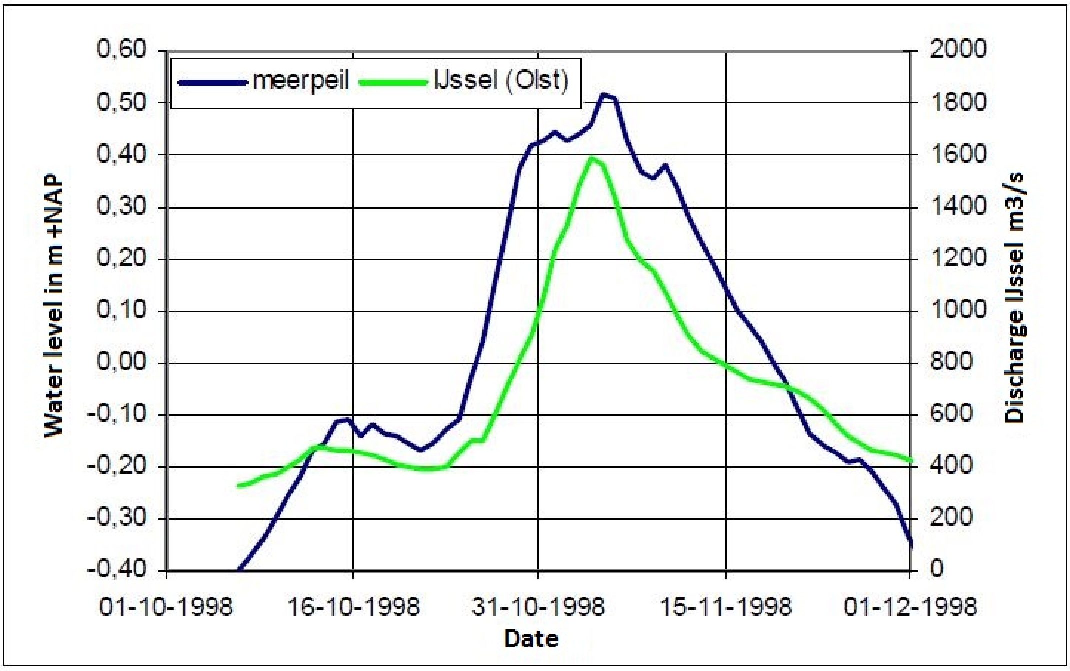


Figure 4.2: Discharge river IJssel and average water level in the IJsselmeer during October/November 1998. Source: [33]

In the same report also the relation between the discharge through the Afsluitdijk and the discharge in the river Rijn was investigated. The results are given in figure 4.3. A timespan of 20 years was used for this relation, making it very plausible the discharge through the Afsluitdijk is dominated by input through the river Rijn, thus the river IJssel [1], thus the water level in the IJsselmeer.

The amount of discharge through the Afsluitdijk depends also on the water level in the Waddenzee. Because the Waddenzee is effected by tide, the water level in the Waddenzee experiences much more frequent fluctuations than the river IJssel. As the spread in discharge of the IJssel is also high, the discharge through the IJssel is seen as the most dominant factor for the discharge through the Afsluitdijk.

[1]The report also shows an almost perfect one on one relation between discharge through the river Rijn and the river IJssel

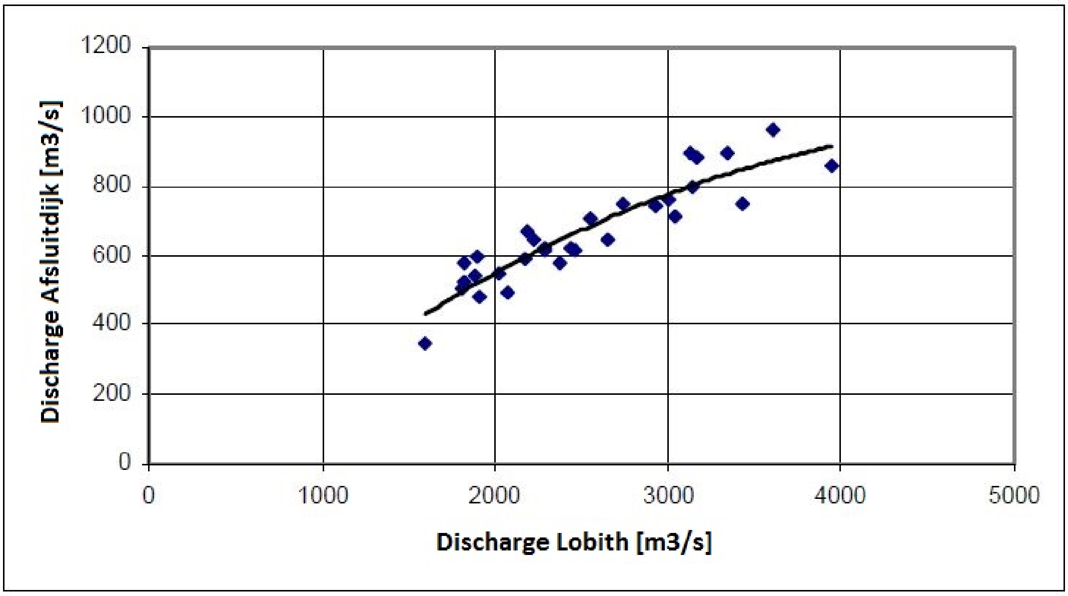


Figure 4.3: Relation discharge through Afsluitdijk and discharge river Rijn at Lobith during 1976-2008. Source: [33]

For both input and output in the water balance of the IJsselmeer the discharge in the river IJssel is seen as the dominant factor. As discharge through the Afsluitdijk hardly influences the Waddenzee water level (the North sea is a far more dominant boundary) the correlation between the Waddenzee water level and the IJsselmeer average water level can be seen as negligible.

4.1.3. CORRELATION AVERAGE AND LOCAL IJSSELMEER WATER LEVELS

The *local* water level in the IJsselmeer is influenced by the average water level and by the obliquity in the IJsselmeer water level due to the effect of wind. Therefore the (average) water level differs from the local water level. As the IJsselmeer is relatively a shallow water basin, the effect of wind on the obliquity is significant.

WATER SET DOWN

The local water level depends on the average water level and a gradient in the water level because of the influence of wind. It can be expressed in formula form as follows:

$$H_{IJsselmeer,local} = \bar{H}_{IJsselmeer} \pm \Delta H \tag{4.1}$$

In this equation $H_{IJsselmeer,local}$ is the local water level (at in this case the location of the Afsluitdijk), $\bar{H}_{IJsselmeer}$ is the average water level in the IJsselmeer, and ΔH is water set up (in the case $\pm$ is a +) or water set down (in the case $\pm$ is a -) caused by the wind.

Wind set up or set down can be calculated by simplifying the balance between shear force caused by wind on the water body and the counter pressure caused by the weight of the water body [34]. This results in:

$$\Delta H = c * \frac{u^2}{g * h} * L \tag{4.2}$$

In this formula, c is a constant for which $3.4 * 10^{-6}$ can be used [34], u is the wind speed, g the gravitational constant, h the depth of the water, and L the fetch length. In appendix I a validation of the use of this formula is done for the IJsselmeer system. It is found that equation (4.2) slightly overestimates measured data.

Because the depth of the IJsselmeer is relatively low (≈ 5 meter) and the fetch length can be long (up to 50 km), water set up or set down can be in the order of meters.

In 1990 Rijkswaterstaat did research to the effect of wind set up and set down near the sluices of the Afsluitdijk [35]. For different wind directions and wind speeds, the water set up and set down was calculated. The results of wind directions 330 and 0 degrees relative to the North (so Northwest and North), gave wind set down in the order of meters. A wind speed of 34 m/s let to -283 cm wind set down (in respect to NAP water level) near the sluices of Den Oever. Wind directions 270 and 300 (wind from the West) gave even more extreme results. At the wind speed of 34 m/s all the water will be blown away leading to complete dry out of the location.

Off course these wind speeds are very unlikely to happen. However, a breach of the Afsluitdijk will occur during an extreme storm. And during such an extreme storm, extreme wind speeds will occur at the Afsluitdijk. The correlation factor between failure of the Afsluitdijk and extreme wind speeds is very high (close to 1). Therefore a big error is made when assuming the water local water level at the inside of the Afsluitdijk is the same as the average water level in the IJsselmeer (thus assuming $\Delta H \approx 0$ in equation (4.1)). To model the inflow through the breach in a correct way this set down must be included.

COMPLEXITY AND SENSITIVITY OF THE CORRELATION

Taking the relation between the three water levels (Waddenzee, Average IJsselmeer, and inside of the Afsluitdijk) into account makes the model a lot more complex. In 2013 a study was done by Deltares and HKV Lijn in Water to define the hydraulic boundary conditions for the design of the new Afsluitdijk [10]. This study resulted into statistics for the head difference over the Afsluitdijk. The study gives an exceedance frequency curve for the water difference between the outside and the inside of the Afsluitdijk. Unfortunately, the study does not give any relation between those water differences and the average water level in the IJsselmeer. For this relation wind direction and wind speed must be known during these extreme water level differences.

Even if the likeliness of certain wind directions and wind speeds during the maximum water level differences is known, equation (4.2) gives only a simplified expression of the wind set up/set down (see appendix I). To get accurate results the wind effect on the IJsselmeer must be modeled with a dynamical model. Because of these reasons the correlation between the 3 water levels is a complex phenomenon and is not easily defined.

Before effort is put into solving this complexity it is investigated first whether this reduced local water level has much effect on the total inflow of water during a breach. In appendix J a sensitivity analysis is done to see if this reduction in local water level has much effect. Result of this appendix is that it has a negligible effect of the total average water level increase in the IJsselmeer. This is mainly because in case of a levee breach more than 90 % of the time free overflow will occur instead of submerged overflow [2], meaning that the downstream water level does not influence the amount of discharge though the breach. And in case of a breach at the sluices, it is found that the downstream water level has no effect at all (only free flow).

CONCLUSION CORRELATION

The Matlab-model takes the average water level in the IJsselmeer also as the local water level to determine the head difference between the outside and inside of the Afsluitdijk. As the effect of wind is significant, this is physically incorrect and an error is made. However, this simplification is allowable as the discharge through the Afsluitdijk is not very sensible to differences in the local IJsselmeer water level (see appendix J).

[2] For definitions of free and submerged overflow see section 4.4.1

4.2. CURRENT WATER LEVEL DISTRIBUTION IJSSELMEER

The water level distributions used in the Matlab model (row 2 in figure 4.1) are gathered from the Hydra models. With these models the return period of average water levels in the IJsselmeer and water levels in the Waddenzee near the Afsluitdijk can be found. This is done in section 4.2.1 for the IJsselmeer and in section 4.3.1 for the Waddenzee.

Subsequently, a distribution is fitted through these gathered value. This is done with the help of the program Excel. Values of the Generalized Extreme Value distributions are found which are then used in the Matlab model.

The Hydra models.
The Hydra models are software models belonging to hydraulic boundary conditions as prescribed by the Dutch regulations [16]. With the software primary water defence systems can be assessed for different failure mechanisms. In the assessment statistics of wind, waves and water levels are used.

The Hydra models can also calculated the hydraulic loads at needed frequency levels. If this frequency level is chosen to be the same as the required statutory norm the hydraulic loads are equal to the hydraulic boundary conditions.

For different parts in the Netherlands different Hydra models are needed. In this thesis Hydra M is used for the hydraulic loads in the IJsselmeer area and Hydra K is used for the hydraulic loads near the coast (Waddenzee). Both models operate independently from each other and even operate differently at some points (partition of the wind directions for example).
(Source: [36])

4.2.1. HYRDA M

With Hydra-m the following water levels in the IJsselmeer near the Afsluitdijk were found for each corresponding return period. This data is a combination of all average IJsselmeer water levels, all wind directions and all wind speeds. Appendix K shows the full results of the Hydra-M computation. A short overview of the relevant information is presented in table 4.1).

Return period [years]	Water level [m +NAP]
10	0.379
25	0.446
50	0.500
100	0.565
250	0.655
500	0.727
1000	0.803
1250	0.830
2000	0.885
4000	0.971
10000	1.091

Table 4.1: Return period of water levels in the IJsselmeer

4.2.2. EXCEL

With the help of the solver function in Excel the data was fitted to the Generalized Extreme Value distribution (see appendix H for the explanation of the Generalized Extreme Value Theory). For the water level in the IJsselmeer a Gumbel distribution seems the best fit. A Gumbel distribution takes the following form:

$$F(x;\mu,\sigma) = exp(-exp(-\frac{(x-\mu)}{\sigma})) \tag{4.3}$$

In this equation x is a water level, μ is the location parameter, σ is the scale parameter, and $F(...)$ is the cumulative distribution function expressing the probability a certain water level x or less than x can occur. For example, if x = 0.6 and $F(0.6)$ has the value 0.9, it means the water level is lower than 0.6 meter in 90% of the cases.

By fitting the data from table 4.1 to the Gumbel distribution values for the location parameter (μ) and the scale parameter (σ) are found. For this data set these values are found to be:

$$\mu = 0.11 \text{ and } \sigma = 0.10$$

4.2.3. MATLAB

For the random simulation of a water level in the IJsselmeer equation (4.3) is rewritten to express x (Note: $F(x;\mu,\sigma)$ is replaced by $P(X \leq x)$ which has the same meaning):

$$x = \mu + \sigma * (-ln(-ln(P(X \leq x)))) \tag{4.4}$$

In the Matlab model this equation is used and $P(X \leq x)$ is replaced by the function *rand* which gives a (standard) uniformly distributed random number between 0 and 1. Together with the values for the location and the scale parameter, random water levels of the Waddenzee can be simulated. Figure 4.4 gives the results of a Monte Carlo run in Matlab. If the number of Monte Carlo runs is high enough, this figure can be seen as a representation of the probability density curve of the IJsselmeer water levels.

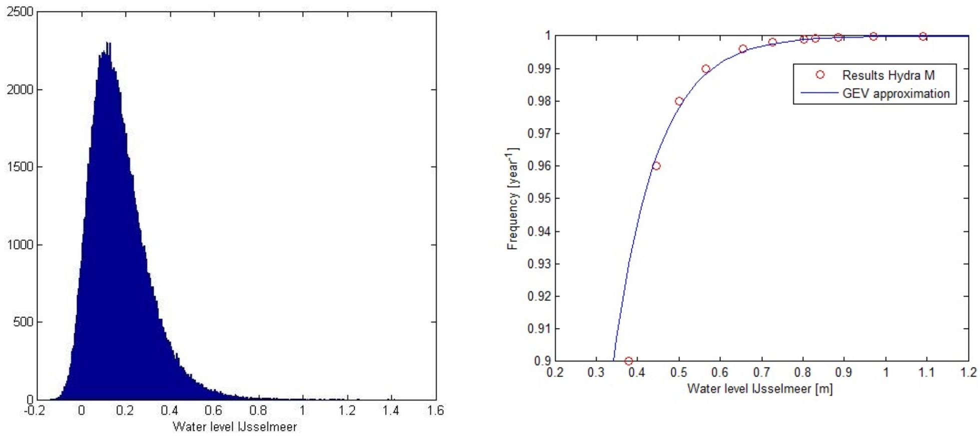


Figure 4.4: Historgram (left) and cdf (right) of the water levels in the IJsselmeer at the location of the Afsluitdijk

4.3. CURRENT WATER LEVEL DISTRIBUTION WADDENZEE

The probability distribution of the water levels in the Waddenzee is gathered in the same way as the probability distribution of the water levels in the IJsselmeer (section 4.2). Instead of Hydra M, Hydra K is used for the return periods and their corresponding water levels. Although these are different models they are much alike. Again, with the help of Excel a Generalized Extreme Value distribution is fitted and this is again implemented in Matlab.

4.3.1. HYRDA K

With Hydra-k the return periods presented in table 4.2 are found for the water levels in the Waddenzee near Den Oever.

Return period [years]	Water level [m +NAP]
1	2.51
3	2.92
10	3.29
30	3.62
100	3.94
300	4.21
1,000	4.52
3,000	4.77
10,000	5.02
30,000	5.24
100,000	5.48

Table 4.2: Return period of water levels in the Waddenzee

4.3.2. EXCEL

The same solver function in Excel was used to fit the data to the Generalized Extreme Value distribution. Where a Gumbel distribution was most suitable for the water level distribution of the IJsselmeer this is not the case for the water level distribution of the Waddenzee. For the Waddenzee a reverse Weibul is used (see again appendix H) as it gives a better fit. A Weibull distribution looks very similar to the Gumbel distribution and takes the following form:

$$F(x;\mu,\sigma,\xi) = exp\left(-(-\frac{(x-\mu)}{\sigma})^{\alpha}\right) \tag{4.5}$$

Again x is a water level, μ is the location parameter, σ is the scale parameter, and $F(...)$ is the cumulative distribution function expressing the probability a certain water level x or less than x can occur. Now also the parameters ξ and α are added. They relate to each other and the shape parameter ξ must be negative in order to validate the Weibul distribution.

$$\xi = -\alpha^{-1} < 0$$

Again a fit is done, this time between the numbers from table 4.2. The location parameter (μ), the scale parameter (σ), and the α value are found. For this data set these values are found to be:

$$\mu = 9.85,\ \sigma = 7.22,\ \text{and}\ \alpha = 22.90$$

4.3.3. MATLAB

For the random simulation of a water level in the IJsselmeer equation (4.5) is rewritten to express x:

$$x = \mu - \sigma * [-ln(P(X \le x))]^{\frac{1}{\alpha}} \tag{4.6}$$

In the Matlab model this equation is used and again $P(X \le x)$ is replaced by the function *rand* which gives a random number between 0 and 1. Together with the values for the location parameter, the scale parameter, and the α-value, random water levels of the Waddenzee can be simulated. Figure 4.5 gives the results of a Monte Carlo run in Matlab. Although this figure looks like it has the same shape as figure 4.4 it is not, because of the difference in distribution (Gumbel and Weibul). This difference is of importance for the high (extreme) values of the distribution. As these values bring the largest consequences the difference in distribution type must be taken into account.

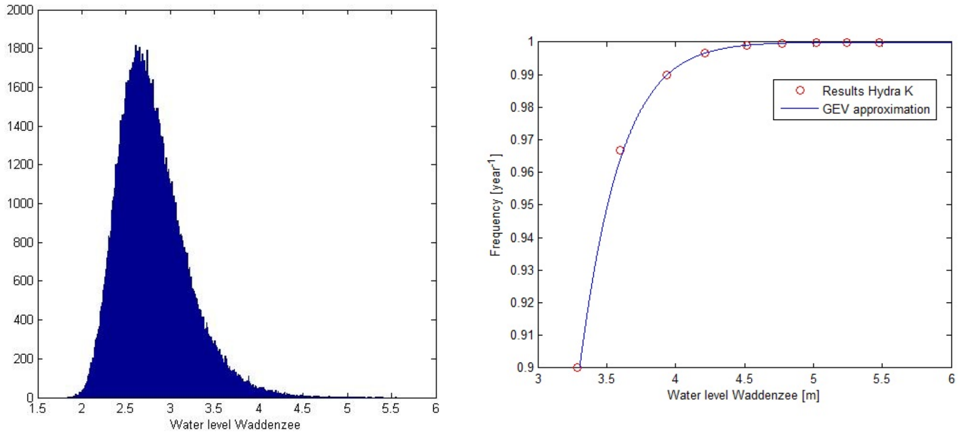


Figure 4.5: Historgram (left) and cdf (right) of the water levels in the Waddenzee at the location of the Afsluitdijk

4.4. POSSIBLE BREACH SCENARIOS

For row 5, 6, and 7 in figure 4.1 the scenario input is used. For each 4 scenarios described in chapter 3 different Matlab scripts are written. The core of each script is the same but in these three steps (rows 5,6, and 7) the 4 scenarios differentiate from each other.

Table 4.3 gives a recap of the results of previous chapter. Probability of failure is included in the Matlab model in row 4. For the differences in width and depth of the breach the variables are simply changed in each scenario script. The differences in the location of failure (breach in the sluices or in the levee) is not simply adjusted. Different formulas are used for the two different breach types. These are explained below (in section 4.4.1 and section 4.4.2).

Characteristic	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Probability of failure	1 / 250 per year	1 / 140 per year	1 / 140 per year	1 / 250 per year
Location of failure:	Sluices	Levee	Levee	Sluices **and** levee
Width of the breach (at an 1/10,000 storm)	336 meter	380 meter	1300 meter	336 / 300 meter
Depth of the breach	NAP -4.40 meter	NAP + 2.00 meter	NAP -0.40 meter	NAP -4.40 m / + 0 m
Development breach (from 0 to full breach)	Immediately	Linear during 12 hours		Immediately / Linear during 12 h
Timing of the breach	During the peak of the storm			
Duration storm	30 more hours after breach			

Table 4.3: The 4 scenarios from section 3.8

For each scenario the width of the breach, depth of the breach, and discharge through the breach are calculated once for each hour. As it is assumed the duration of the storm is 30 hours after the start of the breach (i.e. the peak of the storm) this approach leads to the calculation of 30 widths, depths, and discharges for every storm.

The peak water level of the storm is drawn from the water level distribution. After 30 hours this water level is reduced to NAP + 0 meter which is more or less the average water level (no storm and no tide). It is assumed the development of the water level decreases linearly to this value of 0. Therefore the water levels each hour after the storm are 1/30th lower than the peak water level every hour after the start of the breach. The discharge through the breach, independent of the breach dimensions, decreases as time develops.

4.4.1. BREACH AT THE SLUICES

At a breach at the sluices the water is influenced by the top of the sluice gates (the construction around the gates) (see figure 4.6). Only the gates will 'blow out' during a storm (not the surrounding structure) and the water will flow through the sluices. The flow through the sluices is modeled by equation (4.7).

The crest of the sluices lies at NAP -4.4 m. The top of the construction that holds the gates in place lies at NAP -0.4 m. This means the maximum gate opening is 4 meters. It also means the water will flow under the sluice construction in case of a breach as the breach will occur at water levels higher than NAP -0.4m.

For underflow the discharge formulation can be formed by the equations of Bernoulli and the continuity equation. This results in the following formula for gate discharge for a rectangular cross section:

Figure 4.6: Sluices at the Afsluitdijk during low water (source: [20])

Underflow: $Q = C_d * W * b * \sqrt{2 * g * y_1}$ (4.7)

In this equation Q is the discharge through the gate (m^3/s), W is the width of the gate, b is the (vertical) gate opening, g is the acceleration due to gravity, y_1 is the upstream water level, and C_d is the discharge coefficient (see also figure 4.7 for some of the parameters). C_d depends on different aspects and is treated later on in this section.

For underflow the discharge is influenced by a hydraulic jump caused by the sluice. This hydraulic jump can either be a free hydraulic jump or a submerged hydraulic jump. Figure 4.7 shows the difference between the two hydraulic jumps. Difference is whether or not water stands on the lower side (IJsselmeer side) of the gate. The difference between the two depends on the relation between gate opening (b in the figure), the upstream water level (y_1 in the figure), and downstream water level (y_3 in the figure).

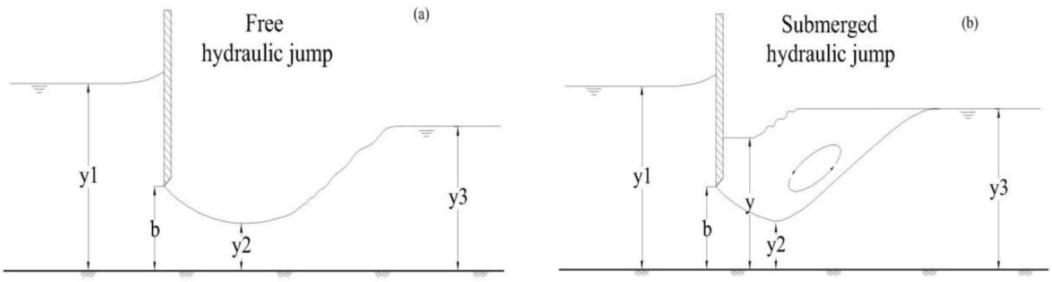


Figure 4.7: Free and Submerged Hydraulic jump (source: [37])

P. Swamee [38] found the following mathematical expression to determine whether the flow is free flow (a free hydraulic jump) or submerged flow (a submerged hydraulic jump):

$$\text{Free flow: } y_1 \geq 0.81 * y_3 * \left(\frac{y_3}{b}\right)^{0.71} \tag{4.8}$$

$$\text{Submerged flow: } y_3 < y_1 < 0.81 * y_3 * \left(\frac{y_3}{b}\right)^{0.71} \tag{4.9}$$

In this equation y_1 is the upstream water level (measured from the crest of the gate), y_3 is the downstream water level (also measured from the crest of the gate), and b is the (vertical) width of the gate (see also figure 4.7).

As the (vertical) width of the gate is relatively big (b = 4 meter), only free flow will occur during a breach. This is important for the determination of the parameter C_d in equation (4.7). The discharge coefficient is different for free and submerged flow. Because only a free hydraulic jump is expected to occur during a breach the determination of C_d is only done for free flow.

Henderson [39] derived the following equation to determine the parameter C_d (in free flow conditions):

$$C_d = \frac{C_c}{\sqrt{1+\eta}} \tag{4.10}$$

In this equation: $\eta = C_c * b/y_1$ and $C_c = y_2/b$ (see figure 4.7 for definition of y_2). Because y_2 is hard to define, $C_c = 0.61$ is used in most researches and is a safe assumption.

4.4.2. BREACH IN THE LEVEE

In row 5 (of figure 4.1) the final width of the breach is calculated. This final width depends on the peak water level in the Waddenzee and the scenario (width at an 1/10,000 storm and development of the breach).

Whether or not the downstream water level influences the discharge is also of importance when looking at over flow. A perfect weir (Dutch: volkomen overlaat) does not 'feel' the downstream water level, an imperfect weir (submerged overflow) (Dutch: onvolkomen overlaat) does. Figure 4.8 shows shows the difference between the two.

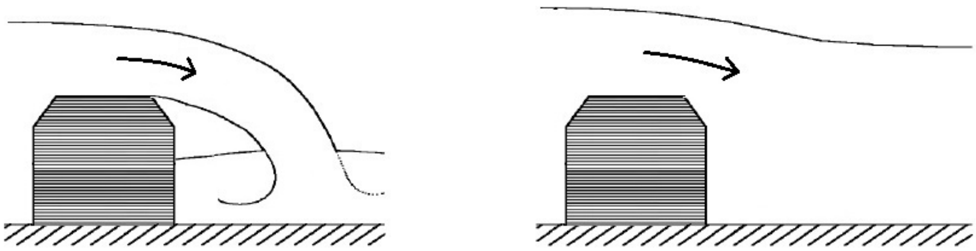


Figure 4.8: Perfect weir (left) and submerged overflow (right) (source: [20])

Figure 4.8 shows a clear difference between the two phenomena. The perfect weir is drawn in such a way the water will spill on the downstream reservoir. At the submerged overflow case, water is 'pushed' in the other reservoir. At the perfect weir the downstream water level does not require to be lower than the crest level of the weir. The transition between perfect and imperfect overflow is at $2/3^{rd}$ of the upstream water level [40]:

$$\text{Perfect weir: } H_{IJsselmeer} < \left(\frac{2}{3}\right) * H_{Waddenzee} \tag{4.11}$$

$$\text{Submerged overflow: } H_{IJsselmeer} > \left(\frac{2}{3}\right) * H_{Waddenzee} \tag{4.12}$$

In these equations, H stands for the water level in relation to the crest level of the weir. As both water levels and the height of the crest level will change over time, it is likely both perfect and submerged overflow will occur. Therefore both corresponding formulas are programmed in the Matlab model.

$$\text{Perfect weir: } Q = m * W * \frac{2}{3} * \sqrt{\frac{2}{3} * (g * H^3_{Waddenzee})} \tag{4.13}$$

$$\text{Submerged overflow: } Q = m * W * (H_{IJsselmeer} - H_{Crestlevel}) * \sqrt{2 * g * (H_{Waddenzee} - H_{IJsselmeer})} \tag{4.14}$$

In this equation Q is the discharge (in m^3/s), m is the flow coefficient (≈ 0.88), W is the width of the gap, and H is the water level at the location of its index.

The depth and the width of the breach develop over time (in contrast to a breach at the sluices). Therefore, for each hour after the peak a value for the width and the depth are computed in the script. As was argued in chapter 3, the development of the breach in the levee can be assumed linear for the first 12 hours after the storm. After 12 hours the final dimensions are reached and no further development of the breach occurs.

4.4.3. DISCHARGE THROUGH THE BREACH

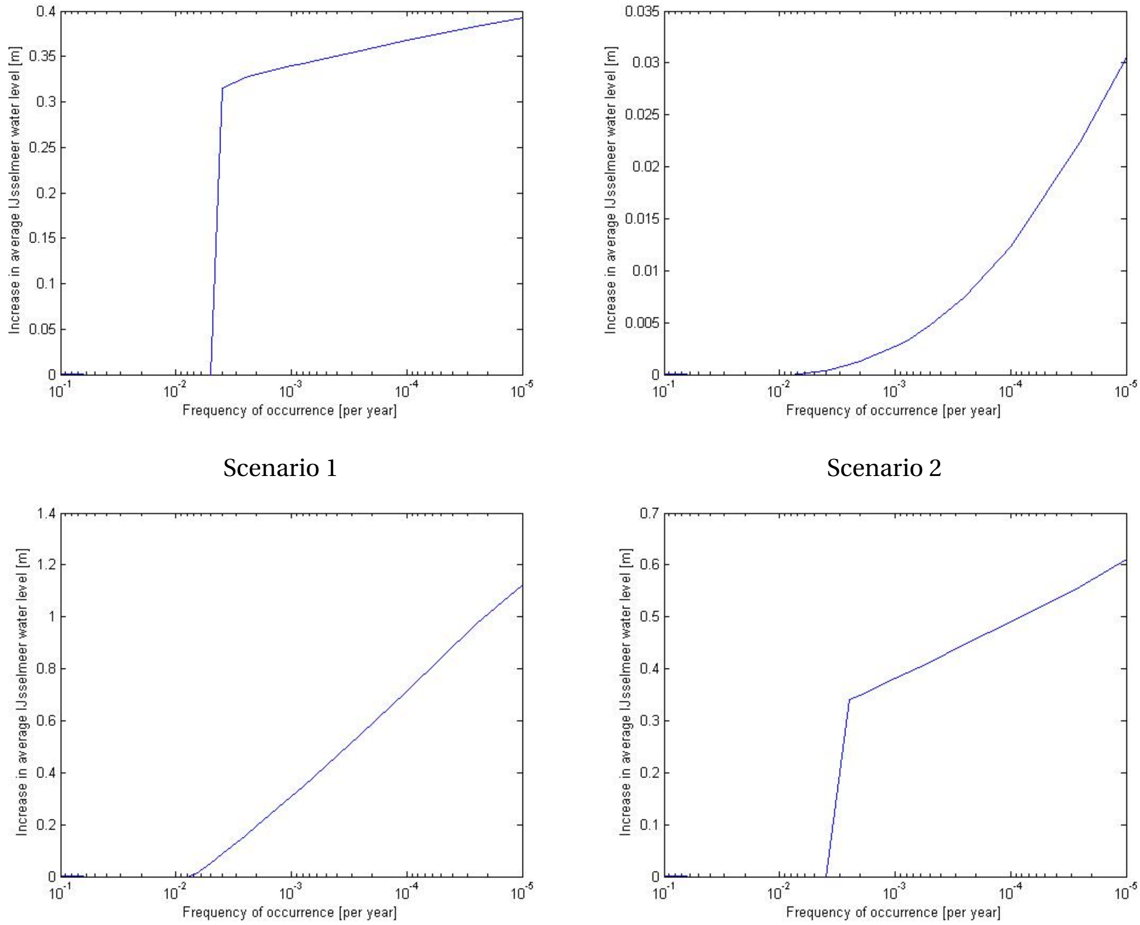
Now, for every hour after the peak of the storm the Waddenzee water level, breach width, breach depth, and IJsselmeer water level [3] are determined. With the formulas explained in section 4.4.1 and section 4.4.2 the hourly averaged discharge through the breach can be calculated.

When these discharges are averaged and multiplied with the duration of the storm the total discharge through the gap is found. This total discharge divided by the surface area of the IJsselmeer (≈ 1100 km^2) and the average water level increase in the IJsselmeer is found.

[3] In the computation, the average IJsselmeer water level is considered to be constant during the full duration of the storm. In other words: no feedback loop to take the effect of the increase in water level on the discharge is modelled in the script. As was shown in section 4.1.3 the IJsselmeer water level has little influence on the discharge through the gap.

4.5. RESULTS OF THE MODEL

The results of the Matlab simulation are given below in figure 4.9. The figures present the increase in average IJsselmeer water level because of the possibility of a breach in the Afsluitdijk. It shows the increase in water level after the last hour of the storm. This is done for each scenario. A detailed presentation and analysis of the results of the model can be found in appendix L. In this appendix also tables are shown with water level increases corresponding return periods.



Scenario 1

Scenario 2

Scenario 3

Scenario 4

Figure 4.9: Results of the matlab model for each of the 4 scenarios

The different developments of the breaches can be seen. At scenario 1 and 4 the sluices will blow out altogether. This explains the big leap in the top left and the bottom right figure in figure 4.9 which is absent in the other two figures. What is also noteworthy is that all the scenario start showing values above 0 only at frequencies smaller than 10^{-2}. Higher frequencies (more to the left in the figures) show no increase in average IJsselmeer water level because at these frequencies the Afsluitdijk will fulfil its function to retain water from the Waddenzee. Finally, the effects of more extreme storms can be seen. The effect of higher Waddenzee water levels has more effect in scenarios 2 and 3 than in scenario 1. This is explainable as the width of a breach in the levee is dependent on the outside water level (see section 3.5). The width of a breach at the sluices is, once it fails, no longer dependent on the outside water level.

For the numbers corresponding with the frequencies of the storms references is made to tables L.1, L.2, L.3, and L.4 in appendix L.

4.6. CONCLUSIONS CHAPTER 4

In this chapter the expected increase in IJsselmeer water level is calculated with the help of a Matlab model. Each of the 4 scenarios described in section 3.8 are used as input for the Matlab scripts. The output of the scripts is the maximum average IJsselmeer water level increase due to failure of the Afsluitdijk. This gives the maximum effect of a breach in the Afsluitdijk on the IJsselmeer but not necessarily means it gives the maximum effect on the IJsselmeer dikes (the wind speeds will probably be low at these maximum water level increases as the maximum water level increases are at the end of the storm). This difference is treated in next chapter (starting in section 5.1).

To get to this increase in average IJsselmeer water level, the current probability distribution functions for the extremes of the water levels in the IJsselmeer and the Waddenzee are described. It is found that for the IJsselmeer a Gumbel distribution is the best fit and for the Waddenzee a Weibul distribution provided the best results.

In this chapter the sensitivity of the effect of wind speed and wind direction on the local IJsselmeer water level was investigated. Although the effect of wind can be very extreme during storm conditions, the sensitivity of the wind set down on the discharge through the Afsluitdijk is very low. Further investigation showed that this low sensitivity is because the flow through the breach is mostly independent of the IJsselmeer water level. The large height of the gate results in only free flow (i.e. flow is independent of the downstream water level) in case of a breach through the sluice. And in the case of a breach in the levee, more than 90% of the flow can be modelled with a perfect weir formula (i.e. flow is independent of the IJsselmeer water level). This dominance of perfect overflow is because most of the time the water level in the Waddenzee is relatively much higher than the water level in the IJsselmeer.

By implementing the development of the breach into the model the exceedance frequency curves for each scenario were found. The results for scenario 1, 2, 3, and 4 are found in figure 4.9. Table 4.4 shows the results of the 4 scenarios for some of the calculated return periods.

Return period [years]	Water level increase [m]			
	Scenario 1	Scenario 2	Scenario 3	Scenario 4
100	0	0	0	0
1,000	0.34	0	0.31	0.38
10,000	0.37	0.01	0.71	0.49
100,000	0.39	0.03	1.15	0.61

Table 4.4: Results of the Matlab simulation for all scenarios for some of the frequencies

The table shows that scenario 3 is the most extreme scenario for the highest return periods. In this scenario, the water level increases can be over one meter. Scenario 1 and 4 result in less extreme water level increases but could still be significant enough to cause problems for the dikes around the IJsselmeer. It is found that for return periods between 250 and 1,000 these scenarios lead to the highest water level increases. Finally, scenario 2 shows hardly any increases in water level. For this result it can be concluded that if the boulder clay layer does not erode at all (as is the assumption in scenario 2), taking failure of the Afsluitdijk into account will not lead to increased risks to the dikes around the IJsselmeer.

5

Chapter 5: Influence on the Noordoostpolder

In this chapter the research question: *What is the effect of this increase of the IJsselmeer water level on the safety of the IJsselmeer dikes?* will be answered.

At the end of chapter 4, increases of IJsselmeer water levels at the end of the storm were presented. In section 5.1 it is argued that these values at the end of the storm will not necessarily lead to the most governing situation as at the end of the storm wind speeds will be very low and local water set up due to wind will be minimal. In the rest of this section the timing of the most governing situation during a storm is investigated. In section 5.2 the effects of a second storm hitting the IJsselmeer system is treated. Also, the adjustment of input for PC-Ring is handled in this section.

Section 5.3 gives a description of dike ring 7 which protects the Noordoostpolder. In this section it is argued why the influences of a possible breach in the Afsluitdijk is only calculated for this dike ring area and why this particular dike ring area. In section 5.4 the probabilities of failure for the Noordoostpolder are given for both no failure of the Afsluitdijk and for the case that a breach is already present in the Afsluitdijk. Section 5.5 treats the consequences of a breach in the dikes protecting the Noordoostpolder. Estimates for the economic damage and loss of human life are given.

In section 5.6 the results of the previous two sections (probabilities and consequences) are combined to express the risks of flooding for the Noordoostpolder. In the last section, section 5.7, the conclusions of this chapter can be found.

5.1. Governing condition during the storm

The water defence line around the IJsselmeer will fail due to a combination of a high water level and high waves standing on the outside of the dike. As was already explained in previous chapter (see section 4.1.3), the local water level in the IJsselmeer is a combination of the average IJsselmeer water level and wind set up. Equation (4.1) gives the formula for this combination. This equation can also be expressed in differences to the original local water level (before the storm):

$$\Delta H_{local} = \Delta H_{average\ water\ level} + \Delta H_{wind} \tag{5.1}$$

For the resistance of the dike against flooding it does not matter if local high water (ΔH_{local}) is a combination of high average water ($\Delta H_{average\ water\ level}$) and no wind set up ($\Delta H_{wind}$) or a normal average water level and high wind set up. However, for calculating the probability of occurrence of local high water, the reasons of the local water level are of importance.

5.1.1. DEVELOPMENT OF INCREASE OF WATER LEVEL DURING THE STORM

The results of previous chapter are the final water level increases at the end of the storm (30 hours after the peak). In these last hours the wind speeds have dropped down to low values in the order of 0 to 10 m/s. These low values give little water set up. If the results from chapter 4 are used for determining the new probability of failure of the dikes around the IJsselmeer it is implicitly assumed the local high water level is dominated by the average water level and not the wind set up. In other words, a high average water level is assumed to be the governing load [1] on the IJsselmeer dikes.

This assumption is incorrect (as will be shown in this chapter). Other possible combinations of wind set up and water level increase due to the breach are investigated. The possible load governing load can be anywhere between the peak of the storm (maximum ΔH_{wind} and no $\Delta H_{averagewaterlevel}$) and the end of the storm (minimal ΔH_{wind} and maximum $\Delta H_{averagewaterlevel}$). Therefore, for every hour (after the peak) of the storm the value of ΔH_{local} is calculated. To achieve this, the average water level set up due to failure of the Afsluitdijk and the water level set up due to wind must be calculated for each hour after the peak of the storm.

Calculating the average water level set up is done with the same Matlab model that was used in chapter 4. As the model already was programmed to calculate the discharge through the breach in time steps of one hour, the water level increases are easily found. In figure 5.2 and figure 5.3 the average water level increases are given for every hour after the peak for respectively scenario 1 and scenario 3. It can clearly be seen that scenario 3 needs the first hours to form the breach at which the discharge is negligible where in scenario 1 the inflow starts right away. In both scenarios the amount of water level increase starts to decelerate as the outside water level decreases in the later stages of the storm. In both figures an 1/10,000 year storm is used.

5.1.2. INCREASE OF WATER LEVEL DUE TO WIND

The other variable in equation (5.1) is the water level increase due to wind set up. For the calculation of this variable equation (4.2) is used. For this equation c is assumed to be 3.4*10^{-6} [34], g is taken as 10 m/s^2, h is taken as 5 m on average and the fetch length is taken as 50 km. This fetch length is approximately the distance between Den Oever and Urk.

The last variable in equation (4.2) is the wind speed. For this a number of assumptions are used. First assumption is that the peak of the wind speed is on the same time as the peak of the storm (peak of the Waddenzee water level). Second assumption is that after the peak of the storm the wind speeds will decrease linearly to an averaged daily wind speed value [2]. When looking at the measurements of the wind during the storm of 1953 (see figure E.1 in the appendix) this linear decrease looks a reasonable assumption. Only in the first hours after the storm the linear decline overestimates the hourly averaged wind speeds measured in 1953.

For the peak value of the wind speeds a closer look is taken at the wind statistics measured at Schiphol. These measurements were translated to find a relation between the maximum potential wind speeds and their return period. Figure 5.1 presents this relation. In this thesis, the 330° values is used as maximum wind speed as the Afsluitdijk will most likely fail during a storm coming from the North West. For the 1/10,000 storm the peak wind speed value of 29 m/s is used.

Note: For wind speeds of 29 m/s (and the assumptions for the other variables the same as explained before) equation (4.2) gives a wind set up of 2.9 meters. The 'Hydraulische Randvoorwaarden 2006' [41] (part of the WTI2006) gives standards between NAP + 1.5 m for the Noordermeerdijk and NAP + 2.5 m for the Zuidermeerdijk (see figure 5.8 for their location). If assumed that the original water level of the IJsselmeer was on its target level of NAP -0.40 m and realizing the 29 m/s corresponds with an 1/10,000 storm and HR2006 uses 'only' an 1/4,000 storm, the calculated wind set up seems a reasonable estimate of the wind set up during an 1/10,000 year storm. See also appendix I for a validation of the calculated wind set up.

[1] Governing load means the combination of load that gives the governing design or assessment criteria. In other words it was assumed that the combination of different load parameters that will give the most trouble to the dikes.

[2] For this 10 m/s is assumed. This corresponds with a moderate to strong breeze.

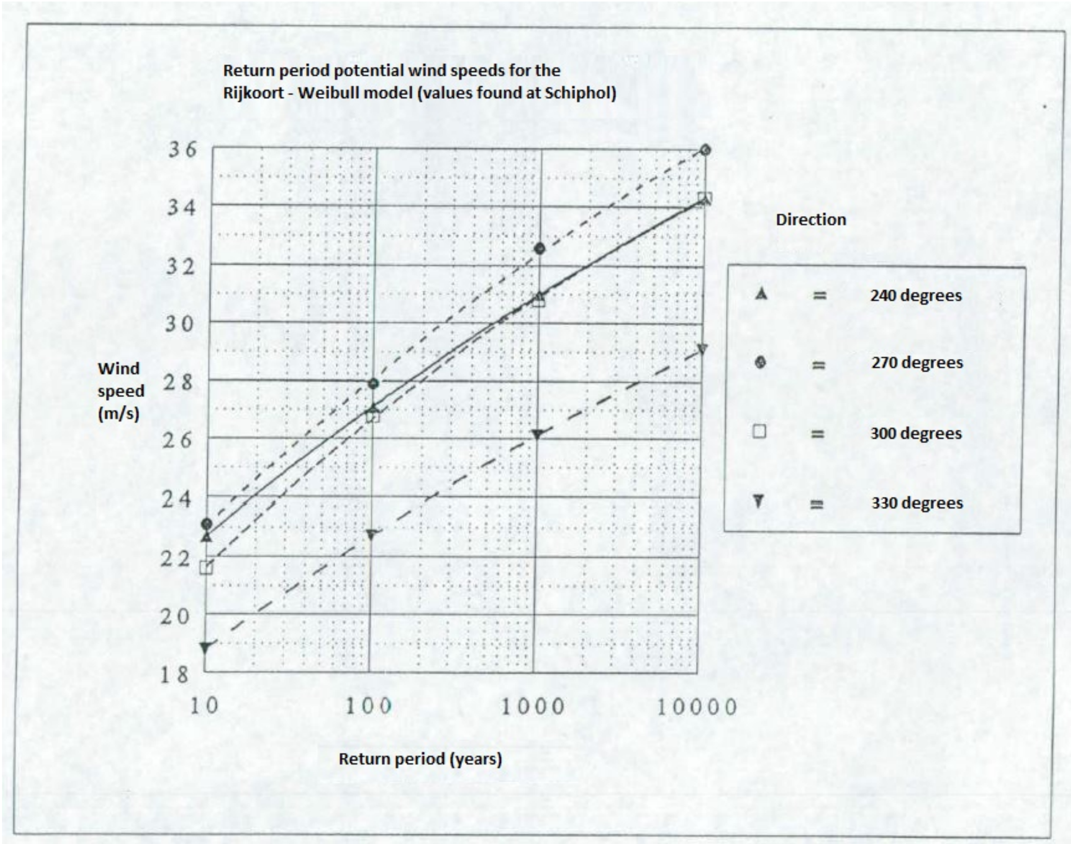


Figure 5.1: Relation maximum wind speeds and return period (source [35])

5.1.3. CONCLUSION INCREASE IN LOCAL WATER LEVEL OVER TIME

Wind speeds are defined for every hour in the storm and equation (4.2) can also be used for each hour after the peak of the storm. These results are also plotted in figure 5.2 and figure 5.3. In these figures also the summation of the two lines, equation (5.1), is plotted. It puts the amount of water set up due to failure of the Afsluitdijk in a remarkable perspective to the amount of water set up due to wind.

It can be seen that the effects of wind set up are far more dominant than the effects of the breach on the average IJsselmeer water level. Not only at peak of the storm (which was expected, as the breach only started to develop) but also at the later hours of the storm. Figure 5.2 shows that even the final average water level increase is smaller than the 10 m/s wind set up. Only for scenario 3 the local water level increase due to the breach will be higher than the wind set up in the last hours of the storm (figure 5.3). But even then, the effect of wind during the peak hours of the storm give a far higher local water level increase than the final hours of the storm in which the influence of the discharge through the breach is at its maximum.

Despite the numerous uncertainties in the assumptions used for finding the values in these figures (for example the fetch length, average water depth, probability of failure of the Afsluitdijk, effect of boulder clay in the Afsluitdijk, linear decrease of wind speed during the storm, neglecting bottom friction and inertia effects, etc. (see also appendix M and R), these differences in maximum wind set up and maximum average water level increase are so large the following statement is safe to conclude:

The IJsselmeer water level increase due to failure of the Afsluitdijk (with its current probability of failure) during an extreme storm has no effect on the probability of failure of the dikes around the IJsselmeer during that same storm.

The inflow of water through the breach is too slow and the retention area of the IJsselmeer is too big to have any significant effect on the local water increases. As the inflow through the breach starts to get to significant values, the wind speeds are decreased to such extent that the total increase in local water level is lower than water set up during the first hours of the storm.

It must be noted that the last addition to the statement ('during that same storm') is very important. This is discussed in next section.

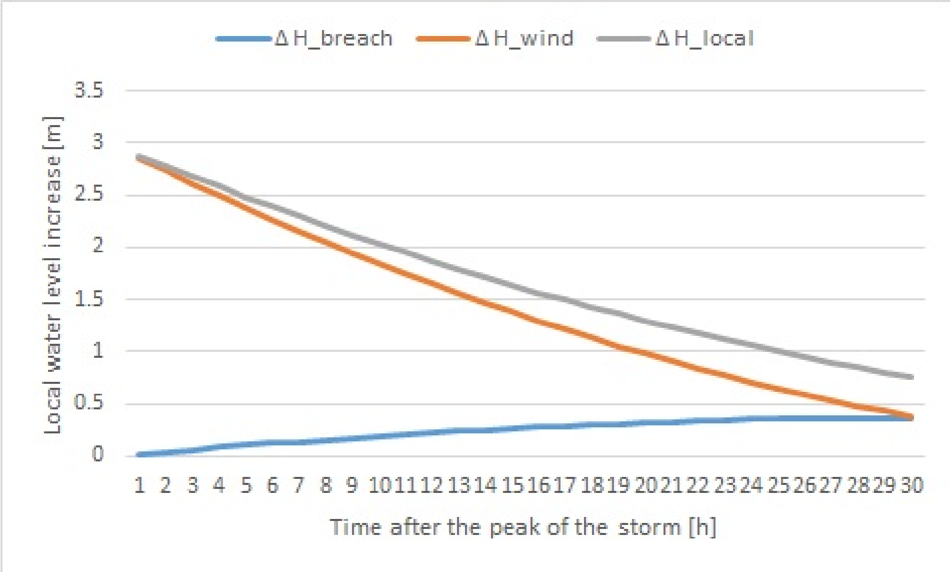


Figure 5.2: Water level increases due to failure of the Afsluitdijk and wind set up for Scenario 1 and an 1/10,000 year storm

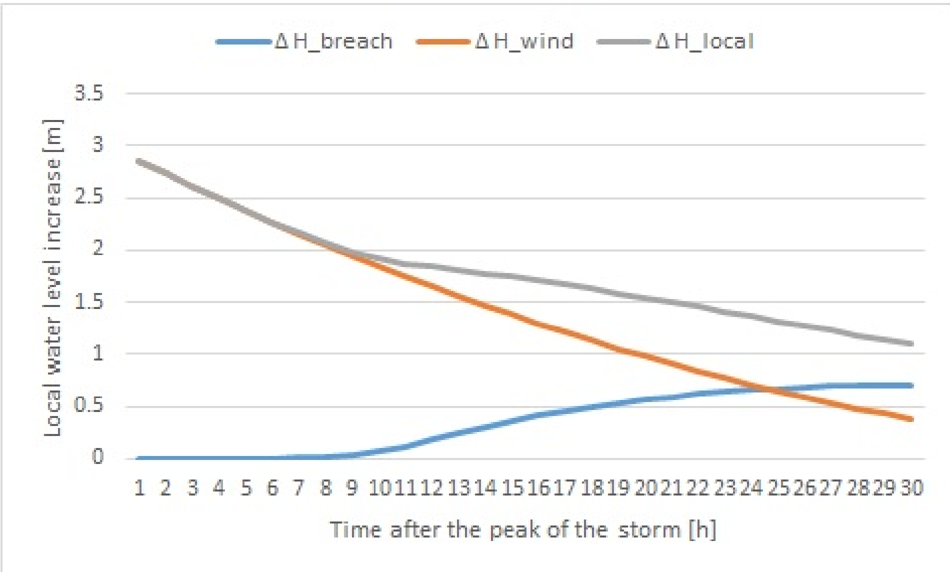


Figure 5.3: Water level increases due to failure of the Afsluitdijk and wind set up for Scenario 3 and an 1/10,000 year storm

NOTES ON THIS CONCLUSION

Two notes about this important conclusion must be mentioned:

One can argue that the extreme wind set up dominance is because of extreme wind speeds are taken into consideration, occurring during an 1/10,000 year storm. However, the conclusion also holds for less extreme storms. The same figure as figure 5.3 is made for an 1/1,000 year storm. This is presented in figure 5.4. During this storm the peak wind velocity is taken as 26 m/s instead of 29 m/s. The effects of this adjustment can be seen in this figure. The dominance of the wind set up is still existent. It might even be argued that the dominance is increased as also the amount of inflow during a less extreme storm is decreased.

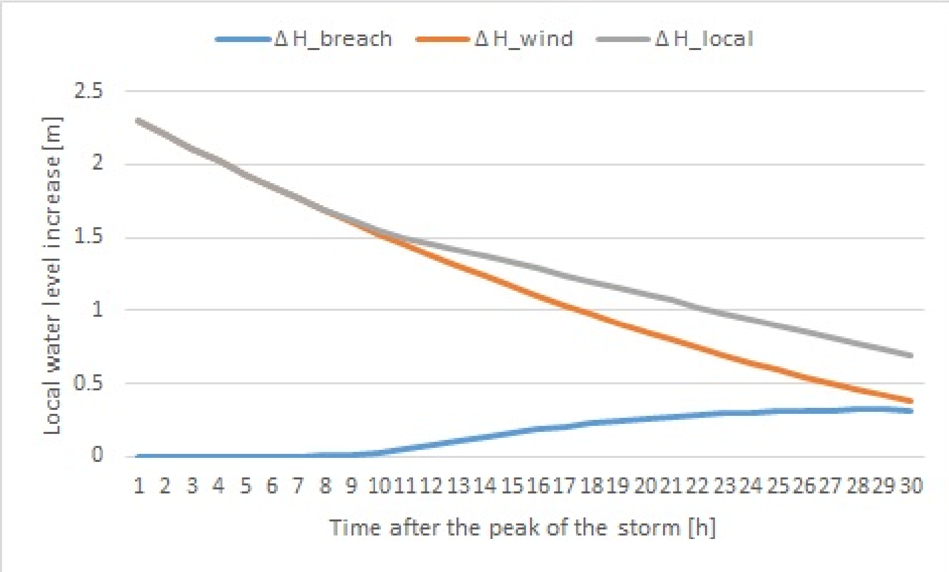


Figure 5.4: Water level increases due to failure of the Afsluitdijk and wind set up for Scenario 3 and an 1/1,000 year storm

Another note that can be made is that two phenomena are passed by during the approach that led to the previous conclusion. The first is the effect of wind on waves. The governing load is assumed in this section as the load at which the local water level is the highest. Local variations in the water level (due to waves) have also a very significant impact on the probability of failure of a dike ring element. Although the effect of waves can be significant, taking waves into account will not change the previously made conclusion. In fact it will only strength it, as bigger waves are expected at higher wind speeds and thus increasing the dominance of wind speed on the governing load.

The other phenomenon relates to the assumption that the whole IJsselmeer is a system in which disturbances (extra inflow of water) are influencing every part of the IJsselmeer at the same intensity and at the same time (the so called 'kombergingsmethode' see also section 5.5.1). This assumption might be unacceptable, as the inflow of water at the Afsluitdijk will take some time to influence locations at the other side of the IJsselmeer (e.g. Flevoland). Because of simplicity it is assumed the whole IJsselmeer acts directly (order of seconds) on this new interaction with the boundary. In reality this is in the order of hours:

Because the IJsselmeer is a relatively shallow lake, disturbances will travel at the speed of a shallow water wave. In shallow water, wave speed is a function of only the water depth [42]: $c = \sqrt{g * h}$ in which c is the speed of the disturbance, g the gravitational constant, and d the water depth. As the average depth of the IJsselmeer lies at 5 meter, the average wave speed is $\approx 7m/s$. And with a length of more than 50 km (distance Den Oever - Urk) this leads to almost 2 hours travel time. This 2 hours travel time causes a delay in the effect of the discharge through the breach.

In appendix I it is stated that for the response time of the water set up on the effects of wind speed are in the same order as the travel time of a wave from the Afsluitdijk to the other side. Therefore the effect of both delays (in water set up due to discharge through the breach and in water set up due to the effects of wind) can cancel each other out when looking at which is the most dominant factor.

5.2. A SECOND STORM

The conclusion in section 5.1.3 had one important addition: the last 4 words of the statement, 'during that same storm'. Up to now it was always assumed the Afsluitdijk was at its full strength if the extreme storm will occur. In this assumption the breaches still need to be developed which causes a negligible amount of discharge through the breach during the crucial first hours of the storm (crucial, because at these hours the outside water level is at its maximum as are the wind speeds). Another effect of this assumption is that storms less extreme than an 1/140 year storm are not influencing the IJsselmeer at all (as the Afsluitdijk will survive during these storms).

The effects of the breach on the days, weeks, and months after the storm are not discussed yet. For the next part of this chapter the effects of a second storm during the period between the first storm that causes the breach and the repairs are investigated. It is assumed the first storm causes the breach and is taken as an 1/250 per year storm (see section 5.4.4 why the first storm is assumed to be the storm with a relatively high frequency). After this first storm a second (less extreme) storm will hit the Afsluitdijk. For the second storm three different conditions apply compared with the first storm:

1. During this second storm water will flow in faster in the first hours after the peak of the storm as the breach is already at its maximum dimensions. For breach development in the levee this has significant consequences.
2. The second storm also causes inflow before the peak of the storm. To model the storm before the peak a linear relation for the water level in the Waddenzee is assumed. From a water level of NAP + 0 m at the beginning of the storm to its peak value at the peak of the storm. The time between the beginning and the peak of the storm is assumed to be 20 hours. This assumption is based on figure 3.3.
3. The intial water level at the IJsselmeer is already at NAP + 0 m as the first storm resulted in an open connection between the IJsselmeer and the Waddenzee. The NAP + 0 m is taken as the equilibrium water level in the Waddenzee.

As water level increases due to the breach are more extreme than during the first storm the governing load will probably not lie at the peak of the storm. The governing load can also happen multiple hours after the peak of the storm. To find this governing load the same method as in the previous section is used.

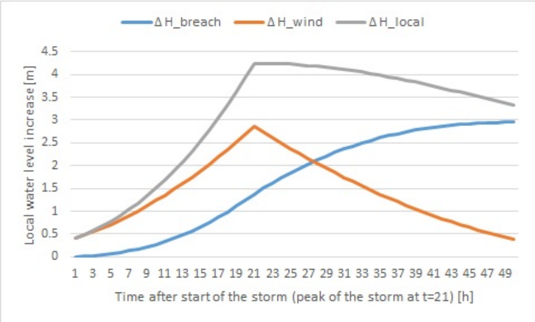
5.2.1. DEVELOPMENT OF INCREASE OF WATER LEVEL DURING THE SECOND STORM

The timing of the governing load during the second storm depends on the intensity of the second storm. Higher wind speeds have more effect on the wind set up than Waddenzee water level, thus on the discharge through the breach. Consequently, the lower the peak wind speed during the storm the less dominant the wind set up gets (i.e. dominance of wind set up over the water set up due to the breach).

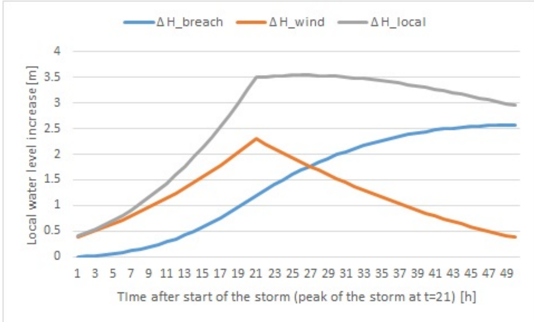
First it is investigated if the timing of the governing load is indeed changed or the results will be the same as the timing during the first storm. The graphs in figure 5.5 show the development of the increase in local water level due to the extra discharge through the breach, wind set up and total local water level increase for different kind of second storms. For all graphs the same first storm of 1/10,000 is used together with scenario 3 (so leading to a breach in the levee with a width of 1,300 m). In these figures the reference of the time on the x-axis is changed compared to the figures in section 5.1. Here the 20 hours before the peak are also shown and the reference is no longer after the peak of the storm but after the beginning of the storm (= t=0). This is done because during the second storm, also in the hours before the peak of the storm water can flow in as the breach is already present (point 2 in section 5.2).

The orange line represents the wind set up again. The assumption that the wind speeds peaks at the peak of the storm (t=21 h) can clearly be seen in each figure. Also the quadratic effect of the wind on the water level set up can be seen. The peak of the wind set up is lower as the storms get more frequent and the assumption that the wind speeds are 10 m/s before and after the storm can also be noticed (at t= 0 and t= 50).

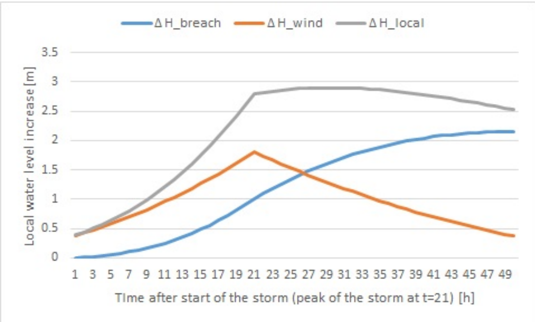
The blue line in figure 5.5 follows a s-curve. The water level increase adds up cumulatively during the storm. The discharge through the gap increases as the storm builds up and decreases after the peak. This explains the s-curve.



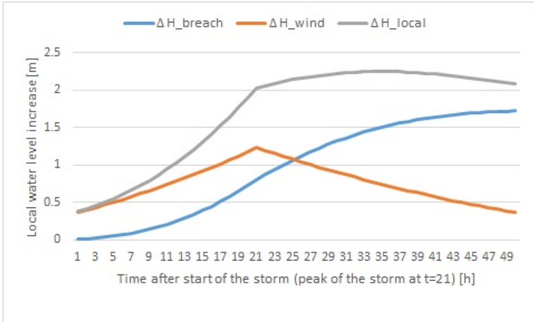
Effects of an 1/10,000 per year storm



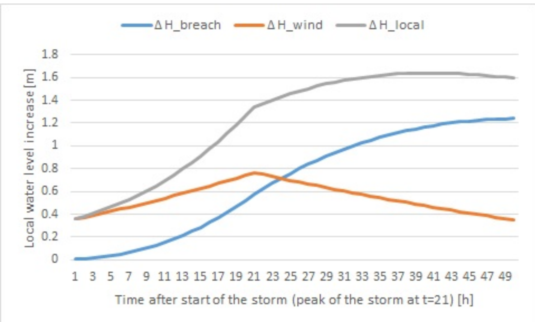
Effects of an 1/1,000 per year storm



Effects of an 1/100 per year storm



Effects of an 1/10 per year storm



Effects of an 1/1 per year storm

Figure 5.5: Water level increases due the effects of a second storm and wind set up Scenario 3 and an 1/10,000 per year storm, an 1/1,000 per year storm, an 1/100 per year storm, an 1/10 per year storm, and an 1/1 per year storm.

The maximum value of the local water level can be taken at the time the governing load takes place. As was discussed in section 5.1.3 the effect of wind speed on waves (1) and the travel time of the water that comes through the breach reaches the other side of the IJsselmeer (2) are not taken into account in the approach used to find the most governing time. Because of the first effect, the governing time taken now is taken too late. In the first hours after the peak of the storm the wind speeds are higher than in later hours and thus waves will be higher [3]. If inertia of the IJsselmeer was also taken into account increase of local water level due to the effect of the breach will be delayed by approximately 2 hours. It is assumed these two effects will cancel each other out (see appendix I). Therefore the highest values found in figure 5.5 can be taken as the time when the governing load takes place.

[3] Waves are taken into account later on in PC-Ring. Only for determining the time at which the governing load occurs waves are not taken into account

5.2.2. THE GOVERNING LOAD IN PC-RING

Unfortunately, the timing of the governing load found in figure 5.5 can not be simply implied in the input files for PC-Ring. It is ideal to use these 5 different storms and 'tell' PC-Ring to take the wind speeds a few hours later than the peak of the storm. Such a function is not available in PC-Ring and therefore these adjustments need to be inserted in a different way. It is done by calculating the expected water set up due to the breach at different wind speeds (so no wind set up) and add these values to the expected local water level (further explained below). Then, also the wind statistics are corrected (see section 5.2.3), to make sure not the peak wind speeds are used in the calculation.

In PC-Ring the hydraulic boundary conditions are entered by multiple data files. For each different locations around the IJsselmeer one data file exists (see appendix N for an example). For dike ring area 7 (see section 5.3) this means 30 different data files. The boundary conditions for locations between two known locations is simply found by interpolating between the two neighbouring values. In a data file different kind of data can be found. For 12 different wind directions (all wind directions split up into parts of 30 degrees), for 5 different initial average IJsselmeer water levels (-0.4, -0.1, 0.4, 1.0, and 1.8 m +NAP), and for 9 different wind speeds (14, 19, 22, 25, 28, 31, 34, 38, and 42 m/s) values are given for wave height, wave period, wave direction, and local water level. These values were calculated with the help of the WAQUA model (for more information on the WAQUA model see [43]). The probabilities of each wind directions, wind speed, and initial average IJsselmeer water level are given in other input files (see next section 5.2.3).

In this research the values of the local water level (the last column in the data files) are adjusted. First, every local water level corresponding with an initial average IJsselmeer water level of NAP -0.4 m is increased with 0.4 meter and every local water level corresponding with an initial average IJsselmeer water level of NAP -0.1 m is increased with 0.1 meter. This is done because it was assumed the average IJsselmeer water level is at least equal to the average Waddenzee water level due to the open connection caused by the first storm (see point 3 in section 5.2). It is important this correction is adjusted in the local water level and not in the initial average water level. As the initial average water levels are connected to probability functions in other input files. In order to get correct results, the probabilities of an initial average water level of NAP -0.4 or -0.1 m in the old situation (Afsluitdijk does not fail) must be added to the probability of an initial water level of NAP + 0 m in the new situation (Afsluitdijk has failed during a previous storm).

Second, the local water level will get an extra increase due to the discharge through the breach during the storm [4]. The amount of increased water level depends on the strength of the storm. In the input file the strength of the storm is expressed in wind speed. With the help of the Excel file (used in section 5.2.3) and the Matlab script (used in chapter 4) the average water level increases are calculated for each different wind speed and direction. With the help of the PC-Ring input file for the probabilities of a certain wind speed given a certain wind direction (see also next section), a probability of exceedance per year is calculated. With this probability of exceedance a Waddenzee water level is found (taking the Volkerfactor (see next section) into account). With this Waddenzee water level together with the wind speed the governing time can be estimated (analogous to the approach in previous section) and the average water level increase can be calculated (analogous to the approach in previous chapter). This increase in average water level is then added to the local water level in the data file corresponding to its wind direction and speed. This process is repeated for all 30 locations.

5.2.3. CORRECTION FOR WIND STATISTICS

The timing of the governing load is no longer taken at the peak of the storm. Previous sections showed that during the later hours after the peak of the storm (so with less wind speeds) the increase in average water level is significant enough to dominate the failure probabilities of the IJsselmeer dikes. If these increases in water level are simply added to the input values of PC-Ring, PC-Ring will calculate the probability of failure with the same wind statistics, thus the same peak wind speeds. This shift (or delay) in the timing of the most governing situation should take lower values of the occurring wind speeds as load for determining the probability of failure of the IJsselmeerdikes. In other words: a correction must take place on the wind statistics.

[4] The increase of local water level is calculated with PC-Ring during the run. Thus, these values does not need to be added manually

To execute this correction, a closer look is taken in the underlying programming of the software in PC-Ring. According to Bastiaan Kuijper, from HKV (personal communication), the wind statistics used as input for PC-Ring can be found in the map WWdataBasis. In this map different databases are found with statistics of wind direction and wind speeds for different locations in the Netherlands (see appendix O for an example). For some of the datasets a so called Volkerfactor is taken into account. For other datasets this factor is not used. The Volkerfactor is used for wind statistics at sea. The factor assumes that only if the high wind speeds occur during high astronomical tide these wind speeds are relevant. The storms that occur during low tide do not contribute to extremely high local water levels and therefore the probability of occurrence of high wind speeds is corrected with a (Volker)factor 2.

According to Kuijper, dataset number 25 corresponds to the location of the IJsselmeer. In this dataset wind directions are split up in sectors of 30 degrees. For each wind direction sector, values for 4 parameters are given in this database. The 4 parameters are P, a_w, b_w, and c_w. Parameter P represents the probability of occurrence of each wind direction sector. Parameters a_w, b_w, and c_w describe the probability density distribution for the wind speed for that particular wind direction. With these four parameters the probability of the wind speeds given a certain wind direction in the data file used in previous section can be determined.

According to the theory manual of PC-Ring [44] a_w, b_w, and c_w are parameters for description of the directional wind statistics. This description is done with the following formula:

$$p = 1 - q = exp[-exp(-Kr(u))] \tag{5.2}$$

In this formula, q is the exceedance probability, p is 1 minus the exceedance probability, and Kr(u) is a function defined below. If $Kr(u)$ is taken as a linear function, equation (5.2) would be a Gumbel distribution. In PC-Ring $Kr(u)$ is taken as a quadratic function in order to provide a more accurate fit over the gathered data at different measuring stations. In PC-Ring $Kr(u)$ is defined as:

$$Kr(u) = au^2 + bu + c \tag{5.3}$$

In this equation u is the wind speed in m/s and a, b, and c are the parameters used to fit the distribution over the gathered data. These parameters are also found in the dataset in PC-Ring as a_w, b_w, and c_w.

In this thesis, the values of a_w, b_w, and c_w are adjusted to correct the delay of the wind statistics for every wind direction. This correction is done with the help of Excel. The parameters are adjusted in such a way the probability of exceedance of an original wind speed are the same as the probability of exceedance of the adjusted wind speed. This correction of the 3 variables is done for every wind direction.

Example: the original wind speed at the peak of the storm is 25 m/s and the adjusted wind speed is needed 2 hours later, so u = 23 m/s (as result of the assumption of a linear decrease in wind speed from the peak to the end of the storm (= 10 m/s) during the duration of 30 hours that is used in this thesis). The original values of a_w, b_w, and c_w show that the probability of exceedance of 25 m/s is $6.55 * 10^{-6}$ for the wind direction of 330°. Now, the values of a_w, b_w, and c_w are adjusted in such a way the probability of exceedance of 23 m/s is adjusted to this same value of $6.55 * 10^{-6}$. The same is done for every wind speed (taking steps of 1 m/s). As the timing of the governing load differs for every possible wind speed (see figure 5.5), different corrections are made for every wind speed (not every correction is 2 hours, e.g. it can also be 0 or 10 hours). With the help of the least square method the values of a_w, b_w, and c_w are found that provide the best fit for the adjusted probability density function.

5.3. DESCRIPTION OF THE INVESTIGATED LOCATION

To answer the research question of this chapter[5], the effects of a breach in the Afsluitdijk must be investigated on every dike ring around the IJsselmeer. Because of the time limitations of this thesis research, not all dike ring areas can be calculated. Therefore only one dike ring area is selected. For this dike ring area the effects of a breach in the Afsluitdijk is investigated. To get the reduced safety of the other dike ring areas the same approach can be followed.

The investigated dike ring area chosen in this thesis is dike ring area 7 (see figure C.2). Dike ring area 7 protects the Noordoostpolder. The Noordoostpolder is the Northeastern part of the province of Flevoland and lies in the Southeastern side of the IJsselmeer.

Dike ring area 7 was chosen because of its geographic position in the Southeast of the IJsselmeer. The Afsluitdijk will fail most likely under the conditions of a Western/Northwestern storm. During such a storm the IJsselmeer dikes in the Southeast of the IJsselmeer (see figure 5.6) are receiving the biggest loads. Because of this, the dike ring areas 7, 8, 10, and 11 are most suitable to investigate (again, see figure C.2 for the location of the other dike ring areas).

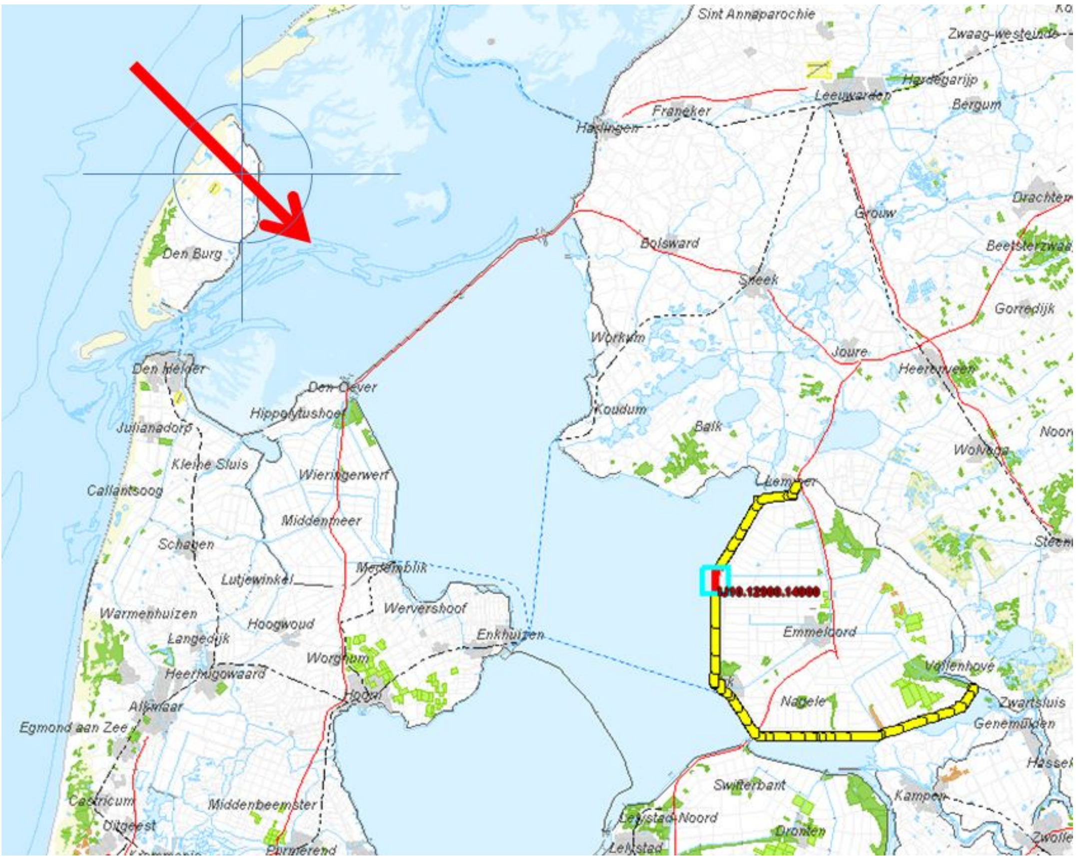


Figure 5.6: Dike ring area 7 in the Southwestern side of the IJsselmeer

Out of these 4, dike ring area 7 is the most simplistic. Only the west and south borders are part of the type-a water defence system. The Northeast border of the dike ring area 7 consist of type-c defence structures which are modelled with a failure probability of 0 as it is assumed this water defence line is not likely to fail. As no other water defence lines lie inside the dike ring area (as for instance is the case in dike ring area 8 with the Knardijk) this area can be simplified as a gigantic bathtub of which the east and south side are effected by the IJsselmeer (and Ketelmeer) system and of which the Southwest side is effected by the Zwarte Meer (see figure 5.8). The Zwarte Meer is highly influenced by the Ketelmeer and thus the IJsselmeer, but can be closed

[5]Which is: *What is the effect of this increase of the IJsselmeer water level on the safety of the IJsselmeer dikes?*

of by the Ramspolkering during storm surges.

The Ramspolkering is a hydraulic structure that can close of the Ketelmeer from the Zwarte Meer. Figure 5.7 shows the Ramspolkering and its location. The Ramspolkering reduces the probability of failure of the water defence elements at the Zwarte Meer, but the Ramspolkering itself can also fail. This makes defining the probability of failure of the whole dike ring a bit more difficult, but this is not problematic (as is shown later on in this report).

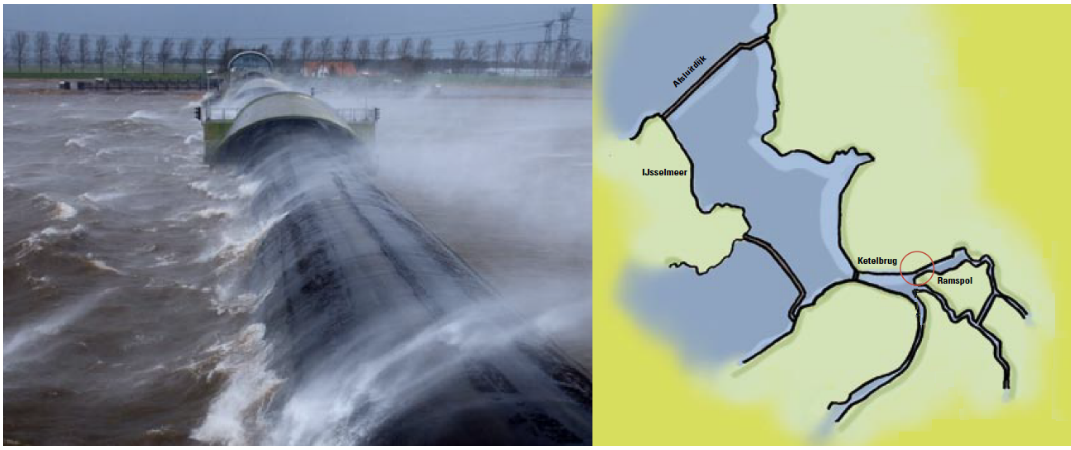


Figure 5.7: Ramspolkering in operation and its location (source: [45])

As the activities in the Noordoostpolder are not that diverse (see subsection 5.3.1), consequences of the failure of the water defence line are relatively easily characterised. Together with the previously described simple primary water defence line, the effects of a change in hydraulic boundary condition in the IJsselmeer can easily be found. The dike ring areas (8, 10, and 11) are a bit more complex. Dike ring area 8 is divided into two parts because of the Knardijk in the middle of the polder and is also influenced by the Markermeer [46]. And the safety of dike ring areas 10 and 11 are influenced by the amount of discharge through the river IJssel [47].

These factors do not only complicate the calculation of failure probabilities and the prediction of flood development through the area once the dikes are breached, but also make expressing the effect of failure of the Afsluitdijk more difficult. Therefore dike ring area 7 is chosen as the dike ring area that will be used in this chapter.

In 2013, project VNK (see the framework on page 60) published a report [14] that investigated the safety of dike ring area 7. Their results will be used in this thesis to compare with the adjusted IJsselmeer water level and adjusted wind statistics.

5.3.1. AREA DESCRIPTION OF THE NOORDOOSTPOLDER

In 1936 preparations were started for reclaiming the Noordoostpolder. Three and a half years later in 1939 the dike between Lemmer and Urk was finished connecting the island Urk to the mainland. In 1940 the dike on the Southern side was finished and in 1942 all the water was pumped out and the Noordoostpolder was reclaimed.

Dike ring area 7 protects the Noordoostpolder from the IJsselmeer in the west, the Ketelmeer in the South and the Zwarte Meer in the Southeast (see figure 5.8). On the East side the polder is bordered to the provinces of Overijssel and Friesland (more to the North). The total surface area of the Noordoostpolder is estimated at 490 km^2. Most of this area is used for agricultural purposes. The Noordoostpolder inhabits 65,000 inhabitants of which 20,000 live in the town Urk which lies on higher ground as Urk is a former island. The current safety standard of the dike ring is 1/4,000 per year [16].

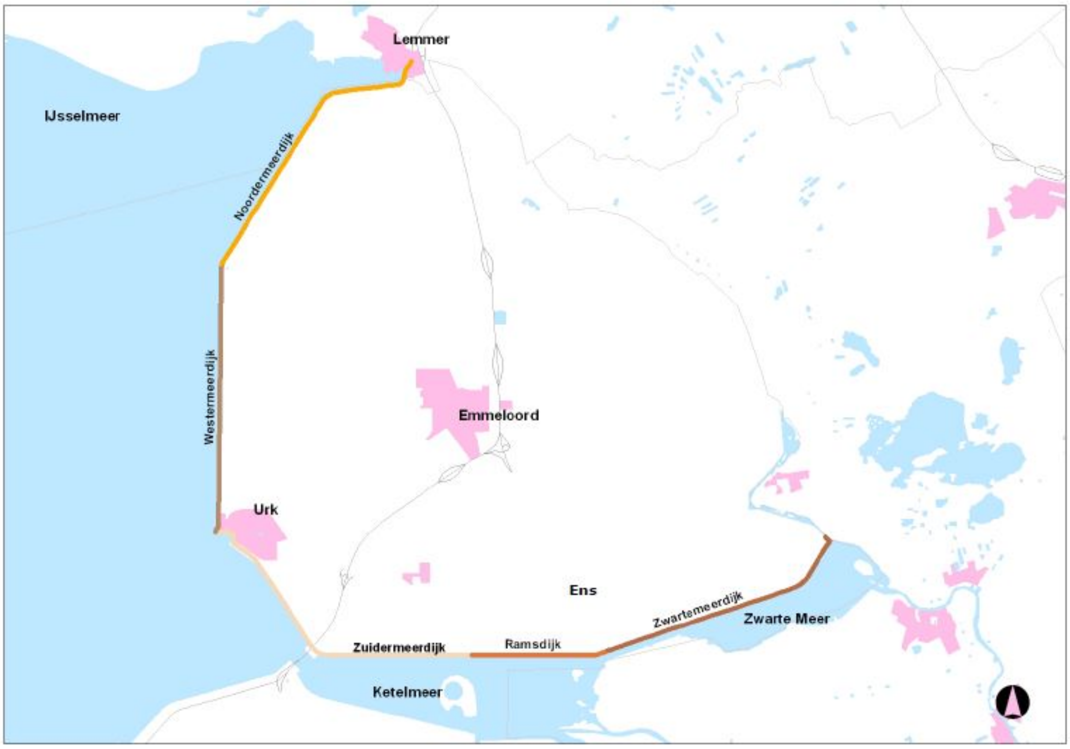


Figure 5.8: Area description Noordoostpolder (source: [14])

Flood Risk in the Netherlands project (Dutch: Veiligheid Nederland in Kaart (or VNK)).
This project analyzed the current flood risk in the Netherlands. Using innovative methods, probabilities, and dike performance, probabilities of failure are being linked to the consequences of flooding expressed in terms of economic damage and casualty numbers.

The results from a VNK analysis can be used to answer questions such as:

- Where is the risk of flooding high or low?
- What are the most vulnerable areas?
- What failure mechanisms are most likely to play a role in a levee breach?
- How can we effectively reduce the risk of flooding?

The project is an initiative of the Ministry of Infrastructure and the Environment, the Association of Regional Water Authorities (Unie van waterschappen), and the Association of Provincial Authorities (Interprovinciaal Overleg).

In the first project (VNK1), 16 dike ring areas were investigated. The second project (VNK2) investigated also the other dike ring areas in the Netherlands. For each dike ring area a report is written. Probabilities of failure of each element in the dike ring are calculated with the help of the PC-Ring software. Also the consequences are investigated by using altitude maps and flooding scenarios. Together with estimates of evacuation possibilities, this gives the consequences of flooding in numbers of economic damage and casualties. When the probabilities of the flooding scenarios are also added, the risks for the whole dike ring can be given.

Because of this stepwise approach, the weakest links (certain failure mechanisms, areas with high economic value, evacuation possibilities, etc.) can be found and the most effective measures can be found. The second project (VNK2) is estimated to finish in the fall of 2014.
(Sources: [48, 49])

5.3.2. ALTITUDES IN THE NOORDOOSTPOLDER

The Noordoostpolder consists of two former islands in the Zuiderzee: Urk and Schokland. These two former islands, together with the area connecting the Noordoostpolder with the mainland, are the only parts of the Noordoostpolder above NAP (see figure 5.9). On average the ground surface level lies on NAP -4.0 meter with the deepest point at NAP -5.0 meter. The altitudes of the area are important for the flood scenarios discussed later on (see section 5.5).

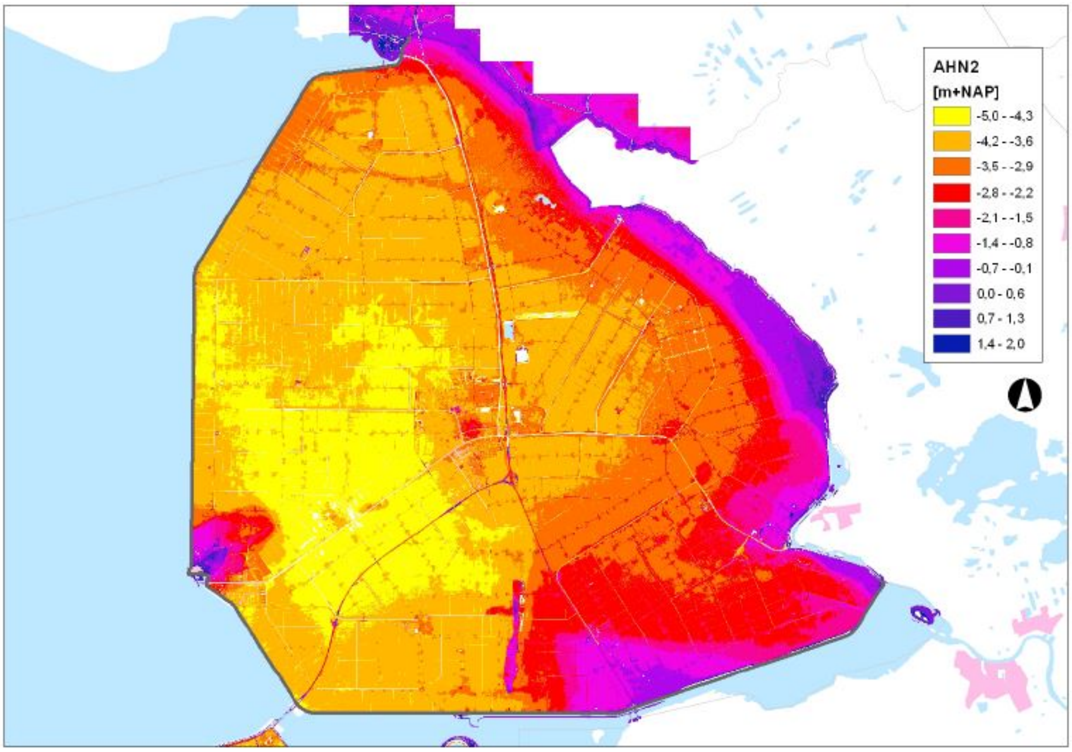


Figure 5.9: Altitude map of the Noordoostpolder (source: [14])

5.3.3. ELEMENTS OF THE WATER DEFENCE SYSTEM

In figure 5.8 the type-a water defence line is split into different parts: Noordermeerdijk, Westermeerdijk, Zuidermeerdijk, Ramsdijk, and Zwartemeerdijk. This division of parts of the system is based on their location and orientation. For determining the probability of failure of the whole dike ring area, further dividing is needed as the error will be too big if partition is simplified to only these 5 divisions.

PC-Ring makes a distinction between dikes, dunes, and hydraulic structure for different failure mechanisms (see also appendix B). Therefore the dike ring must at least be divided into different parts for dikes, dunes, and hydraulic structures. Then, the parts must be divided into elements with the same strength and load characteristics.

Water board Zuiderzeeland (the water board in the Noordoostpolder) made such a division. Water board Zuiderzeeland provided the characteristics of the water defence elements of dike ring 7 for this thesis (personal communication, September 24th, 2014). In this division the whole water defence line is split up in 40 dike ring elements, 7 hydraulic structures, and no dunes. The dike ring elements are more or less evenly divided over the water defence line (see also the yellow pieces in figure 5.6) and the hydraulic structures are situated at Lemmer (4), Urk (2), and Ramspol [6] (1) [7]. These hydraulic structures are sluices, culverts, and pumping stations. Figure 5.10 gives the 47 (40+7) elements of dike ring 7.

Figure 5.10: Dike ring elements in dike ring area 7

[6] For the location of Ramspol see figure 5.7

[7] This hydraulic structure is not the Ramspolkering but a culvert in the Ramsdijk

5.4. PROBABILITIES OF FAILURE

Here the probability of failure of the whole dike ring is presented. These probabilities are the results of the PC-Ring runs for the different events and scenarios.

Section 5.4.1 describes the calculated failure mechanisms in PC-Ring. In section 5.4.2 the current probability of failure is calculated and presented. Section 5.4.3 will give the probability of failure if the Afsluitdijk fails and the differences are compared in section 5.4.4.

5.4.1. USED FAILURE MECHANISMS

In appendix A, the failure mechanisms for water defences are described. PC-Ring calculates only a few of them. For dikes these failure mechanisms are overflow and wave overtopping, sliding of inner slope, piping and heave, and instability of outer slope protection. For structures, the failure mechanisms overflow and wave overtopping, failure of closing, piping, and failure of the construction are considered. For dunes, only dune erosion can be calculated.

All other failure mechanisms can not be calculated with PC-Ring because of several reasons: some of the mechanisms do not lead directly to failure of the element, for other failure mechanisms there is not enough knowledge to be able to calculate the probability of failure, and some failure mechanisms depend heavily on load development in time which is not included in PC-Ring.

Fortunately for dike ring 7, the limited calculation possibilities of PC-Ring are not a problem. In the third assessment [3] all failure mechanisms, that can not be calculated with PC-Ring, received the mark 'good', which means these failure mechanisms provide (more than) enough protection. Because of this, VNK assumes these failure mechanisms are not relevant compared to the failure mechanisms that can be calculated [14].

5.4.2. PROBABILITY OF FAILURE IF THE AFSLUITDIJK WILL NOT FAIL

The effects of a failure of the Afsluitdijk will be compared with the effects of no failure of the Afsluitdijk. As stated before, no failure of the Afsluitdijk is the current starting point for the calculations of failure probabilities of the IJsselmeer dikes. To find this probability of failure of dike ring area 7, PC-Ring is used. If done correctly the results will be the same as in the VNK2 report [14]. This provides a good validation of the results found with PC-Ring [8].

With PC-Ring it is found that **the total probability of failure** of the current dike ring area, provided **the Afsluitdijk will not fail**, amounts **1/1,000 per year**. At the moment this system is being improved to provide more safety against flooding. In this thesis, the current system is worked with and the results of the other PC-Ring calculations are compared with the current probability of failure of 1/1,000 per year.

The total probability of failure of 1/1,000 per year is calculated by calculating the probability of failure for each failure mechanism and for each dike ring element (see appendix B on how this is done). Below, the 10 weakest dike ring elements that contribute mostly to the total probability of failure are presented. It is found indeed that the results of the PC-Ring run are the same as in [14]. The results for the other dike ring elements are given in appendix P. In this appendix also a detailed analysis is given.

DOMINANT ELEMENTS AND FAILURE MECHANISMS

In table 5.1 the weakest 10 elements in the dike ring are presented together with their probability of failure. The 10 weakest elements are all dike elements, so the hydraulic structures are not governing for the total probability of failure.

These 10 weakest elements are all situated on the west part of the Noordoostpolder (see figure 5.10 for their location) and the failure probability of all the elements is dominated by the failure mechanisms overflow and wave overtopping (see appendix P). The combination of the probability of failure of these elements (and the other 37 stronger elements) gives a total probability of failure of 1/1,000 per year.

[8] As PC-Ring is a complex program and the writer of the thesis has no experience with PC-Ring this is a welcome check.

Element of the dike ring	Return period [year]
IJ09.09900.12900	1,600
IJ06.05100.07500	2,400
IJ10.12900.14900	2,600
IJ20.28200.29900	3,100
IJ11.14900.18400	4,000
IJ13.21100.23000	4,100
IJ19.26800.28200	4,300
IJ12.18400.21100	5,700
IJ08.08600.09900	5,700
IJ21.29900.31300	9,700

Table 5.1: Results of PC-Ring, weakest 10 elements of the dike ring (no failure Afsluitdijk) (element numbers can be found in figure 5.10)

5.4.3. PROBABILITY OF FAILURE IF THE AFSLUITDIJK WILL FAIL

A second run is done with PC-Ring to find the probability of failure of the dike ring area, but in this second run the Afsluitdijk has failed during a previous storm. For this previous storm (the first storm) any kind of magnitude can be assumed. However, the more extreme the first storm will be, the less likely a combination of the first and the second storm will be (see section 5.4.4 for the probability of the combination of the two storms). Secondly, higher frequencies will lead to minor increases in discharge at the cost of major increases in return period (also for scenarios 2 and 3). Therefore, the combination of a first storm with a frequency of 1/250 per year with the second storm will be used to investigate the effects of a second storm on the safety of the dikes of the Noordoostpolder.

As was shown in chapter 4, scenario 3 starts to be the dominating scenario for storms extremer than 1/1,000 and only at storms more extreme than 1/10,000 the discharges through the breach are significantly higher (leading to differences of a few decimeters). In scenario 1 and 4, the storm blows out the sluices completely leading to breach width of 336 meters. This width is much wider than the width of a breach in the levee at the end of an 1/250 year storm. Because of this, scenario 1 and 4 (both leading to the same values for an 1/250 storm) are used and not scenario 3 (or 2).

Note: It is interesting to see the probability of failure of multiple magnitudes of the first storm (e.g. 1/250, 1/1,000, and 1/10,000) and every scenario (1, 2, 3, and 4). Unfortunately, calculation of these storms and scenarios requires a lot of time. For each combination of storm and scenario, different discharges through the breach are found and thus different increases in water level. These increases in water level need to be adjusted for 30 locations around the Noordoostpolder, for 12 different wind directions, and 9 different wind speeds. Also the wind statistics must be corrected as the timing of the governing load differs for the combination of storm and scenario (which also needs to be found first). All these computations require much time and will most likely lead to less interesting values than the combination of the 1/250 per year storm and scenario 1 or 4. Therefore, these other combinations are not executed for this thesis.

After implementing these starting points in the input files of PC-Ring, **the total probability of failure** of the current dike ring area **provided the Afsluitdijk has failed** during a previous 1/250 storm is found to be **1/450 per year**. Below, the results of second run are given.

DOMINANT ELEMENTS AND FAILURE MECHANISMS

As was done in section 5.4.2 the weakest 10 elements are also presented here. Table 5.2 show the weakest 10 elements given that the Afsluitdijk fails. Also here the 10 weakest elements are all dike elements, so the hydraulic structures are still not governing for the total probability of failure.

9 of the 10 weakest elements presented in table 5.1 are also found in table 5.2. Only element IJ21 (previous number 10) is replaced by IJ05 (the new number 9). This change is not very noteworthy as element IJ21 is now just fallen out of the top 10 and element IJ05 used to be just outside the top 10.

If this table is compared with table 5.1 one can see that all the dike elements have an increased probability

Element of the dike ring	Return period [year]
IJ20.28200.29900	900
IJ09.09900.12900	1,300
IJ10.12900.14900	1,900
IJ06.05100.07500	1,900
IJ11.14900.18400	2,400
IJ13.21100.23000	2,800
IJ12.18400.21100	3,500
IJ19.26800.28200	4,000
IJ05.04300.05100	4,200
IJ08.08600.09900	5,100

Table 5.2: Results of PC-Ring, weakest 10 elements of the dike ring (failure of the Afsluitdijk) (element numbers can be found in figure 5.10)

of failure. One difference stands out compared to the others. This is element IJ20. IJ20 suffers more from the increases in loads than the other elements as this element has a relatively weak outer slope protection (see table P.2 and table P.5 in the appendix). For this element both probabilities of failure for failure mechanisms overflow and wave overtopping and instability of the outer slope are increased. This results in a much higher combined probability of failure.

5.4.4. DIFFERENCES COMPARED

When the total probabilities of failure of the dike ring are compared it can be concluded that failure of the Afsluitdijk has a negative effect on the safety of the dikes of the Noordoostpolder. The probability of failure with the Afsluitdijk intact is 1/1,000 per year and the probability of failure of the Noordoostpolder given that an 1/250 per year storm has already occurred (at the Afsluitdijk) is 1/450 per year.

Difference between these two values is the conditionality of the probabilities of failure. The first run is the probability of failure of the Noordoostpolder given that the Afsluitdijk does not fail and the second run results in the probability of failure of the Noordoostpolder given that the Afsluitdijk has failed already in a previous storm. In order to compare the influence of failure of the Afsluitdijk, this conditionality has to be taken into account. The total (conditional) probability of failure, thus including the effects of failure of the Afsluitdijk during a previous storm, can be calculated with the following formula:

$$P(F_N \cap F_A) = P(F_N|F_A) * P(F_A) \tag{5.4}$$

In this equation $P(F_N \cap F_A)$ is the probability that both the dikes of the Noordoostpolder and the Afsluitdijk fail, $P(F_N|F_A)$ is the probability that the dikes of the Noordoostpolder fails given that the Afsluitdijk has already failed (this is calculated in section 5.4.3 and is found to be 1/450 per year), and $P(F_A)$ is the probability that the Afsluitdijk fails (1/250 per year).

If the found values of F_N and F_A are filled into the equation the following results for the probability of failure of the dike ring area during the second storm are found.

$$\begin{aligned} P(F_N \cap F_A) &= P(F_N|F_A) * P(F_A) \qquad (5.5)\\ P(F_N \cap F_A) &= 1/450 * 1/250 = 8,9 * 10^{-6} \approx 1/112,500 \end{aligned}$$

These calculations show that the probability of failure of dike ring area 7 due to a second storm is ±112 times lower than the current probability of failure of dike ring area 7. This means the governing situation for dike ring area 7 is the current situation and not the combination of 2 storms in one winter period. The main reason for this difference in failure probability is the unlikely occurrence of an 1/250 storm before the disastrous second storm.

5.5. CONSEQUENCES OF A FLOOD

This section presents the consequences of failure of dike ring area 7. These consequences can be seen to be independent of the storm conditions under which it fails. This makes the consequences the same for every storm and every of the 4 scenarios at which the dikes of the Noordoostpolder fail.

This independency is caused by the fact that the storm has a relatively short duration compared to the time that is needed to inundate the Noordoostpolder (because of its large surface area). According to VNK [14] the height of the water in the final situation in the polder can be determined on the average water level of the IJsselmeer before the storm. After the storm it is assumed the average water level will go back to its original average water level before the storm.

The consequences investigated in the VNK report can be used for no failure of the Afsluitdijk. In the case of failure of the Afsluitdijk water level in the IJsselmeer will be a little bit higher and thus the consequences will also be larger. This effect needs to be accounted for.

5.5.1. APPROACH FOR CALCULATING THE CONSEQUENCES

To calculate the consequences of the flood in the Noordoostpolder PC-Ring uses a basin capacity method (Dutch: kombergingsmodel) [9]. In this method it is assumed the final water level in the Noordoostpolder is in balance with average water level of the IJsselmeer. This water level is calculated with the following simplified formula [14].

$$m_{final} = \frac{m_{start} * A_{lake} + h_{polder} * A_{polder}}{A_{lake} + A_{polder}} \tag{5.6}$$

In this equation m_{final} is the average water level in the balanced (=final) situation [m+NAP], m_{start} is the average IJsselmeer water level at the beginning of the storm [m+NAP], A_{lake} is the surface area of the IJsselmeer [m^2], A_{polder} is the surface area of the Noordoostpolder [m^2], and h_{polder} is the average height of the ground level of the Noordoostpolder [m+NAP].

Crucial in formula (5.6) is m_{start} (the other parameters are more or less constant). Ideally for every possible average IJsselmeer water level the final water level in the polder should be calculated. Practically this is not possible. VNK uses 5 different initial water levels for the IJsselmeer:

- NAP -0.4 m
- NAP -0.2 m
- NAP 0 m
- NAP +0.2 m
- NAP +0.4 m

For one of these initial water levels also a SOBEK simulation was executed (for more information about SOBEK see [50]). In this way the rising velocity of the water in the polder is estimated and the found effects are validated.

5.5.2. DIFFERENT RING PARTS

For estimating the consequences of inundation of the Noordoostpolder the polder is split up into two different flood scenarios. Each flood scenario consists of the part of the dike ring area for which the consequences in economic damage and casualties are independent of the breach location in that part of the dike ring area.

Here the simplicity of dike ring area 7 is again very convenient. As (almost) no high elements that can hinder the inundation process are present, a breach at the location of the IJsselmeer or Ketelmeer has more or less the same consequences. Therefore, the Noordermeerdijk, Westermeerdijk, Zuidermeerdijk, and Ramsdijk (see again figure 5.8) can all be simplified to one dike ring part. The other dike ring part is at the Zwartemeerdijk where the Zwarte Meer influences the probability of failure and the consequences of the flood.

[9] In this method the movement of water is simplified by assuming that inertia and friction of transport between and in the two basins is neglected

The probability of failure of the Zwartemeerdijk is extremely low (<1/1,000,000, see the values for Z08-Z01 in table P.2 in the appendix). Therefore, the risk for the scenario that the Zwartemeerdijk will fail will also be extremely low. As the consequences of flooding will probably be in the same order at both dike ring parts, the risks of flooding at the first dike ring part will be dominant. This means the consequences of only one flood scenario should be investigated. This is the scenario of a breach at the West or the South side of the dike ring area.

5.5.3. EVACUATION

The number of victims (and to a less extent economic damage) is very much influenced by the fraction of the people that can be evacuated. VNK uses 4 different evacuation scenarios to find a range of expected number of casualties. For each evacuation scenario different evacuation fractions (i.e. the % people being evacuated) and the probability for that evacuation fraction (in respect to the other evacuation probabilities) are calculated. VNK found the presented values in table 5.3.

Evacuation scenario		Evacuation fraction	Probability
Unexpected flood	No evacuation	0.00	0.20
Unexpected flood	Disorganised evacuation	0.40	0.08
Expected flood	Disorganised evacuation	0.67	0.40
Expected flood	Organised evacuation	0.80	0.32

Table 5.3: Evacuation fractions and probability per evacuation scenario

The 4 different scenarios are differentiated between an unexpected or expected flood, meaning whether or not the breach in the Noordoostpolder was expected beforehand. Then, a division is made between no evacuation possibilities, a disorganised evacuation or an organised evacuation. The combinations of no evacuation during an expected flood and an organised evacuation during an unexpected flood are not seen as possible (probability is 0).

With the help of population data and table 5.3 the number of casualties of a flood in the Noordoostpolder can be estimated. Because of the uncertainty of the evacuation fraction a range for the expected number of casualties will be given.

5.5.4. RESULTS OF THE CALCULATION

The results of the PC-Ring model are presented in figure 5.11. For each of the 5 initial water levels the water level in the polder, the expected economic damage and the range of the number of casualties is given. Also it can be seen that almost the whole part of the Noordoostpolder will be flooded. Only Urk and Schokland are not inundated (see section 5.3.2). In other areas the water level can be even higher than 3 meter above the ground level.

In the starting point of the previous 5 initial water levels it was assumed the discharge of the rivers IJssel and the Vecht can be drained away directly. VNK made a 6th scenario in which this drainage is hindered (for any kind of possible reason). They called this scenario the 'maximum scenario' as this gives the highest economic damage and highest expected number of casualties. Figure 5.12 presents the result of PC-Ring for the maximum scenario.

Initial water level	Final water level		Economic damage [M€]	Casualties
NAP -0.4 m Water depth [m] No water 0 - 0.2 m 0.2 - 0.5 m 0.5 - 1 m 1 - 1.5 m 1.5 - 2 m 2 - 3 m > 3 m	NAP -1.31 m		3625	60 - 290
NAP -0.2 m	NAP -1.17 m		3835	60 – 310
NAP 0.0 m	NAP -1.02 m		4070	65 – 335
NAP +0.2 m	NAP -0.88 m		4300	70 – 355
NAP +0.4 m	NAP -0.74 m		4545	75 - 375

Figure 5.11: Consequences of failure of the Noordoostpolder (source: [14])

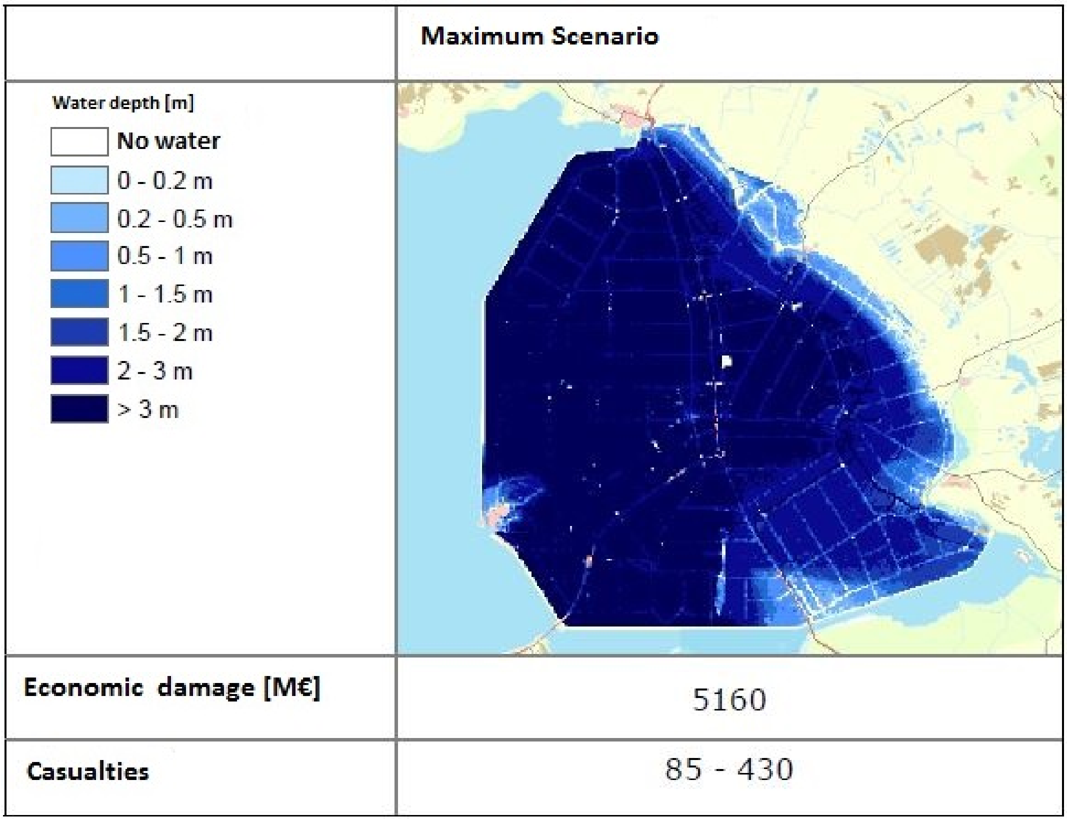


Figure 5.12: Consequences of failure of the Noordoostpolder for the maximum scenario (source: [14])

5.6. FLOOD RISK

When values of the probability of failure and the consequences are known (see section 5.4 and section 5.5) the risks can be calculated (see also appendix C). In a simplistic way the risks can be calculated with the following relation:

$$\text{Risk} = \text{Probability} \times \text{Consequences} \tag{5.7}$$

If the probability is expressed in a probability per year and the consequences are expressed in a monetary value (e.g. euros), the risks can be expressed in €/year. These units are used in this thesis to express the economic risk. Also the expected number of casualties was calculated in section 5.5. This risk is expressed in number of casualties per year.

In PC-Ring the risks of flooding is expressed in the following 6 different measures:

1. **Expected value of the economic damage** The expected value of the economic damage is the damage a flood caused multiplied by the probability of the flood (see equation (5.7)).
2. **A damage function** This function presents the probability per year that a flood causes a certain damage in the form of a FN-curve. As was stated in section 5.5 the consequences are independent of the location of the breach in the dike ring (only one flood scenario). This means the expected economic damage is always the same for every flood (circa 4,500 million euros [14]).
3. **Expected value of the number of deaths** The number of casualties shows a spread in section 5.5. This spread is caused by the uncertainty in the possibility of evacuation. For determining the risk an expected value of the number of casualties is calculated. Using the values of figure 5.11 and table 5.3 (the evacuation fractions and corresponding probabilities) the expected number of casualties is found to be approximately 167.

$$\begin{aligned}
375 * (1 - 0.80) * 0.32 &= 24.0 \\
375 * (1 - 0.67) * 0.40 &= 49.5 \\
375 * (1 - 0.40) * 0.08 &= 18.0 \\
375 * (1 - 0.00) * 0.20 &= \underline{75.0 \quad +} \\
\text{Expected number of casualties} &= 166.5
\end{aligned}$$

   This expected number is multiplied with the probability of a flood to arrive at the risk.
4. **Local risk** Local risk is the probability that a person dies because of a flood if it would stay on a certain spot during the whole year. It expresses the risks of a flood for a location in the dike ring area. It depends on the water depth during a flood and the rising velocity of the water at that location. It is independent of the number of inhabitants. This measure is expressed as a map of the area. The highest local risks can be found just behind the weakest dike ring elements.
5. **Local individual risk** Local individual risk is the same as the local risk, but now also the possibilities of evacuation are taken into account. For dike ring area 7 this means the local individual risk is a factor 2.3 [10] lower than the local risk at every location.
6. **Group risk** Also for the number of casualties a FN-curve is constructed. Figure 5.13 gives the FN-curve for the current situation of the expected number of casualties. The differences in number of casualties are because of the different evacuation scenarios.

Numbers 1 and 2 relate to the economic risk and numbers 3 to 6 give an expression for the number of casualties. For comparing the effect of failure of the Afsluitdijk with the current approach numbers 1 and 3 are the most relevant. These values are estimated in next sections.

5.6.1. RISK IF THE AFSLUITDIJK WILL NOT FAIL

If the Afsluitdijk will not fail, the current safety of the Noordoostpolder and the current consequences give a total expected value of the economic damage of **4.5 million euro per year**. This value is found by using equation (5.7). The highest consequences from the VNK report are used (see 5.11) which is approximately 4500 million euro. The probability of failure of the current dike ring area (not taking failure of the Afsluitdijk

[10] 2.3 = 1/(1 - (0.80 * 0.32 + 0.67 * 0.4 + 0.40 * 0.08 + 0.00 * 0.20)). Values from table 5.3

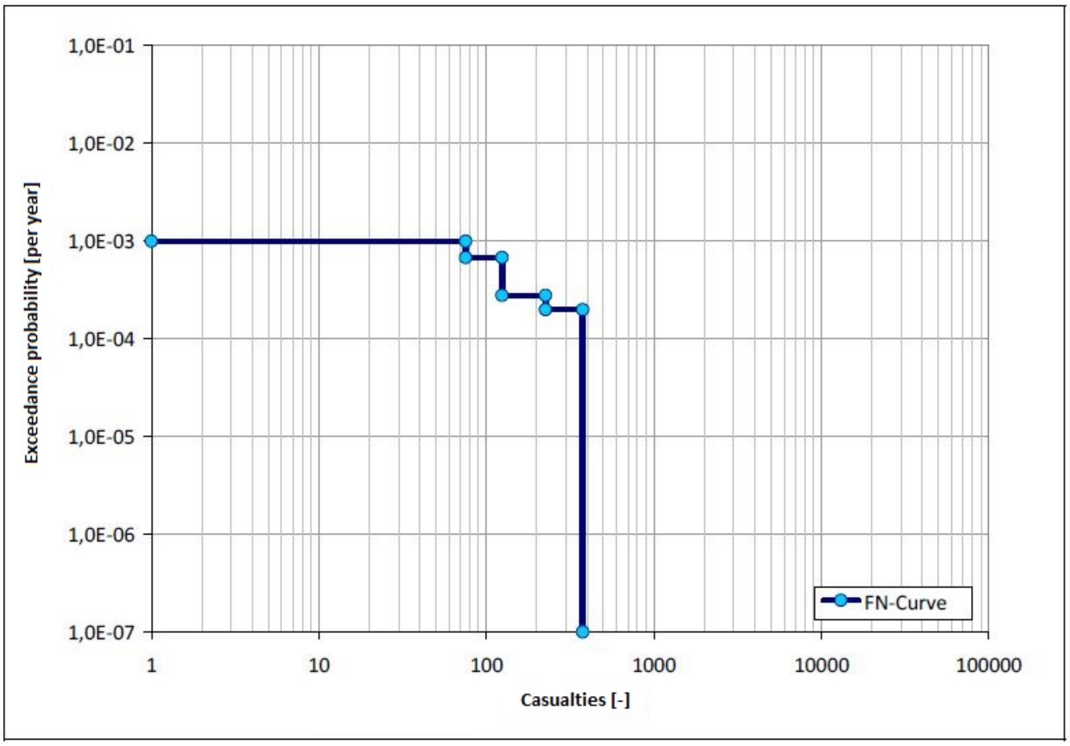


Figure 5.13: FN-curve for the expected number of casualties in dike ring area 7 (source: [14])

into account) is 1/1,000 per year. This leads to a total risk of 4.5 million euro per year.

When looking at the regional economic risk the biggest values of economic risk (per acre) are found in the residential areas (except the center of Urk, as it lies above NAP).

The risk of casualties is estimated to be **0.17 casualties per year**. This value is found if the expected number of casualties (i.e. 166.5) during a flood is multiplied with the probability. Again the regional economic risks is the highest at the residential areas (except the center of Urk).

5.6.2. LONG TERM AND SHORT TERM RISKS

If the probability of failure of the Afsluitdijk is taken into account, the risks of flooding of the Noordoostpolder can be expressed in two ways. The first option is calculating the risk of flooding of the Noordoostpolder due to a second storm after a first storm damaged the Afsluitdijk. As the probability of failure of the Afsluitdijk is relatively low, the total probability of this option (i.e. the combination of a first and the second storm) is much lower than the probability of flooding for the first storm (see section 5.4.4)). The estimated risk of flooding of this option will therefore also be much lower than the risks of failure during the first storm (so if the Afsluitdijk will not fail). Taking the possibility of failure of the Afsluitdijk into account will not increase the risks of flooding of the Noordoostpolder.

This first option gives values for risks which can be used for long term policy. For questions such as: Should we increase the strength of the sluices in the Afsluitdijk? or Should the height of the dikes in the Noordoostpolder be increased? these values of risks are useful. For this 'long term' risk it can be stated that reducing the probability of failure of the Afsluitdijk will not have much effect on a reduction of the risks of flooding for the Noordoostpolder. In other words the IJsselmeer system is robust enough to provide enough safety for flooding of the Noordsoostpolder due to influences of the Waddenzee/ North sea. The low probability of failure of the Afsluitdijk, the large retention area of the IJsselmeer (high discharges cause only little average water level increase), and the long distance between the Afsluitdijk and the Noordoostpolder result in a very robust IJsselmeer system.

In another way (the second option) the an estimation of the risks can be worth investigating. That is, if the risks of a flood in the Noordoostpolder are calculated given that the Afsluitdijk already has a breach caused by a previous storm. Difference between this option and the first option previously explained is that the risks calculated in this second option are useful for short time policy where the previous option focussed on long time policy. The expression for this risk is useful for ad hoc decision making: 'Okay, so we have a breach in the Afsluitdijk, what do we do? Do we accept this extra risk of flooding of the Noordoostpolder or are we going to take short time measures to reduce the risk? Options are (among others) to increase the intensity of inspections of the dikes of the Noordoostpolder, use emergency repairs for the breach in the Afsluitdijk (see section 6.2), or evacuate the Noordoostpolder already for the rest of the winter season. One can imagine that the outcome of this consideration on how to act is not easy to make. A very useful tool for helping to make this decision is to know how high the risks are given that the Afsluitdijk has already failed.

5.6.3. RISK IF THE AFSLUITDIJK HAS FAILED

The consequences of failure are a little bit higher as in the case the Afsluitdijk has not failed. The average IJsselmeer water level is now increased as an open connection is present between the Waddenzee and the IJsselmeer. Therefore the average IJsselmeer water level in equilibrium (so days after the breach of the Noordoostpolder) will lie around NAP + 0 m. This is also the case in the maximum scenario in section 5.5. For this maximum scenario, it was assumed the discharge of the rivers IJssel and Vecht were no longer be able to be flushed out trough the Afsluitdijk to the Waddenzee for any reason. A breach in the Afsluitdijk is one of these possible reasons, therefore this maximum scenario is used as consequence for failure of the Noordoostpolder given that the Afsluitdijk has already failed.

The total expected value of the economic damage is estimated to be 5,160 million euro. Together with the probability of failure of 1/450 per year this means the economic risk are **11.4 million euro per year**. This is a factor of approximately 2.5 higher than the risks of flooding if the Afsluitdijk has not failed (yet).

The number of casualties has a spread between 85 and 430. The same calculation as at point 3 in section 5.6 is used to find the expected number of casualties.

$$
\begin{aligned}
430 * (1 - 0.80) * 0.32 &= \ 27.52 \\
430 * (1 - 0.67) * 0.40 &= \ 56.76 \\
430 * (1 - 0.40) * 0.08 &= \ 20.64 \\
430 * (1 - 0.00) * 0.20 &= \underline{\ 86.00 \quad +} \\
\text{Expected number of casualties} &= 190.92
\end{aligned}
$$

The risks of casualties due to flooding is then calculated (again using equation (5.7)) to be **0.42 casualties per year**. This is also a factor of 2.5 higher than the risks of casualties due to flooding if the Afsluitdijk is still intact.

5.7. CONCLUSIONS CHAPTER 5

In this chapter the effects of the increase in IJsselmeer water level due to failure of the Afsluitdijk on the safety of the IJsselmeer dikes is answered. In the section 5.1 it was stated that the increases in average IJsselmeer water level as found in chapter 4 can not simply be used when investigating the increased amount of risks the IJsselmeer dikes receive due to failure of the Afsluitdijk. The results of chapter 4 show the water level increases at the end of the storm. At that time the wind speeds will be very low and local water set up due to the effect of wind will consequently also be very low. This will lead to lower local water levels than during the storm.

5.7.1. THE FIRST STORM

To find at what time during the storm the highest local water levels occur the local water set up due to wind and due to the breach in the Afsluitdijk is calculated for every hour after the peak of the storm. In section 5.1 it was found that for every kind of storm and every scenario the highest local water levels (at the Noordoostpolder) were found during the peak of the storm. Thus when the breach in the Afsluitdijk just starts to develop and consequently no inflow of water has occurred yet. After the peak of the storm, former Waddenzee water will start influencing the local water level. But as the wind set up decreases at a much faster rate than the increase of average IJsselmeer water level, the local water level will only decrease after the peak of the storm. From this it can be concluded that:

The IJsselmeer water level increase due to failure of the Afsluitdijk during an extreme storm has no effect on the safety of the dikes around the IJsselmeer during that same storm.

That the highest local water level can be found at the peak of the storm can be explained by the dominance of the wind set up over the increase in water level due to the breach. Wind set up is relatively high because of the long fetch length and shallow water in the IJsselmeer. Increase in water level due to the breach is relatively low because of the large retention area of the Afsluitdijk, the long time it takes to develop the breach (in case of a levee breach), and the time it takes for water from the Waddenzee to reach the other side of the IJsselmeer. In other words it can be stated that:

The whole IJsselmeer system is robust enough to provide enough safety for the dikes of the Noordoostpolder during a storm at which the Afsluitdijk fails.

5.7.2. THE SECOND STORM

In section 5.2 the effects of a second storm hitting the IJsselmeer was investigated. For this second storm it was assumed a first storm had already hit the Afsluitdijk that winter season which caused a breach in the Afsluitdijk. In this event (i.e. a second storm following an extreme first storm), discharges through the breach start already in the hours before the peak of the storm (1) and as the breach is already fully developed the discharges will only be limited by the Waddenzee water level (2). Other effect of this first storm is that the initial average water level of the IJsselmeer is increased as an average water level of NAP -0.4 m can no longer be maintained due to the open connection between Waddenzee and IJsselmeer (3). These three factors increase the importance of the water set up due to the breach in respect to the wind set up. It was shown that for storms more frequent than approximately 1/10,000 per year the highest local water levels are found after the peak of the storm. More extreme (i.e. less frequent) storms still have the highest water levels at the peak of the storm.

By using the PC-Ring software it is calculated in section 5.4.2 that the current probability of failure of the dike ring area protecting the Noordoostpolder (not taking failure of the Afsluitdijk into account) is 1/1,000 per year. It was found that for this dike ring area the failure mechanism overflow and wave overtopping is the dominant failure mechanism and the weakest dike ring elements can be found on the West side of the Noordoostpolder.

Given that the Afsluitdijk already suffered from an 1/250 storm and breach scenario 1 or 4 had occurred, the total probability of failure of the dike ring area of the Noordoostpolder is increased to 1/450 per year (see section 5.4.3). This is a factor 2.2 less safe than the probability of failure given the Afsluitdijk is still intact at the beginning of (any) storm.

However, a governing second storm can only occur if a first storm, strong enough to cause a breach in the Afsluitdijk, has occurred. As this first storm has at least a probability of 1/140 per year (for scenario 2 and 3, 1/250 per year for scenario 1 or 4) the combination of two storms lead to a much lower risk than the risks at failure of the Noordoostpolder at its original probability of failure of 1/1,000 per year. This means that:

Even if the possibility of multiple storms are taken into account the probability of flooding of the Noordoostpolder do not increase.

5.7.3. RISK GIVEN THAT A FIRST STORM HAS ALREADY OCCURRED

In the case that the Afsluitdijk does not fail section 5.6 estimates that the economic risks due to flooding is 4.5 million euro per year and 0.17 casualties per year. This is the current risk of flooding for the Noordoostpolder with the current probability of failure of 1/1,000. Because the probability of failure does not increase if failure of the Afsluitdijk is taken into account these risks will stay the same.

In the case the Afsluitdijk has already failed the risks do increase. However, this is a conditional risk. This risk only occurs once an 1/250 storm has already hit the Afsluitdijk. When looking at the long term the risks do not increase (as flooding given that an 1/250 per year storm has already hit the Afsluitdijk is very unlikely). For short time decisions (decision making if the breach has been developed in the previous storm) this conditional risk can be an useful tool. It is found that given the Afsluitdijk already has a (1/250 per year and scenario 1 or 4) breach the economic risks is estimated at 11.4 million euro per year and 0.42 casualties per year. This comes down to risk of a negligible amount of casualties per day and a risk of roughly 62,500 euro per day [11] which is also relatively low.

Note: These values are calculated for a first storm with a frequency of 1/250 per year. If the first storm is more extreme, the risks of not repairing increases. So in the case of a breach in the Afsluitdijk the calculations in this chapter should be executed again with the breach dimensions corresponding with the magnitude of the occurred storm.

[11] The probabilities of failure are expressed in probability per year. As the extreme storms are assumed to occur only during the winter half year the risks per day is 365/2 times lower than the risks per year.

III

Part III: Discussion

6

Chapter 6: Discussion of the results

Chapter 5 finished with some important conclusions. It was stated that the IJsselmeer system was robust enough to provide enough safety against flooding due to influences from the Waddenzee. Chapter 5 also showed that even if a breach is formed in the Afsluitdijk, the risks of flooding of the Noordoostpolder only increases with a factor of 2.5.

The unexpected strong positive effect of the whole IJsselmeer system on the reduction in risks of flooding, due to effects on the Waddenzee, strengthens the debate on the required strength of the Afsluitdijk. As it is found that the Afsluitdijk does not provide much extra safety to the dikes of the Noordoostpolder, increasing the strength of the Afsluitdijk to withstand an 1/10,000 per year storm seems not very necessary. However, the necessity of improvements can not be argued solely on the conclusions of previous chapters. There are multiple reasons for this.

Two of the most important ones are the fact that the Afsluitdijk also has other functions and a complete cost benefit analysis should be done in which the possibilities of repairs should also be taken into account. To place the drawn conclusions into perspective, these aspects are treated briefly below in respectively section 6.1 and section 6.2.

6.1. Other functions of the Afsluitdijk

This thesis focuses on the probability of failure of the Afsluitdijk in respect to its ability to protect the hinterland against floods. Failure of the Afsluitdijk was not seen as a problem as long as the dikes surrounding the IJsselmeer did not get an increase in probability of failure. Because of the focus of this thesis (which is to get more insight into flooding probabilities), it is logical the other functions of the Afsluitdijk and the IJsselmeer were left out of consideration. Only briefly, in section 2.2.4, other functions of the Zuiderzeeworks were addressed. All of these other functions are impacted heavily if a breach is formed in the Afsluitdijk. To give an idea about this impact, a few of the impacts are discussed below.

6.1.1. Salt intrusion

Starting with the most important secondary function of the IJsselmeer system: its water retaining function. As was stated in section 2.2.4, its function to separate salt water from fresh water is very important for the Northern part of the Netherlands. The amount of salinisation is an important parameter to determine whether or not it can be used for drinking water or agricultural purposes. At the moment, the IJsselmeer system already experiences the negative effects of salinisation due to salt intrusion at the sluices. In the dry summer and autumn of 2003, Chloride-values of 180 mg/L were found in the IJsselmeer which is higher than the prescribed norm of a maximum concentration of 150 mg/L Chloride in drinking water [51]. As it is a difficult process to remove Chloride out of water, this is a serious issue.

Multiple studies have been done about Waddenzee water overtopping the Afsluitdijk. These studies were done to see the effects of allowing water overtopping over the Afsluitdijk. One of these studies showed that if overtopping occurs at an 1/300 per year storm, Chloride concentrations will never reach higher than 150 mg/L near Andijk (where drinking water is retrieved for Noord-Holland) and will get to concentrations of 250-300 mg/L near Makkum, Stavoren, and Lemmer (where water is retrieved for agriculture in Friesland) [52]. Thus, for only overtopping, the amount of salt in the IJsselmeer is close to the acceptable limits.

Once a breach is formed, it depends on wind speeds, wind directions, and discharge of the river IJssel how fast the amount of salt intrusion will be. However, the average amount of Chloride in the Waddenzee lies around roughly 30,000 mg/L [53]. And as the IJsselmeer system already has trouble keeping the amount of Chloride low an open connection with the Waddenzee will have disastrous consequences. Although the average retention time of water in the IJsselmeer is 5 months in the Summer and around 3 to 4 months in the Winter [54], a complete flush out of the Chloride will take much longer. Back in 1932 it took a total of approximately 5 year after closing to turn the IJsselmeer from a salt water lake to a fresh water lake [55].

As 30% of the Netherlands is dependent of the IJsselmeer system for their fresh water source (see section 2.2.4) the effect of this amount of salt intrusion might be even more disastrous than the increase in the probability of flooding of the polders.

6.1.2. ECOLOGY

For the (other aspects of) ecology, the effects of a breach in the Afsluitdijk are not necessarily negative. At the moment there are multiple plans to improve the quality of nature and ecology by creating a fish migration channel through the Afsluitdijk or even open the Afsluitdijk permanently to create a brackish water basin [56] [57]. Multiple environmental organisations like the Waddenvereniging plead for an open connection between the Waddenzee and the IJsselmeer. In this way a natural balance will be found to exchange sea water, fresh water, sediment, and aquatic animals.

In ecological perspective, a breach in the Afsluitdijk could turn into a desirable situation. According to Guus Kruitwagen, from Witteveen + Bos, there are many diadromous fish species that are currently travelling through the sluices in the Afsluitdijk (personal communication, 21-11-2014). Examples are the eel and the salmon. These species need both fresh and salt water. However, as it is tried to minimize the amount of salt intrusion through the sluices, the sluices are only opened if the water level difference (between Waddenzee and IJsselmeer) is at least 10 cm. This leads to relatively high flow speeds. Most fish need low currents (approximately < 1 m/s) and therefore the time window of the fish to travel through the Afsluitdijk is currently very small. In case of an open Afsluitdijk this time window will be much larger and thus the fish will benefit from the breach.

Also the sudden transition from fresh to salt water is a problem at the moment. If the sluices open to let fresh water out, many fresh water fish are swimming through the sluices. Once the sluices close, the fish can not go back and they die because of the high concentration of salt in the Waddenzee. If the transition is more gradually, which is the case in an open connection, fresh water fish can turn around back to the river IJssel once they notice higher concentrations of salt in the water.

When looking at the ecosystem of the IJsselmeer an open Afsluitdijk will be much more favourable than the current situation. However, because of safety reasons a breach will be repaired at last during the following summer. In this way the original situation will be achieved again and thus the balance gets disturbed temporally.

6.1.3. MOBILITY AND RECREATION

Two other functions presented in section 2.2.4 are mobility and recreation. Especially the mobility function also experiences the negative effects of a breach in the Afsluitdijk. A breach in the levee will cause also a gap in the road and therefore blocking the highway A7, connecting Den Oever with Friesland.

6.2. REPAIRING A BREACH

For all of the conclusions stated in this thesis it is assumed that once a breach develops during a storm the breach will be repaired in the summer months. Executing such repairs is not an easy task and this difficulty should be kept in mind.

Closing the last part of a dam is very difficult and therefore costly and thus the amount of times a breach needs to be closed and the intensity of the repair works should be kept to an absolute minimum. The costs for repairing the breach should also be taken into account when considering the required probability of failure of the Afsluitdijk. Not doing any improvements, thus neglecting the Afsluitdijk, will lead to higher probabilities of failure of the Afsluitdijk and thus to a higher probability that repairs are needed.

6.2.1. CLOSURE IN THE SUMMER

Closing the final part of a sea dike or dam is difficult because of the high flow velocities found at the place that needs to be closed (see also the picture on the cover of this report). The narrower the final gap gets the higher the flow velocities will be. In the Waddenzee the water level can range from NAP -1.0 m during low tide to NAP +1.0 m during high tide. These tidal differences lead to flow velocities that can erode the construction material right after it is placed leading to high losses of construction material. Most of the time the final stages of the closure are timed between high and low tide at which time the water difference between the two sides are minimal. Also it is preferable to construct the final part during good weather conditions (especially low wind speeds are needed). As the chance on low wind speeds is higher during summer constructing repairs is ideally done during this period.

Closing the dam is not only difficult because of the high flow velocities. Also getting the material and equipment at the right place is a difficult job. For this, ships can be used (as was done in 1932 at the Afsluitdijk), but at the location of the dam low water depths are found (because of the foundation of the dam) which means only small and light ships can be used. Other option is to use equipment and supply from land, so on the dam itself.

For construction material, clay and stones can be combined with mattresses to increase the stability of the new dam. By stacking up mattresses made of willow faggots or synthetic material, closure is realized by successively dropping the mattresses onto each other. In between, the mattresses are ballasted by the stones or clay.

Another option, not using stones or clay, is sudden closure. The modern type of doing this is to make use of caissons. These caissons are constructed somewhere else and then are sailed to the gap where it will be sunk down. Constructing these caissons might require a lot of time and therefore other solutions might be needed. In case of an emergency, this could be the sinking of vessels and surrounding them by clay or stones to prevent scour around the vessel. Sinking of vessels was (some of) the old way(s) of closing the final gap in a dam. This could also be an emergency repair measure (see also section 6.2.2).

Sources used in this section: [58] and [59]

6.2.2. EMERGENCY CLOSURE

Emergency closure of a breach in a dike or dam is characterized by improvisation. Emergency closures should be executed within days or even hours. The time to act is extremely low and thus there is no time to make any designs or fully developed plans. The idea is that a quick closure prevents escalation of the conditions like the breach growth in Walcheren. Also for the Afsluitdijk, a quick closure of a breach might prevent much of the water flowing in if the breach is closed during the storm. However, it is expected such repairs can only be executed in the later hours of the storm at which (it is assumed) the breach is already fully developed. During the peak of the storm wind speeds and wave heights will be too high to safely manoeuvre with ships and equipment. Especially at the Afsluitdijk repairing during the storm is not a good option as ships might strand, equipment can be washed of the Afsluitdijk, or people can get enclosed by the formation of multiple gaps as there is no hinterland to escape to (which is available at sea dikes protecting the land).

For the Afsluitdijk emergency repairs are mostly suitable to reduce the risks of flooding around the IJsselmeer after the storm. If a closure is realised in the first days after the storm, the average IJsselmeer water level can quickly recover to its old target level and thus reduce the probability of failure of the IJsselmeer dikes. Also the amount of salinisation can be limited in this way. This will reduce the economic damage of a breach in the Afsluitdijk as well.

For this, a simple cost benefit analysis can be made. If the costs of a repair of the breach in the Afsluitdijk are lower than the increased risks of flooding for all the dike ring areas around the IJsselmeer the benefits of emergency repairing the Afsluitdijk outweigh the cost of executing these repairs.

As no fully developed plans will be made for emergency closures, these closures have a relatively high risk of failure (for instance if a ship is used as temporally obstruction, water can easily flow around the ships hull which might lead to increased flow velocities and increased local scour of the dam, see figure 6.1). Also the repairs most likely still need to be strengthened during the summer. Both the high risks of failure and need of improvement later on should also be taken into the cost aspect of the cost benefit analysis.

Figure 6.1: Emergency repairs for a near dike breach in Ouderkerk aan den IJssel in 1953 (source: [59]

On the other side, the benefits of emergency repairs should also include the benefits for the other functions (see section 6.1). Especially if the amount of salinisation can be prevented to acceptable levels (i.e. to still be able to use the water from the IJsselmeer as drinking water) the benefits can be huge. And, of course the reduction in risks for the other dike ring areas are also needed in the analysis.

As the effects of salinisation during a breach (and the development of salinisation) are not within the scope of this thesis, a reliable cost benefit analysis can not be made. As the risks of flooding are relatively low (see section 5.7.3) the damage due to salt intrusion through the breach is expected to be a relatively big contributor to the total estimation of the benefits. In order to say something about executing emergency repairs or not these effects should be quantified. It is also advisable to do this for different dimensions of the breach as this will influence the exchange of water between Waddenzee and IJsselmeer. The effect of these different dimensions on the increased risks of flooding during a second storm must also be calculated in order to arrive at a reasonable cost benefit analysis.

Sources used in this section: [58] and [59]

7

Chapter 7: Conclusions and Recommendations

This chapter contains the conclusions (section 7.1) and recommendations (section 7.2) that result from the research that has been done in this thesis. The conclusions in perspective to literature are presented in appendix R.

7.1. Conclusions

The conclusions of this thesis are organized by chapter (and thus by research question). Chapter 2 gave a summary of the IJsselmeer system. The conclusions on the other three research questions are presented below in section 7.1.1, 7.1.2, and 7.1.3. The three most important conclusions are presented in section 7.1.4.

7.1.1. Possible breach scenarios

Chapter 3 treated the second research question: *What possible scenarios of breaching of the Afsluitdijk can be expected?* The chapter led to the following conclusions:

- Both the sluices and the levee itself are prone to failure. The Afsluitdijk can fail at either (or both) of these locations.
- In the levee, a boulder clay layer is present. It is difficult to predict the retardant effect of this boulder clay layer on the development of the breach once the Afsluitdijk fails at its levee.
- The probability of failure of the Afsluitdijk is much higher than the standard of 10 % failure in an 1/10,000 per year storm event. This thesis takes a probability of failure of 1/250 per year for the sluices and 1/140 per year for failure at the levee.

Together with other important characteristics, which were based on literature, this led to 4 different breach scenarios presented in table 7.1.

7.1.2. Influence on the IJsselmeer

The third research question: *What is the influence of a breach in the Afsluitdijk on the IJsselmeer and what kind of water level increases can be expected?* is answered in chapter 4. The following conclusions were found:

- The water level in the IJsselmeer (both local and average water level) does not have a significant influence on the discharge through a breach in the Afsluitdijk. This means the amount of discharge is mostly affected by the dimensions of the breach and the water level in the Waddenzee.

- By means of a Matlab model, using the probability density function of the Waddenzee water level, and using the 4 different breach scenarios, exceedance frequency curves for each scenario were found. These exceedance frequency curves show a continues relation between frequency and increased water level of the IJsselmeer due to the effects of a breach. Table 7.2 shows the results of the 4 scenarios for some interesting return periods.

Characteristic	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Location of failure:	Sluices	Levee	Levee	Sluices **and** levee
Probability of failure	1 / 250 per year	1 / 140 per year	1 / 140 per year	1 / 250 per year
Width of the breach (at an 1/10,000 storm)	336 meter	380 meter	1300 meter	336 / 300 meter
Depth of the breach	NAP -4.40 meter	NAP + 2.00 meter	NAP -0.40 meter	NAP -4.40 m / 0 m
Development breach (from 0 to full breach)	Immediately	Linear during 12 hours		Immediately / Linear during 12 h
Timing of the breach	During the peak of the storm			
Duration storm	30 more hours after breach			

Table 7.1: The 4 scenarios used in this thesis

Return period [years]	Water level increase [m]			
	Scenario 1	Scenario 2	Scenario 3	Scenario 4
100	0	0	0	0
1,000	0.34	0	0.31	0.38
10,000	0.37	0.01	0.71	0.49
100,000	0.39	0.03	1.15	0.61

Table 7.2: Results of the Matlab simulation for all scenarios for some of the frequencies.

- If the boulder clay layer will not erode away during the breach process, a levee breach in the Afsluitdijk will hardly have any effect on the IJsselmeer.
- In the case of a breach at the sluices an abrupt change in the amount of discharge is found at the probability of failure of the sluices. This leads to relatively high discharges at relatively low return periods (roughly between 1/250 and 1/1,000 per year storms).

7.1.3. INFLUENCE ON THE NOORDOOSTPOLDER

The fourth research question was investigated in chapter 5: *What is the effect of this increase of the IJsselmeer water level on the safety of the IJsselmeer dikes?*

- The effect of wind set up is far more dominant than the effect of water increases due to a breach in the Afsluitdijk in an extreme storm at which the Afsluitdijk fails. It is shown that the governing conditions occur during the peak of the storm when no Waddenzee water has flown in yet. Therefore it can be stated that: *The IJsselmeer water level increase due to failure of the Afsluitdijk during an extreme storm, has no effect on the safety of the dikes around the IJsselmeer during that same storm.*
- The effect of a breach in the Afsluitdijk on the probability of failure of the dikes of the Noordoostpolder is negligible is because of the following reasons:
  - The retention area of the IJsselmeer is relatively large compared to the dimensions of a possible breach in the Afsluitdijk (it requires much water inflow to achieve minor water level increases).
  - In the case of a levee breach it takes a long time for the breach to grow to significant dimensions (at which time the peak storm event has already passed).
  - The large distance between the Afsluitdijk and the Noordoostpolder (approximately 50 km) results in a delay in Waddenzee water effecting the local water conditions at the Noordoostpolder.

7.1.4. FINAL CONCLUSIONS

The four most important conclusions that together answer the main question: *What are the consequences of failure of the Afsluitdijk on the IJsselmeer and its surrounding dike ring areas?* are stated below.

- The current probability of failure of the Afsluitdijk is much higher than the current governing standard.
- Although the current probability of failure is low, the whole IJsselmeer system is robust enough to provide enough safety for the dikes of the Noordoostpolder during a storm at which the Afsluitdijk fails.
- In case multiple extreme storms occur during one winter half year, a breach in the Afsluitdijk does effect the probability of failure of the dike of the Noordoostpolder. However, as the probability of a combination of multiple extreme storms is very low, the risk of flooding of the Noordoostpolder do not increase.
- If this conditionality is not taken into account (useful for ad hoc decision making) the risk does increase. Given that an 1/250 per year storm has already hit the Afsluitdijk and given that this storm caused a complete blow out of all the sluices in the Afsluitdijk, the risk of flooding of the Noordoostpolder increases with a factor of 2.5 as long as this breach is not repaired.

7.2. RECOMMENDATIONS

The following recommendations result from this research:

- The retardant effect of *the boulder clay layer* in the Afsluitdijk should be investigated. This should be done together with a detailed investigation on the development of the water flow velocities through the breach in the mid-to-later hours of the storm. If the boulder clay layer has any effect on the prevention of erosion of the levee breach (as is expected in [28] and [13]) this increases the likelihood of scenario 2.
- Later on in the research (section 5.1) it was found that *the effects of wind* are very important for the risk of flooding around the IJsselmeer. Relatively rough assumptions and simplifications are used to model the effects of wind on the local water level. These assumptions are validated in appendix I and it is shown that the simplifications slightly overestimate measured data. In order to improve the findings of this thesis, the effects of wind on water set up and waves should be modeled with greater accuracy. Especially unsteady responses (e.g. resonance) of the water level to wind are of interest as they show even more extreme results in the measurements than the steady responses.
- *The adjusted probability density curves for the wind statistics* used as input for PC-Ring should be improved. Although great care is taken in this process of letting PC-Ring work with the most governing conditions, adjusting these input files are done by manually adding water level increases and adjusting the probability density curves for the wind speeds. This process could be improved.
- For the effects of a breach in the Afsluitijk on the dikes around the IJsselmeer the location of the Noordoostpolder was chosen in section 5.3. It was assumed a breach in the Afsluitdijk has the biggest influence on the Noordoostpolder. It could be that *the risk of flooding for the dikes at other locations around the IJsselmeer* increase more than the risk of flooding for the Noordoostpolder as other wind directions are significantly affecting the probability of failure of these dikes.
- The numbers found in section 5.6.3 are calculated by assuming *the magnitude of the first storm* corresponds with an 1/250 per year storm. The risk given a storm has already hit the Afsluitdijk are much higher if this first storm is much stronger. In case a cost benefit analysis is carried out on whether emergency repairs are needed (see also section 6.2), the correct breach dimensions should be used. Therefore it is recommended that these calculations should also be done for different kind of dimensions of the breach.
- Although the Afsluitdijk is robust and protects against flooding from the Waddenzee, even if a breach has formed, a breach in the Afsluitdijk causes damage to *the other functions of the Afsluitdijk* (see section 6.1). It is recommended this aspect should be analysed further. Especially the effects of the salt Waddenzee water intruding the fresh IJsselmeer water can have very big consequences for the fresh water storage and ecological function of the IJsselmeer.
- The expected numbers for probability of failure, consequences, and risks are indications of the real probabilities of failure, consequences, and risks. As many assumptions and simplifications are made in this thesis, the final values of (for instance) the dimensions of a breach and the increases in water level during extreme storms are not accurate. A summary of the important assumptions used in this thesis is given in appendix Q.

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IV

Part IV: Appendices

A

FAILURE MECHANISMS OF DIKES

A dike or dam can fail at different failure mechanisms, a few of these are important in this thesis (see section 1.1 and 5.4.1). Figure A.1 shows the failure mechanisms as shown in [7]. It presents the most important failure mechanisms to dikes.

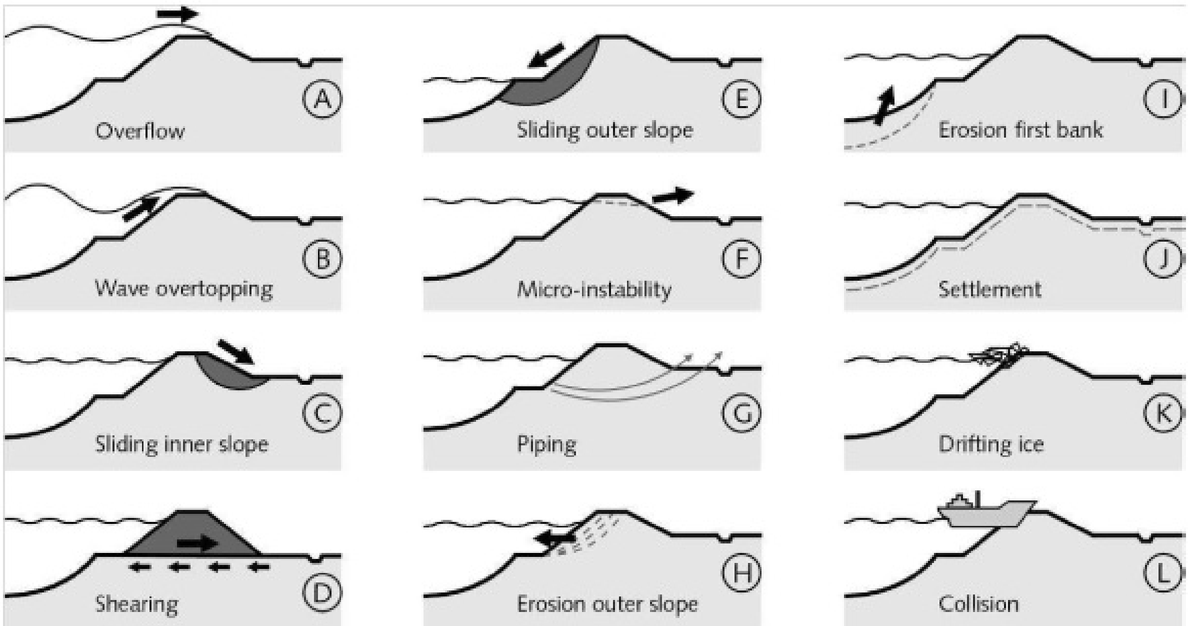


Figure A.1: Failure mechanisms

Below, the failure mechanisms are explained. Not all of the 12 failure mechanisms presented in figure A.1 are currently in the Dutch regulations [16]. Horizontal shearing, ice drift, collisions and other external factors, and heave and bursting (not in the figure) are not found in the regulations. However, a designer/constructor of the dike needs to take them into account.

A.1. (WAVE) OVERFLOW

The main function of the water defence line is to retain water. Water flowing over the dike by either a too high water level or too high waves (or run up) is the first obvious failure mechanisms for a dike (A and B in the figure). However, almost always some discharge is allowed as long as the stability of the dike is safe enough. This depends on the protection against eroding of the crest and the inner part of the dike. The amount of water the inner slope can withstand is defined as a limit state. Run-over and wave overflow are not 'real' failure mechanism. They are causes of dike failure as they lead to failure of the inner slope (see for in depth explanation section B.1.1).

A.2. INSTABILITY OF INNER SLOPE

Whenever wave overflow occurs, the top layer of the inner slope of the dike will get infiltrated with water (C). This results in higher water pressures and thus in a lower effective ground pressure resulting in low resistance against shearing. Also the weight of the top layer will increase and thus creating a larger load on the dike. Both effects (reduction in the resistance and the increase of the load) have a negative impact on the stability of the dike. This instability will lead to (horizontal) cracks in the top layer, deformation, and erosion. Crack formation will increase the rate of infiltration and thus increasing the speed of the failure.

A.3. HORIZONTAL SLIDING

A special case of macro stability failure is horizontal sliding (D). In this mechanism the complete dike is pushed aside by the water pressure. No regulations for safety are used in the Netherlands as this failure mechanism is not seen as a governing mechanism [16]. However, past levee breaches in Zoetermeer (1947), Oostzaan (1960), Bleiswijk (1990), and (probably the most famous) Wilnis (2003) show that horizontal translation can be a failure mechanism [8].

A.4. SLIDING OUTER AND INNER SLOPE DUE TO MACRO INSTABILITY

Due to macro instability large parts of the slopes are slid away (E). It means failure of the levee body and failure of the soil layers underneath the dike. As figure A.1 shows, the section of soil that moves during the unstable situation can be simplified as a sliding circle. A high water level outside will lead to an increase of the phreatic water level inside the levee and an increase of the water pressures inside. Again this results in low resistance against sliding. The crest of the dike will collapse and the bottom of the levee will move upwards (or horizontal if a ditch is present, filling the ditch).

Macro instability of the inner side can occur with extreme high water levels and macro instability of the outer side can occur when the water in front of the dike drops very fast.

A.5. MICRO-INSTABILITY

Micro-instability is instability due to washing out of the dike core sediment (F). The threat comes from within the dike due to a high phreatic water level. In a sand dike, washing out of sand is most likely to happen at the toe of the dike as the phreatic water level can grow higher than the levee itself. The connection of the phreatic line inside the dike body with the surface of the inner slope, is considered the limit state of this mechanism [16]. A cover of clay can prevent the outwash of sand. However, when water pressures get high enough, the clay cover can burst open or it can be pushed away. A levee consisting of only clay doesn't have this problem as groundwater flow in clay is very low. Difference between micro-instability and instability of the inner slope due to infiltration and wave overtopping is the direction of the threat. At micro instability the water seeps from the core of the levee trough the inner slope and at infiltration (due to, for instance, wave overtopping) the water infiltrates from outside.

A.6. PIPING

Piping is the failure mechanism of wash out of sand underneath the dike due to the formation of 'pipes' (G). When long lasting high water occurs, the difference in water level on the outer and on the inner side of the dike will create a high water head difference. This water head difference will increase the speed of the ground water flow. When the seepage velocity is high enough erosion can occur. The seeping water removes soil, starting from the exit point of the seepage, and erosion advances to the river/sea side of the dike. The appearance of piping can be noticed because the sand is 'boiled' out of the ground creating a 'sand boil'. If the pipes get big enough the dike will collapse due to lack of support of its foundation.

A.7. HEAVE AND BURSTING

Heave is lifting of and liquefaction of the inner sand layer behind the dike by vertical groundwater flow upwards. Bursting is the forcing up of the top clay layer behind a dike by high pore water pressures in the sand layer below the clay. Both are results of high water pressures behind the dike due to high gradients in the ground water. Whether this results in heave or bursting depends on the difference of the soil type on the inside of the dike.

A.8. INSTABILITY OF OUTER AND INNER SLOPE PROTECTION

The protection of the outer slope, bank, crest, and inner slope provides protection against erosion of the levee. Protections on the outer slope can fail due to wave attacks, currents (tides, waves, river discharge, etc.), or high (static) water pressures. Protections on the inner slope of the levee can fail due to discharge over the dike (H). After the protection fails the waves and currents can attack the levee directly.

A.9. INSTABILITY OF THE SHORE LINE

If the soil in front of the dike consists of weak clay, peat, or loose sand, the soil can become unstable and sliding or compaction of the soil can occur which might have an impact on the stability of the foundation of the levee (I).

A.10. SETTLEMENTS

Settlement of the dike can reduce the retaining height of the dike (J). Especially on clay or peat layers settlements can be significant. Abroad, this failure mechanism is a major problem as extraction of ground water causes settlements much higher than the expected sea level rise (due to climate change). The deformations of constructions based on geotechnical processes can happen very fast (seconds) or very slow like creep and consolidation processes. For current designs in the Netherlands, the limit state for this phenomenon is the deformation during 50 years.

A.11. ICE DRIFT

Ice drift used to be a big problem in the past [67] (K). Ice affected the dike by physically hitting the dikes and by blocking the river causing higher water tables. Because of the salinity of sea water and increasing temperatures of the river waters, ice drift is no longer a big problem in the Netherlands.

A.12. EXTERNAL FACTORS

External factors can be caused by humans or animals. Human actions include, for example, bombing (war/terrorism), ship collision or leakage of water pipelines (L). Animals can also harm the stability of dikes. They include, for example, rats creating tunnels and cattle farming on grass revetments. No strict regulations for dikes are found for dealing with these external factors. However, these factors can be an important cause of dike failure and therefore should be taken into account when designing a dike.

A.13. RECOMMENDED OTHER LITERATURE

Recommended other literature on the different failure mechanisms of dikes is presented in table A.1

Year	Title	Author(s)
2007	VTV2006, Voorschrift Toetsen op Veiligheid primaire waterkeringen	Minsterie van Verkeer en Waterstaat
2007	HR2006, Hydraulische Randvoorwaarden voor het toetsen van primaire waterkeringen	Minsterie van Verkeer en Waterstaat
2007	Failure Mechanisms for Flood Defence Structures	FLOODsite
2007	Failure Mechanisms of sea dikes - inventory and sensitivity analysis	Mai Van, C.; Van Gelder, P.H.A.J.M.; Vrijling, J.K.
2008	De dijk van de toekomst? Quick scan doorbraakvrijde dijken	Silva,W.; Velzen,E. van
2014	Wave overtopping resilient Afsluitdijk	Landa, P.M.
2014	On reducing piping uncertainties. A bayesian decision approach	Schweckendiek, T.

Table A.1: Recommended other literature

B
DESCRIPTION OF PC-RING

As was described in section 1.3 and in chapter 5, the model PC-Ring is used for investigating the amount of reduced safety of the IJsselmeer dikes. This section explains the model background of the software PC-Ring. An example of a failure mechanism is shown and the statistical models and mathematical techniques are explained.

The PC-Ring model is a model that uses a probabilistic method to calculate the probability of failure of a complete dike ring area. In PC-Ring, probabilities of failure can be calculated for dikes, dunes, and structures. First, for each failure mechanism a limit state function is formed to calculate the failure probability of one single failure mechanism on one single part of the dike ring. Then, all probability of failures of all failure mechanisms for one single part of the dike ring are taken together. This leads to one single probability of failure for one part of the dike ring. Lastly, all probability of failures of all parts of the dike ring are taken together to arrive at one probability of failure.

This method of first defining all single probability of failures gives the positive side effect that it can easily be seen where the weak spots in the dike ring are found. This is useful for effectively improving the safety of the dike ring area.

A complete description of the PC-Ring model is given in a series of reports. These reports can be divided into three main catergories:

- **The user manual** This manual is used to understand the possibilities of the program and how to use it. Calculation possibilities, how to put in input files, how dike ring areas are schematized, and a couple of examples are treated here.
- **The theoretical manual** The theoretical manual explains the theoretical idea behind PC-Ring.
- **The programming manual** In this manual information is given about the construction of the software. This manual is designed for programmers who would like to adjust and improve the program. This manual is not of relevance to this thesis.

The second manual, the theoretical manual, is the most relevant and the most interesting for this thesis as it gives the theoretical background of the model. The theoretical manual consists of three parts. Part A: Mechanism description [60], part B: Statistical models [61], and part C: Mathematical techniques [62]. On the basis of these reports the theory behind PC-Ring is described below.

B.1. MECHANISM DESCRIPTION

The failure mechanisms that are used in the calculations of PC-Ring are described in part A of the theoretical manual. The following failure mechanisms are used for calculating the total failure probability:

- Overflow/overtopping
- Sliding inner slope and outer slope
- Heave and piping
- Damage to bank protection and erosion
- Piping at structures
- Failure of closing of structures
- Dune erosion

Other failure mechanisms (see appendix A) are not seen as dominant failure mechanisms and are therefore neglected in PC-Ring.

For each failure mechanism, limit state functions are described. To get an idea about the modelling of these failure mechanisms one of these models is described in this thesis. Below a description of the model that is used to take the failure mechanism overtopping into account is described. Overtopping is chosen as it is seen as the most dominant failure mechanism for the levee of the Afsluitdijk [21, 67, 68].

B.1.1. LIMIT STATE FOR THE FAILURE MECHANISM OVERTOPPING

The limit state function is based on the fact that failure will occur if the water that is topped over the dike due to waves is too much for the crest and inside of the dike to handle. If this happens erosion will take place and a breach can be formed.

In other words. The occurring discharge over the dike should be lower than the critical discharge of the dike: $q_o < q_c$. The limit function than is described as:

$$Z = m_{qc} * q_c - m_{qo} * q_o \tag{B.1}$$

In this function Z is the limit function. If Z is lower than 0 the limit is reached and the dike fails, if Z is bigger than 0 the dike holds (= no failure). Two other factors are added to the function m_{qc} and m_{qo}. They are used as model factors for respectively q_c and q_o the express the uncertainty in the models in which the critical and the occurring discharges are determined.

The variable q_o is determined by doing a train of multiple models after each other. The trail of this train is schematized in figure B.1.

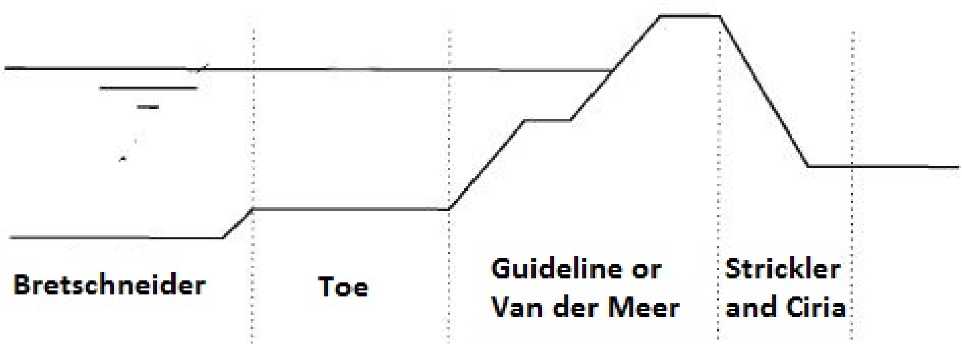


Figure B.1: Trail of the models determining occuring and critical overtopping discharge (Source: [60])

First, local water levels and boundary conditions are gathered as input (left of figure B.1). Then these water levels are translated to waves with the help of the Bretschneider wave growth model [69]. When the waves hit the toe of the dike, the toe will influence the waves. These waves will attack the dike and overtopping discharges are determined with the help of guide lines or the Van der Meer formula [70].

The right part in figure B.1 is used for determining q_c. Strickler's formula is used for the determination of the roughness coefficient [71] and Ciria formulas are used for determining the strength of the grass layer [72]. The Ciria formula is as follows:

$$q_c = \frac{{v_c}^3}{tan\ \alpha_i * C^2} \tag{B.2}$$

In this equation q_c is the critical discharge again. v_c is the critical velocity, α_i is the angle of the slope, and C is the roughness coefficient of Chézy.

PC-Ring uses the Strickler formula to determine this roughness coefficient.

$$C = 25 * (\frac{q_c}{k * v_c})^{\frac{1}{6}} \tag{B.3}$$

In this formula k is a representative value for the roughness of the slope.

Now, equation B.2 and equation B.3 can been taken together to get to the following formula:

$$q_c = \frac{v_c^{5/2} k^{1/4}}{125\ tan\ \alpha_i^{3/4}} \tag{B.4}$$

This leaves the critical flow velocity (v_c) as the only unknown. For this, an empirical formula is used that determines failure of the grass cover after a certain time t_o (in hours). For t_o the duration of the storm should be used. The empirical formula used in PC-Ring is as follows:

$$v_c = f_g * \frac{3.8}{1 + 0.8\ log\ t_o} \tag{B.5}$$

Final unknown is f_g. This variable is a measure for the quality of the grass layer (gaps, depth of the roots, etc.) and varies between 0.7 for bad grass layers to 1.4 for good quality layers.

B.2. STATISTICAL MODELS

For each failure mechanism a limit state function is described in PC-Ring. The limit state functions from previous section and the local parameters are put together with the statistical models described here. In this way for every part of the dike and for every failure mechanisms failure probabilities are calculated.

The set-up limit state functions all have variables that have an uncertainty. This uncertainty in the variables is described by a probability density function (see also appendix H). PC-Ring uses different forms of probability density functions. A few of these are the normal (Gaussian), lognormal, exponential, Gumbel, Weibull, and Pareto distributions. Together with this type of distribution (at least) an average and a standard deviation (see again appendix H) is needed to define the probability density function.

B.2.1. SPACIAL SPREAD AND SPREAD OVER TIME

Most of the variables show differences over time or over distance. If the variable is known exactly (no uncertainty) at place x and time t, it does not mean there is no uncertainty in the variable at a different place or time. However, it is likely the variable close (in both space and time) to the known variable shows more likeliness to the known variable than a variable further away (in both space and in time). For instance, if the variable 'height of crest' is known at place x_1 and t_1 it is likely the height of the crest is more or less the same 1 meter away from x_1. The height of the crest 1,000 m away from x_1 is much less likely to show this same resemblance to the known parameter.

For determining the way a variable influences the same variable at a different place a correlation function is used. This function gives the correlation coefficient as a function of the distance between two points. In PC-Ring the following correlation function is used:

$$\rho(\Delta x) = \rho_x + (1 - \rho_x) e^{-\frac{\Delta x^2}{d_x^2}} \tag{B.6}$$

In this equation $\rho(\Delta x)$ is defined as the correlation coefficient as a function of Δx. Δx is the distance between two points. ρ_x and d_x are the two new parameters used as input. ρ_x is the constant correlation and gives the 'minimum' correlation or the correlation independently of the distance. d_x is the correlation distance parameter and expresses the distance at which the known variable influences the other variables. A high correlation distance means the variables a far distance are influences by the known variable. A low correlation distance means the variable hardly influences the variables close by. Figure B.2 shows the general form of equation B.6.

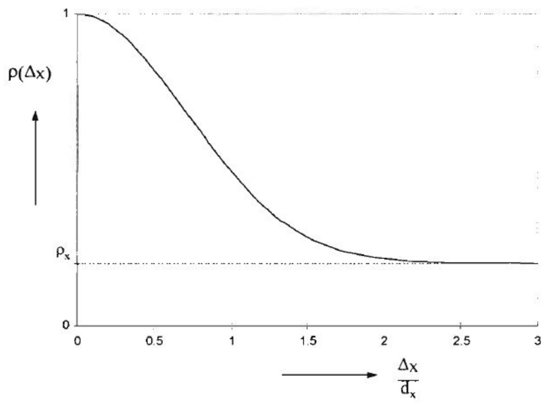


Figure B.2: General form of the distance correlation function (Source: [61])

Correlation functions for time instead of space follow a similar route. Instead of d_x, Δt is used for the time intervals.

B.2.2. VARIABLES IN PC-RING

The variables in PC ring are described by the following aspects:

- Distribution type
- Average (μ)
- Standard deviation (σ)
- Correlation function for spatial spread ($d_x and \rho_x$)
- Correlation function for spread over time ($\Delta t and \rho_x$)

For each failure mechanism in PC ring these 5 aspects (= 7 parameters) are defined for each of the variables of the limit state function corresponding to the failure mechanisms. Again only for the failure mechanism overtopping these aspects are given.

Table B.1 gives an overview of the variables needed for the limit state function of overtopping that are already programmed in PC-Ring. The uncertainty factors in the parameters f_b *and* f_n are gathered from lab results. Uncertainties in these parameters (lab to reality) are in turn taken into account in m_{qo}. The other values in the table are based on estimations.

	Discription	Type	μ	spread	d_x	ρ_x
k	Roughness inner slope	lognormal	0.015	V = 0.25	300 m	0.5
f_b	Factor for q_b	normal	5.2	σ = 0.55	-	-
f_n	Factor for q_n	normal	2.6	σ = 0.35	-	-
m_{qc}	Model factor for q_c	normal	1.0	σ = 0.50	1500 m	0.4
m_{qo}	Model factor for q_o	normal	1.0	σ = 0.50	per part	0.7

Table B.1: Variable for overtopping in PC-Ring

B.3. MATHEMATICAL TECHNIQUES

Result of previous section, statistical models, are multiple failure probabilities per parts of the dike ring (dike ring sections). Each part of the dike now has a single probability of failure for every failure mechanisms. These different failure probabilities are taken together to arrive at the total probability of failure for the whole dike ring. This compositions is described in this section.

B.3.1. FAILURE OF A SINGLE ELEMENT

The probability of failure of the Z function (Z<0) can be written formally as:

$$P(F) = P(Z(\boldsymbol{X}) < 0) \tag{B.7}$$

In this equation P is the probability, F is failure, Z is the limit state function and $\boldsymbol{X}$ is a vector which includes all the stochastic variables.

As $\boldsymbol{X}$ consists of different stochastic variables equation B.7 is not easily solved. To solve the equation multiple mathematical techniques can be used. All methods have their advantages and disadvantages. PC-Ring uses FORM-analysis (First Order Reliability Method), SORM-analysis (Second Order Reliability Method), and Directional Sampling (variant of the Monte Carlo method).

The FORM method has as advantage that it requires little computational time and that it gives a measure for the influence of each of the stochastic variable (expressed as α). This last aspect is very useful for determining the correlation factors of different limit state functions later on.

The first two letters of the abbreviation FROM stand for First Order. The method is first order because the limit state is linearised in a point that is called the Design Point. This design point is defined as the point on the line Z=0 which has the highest probability density. In other words: the conditions (=coordinates of the point) which are the most likely to occur given the event of a failure. This design point needs to be found during the FORM analysis by means of an iteration process.

In PC-Ring the stochastic variables in $\boldsymbol{X}$ are translated to standard normal distributed variables (called $\boldsymbol{u}$). These standard normal distributed variables have (by definition) a mean of 0 and a standard deviation of 1.

Variables from $\boldsymbol{X}$ that follow a normal distribution can be transformed simply:

$$X = \mu_X + u\sigma_X \tag{B.8}$$

Variables from $\boldsymbol{X}$ that follow another distribution than a normal distribution need to be transformed by using its exceedance probability function $F_X(x)$. To transform this, the exceedance probability function should be equal to the exceedance probability function of the standard normal distribution:

$$F_X(x) = \Phi(u) \tag{B.9}$$

In this equation Φ is the probability density function of the standard normal distribution.

Because of this transformation all stochastic variables are scaled the same. In case of 2 variables the interpretation of the design point can now easily be found. Figure B.3 shows that the design point can be found as the shortest distance to the origin of the u-space.

As the limit state function is linearised it can be written as follows:

$$Z_L = B + A_1u_1 + A_2u_2 + A_3u_3 + \ldots \tag{B.10}$$

In this equation Z_L is the limit state function, B and A are constants of the limit function, and their indexes give the index of the original stochastic variable X.

Because of the properties of the u-variables the reliability index β can be found as follows:

$$\beta = \frac{\mu(Z_L)}{\sigma(Z_L)} = \frac{B}{\sqrt{\sum A_i^2}} \tag{B.11}$$

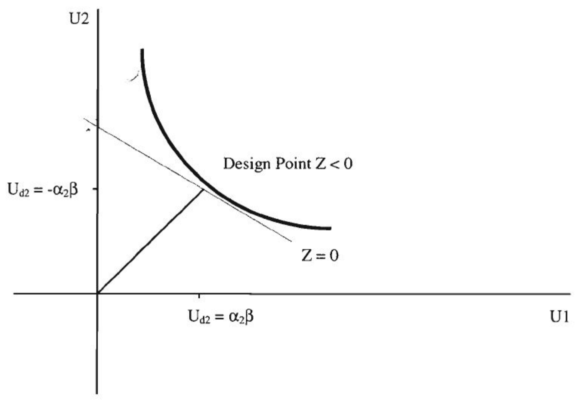


Figure B.3: Design point in u-space (Source: [62])

In this equation i is the index for the original stochastic variable again. This results in the probability of failure as:

$$P(Z<0) \approx P(Z_L<0) = \Phi(-\beta) \tag{B.12}$$

If the limit state function (see equation B.10) is divided by $\sqrt{\sum A_i^2}$ the limit state function can be written as follows:

$$Z_L = \beta + \alpha_1 u_1 + \alpha_2 u_2 + \alpha_3 u_3 + ... \tag{B.13}$$

Z_L stays Z_L as it can be multiplied/divided freely with any given positive number, as the only interesting part of Z is whether it is a positive or negative value. In this function a new variable is introduced: $\alpha_i = A_i/\sqrt{\sum A_i^2}$. The design point u_d can now be written as $u_{d,i} = -\alpha_i * \beta$ (see figure B.3 again). Variable α_i is also known as the influence coefficient and shows the relative influence of the stochastic variable with index i.

B.3.2. PROBABILITY OF FAILURE OF A SYSTEM

PC-Ring assumes that the total probability of failure can be schematized by a series-system:

$$P(F) = P(Z_1 < 0 \text{ or } Z_2 < 0 \text{ or } ...Z_m < 0) \tag{B.14}$$

In this general equation m can be the number of dike ring sections. If that is indeed the case, the total probability of failure is a combination of all the dike ring sections. To calculate this, first a FORM-analysis must be executed for all the elements $Z_i < 0$. This will lead to the reliability index β_i and influence factors $\alpha_{i,k}$ for every stochastic variable i and mechanisms k.

A first estimation of the total probability of failure can be given by a lower bound and an upper bound:

$$\max P(Z_i < 0) < P(F) < \sum_{i=1}^{m} P(Z_i < 0) \tag{B.15}$$

This means the probability of failure is at least larger than the weakest element and at most the probability of all elements together.

In a couple of cases these upper and lower bounds are lying close to each other and in that case this estimation is satisfying. Unfortunately this does not happen in most cases. Therefore PC-Ring uses an estimation method to calculate the failure of the system.

Whether the probability function leans towards the upper bound or the lower bound depends on the correlation between all the limit functions. If there is no correlation, the upper bound can be taken as the probability of failure. If the complete correlation is found the lower bound should be taken.

The amount of correlation can be expressed as a correlation coefficient (0 is no correlation, 1 is fully correlated). The correlation coefficient of two functions Z_i and Z_j can be calculated with the following formula:

$$\rho(Z_i Z_j) = \sum \alpha_{ik}\alpha_{jk}\rho_{ijk} \tag{B.16}$$

In this formula i and j are the elements (failure mechanism or dike section) and k is the index for the stochastic variable. The sum is taken for every stochastic variable (every k). This equation shows that if the influence factor of a stochastic variable is large, the correlation of the same stochastic variable between two elements is also larger (which makes sense).

To solve equation B.14, the equation is first simplified to a series system with two elements. Because now only two elements are taken into account this equation can be written one step further:

$$P(F) = P(Z_1 < 0 \text{ or } Z_2 < 0) = P(Z_1 < 0) + P(Z_2 < 0) - P(Z_1 < 0 \text{ and } Z_2 < 0) \tag{B.17}$$

For our case this means the probability $P(Z_i < 0 \text{ or } Z_j < 0)$ can be replaced with an equivalent probability $P(Z^e < 0)$. This is done by solving equation B.17 in which the last term $P(Z_1 < 0 \text{ and } Z_2 < 0)$ is solved by the method Hohenbichler. In this method the correlation coefficient from equation B.16 is needed. Method Hohenbichler is not explained here. Reference is made to his report [73]. The method results besides the equivalent probability ($P(Z^e < 0)$) also in its corresponding β^e and α^e values. Figure B.4 shows a visualisation of this result.

This result means that the original problem of n different probabilities are now reduced to n-1 different probabilities. Because new β and α values are also calculated, this process can be repeated again (with Z^e and Z_3). The process can be repeated n-1 times to get a total probability of failure that is equivalent for the whole system.

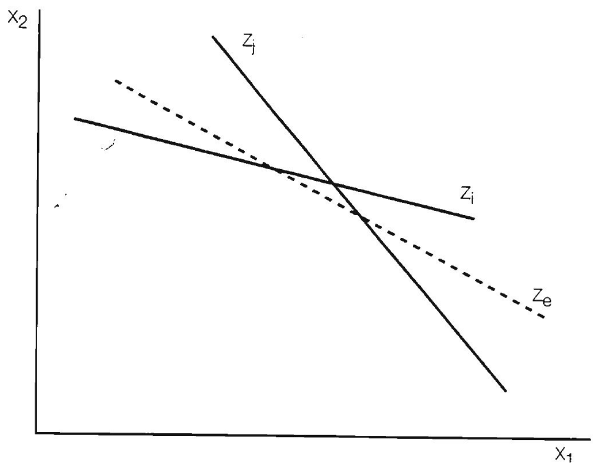


Figure B.4: Equivalent limit state function (Source: [62])

Previously, the term element was used, as the method is the same for the element 'failure mechanism' and the element 'dike section'. Both elements should be used in the process, but this must be done one after the other. Two possible paths could be taken:

- **Path 1.** First, take all the probabilities of all failure mechanisms to arrive at a probability of failure per dike ring section. Then, take all these probabilties together to arrive at one probability of failure of the whole dike ring area.

  So: all failure mechanisms -> probability per section -> total failure probability.

- **Path 2.** First, take all the probabilities of all sections for one failure mechanism. This gives a total probability of failure of the whole dike ring area for one failure mechanism. Then, do this also for the other failure mechanisms and add those probabilities together to arrive at one probability of failure for the whole dike ring area.

So: all sections -> probability per failure mechanism -> total failure probability.

PC-Ring uses the first option, path 1, as path 2 shows a disadvantage. Correlation coefficient between the failure mechanisms for the total dike ring area are not easily defined. Therefore it is hard to combine these limit state functions. The correlation coefficient between the failure mechanisms per dike ring section are easer found and thus path 1 is preferred over path 2. The other step, combining the dike ring sections, is independent of the failure mechanisms taken together or the failure mechanisms not yet taken together. This means it doesn't matter if the dike sections are combined first or later. As the correlation coefficients are different for the case that all sections are taken together or per section, failure probabilities should be combined first per section for all failure mechanisms (thus, path 1).

C

New Dutch stadards

In 2006, the project VNK2 (Flood Risk in the Netherlands part 2, Dutch: Veiligheid Nederland in Kaart 2) was started [49] (see also the framework in section 5.3.1). The project tries to give insight into the risks of flooding in the Netherlands. Another aspect of this project is a revision of the current norms used for floods, they are improved to the 'new' norms. This revision results in an advice to the Dutch minster of Infrastructure and Environment. The project was initiated by the Ministry of Infrastructure and Environment, the Dutch Water Authorities (Dutch: Unie van Waterschappen) and the Inter-provincial consultation (Dutch: Interprovenciaal Overleg). Executed by Rijkswaterstaat, the project is aimed to finish in 2015.

The new norms use a risk based approach instead of the current probability of exceedance of a certain water level. The following sections describe the difference between those two approaches (C.1 & C.3) including a step in between (C.2).

C.1. Probability of exceedance

The probability of exceedance is currently the norm that is in use in the legislations for the assessment of dikes. This probability of exceedance is the probability that the maximum water level occurs which the dike can retain (see figure C.1). In the legislation of the dikes the dikes are assessed to this failure mechanism.

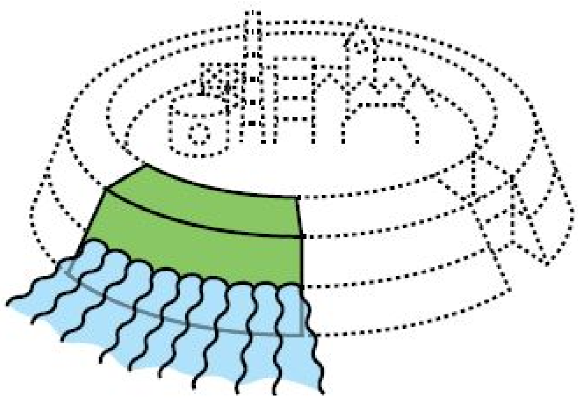


Figure C.1: Probability of exceedance (source: [63])

The origin of the current norms are found around the time of the first Delta committee (~1960). At that time it was a lot harder to assess the probability of flooding, the consequences of flooding and therefore the risks of flooding. For each dike ring area a safety norm was given by the Delta committee ranging from 1/250 per year for the dike ring areas with low consequences of flooding to 1/10,000 per year for the provinces of North and South Holland (see figure C.2). The difference in norms for the dike areas are caused by an estimation of the consequences of flooding of the dike ring area. However, as these norms are approximately 50 years old, an update of the norms is desirable as consequences have changed.

Possible outcomes of this assessment are: sufficient, not sufficient, and no judgement. If a dike is assessed as not sufficient, improvement of the dike is necessary.

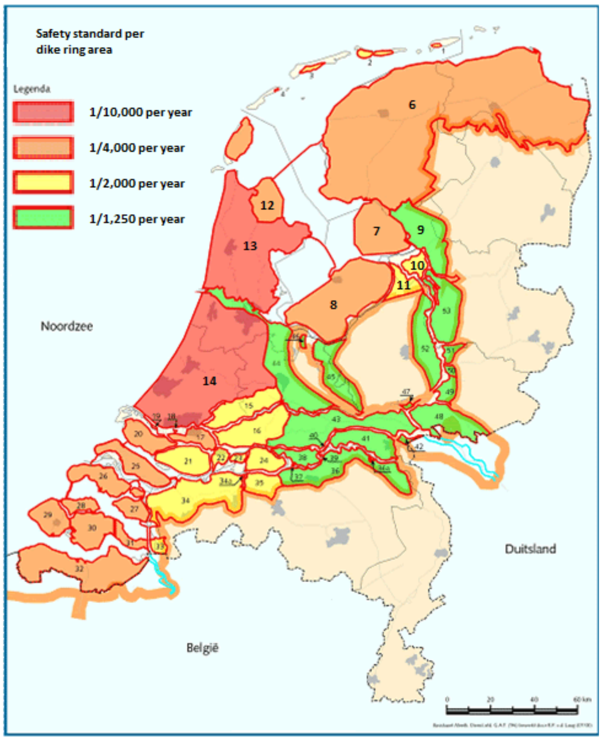


Figure C.2: Dike ring areas and its norm (source: floodsite.net)

C.2. PROBABILITY OF FLOODING

The probability of flooding gives the probability of actual flooding of a dike ring. For the calculation of the probability of flooding of the whole dike ring, all dike compartments in the ring are assessed and all failure mechanisms are considered (see also figure C.3). In this calculation the parameters for the strength of the dike and foundation, the loads of the water on each compartment, and the uncertainties in these parameters are used as input.

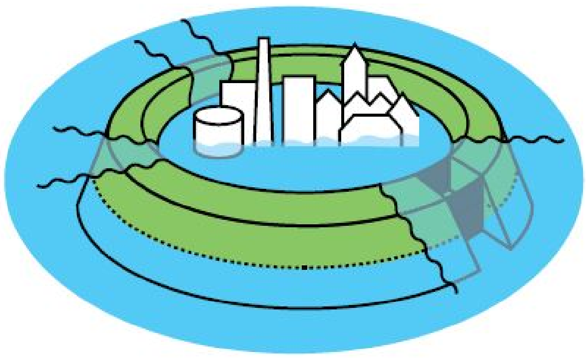


Figure C.3: Probability of flooding (source: [63])

Advantage of this approach is that with the outcome it can easily be seen which dike compartment is weak and which failure mechanism is dominant. This way improvement of the dike ring area can be more effective and efficient. Disadvantage of this approach (in respect to the previous one) is the need of more strength parameters as the assessment gets more complex.

The output of the probability of flooding approach is a probability of failure (usually expressed in probability per year).

C.3. RISKS OF FLOODING

Probability combined with consequence gives the risk. The risks of flooding approach gives also an indication of how big the economic damage and casualties will be if a dike ring fails (see figure C.4). For this, it also takes into account which part of the dike ring fails, as the consequences of flooding depend on the location of the dike breach/overflow. By combining the probability and consequences, new norms can be used based on individual risk, public risk, and economical risk. Disadvantage of this approach is that a lot of data of the strength, load, and consequences are needed.

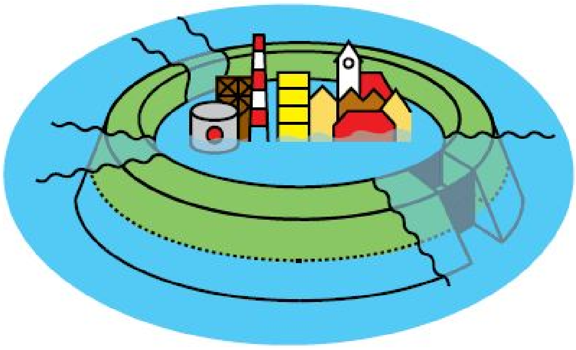


Figure C.4: Risks of flooding (source: [63])

Outcome of the risks of flooding approach is a risk expressed in damage/year. Damage is expressed in a monetary value (e.g. Risk = €/ year).

Source used in this section: [63]

D

A,B, AND C-TYPE DEFENCES

As previously mentioned in section 3.2, the Dutch law ‘Waterwet’ states that once in 5 years all Dutch primary water defences should be assessed. For this, the term ‘primary water defence’ is needed to be explained.

A primary water defence structure is a water defence structure (dike, weir, dam, retaining wall, etc.) that provides protection against floods in a dike ring area. It can fulfil this function either directly by being part of the dike ring or indirectly by connecting multiple dike rings. This introduces the term dike ring area. According to the same law, a dike ring area is defined as an area that, due to a system of water defence structures, is protected against floods. The floods around the IJsselmeer area can occur due to extreme storm surges, extreme water discharges in the main rivers, high water in the IJsselmeer or Markermeer, or a combinations of these aspects. The system of water defence structures that protects against flooding can exist of primary water (artificial) defences systems or (natural) high grounds.

As primary water defences can fulfil their function in different ways (protecting from inside and outside, permanent or temporarily, etc.) a division is made between 4 categories of primary water defence. They are numbered as type a, b, c, and d.

- **Category a** This type of water defence consists of the main parts of the dike ring area that directly retains water from the outside. They encircle the dike ring areas and close them off. This category of water defences are of such importance that if a failure occurs, the hinterland is flooded directly. The dikes (not dams) around the IJsselmeer and the Markermeer are categorised as type a.
- **Category b** Primary water defences that lie in front of a dike ring or connect two or more dike ring areas are categorised as type b. Failure of this type of defence is not a direct cause of flooding. However, failure of type b water defences increase the loads on type a water defences. In most cases these type a water defences are designed in a way taking the extra defence of the type b defence into account. This is currently the case for the Afsluitdijk and the Houtribdijk (both type b) and the IJsselmeer dikes (type a).
- **Category c** Primary water defence systems that are part of a dike ring, but do not retain water directly are category c type defence systems. These water defence systems become active once a type a or b water defence system fails. Examples of c type defences are the Knardijk in the Flevopolder and the dike on the border of the Noordoostpolder and Overijssel.
- **Category d** The term category d is not much used. This category has all the functions of type a, b, and c but it is different from the previous categories because they lie outside the Dutch national borders. Category d water defences are located at the Belgium North Sea coast, the Belgium river dikes along the Schelde, and the German river dikes along the Rijn.

In figure D.1 a schematisation is given of the first three different categories of primary water defence systems.

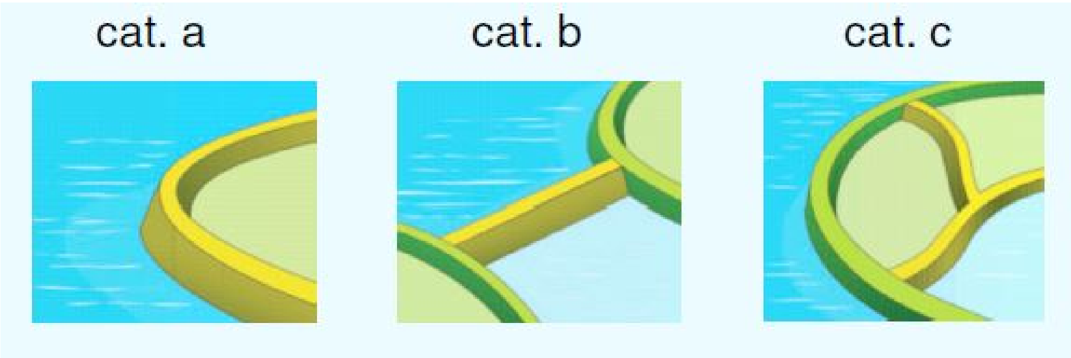


Figure D.1: A,b, and c type defences (source: [64])

E

DESCRIPTION OF THE STORM OF 1953

Section 2.1 introduced the event on January 31th and February 1st 1953. This appendix will focus on the description of the water levels over time, the influence of the tide, and the influence of the wind in the storm on the water level during this storm.

For the description of this disaster, reference is made to a report about the flood in 1953 published by Rijkswaterstaat in 1961 [22]. As the storm had a disruptive effect on the Netherlands much effort in investigating this storm was done. In the report focus was laid on the location of Vlissingen as at this location the storm had the most consequences. As the storm was investigated heavily, also research is done to other locations like England, IJmuiden, Den Helder, and (most relevant) Den Oever and Harlingen (close to the Friesland side of the Afsluitdijk).

The storm of 1953 was a so called storm surge. For the Netherlands, a storm surge means a coincidence of water set up in the North Sea due to the storms wind and astronomical high water (high tide). The summation of the wind set up and the astronomical tide gives the occurring water level. Because both are time dependent and the tide is a sinusoidal phenomenon the maximum water level does not have to occur when tide and wind set up are maximum. Simply adding the maximum tide and the maximum wind set up together will lead to an overestimation of the occurring water level.

E.1. WATER SET UP

Figure E.1 shows the wind speeds and wind directions during January 31th and February 1st 1953. On the right axis the wind speed is given (in m/s). For every hour the maximum wind speed velocities are given (scaled on the right axis). More relevant is the continuous line. It shows the average wind speed over the hour. The average wind speed determines the water set up (not the maximum peaks).

E.2. ASTRONOMICAL HIGH WATER

The storm of 1953 was not only influenced by the water set up caused by the wind. The timing of the peak of the maximum set up is also very relevant. If the maximum set up is timed during low tide, the maximum water level will not be as high as it could be during high tide. Because of this, a difference in a couple of hours can make a big difference in the occurring water levels.

Also important is the timing of the storm in respect to the occurrence of spring tide. The difference between high tide and spring tide can clearly be seen in figure E.2. This figure shows the water levels at Den Helder and at Vlissing in the weeks around the disaster of 1953. The short harmonic waves are caused by the tide. Every (roughly) 12.5 hours a maximum and a minimum peak occur. These are the high tide water level and the low tide water level. This 'daily' tide is enveloped in the figure to indicate a trend in the high and low water peaks. This trend is also a sinusoidal function and shows three maximum peaks and two minimum peaks in the time interval of 1,5 month of the figure. These maximum peaks occur when earth, sun, and moon align. This can happen in two forms: Full moon (the moon is on the other side of the earth as the sun) and

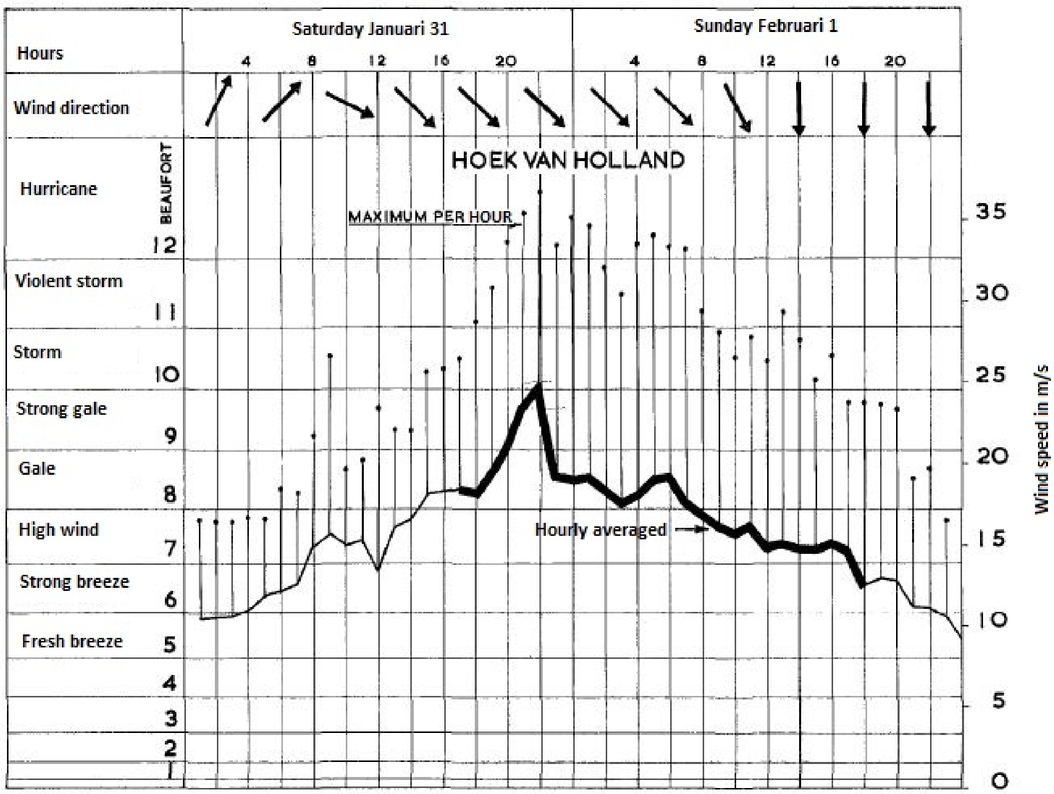


Figure E.1: Wind direction and wind speed during the storm of 1953 (source [22])

new moon (the moon is between the earth and the sun). Full moon and new moon increase the effect of the tide. These maximum peaks are called spring tide and have a frequency of two in approximately 29.5 days.

Because of various reasons (mainly the travel time from the ocean trough the North sea) a delay in spring and neap tide of approximately 2.25 days occurs. This is also shown in figure E.2. Full moon occurred on January 30th 1953 and thus spring tide in the Netherlands coincided with the storm surge on February 1st. Because of other reasons (i.e. the temporary large distance moon-earth) spring tide on February 1st was relatively low. In relation to the previous spring tide (February 18th) it was even 50 to 60 centimetres lower, depending on the location.

E.3. COMBINATION OF SET UP AND TIDE

In figure E.3 the water levels at the locations around the Waddenzee are given. In the top figure also the average tidal water levels (during no storm) are shown. In this way the effect of the storm can clearly be seen. One can see in the top figure that in the beginning of January 31th and at the end of February 2nd the water levels do not differ much from their average values. Here the storm has no effect on the water level. The effect of the storm can be extracted from the top figure. This is shown in the bottom figure of figure E.3. This is the effect of the storm due to wind set up and shows the water level without the influence of the tide.

Splitting the occurred water level into the two parts, tide effect and storm effect, is of importance for analysing future storms. Both are independent phenomenon. The main cause for future extreme water levels lies in the occurrence of storms and thus extreme wind set up. If this coincides with spring tide this leads to even more extreme water levels. This is only a magnifying factor and the real treat comes from the wind set up.

Also, during spring tide much less fluctuations in the Waddenzee water levels are found than the fluctuations in storm effect. Therefore the extremes in storm set up are much more interesting.

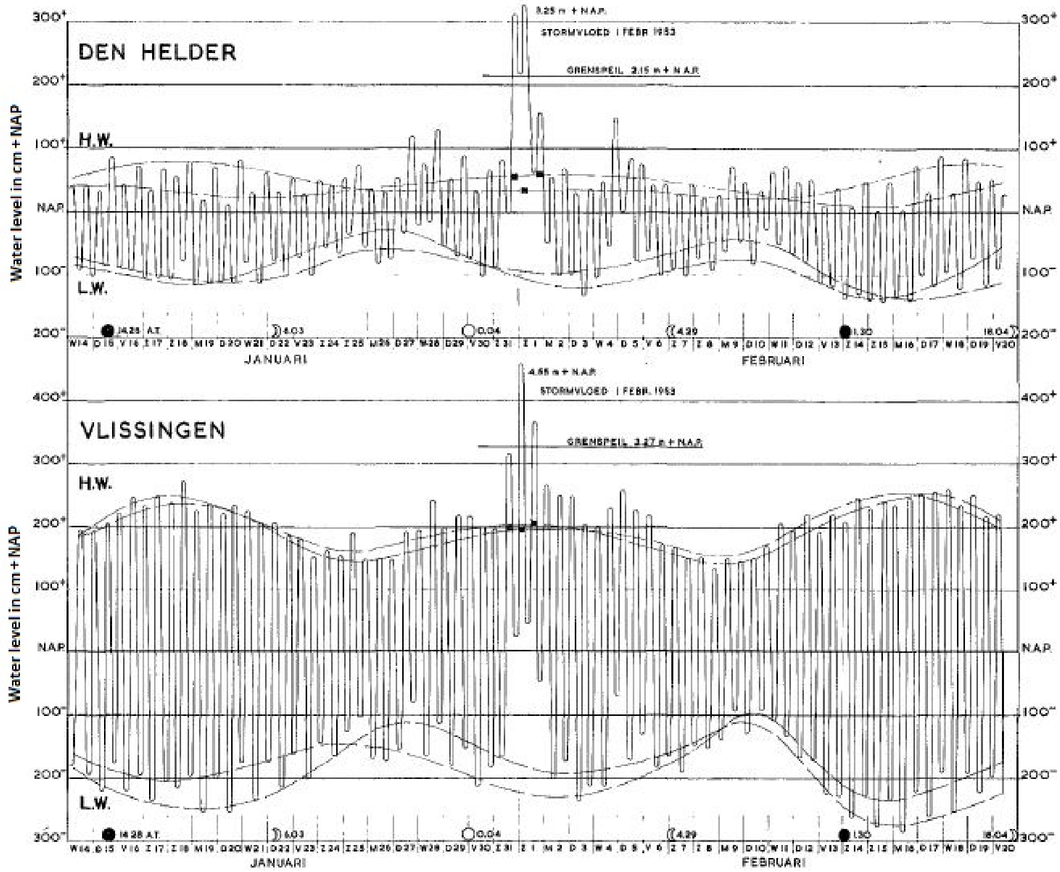


Figure E.2: Water levels at Den Helder and Vlissingen Januari and Februari 1953 (source [22])

E.4. UPSCALING OF THE STORM

For simulating more extreme storms than the 1953 storm, the extreme water levels can be extrapolated. The water levels before and after the extreme water level can not be determined by extrapolating. These are dependent on the extreme water level occurring during the same storm. For defining all the water levels during a storm surge, the storm set up is utilized. The storm set up will be scaled up to account for more extreme storms.

Figure E.4 shows a schematization of the storm effect during the 1953 storm. During a period of 20 hours the storm effect will build up to its maximum after which it will build down again during a period of 30 hours.

This schematization will be used for estimating future extreme storms. First, the most extreme water level will be extrapolated from current data. This extreme water level will determine the peak in the schematized storm effect. Second, it is assumed built up and reduction of the storm will take the same time as the extreme storm of 1953, independently of the extreme water level. This means only the y-axis, in figure E.4, will be scaled up (the water level) and the x-axis will stay the same (the duration).

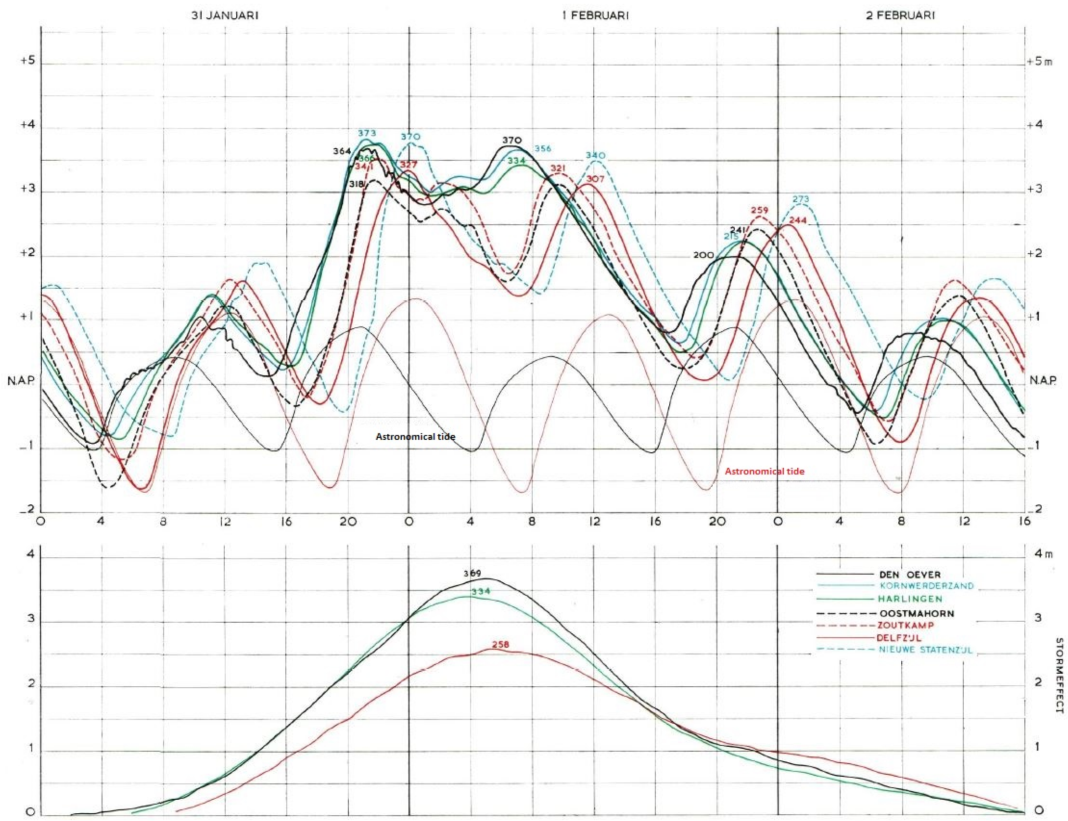


Figure E.3: Water levels during the storm of 1953, contribution of tide and storm effect (source [22])

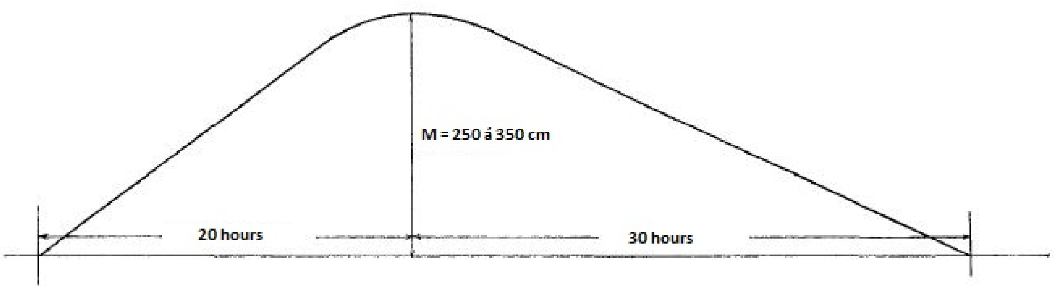


Figure E.4: Schematized storm effect (source [22])

F

EVENT DEPENDENT DIMENSION

The presented dimensions of the breaches in the levee (see section 3.4) are the ultimate values for a 10^{-4} storm. These expected values are the (depth-averaged) width and the average depth of the breach when such a storm hits the Afsluitdijk. As was found in section 3.2, the Afsluitdijk is likely to fail at much less extreme storms (in the order of 1/140 to 1/250 per year). If such a storm occurs it will not cause the same damage as the effect of a 1/10,000 per year storm. On the other hand, even more extreme storms (e.g. 1/100,000) are also able to hit the Afsluitdijk. If such a storm occurs the dimensions described in previous sections will be surpassed.

The dimensions of the breach are strongly dependent on the event that occurs (i.e. the water level standing on the outside of the breach). To find the less and more extreme dimensions of the breach at the 1/10,000 per year storm, a closer look is taken at the model used to find the breach dimensions at the once in a 10,000 year storm.

F.1. STAGES IN THE BRES-MODEL

The BRES-model is a simplified model that describes the development of a breach in a sand dike after it fails. According to Visser [29], the development of the breach can be divided into 5 different stages (see figure F.1).

- **Stage I** $t_0 < t \leq t_1$ The inner slope of the dike will erode and starts to steepen up to the critical slope angle β_1 at $t = t_1$
- **Stage II** $t_1 < t \leq t_2$ Erosion continues. The inner slope stays at a constant slope angle β_1 and retreats towards the outer side of the dike. The crest width of the dike decreases until the original crest vanishes at $t = t_2$. At this point the breach inflow starts to increase (the crest height on the outside starts to decrease).
- **Stage III** $t_2 < t \leq t_3$ The top of the dike decreases in height with a constant angle of the breach side-slopes. This is equal to a critical value named γ_1 in the literature. At this stage, the width of the breach starts to increase. At $t = t_3$ the dike at the breach is completely washed out and the maximum depth is reached.
- **Stage IV** $t_3 < t \leq t_4$ The critical flow stage. Vertical erosion stops and the breach continues to grow only laterally with the side-slope angles remaining at critical value γ_1. At $t = t_4$ the flow through the breach changes from critical to subcritical flow.
- **Stage V** $t_4 < t \leq t_5$ Subcritical flow stage. Still mainly laterally growth of the breach (increase in width) and side-slope angles remain at the same critical value γ_1. At $t = t_5$ the flow through the breach is too low to cause any more erosion and the ultimate dimensions are reached (this reduction in flow is because the outside water level goes down or the inside water level goes up). From $t_5 < t \leq t_6$ water will still flow through the breach but no further erosion occurs. At $t = t_6$ discharge through the gap is 0.

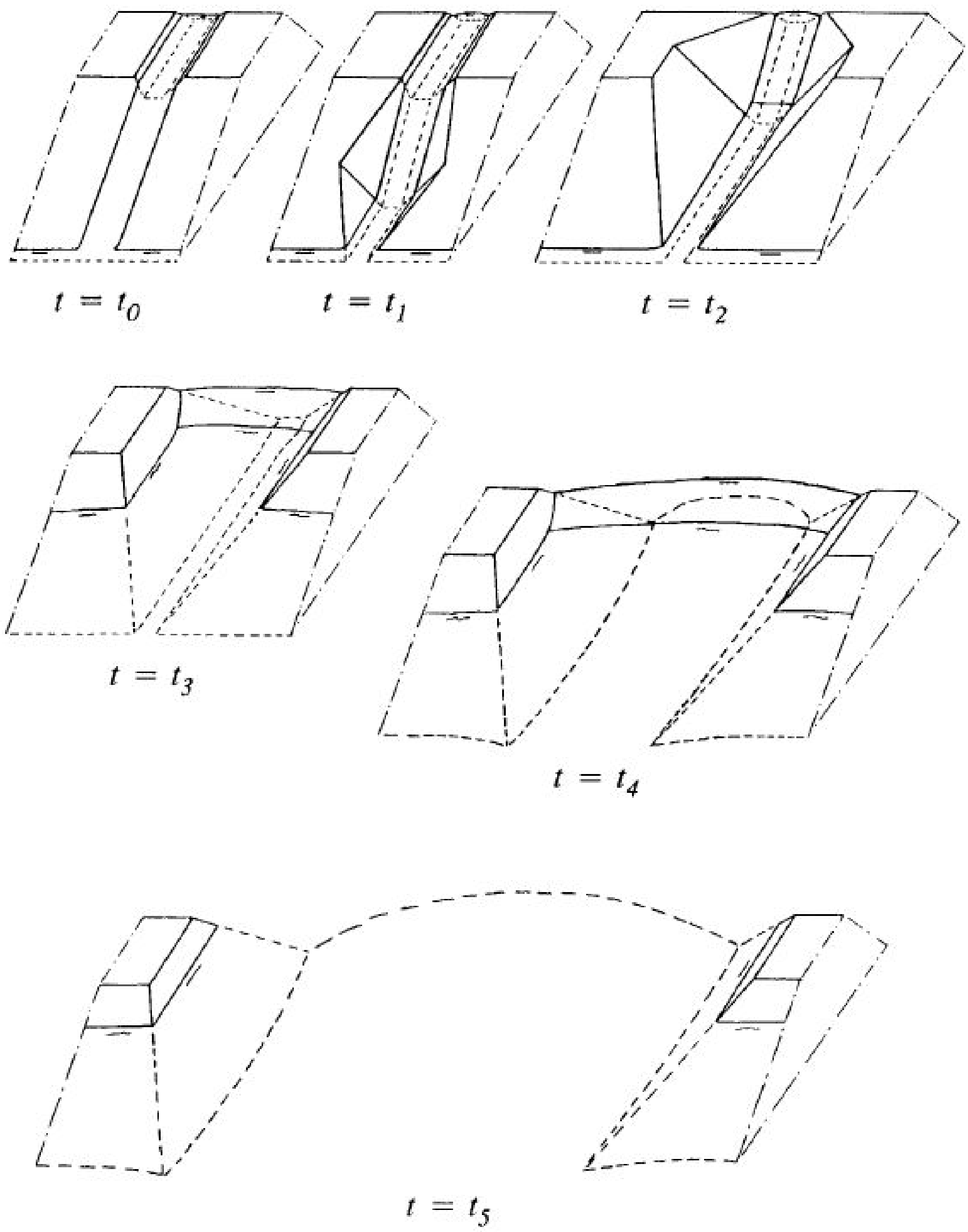


Figure F.1: The 5 different stages of the development of a breach in a sand dike. The index of t represents the time at its specific stage (t_0 = starting point) (source: [29])

Stages I, II, and III represent the initial stage of the breach and are the stages where the breach cuts itself through the dike. Not much discharge through the gap takes place here. The majority of the discharge takes place in stage IV and stage V. Difference between the last two stages is the effect of the water level on the inside of the dike. As is shown in this thesis (see end of 4.1.3) the effect of the IJsselmeer water level on the amount of inflow is negligible. Therefore stage IV will be the most dominant stage [1]

[1]Visser's research [29] focused on river dikes. Here, the outside water level on the dike stays more or less constant and the polder side water level will rise to such extend that it influences the amount of inflow. The IJsselmeer, however, has such a big storage capacity that the water level rise will be so small that the change of flow from critical to subcritical flow is not likely to happen.

F.2. BREACH DIMENSIONS AT DIFFERENT STORMS

To find the dimensions of the breaches during different storms (e.g. 1/140, 1/1,000, and 1/100,000) the BRES-model could be used. This approach has two disadvantages. First, to get familiar with the BRES-model would require some time which could (better) be used at other aspects in this thesis. Second, and more importantly, this approach will give discrete values for the dimensions during certain storms. It would be more relevant to find a continuous relation between water level on the outside of the Afsluitdijk and dimensions of the breach. In this way for each generated storm in chapter 4, dimensions of a breach can be determined.

This continuous relation between ultimate breach width and Waddenzee water level can expected to be as follows:

$$B(H_w) = a + b * (H_w)^c \tag{F.1}$$

In this equation $B(H_w)$ is the ultimate width of the breach after the final hours of the storm, the width depends on the water level standing on the outside of the Afsluitdijk expressed as H_w. a, b, and c are shape factors that determine the shape of the relation between the two variables. In these numbers all the other variables effecting the breach dimensions are represented. These variables are things like height of the crest, bed porosity, side slope angle, particle size, etc. These variables are independent of H_w, can therefore be seen as constants in the relation $B(H_w)$, and can therefore be represented by a, b, and c in equation (F.1).

The values of a and b can easily be found by interpolating the known breach width during the 1/10,000 storm (see section 3.4.1) and assuming the breach width is 0 at the smallest storm at which the Afsluitdijk fails (i.e the 1/140 storm).

The value of c is harder to define. To find this value, the proportionality between water level and breach width has to be found. For this, the 5 different stages defined in Vissers thesis [29] are further examined.

At stage III the breach growth accelerates drastically as the breach starts opening causing an increase in inflow of water. This also means the sediment transport through the breach will increase and thus the dimensions of the breach will enlarge. In stage III the side-slope erosion is entirely controlled by the erosion at the bottom of the breach (E_{bo}). Therefore the rate of increase of the breach width is controlled by the rate of erosion at the breach bottom.

The flow at the beginning of stage III ($t = t_2$) is super critical with a Froude number [2] of $Fr >> 1$. At the end of stage III the flow is still supercritical but with a Fr slightly above 1. This change in Froude number is caused by the increase in gap size (depth) of the breach. Also the sediment transport changes from sheet flow transport (Shields' mobility parameter in the orders of 10 and 100) to suspended load sediment transport (Shields' mobility parameter in the order of 1).

There are two different mechanisms in the breach process. In stage I, II, and III, the increase of the breach will mainly be in reducing the crest level and is dominated by erosion at the breach bottom (E_{bo}). In stage IV and V, the increase of the breach will mainly be in increasing the width of the breach and is dominated by erosion at the slide-slopes (E_{sl}). These two different erosion mechanisms can be described by erosion transport formulae. Visser [29] validated a selection of sediment transport formulae by comparing experimental data with the predicted values. He found that for breaches in sand dikes the BRES-model gave good results by using the Bagnold-Visser (1989) or the Wilson (1987) erosion functions in stages I, II, and III and using the Van Rijn (1984) or Engelund-Hansen (1967) erosion functions in stages IV and V.

Stages I and II contribute relatively little to the total flow through the breach compared with stages IV and V, where most of the breach erosion takes place. Because the breach width will increase the most at the last two stages the relation between breach width and water level will be dominated by these stages.

[2] Froude number is a dimensionless number defined as the ratio of the flow velocity to the square root of the gravity acceleration times the flow depth: $Fr = u/\sqrt{g * h}$. It expresses the ratio of the flow velocity to the propagation velocity of a shallow water wave.

F.3. STAGE IV

According to Visser [29] (eq 4.83) the rate at which the breach width increases (in stage IV) can be expressed as follows:

$$\frac{dB}{dt} = 2\frac{d}{h}\frac{s_s}{(1-p)l_a tan\gamma_1} \tag{F.2}$$

In this equation, B is the width of the breach, t is the time, d is water depth in the breach (see equation (F.5) for its definition), h is the difference between original dike height (H_d) and crest level in the breach (Z_{br}), s_s is the suspended sediment transport, p is the bed porosity, l_a is the adaptation length, and γ_1 is the critical value of side slope angle γ.

This formula introduces many factors of which not all are relevant for the relation breach width versus water level. For instance the bed porosity and the critical slide slope angle are independent of the water level in the Waddenzee. Also the difference between original dike height and crest level in the breach is independent of the water level at stage IV and V, as the depth will hardly increase during these stages. These factors are represented in a and b in equation (F.1) and this simplifies the proportionality in equation (F.2) to:

$$\frac{dB(H_w)}{dt} \propto d * \frac{s_s}{l_a} \tag{F.3}$$

This proportionality means the rate of increase of the width of the breach is positively influenced by the amount of sediment transport and water depth in the breach, but is negatively influenced by the adaptation length.

The adaptation length is the length that is needed for the flow to have reached the maximum sediment capacity. In other words, before $x = l_a$ the suspended sediment transport is not at its maximum. The shorter the adaptation length, the sooner much sediment can be transported. The adaptation length can be approximated by:

$$l_a = \xi\frac{d}{h}\frac{Ud}{w_s cos\beta} \tag{F.4}$$

In this equation ξ is a coefficient of ≈ 0.4 in stage IV, U is the depth averaged velocity of the water, w_s is the fall velocity of a sediment particle (= independent of the water depth), and β is the slope of the crest level in direction of the flow (so $\beta = 0$ in stage IV). In stage IV, as was explained before, the Froude number is around 1. This means the water depth is equal to the critical water depth ($d = d_c$) and the flow velocity is equal to the critical flow velocity ($U = U_c$).

$$d = d_c = \frac{2}{2 + B/B_w}(H_w - Z_{br}) \tag{F.5}$$

$$U = U_c = \sqrt{g d_c B/B_w} \tag{F.6}$$

In these equations g is the acceleration due to gravity, B is the depth averaged width of the breach and B_w is the width of the breach at the water line. According to Visser [29] (equation 4.66 and equation 4.67) these values of B can be expressed as follows:

$$B = b_o + \frac{d}{tan\gamma_1} \quad \text{and} \quad B_w = b_o + \frac{2d}{tan\gamma_1} \tag{F.7}$$

Here b_o is the initial breach width. Although B and B_w are dependent on the water depth the effect of d on the relation B/B_w is negligible. Therefore it is simplified that B/B_w is independent of the water level.

By substitution of equation (F.5) and equation (F.6) in equation (F.4) (and removing the parameters that are independent of the water level) the following proportionality can be found:

$$l_a \propto d * U * d \propto d * d^{0.5} * d = d^{2.5} \propto H_w^{2.5} \tag{F.8}$$

Last unknown proportionality in equation (F.3) is the suspended sediment transport (s_s). Visser showed that for stages IV and V, the sediment transport formulas of Van Rijn or Engelund-Hansen are best to be used.

As Van Rijn showed the best results in Vissers validation experiments [29] the proportionality of Van Rijn's suspended sediment transport formula with the Waddenzee water level is investigated. Van Rijn's (simplified) suspended sediment transport formula can be written as follows [74]:

$$s_s = \alpha_s * \rho_s * U * d_{50} * M_e^{2.4} * D_*^{-0.6} \tag{F.9}$$

In this equation α_s is an empirical coefficient, ρ_s is the sediment density, U the flow velocity, d_{50} is the median particle size, M_e is the mobility parameter (see below), and $D_*^{-0.6}$ is the dimensionless particle size.

Of all these values only U and M_e are dependent on the Waddenzee water level. Where U is described in equation (F.6), M_e needs to be specified further:

$$M_e = \frac{U - U_{cr}}{[(s-1) * g * D_{50}]^{0.5}} \tag{F.10}$$

Here, s is the specific density $\frac{\rho_s}{\rho_w}$ in which ρ_w is the density of water, D_{50} is the diameter of the sediment of which 50% of the sediment is finer than this value. U_{cr} is the critical Shields velocity. For high Reynolds numbers [3] (>200) the flow near the sediment particles is fully turbulent and independent of the viscosity. The critical velocity may then be computed by means of the Chézy equation.

$$U_{cr} = U * \frac{\sqrt{g}}{C} \tag{F.11}$$

In which C is the Chézy coefficient, $C = 18 * log(12 * d/D_{50})$.

Now, the numerator in equation (F.10) can be written as: $U - U_{cr} = U - U * \sqrt{g}/C = U * (1 - \sqrt{g}/C)$. Tough the Chézy coefficient in equation (F.11) is proportional to the water level the flow velocity will dominate the value of M_e (because of the logarithm in the Chézy coefficient). Therefore the proportionality of the value of M_e in equation (F.10) can be approximated by:

$$M_e \propto U - U_{cr} \approx U \propto d^{0.5} \tag{F.12}$$

Equation (F.9) rewritten shows the proportionality of the suspended sediment transport to the Waddenzee water level:

$$s_s \propto U * M_e^{2.4} \propto d^{0.5} * (d^{0.5})^{2.4} = d^{0.5} * d^{1.2} = d^{1.7} \tag{F.13}$$

Now equation (F.13) and equation (F.8) can be filled in to equation (F.3):

$$\frac{dB(H_w)}{dt} \propto d * \frac{s_s}{l_a} \propto d * \frac{d^{1.7}}{d^{2.5}} = d^{0.2} \tag{F.14}$$

It is expected bigger storms have higher peak water levels, but will have more or less the same duration as the storm of 1953 (see also section E.4). Therefore the *difference* in maximum breach width is independent of time. The proportionality of the maximum breach width in equation (F.14) can now be rewritten as (also making use of equation (F.5)):

$$B(H_w) \propto d^{0.2} \propto H_w^{0.2} \tag{F.15}$$

F.4. CONCLUSION RELATION WATER LEVEL AND BREACH WIDTH

It is expected the effect of the outside water level on the final width of the breach is dominated by stage IV in the breach model. To find the relation between outside water level and final breach width the proportionality of equation (F.15) was found. It gives the proportionality between the width of a breach in the levee and the water level in the Waddenzee after a storm at which the Afsluitdijk fails. This exponent in equation (F.15) is equal to the value of c in equation (F.1) and equation (F.1) can therefore be written as:

[3] The Reynolds number is a dimensionless number that is used to define flow patterns in fluid flow situations. $Re = \frac{U*R}{v}$, in which U is the velocity of the water, R the hydraulic radius, and v the kinematic viscosity

$$B(H_w) = a + b * (H_w)^{0.2} \tag{F.16}$$

The values of a and b depend on the scenario and are found by using the following values for B and H_w:

Storm	H_w	Width scenario 2	Width scenario 3	Width scenario 4
1/140	4.02m +NAP	0 m	0 m	-
1/250	4.18m +NAP	-	-	0 m
1/10,000	5.02m +NAP	380 m	1300 m	300 m

Table F.1: Values used to solve equation (F.16)

A description of the different scenarios in explained in section 3.8. Scenario 1 is not included in the table as the breach is expected to be at the sluices in this scenario. Therefore it will not be described with this relation. At scenario 2 and 3 the Afsluitdijk fails at a 1/140 storm and at scenario 4 at a 1/250 storm. Therefore the data points used for fitting equation (F.16) are a width of 0 and a Waddenzee water level corresponding to the most frequent storm at which the Afsluitdijk fails [4].

Solving equation (F.16) for all scenarios takes place in the Matlab model in chapter 4. To give an idea about the relation equation (F.16) is plotted in figure F.2 for the value of scenario 2. To be able to fit the equation through the two known points (from table F.1) high values of a and b are needed. This results in a relation that does not differ much from a linear (c=1) relation. This means the water level - breach width relation is not very sensitive to assumptions made in section 3.5 and this appendix. The values from table F.1 are most important for defining the ultimate width for different storms. If more accurate values are needed it is advised to improve the reliability of these points.

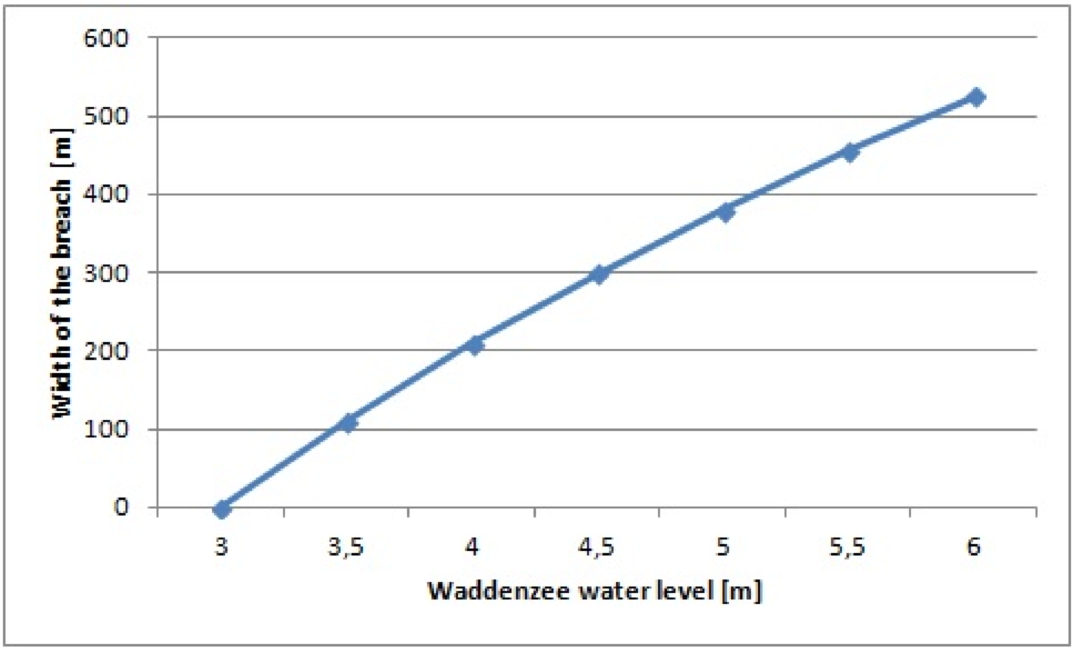


Figure F.2: Relation ultimate width of the breach to the Waddenzee water level in scenario 2

[4]The - marks in table F.1 are used to indicate that those points are not relevant for fitting equation (F.16)

G

THE USED MATLAB CODE

Below the Matlab code that is used in chapter 4 is presented. (This is the script for scenario 4, the other scenarios are similar to this scenario).

```
close all
clear all
clc

%Input parameters Scenario 4
FailureFrequency=250; %once in x year
FailureWaterLevel=4.1750; %water level corresponding with 1/250 per year exceedence
    freq
WidthGapSluices= 336; %Width of gap in the Afsluitdijk at the sluices
WidthGap10000= 300; %Ultimate width of gap in the levee of the Afsluitdijk at
    1/10000 per year storm
DepthGapSluices= 4; %Ultimate depth of gap in the Afsluitdijk, crest level at -4.4m
    max opening at -0.4m
DepthGapLevee= 0; %Ultimate depth of gap in the levee of the Afsluitdijk, at +NAP

%Other input parameters
A=1100.0e6; %surface area ijsselmeer
FlowCoefficient=0.88; %flow coefficient overflow
Cc=0.61; %contraction coefficient underflow
m=80; %change this to adjust number of runs (always even number) (>2)
n=100000*m; %number of Monte Carlo runs (n>200,000)
d=30; % duration of the storm
Nrfail=0; %start at 0
heightAfsluitdijk= 7.75; %average height of the Afsluitdijk

%Parameters needed for water level distribution(s)
MuWaddenzee=9.848609608;
SigmaWaddenzee=7.219882954;
AlphaWaddenzee=22.90035699;
MuIjsselmeer=0.110486793;
SigmaIjsselmeer=0.10237531;
AlphaIjsselmeer=0; %not relevant for gumbel

NrSubmerged=0;
NrClear=0;

```

```
%Event dependent gap. Fit for WidthGap(iStep)=a+b*WaterstandWaddenzee(iStep)^0.2
b=WidthGap10000/(-(FailureWaterLevel^0.2)+(5.02^0.2));
a=-b*(FailureWaterLevel^0.2);

%Start monte carlo
for iStep=1:n
    %Row 3 - Create values for water levels
    WaterstandWaddenzee(iStep) = MuWaddenzee - SigmaWaddenzee * (-log(rand))^(1/
        AlphaWaddenzee); %Reverse weibul

    WaterstandIjsselmeerCurrent(iStep) = MuIjsselmeer + SigmaIjsselmeer *(-log(-log(
        rand))); %Gumbel

    %Row 4 and 5
    if WaterstandWaddenzee(iStep)>FailureWaterLevel && WaterstandWaddenzee(iStep)>
        WaterstandIjsselmeerCurrent(iStep) %Afsluitdijk fails

        %Calc width of the gap after the storm
        WidthGap(iStep)=a+b*WaterstandWaddenzee(iStep)^0.2;

        %Row 6
        %Different width and water level Waddenzee each hour after the storm
        for j=1:d; %1 point every hour

            hw(j)=WaterstandWaddenzee(iStep)*(1-(j/30)); %max water level lineair
                lowering to 0 in 30 hours

            %Discharge flow through gates = underflow
            if hw(j)>WaterstandIjsselmeerCurrent(iStep)
                Cd(j)= Cc/(sqrt(1+(Cc*DepthGapSluices/hw(j))));
                QsubSluices(j)=Cd(j) * WidthGapSluices * DepthGapSluices * sqrt(2 *
                    9.81 * hw(j));
            else
                QsubSluices(j)=0;
            end

            %Discharge flow through levee = overflow
            if j<=12 %needs 12 hours to develop to ultimate dimensions
                WidthGapVar(j)=(j/12)*WidthGap(iStep);
                DepthGapVar(j)=heightAfsluitdijk + (DepthGapLevee -
                    heightAfsluitdijk) * (j/12); %in m +NAP
            else %after 12 hours full dimensions
                WidthGapVar(j)=WidthGap(iStep);
                DepthGapVar(j)=DepthGapLevee; %in m +NAP
            end

            if hw(j) > DepthGapVar(j) && (hw(j)-DepthGapVar(j)) > (
                WaterstandIjsselmeerCurrent(iStep)-DepthGapVar(j)) %Waddenzee water
                level should be higher than gap for inflow
                %Clear or submerged overfall
                if (WaterstandIjsselmeerCurrent(iStep)-DepthGapVar(j)) < (2/3) * (hw
                    (j)-DepthGapVar(j)) %true = clear overfall
                    QsubLevee(j)= FlowCoefficient * WidthGapVar(j) * (2/3) * sqrt(2
                        * 9.81 * (hw(j)-DepthGapVar(j))^3);
                    NrClear=NrClear+1;
                else %submerged overfall
```

```
                    QsubLevee(j)= FlowCoefficient * WidthGapVar(j) * (WaterstandIjsselmeerCurrent(iStep)-DepthGapVar(j)) * sqrt(2 * 9.81 * ((hw(j)-DepthGapVar(j)) - (WaterstandIjsselmeerCurrent(iStep)-DepthGapVar(j))));
                    NrSubmerged=NrSubmerged+1;
                end
            else
                QsubLevee(j)=0;
            end
        end
        Q(iStep)=mean(QsubSluices)+mean(QsubLevee);

        %Row 7 = Water level rise through gap
        DeltaH(iStep) = (Q(iStep)/A)*d*3600;
        Nrfail=Nrfail+1;
    else %Afsluitdijk doesn't fail
        DeltaH(iStep) = 0;
    end

    %Row 8
    WaterstandIjsselmeerAdjusted(iStep)=WaterstandIjsselmeerCurrent(iStep)+DeltaH(iStep);
end %End Monte Carlo

%Info about run, how many failures (does this correspond with FailureFrequency?
Nrfail=Nrfail*100/n;
ReturnFail=100/Nrfail;
ClearSubmergedratio=NrClear/(NrClear+NrSubmerged)

%Create exceedance probability curves for extremes
f=[100000; 40000; 10000; 4000; 2000; 1250; 1000; 500; 400; 250; 200; 150; 125; 100; 50; 25; 10];
DeltaH= sort(DeltaH,'descend');
Wc(1)=DeltaH(1,m*1);        %1/100000
Wc(2)=DeltaH(1,m*2.5);      %1/40000
Wc(3)=DeltaH(1,m*10);       %1/10000
Wc(4)=DeltaH(1,m*25);       %1/4000
Wc(5)=DeltaH(1,m*50);       %1/2000
Wc(6)=DeltaH(1,m*80);       %1/1250
Wc(7)=DeltaH(1,m*100);      %1/1000
Wc(8)=DeltaH(1,m*200);      %1/500
Wc(9)=DeltaH(1,m*250);      %1/400
Wc(10)=DeltaH(1,m*400);     %1/250
Wc(11)=DeltaH(1,m*500);     %1/200
Wc(12)=DeltaH(1,m*667);     %1/150 667 should be 666,666..
Wc(13)=DeltaH(1,m*800);     %1/125
Wc(14)=DeltaH(1,m*1000);    %1/100
Wc(15)=DeltaH(1,m*2000);    %1/50
Wc(16)=DeltaH(1,m*4000);    %1/25
Wc(17)=DeltaH(1,m*10000);   %1/10

f=1./f;
semilogx(f,Wc,'b'); set(gca,'XDir','reverse'); xlabel('Frequency of occurrence [per year]'); ylabel('Increase in average IJsselmeer water level [m]');
```

H

THE PROBABILISTIC APPROACH

This appendix describes some basic probabilistic theories that are used (or are underlying) in different aspects used this thesis. First, a definitions of probability is given (section H.1). In section H.2, the definition of mean and variance are described. The chapter continues with the explenation of the limit state function in section H.3 and the 3 levels of probabilistic approach in section H.4. In the last section (H.5) the use of extreme value distributions is briefly explained.

H.1. DEFINITION OF PROBABILITY

In the early years of the 19th century the 'classical' definition of probability was formed by Laplace. Laplace defined 'probability' as follows:

> *"The probability of an event is the ratio of the number of cases favourable to it, to the number of all cases possible when nothing leads us to expect that any one of these cases should occur more than any other, which renders them for us, equally possible."* [75]

This can be written in formula form as:

$$\text{Probability} = \frac{\text{Number of cases the event occurs}}{\text{Number of all cases possible (event and no event)}} \tag{H.1}$$

This definition of Laplace has a problem. This is because the definition assumes 'nothing leads us to expect that any one of the cases should occur more than any other' leading to the problem that no definition of chance is given when one (or more) of the cases do occur more than the others. It also has a paradox as probability is used within its own definition.

Motivated by this problem and this paradox of the classical definition of probability the frequentist probability definition was found. Seeing equation H.1 as an approximation, this definition defines an event's probability as the limit of its relative frequency in a large number of cases. This can be written as:

$$P(e) = \lim_{n_t \to \infty} \frac{n_e}{n_t} \tag{H.2}$$

In which $P(e)$ = Probability event occurs, n_e = Number of cases the event occurs and n_t = Total number of cases Drawback of this definition is that needs a lot of cases to define the probability an event occurs.

In practice a mathematical description of chance is not always possible. This is because statistical data is not always available. Even when statistics are used, some data can be unknown or simplification in the data is used, an objective mathematical description can turn out to be impossible. In practise, the determination of the probability is influenced by emotional considerations.

If the value of a variable is subject to variations due to chance this variable is called a random or stochastic variable. A stochastic variable can have different values characterised with an associated probability or probability density function. Stochastic variables can be either discrete (taking any countable value in a list of values) or continuous (taking any numerical value in an interval). For discrete stochastic variables it is possible to create a probability mass function: $p_x(X) = P(Y = X)$. This gives an expression for the chance a variable X takes the value Y (for example this is the case for the outcome of a dice). For a continuous stochastic variable a probability mass function is not possible, as an continuous stochastic variable can take an infinite amount of values. The chance the variable X takes the (exact) value Y is thereby zero. A solution for the continuous stochastic variables is the use of a probability density function (pdf). The pdf indicates the density of probability in a neighbourhood around a given value.

H.2. METHOD OF MOMENTS

A stochastic variable is defined by its corresponding probability function. This probability function is often based on a small number of parameters. The stochastic variable is then completely based on the type of probability distribution and the parameters. An example of such a parameter is the average value. This average value can be found by using the mathematical concept of expected value of a stochastic variable. The expected value is defined as:

$$E(X) = \frac{\int_{-\infty}^{\infty} x f_X(x) dx}{\int_{-\infty}^{\infty} f_X(x) dx} \tag{H.3}$$

In which $f_X(x)$ is the probability density function of X.

With equation H.3 the weighted average of X is determined. This can also be seen as the X-coordinate of the center of gravity and is therefore seen as the average value of X, denoted as μ_X. Because $\int_{-\infty}^{\infty} f_X(x) dx = 1$ it follows that $E(X) = \int_{-\infty}^{\infty} x f_X(x) dx$. This expected value of X is also expressed as the first moment of the probability density function as it can be seen as the surface of the function times the length (the arm) to the axis X = 0 (see figure H.1).

In the same way, higher order moments can be found as the expected value of X^2. This second moment is called the moment of inertia. In formula form it written as:

$$E(X^2) = \int_{-\infty}^{\infty} x^2 f_X(x) dx \tag{H.4}$$

Of course these rules can be carried on for higher moments, resulting in other properties of the function.

Previous equations H.3 and H.4 are moments in relation to the axis X = 0. As we will see later on it might be handier to relate this to the axis X = μ_X (see figure H.1). These kind of moments are called the central moments. In formula form (k = 1, 2, 3, ..., n^{th} moment):

$$E((x-\mu_X)^k) = \int_{-\infty}^{\infty} (x-\mu_X)^k f_X(x) dx \tag{H.5}$$

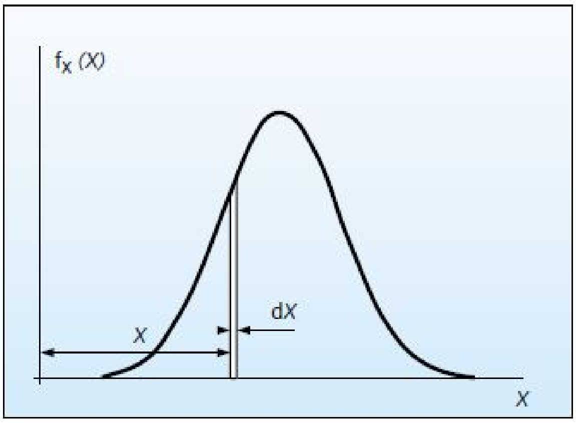
When relating to axis $X = \mu_X$ The first order central moment is 0 per definition. The second order moment, called variance, is of higher interest and is expressed as:

$$Var(X) = E((X-\mu_X)^2) \tag{H.6}$$

As the variance has dimension of X squared, another parameter is introduced called standard deviation (σ_X). In order to get the same dimensions as X the relation of σ to the variance is simple: $\sigma_X = \sqrt{Var(X)}$. The higher the standard deviation the more variance around the expected value is found and thus an higher uncertainty.

H.3. LIMIT STATE FUNCTION

The situation at which just no failure occurs is called the limit stated. Beyond this limit state the structure no longer fulfils its function. The limit state can be written as a function of the strength of the structure and the load on the structure. In a very simplified way it can be said the structure is safe when the strength is higher than the load and fails when the load is bigger than the strength:



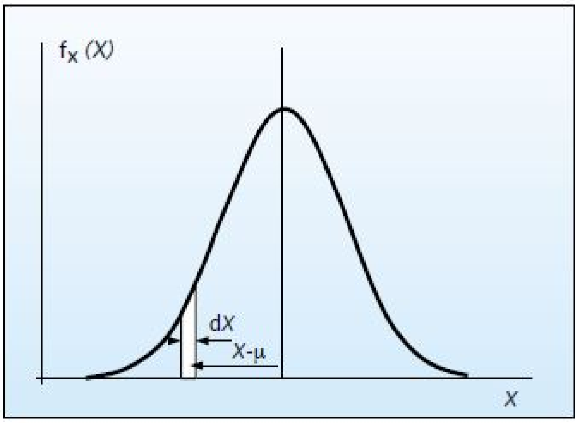


Figure H.1: First order moment (left) and first order central moment (right) (Source: [65])

$$Z = R - S \tag{H.7}$$

In this equation, R represents the resistance and thus the strength and S represents the solicitation which means load. If $S > R$, Z will be come negative indicating the structure fails.

A simple example is the failure of a dike against the mechanism overflow. If the water level (S) rises higher than the height of the dike (R) the dike fails to retain the water and overflow occurs.

The probability that the function Z stays above 0 can be expressed in a reliability function. The probability of failure can be expressed as:

$$P_f = P(Z < 0) = P(S > R) \tag{H.8}$$

The reliability function is the opposite of this function:

$$P(Z \geq 0) = P(S \leq R) = 1 - P_f \tag{H.9}$$

As simple as equation H.7 looks, its execution is not. Both the parameter strength and load are hard to define. Depending on the problem a various amount of factors are influencing the strength of a structure. Quality of constructing, location, time, and climate can have great influence on the strength. As the influences of these factors are most of the time unknown the variable strength is seen as a stochastic variable. The probability function of this variable is most of the time retrieved from gathered data.

Like strength, the load can depend on different factors. Water levels, wave height, weight of the structure, wind loads, and groundwater pressure are examples of parameters that influence the load. Also for strength, a value for the load is hard to predict and thus strength is seen as a stochastic variable. The type of distribution, corresponding with the stochastic variable, depends on the nature of the load. Waves, for instance, can be modelled by using a Rayleigh distribution. Loads can be classified as permanent, variable, and exceptional loads. Permanent loads are loads that are always present and do not fluctuate much in time. For example the structures own weight. Variable loads fluctuate heavily and might even be zero at times. An example is the water level on the outside of the dike. Exception loads occur only in exceptional situations. At almost all times this load does not occur, but there may come a time it will. An example of this is the collision of a ship with a structure.

H.4. PROBABILISTIC LEVEL I, II, III

For using the limit state function (equation H.7), 3 different methods of probabilistic calculation can be used. These methods are expressed in levels and for each higher level the probabilistic approach becomes more prominent. Ideally, the Level III method is used as it is the most accurate. Level II en Level I are approximations of the Level III approach.

- **Level I** This method is the least probabilistic of the three (actually this is more a deterministic approach). No failure probability is calculated. Simply an assessment of safe or not safe is the result of the level I approach. Norms are used in which safety factors are described. These safety factors are used on the loads (safety factor increasing the load) and on the strength (safety factor reducing the strength). If Z is greater than 0 the structure is safe. See equation H.10. In this equation γ is the safety factor.

$$Z = \frac{R}{\gamma} - S * \gamma \;\; (\gamma > 1) \tag{H.10}$$

- **Level II** For this level of approach the use of probabilistic measures increases. Result of this approach is a probability of failure. The limit state function is linearised in a design point. All random variables are approximated by normal distribution functions. Examples of level II methods are FORM and SORM analysis.

- **Level III** The level III method is the most accurate form of the three. The method computes the exact probability of failure of the whole structure using the exact probability density function of all random variables. Examples of level III are the Monte Carlo approach and Numerical Integration.

As the level III is the most accurate form, it is preferred to the other methods. However, the level III method has a few requirements: all the variables and parameters driving the structures' behaviour are known, the parameters' exact distribution is known, and the integration of the joint probability on the safe domain is exact. These assumptions show that the level III approach is not always executable. For this reason the other levels are also used. With increase in level, the accuracy increases, but the complexity increases as well.

H.5. EXTREME VALUE DISTRIBUTIONS

For many cases only the extreme values are of any interest. For example, a designer of a dike needs to design a dike against extreme events like extreme water levels. It in his/her interest to know or predict this value and its corresponding probability or frequency of occurrence. Extreme values of variables can be written as special functions called extreme value functions. Extreme values give the maximum or minimum values of a selection of variables.

$$\begin{aligned} Max(X_1, X_2, X_3, ..., X_n) &= \text{largeste value of the variables } X_1, X_2, ...X_n \\ Min(X_1, X_2, X_3, ..., X_n) &= \text{smallest value of the variables } X_1, X_2, ...X_n \end{aligned} \tag{H.11}$$

If the variables $X_1, X_2, ...X_n$ are stochastic variables, then the extreme values are also stochastic variables. Where the probability distribution of $X_1, X_2, ...X_n$ are the original distribution the probability distribution of the extremes has its own probability distribution called the extreme value distribution.

With the help of some mathematics, it is possible to retrieve an extreme value distribution out of the original distribution. Of course, to do this, the type of distribution of the original distribution must be known. For n samples, the probability function of the minimal values of X is as follows:

$$P(X_1 \le x \cap X_2 \le x \cap ... \cap X_n \le x) = P(X_1 \le x)P(X_2 \le x)...P(X_n \le x) \tag{H.12}$$

And because $P(X_1 \le x) = P(X_2 \le x) = ... = P(X_n \le x) = F_X(x)$ it can be concluded that:

$$P(X_1 \le x \cap X_2 \le x \cap ... \cap X_n \le x) = \left(F_X(x)\right)^n \tag{H.13}$$

This result is equal to the probability distribution of maximum values. Also written as:

$$\left(F_X(x)\right)^n = F_{X_n^n}(x) \tag{H.14}$$

Out of this probability distribution of extreme values the probability density functions can be retrieved by differentiating to X:

$$f_{X_n^n}(x) = \frac{dF_{X_n^n}(x)}{dx} = nF_X(x)\left(F_X(x)\right)^{n-1} \tag{H.15}$$

As also is shown in figure H.2 the probability density function changes depending on the value of n. The extreme value distribution does not have to be the same type of distribution as the original distribution.

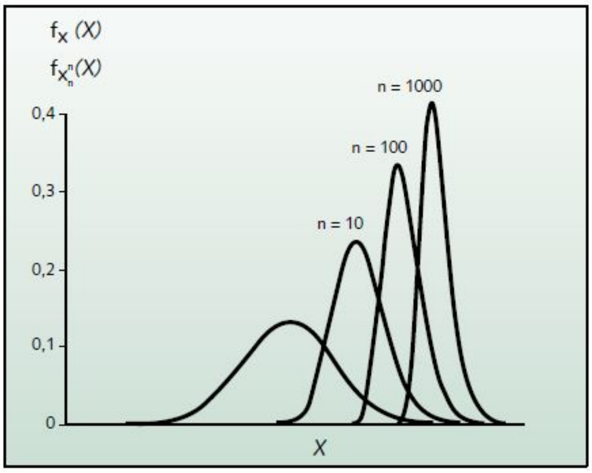


Figure H.2: Probability density function of the maximum values of X (Source: [65])

H.5.1. Annual Maxima Series (AMS) and Peak Over Threshold (POT)

For practical use of equation H.11 two approaches exist. The data can be split up in blocks of which the maximum (or minimum) value is taken. If data during a certain time is measured these blocks can have the length of a certain time interval. If the length of this interval is taken as one year this block maxima series is called the Annual Maxima Series (AMS). For every year this creates one maximum (or minimum) value. Disadvantage of this method is that multiple years of measurements are necessary to create enough data for the extreme value distribution.

Another method that can be used is the Peak over Threshold (POT) method. Over a record all the peak values are extracted which are higher than a certain threshold. If the threshold is set too high, no values might be extracted in any given year. If the threshold is adjusted, then several values can be extracted. This might lead to more extreme value data than the AMS method.

H.5.2. Generalised extreme value theory

The probability function of the extreme values depends on the number of samples (n) that are used (see equation H.13). If the value of n reaches bigger values the extreme value distribution of the stochastic variable is limited to one family of distributions named as the generalized extreme value (GEV) distribution. The GEV distribution consists of the continuous probability distributions Gumbel, Fréchet, and Weibull. These distributions are also known as the asymptotic extreme value distributions. Each of the GEV distribution might also be referred to as type I (Gumbel), type II (Fréchet), and type III (Weibull). Type I and type II are used for maxima and type III for minima.

Source used in this section: [65]

H.6. RECOMMENDED OTHER LITERATURE

Recommended other literature regarding the probabilistic approach is presented in table H.1

Year	Title	Author(s)
2011	Safety standards of flood defenses	Vrijling, J.K.; Schweckendiek, T.; Kanning, W.
2001	Probabilistic risk analysis: foundations and methods.	Bedford, T.; Cooke, R.
1997	Kansen in de Civiele Techniek. Deel 1: De theorie van het probabilistisch ontwerpen	CUR
2010	Risk analysis for flood protection systems	Vrouwenvelder, A.C.W.M., et al.
2004	Reliability analysis of flood defence systems	Steenbergen, H.M.G.M., et al.
1983	First-order concepts in system reliability	Hohenbichler, M.; Rackwitz, R.
2003	Lengte-effect. Memorandum voor het projectbureau VNK2	Thonus, B.
1983	Random fields, Analysis and Synthesis	Vanmarcke, E.
2001	Safety of arches, A probabilistic approach	Schueremans, L., Smars, P., Gemert, D. van
2004	Bayesian probability theory	Olshausen, B.A.
2005	Sampling-based flood risk analysis for fluvial dike systems	Dawson, R., Hall, J., Sayers, P., Bates, P., and Rosu, C.
2012	Monte Carlo-based flood modelling framework for estimating probability weighted flood risk	Kalyanapu, A.J., Judi, D.R., McPherson, T.N., and Burian, S.J.
2006	Adaptive importance sampling for risk analysis of complex infrastructure systems	Dawson, R., and Hall, J.
1997	Optimal dike height under statistical-, damage- and construction-uncertainty	Slijkhuis, K.A., van Gelder, P.H.A.J.M., and Vrijling, J.K.

Table H.1: Recommended other literature

I

VALIDATION OF THE EFFECT OF WIND ON THE WATER SET UP

In section 4.1 and in chapter 5 the effect of wind speed on the water of the IJsselmeer was calculated in a simplified way. By using the following equation the local water set up was calculated (also equation (4.2)):

$$\Delta H = c * \frac{u^2}{g * h} * L \tag{I.1}$$

In this equation the local water level set up due to wind (ΔH) is determined by the wind speed (u), the water level in the IJsselmeer (h), the fetch length (L) and two constants (c and g). For the average water level in the IJsselmeer a value of 5 meter is assumed and for the fetch length a length of 50,000 meter is used. This length is approximately the distance between Urk and Den Oever.

To get an indication of the local water level set up, equation (I.1) gives a good approximation. However, the depth of the IJsselmeer is not constant, the fetch length is currently taken as large as possible (other dike ring elements have shorter fetch lengths) and only from one direction (direction Urk - Den Oever). Apart from the correct values of the parameters, also the approximation of the formula itself is noteworthy. In equation (I.1) no time effect is taken into account. The currently used equation assumes a stationary equilibrium between bottom friction and shear force of wind on the water. Of course, wind speeds will change over time as does the wind direction. To arrive at more accurate results, the non stationary behavior of the wind effect should also be taken into account.

In this appendix, the usage of equation (I.1) is validated by looking at measured values of water levels and wind speeds in the IJsselmeer.

I.1. MEASURED DATA

In 2007, Rijkswaterstaat presented a report titled 'Measured wind-wave climatology Lake IJssel (NL)' [13]. This report presents the results of an extensive wind and wave measuring campaign in the IJsselmeer in the period 1997-2007. The report focused on documenting wind and wave measurements for a range of fetch, depth and wind conditions. Also some measurements are presented about the amount of water level set up or surge.

One of the conclusions of the report was that storm surges for wind speeds of 17-19 m/s (relatively weak storms) are in the order of 30-50 cm for near shore locations. It was also found that the water set up is approximately proportional to $u^{2.2}$. For these conclusions, measured data is used from the stations presented in figure I.1.

If a wind speed of 18 m/s is filled in equation I.1 (and the other variables stay the same), a wind set up of 110 cm is found. This is more than twice than the conclusions of the Rijkswaterstaat report. Partly this difference can be explained by the used fetch length. The 50 km taken in this thesis is the worst possible fetch

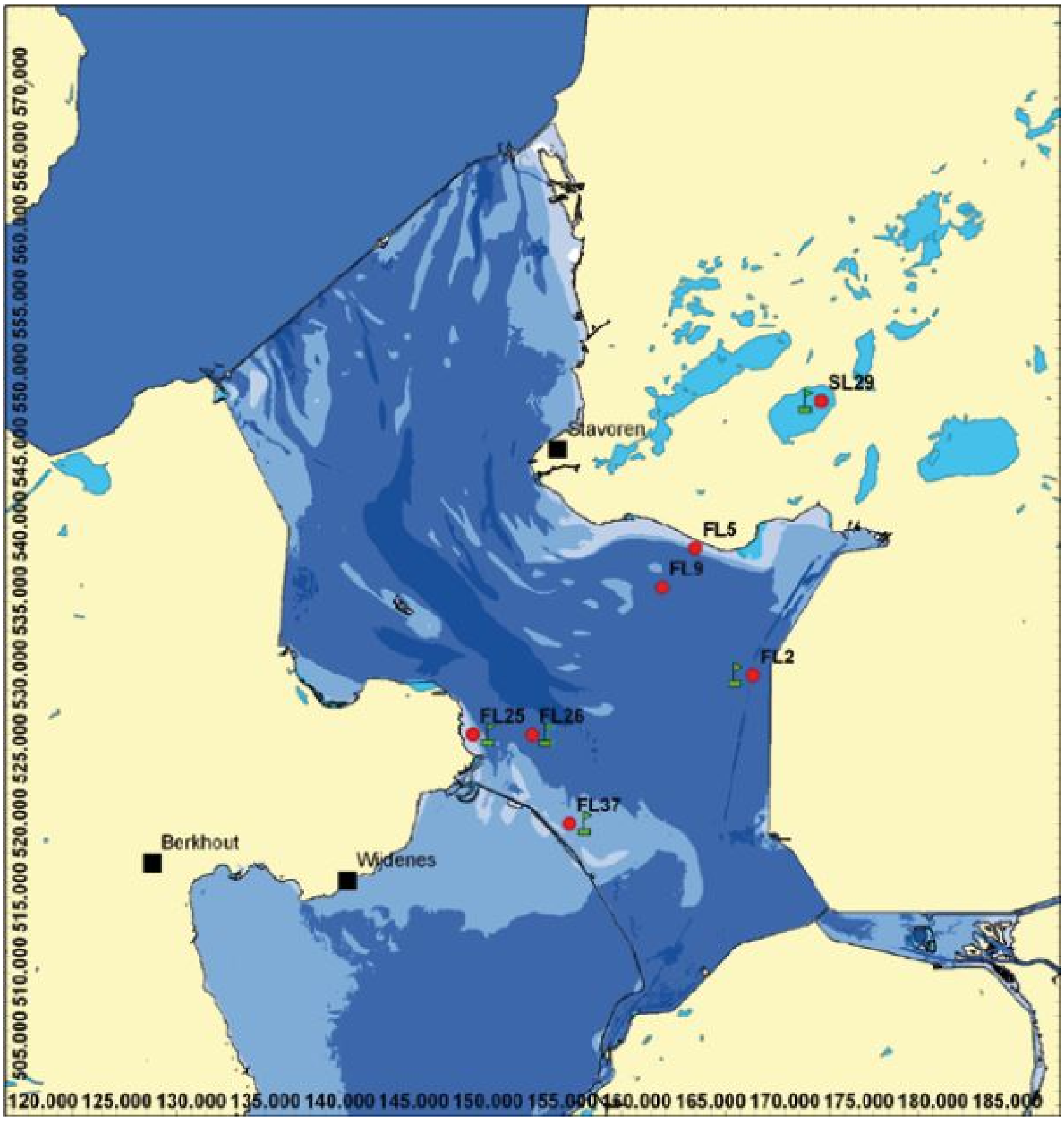


Figure I.1: Used measuring stations in [13]

length (i.e. the longest). It is the longest possible straight line in the IJsselmeer. As can be seen in figure I.1, no (used) measuring stations are situated in the Southeastern point of the IJsselmeer. For the used measuring stations another fetch length should be used. The distance from station FL2 (see figure) to the other side of the IJsselmeer is in the order of 30 km. This changes the wind set up in equation I.1 to 66 cm. This is still higher than the conclusions of Rijkswaterstaat.

In the Rijkswaterstaat report, data is plotted about the surge level and the wind speeds for station FL2. This plot is also given in figure I.2. It only presents wind directions of 240-280 degrees, which is the direction with the longest fetch length. The data is split into three different categories. One category for the average IJsselmeer water level lower than NAP, one category for average IJsselmeer water levels above NAP, and one category for wind speeds with high fluctuating wind speeds over time.

Figure I.3 shows the measured data together with the assumed relation in this thesis. It can clearly be seen that equation (I.1) approximates the measured surge levels. However, the simplified equation overestimates the water level increases.

The measuring campaign of Rijkswaterstaat resulted in a good insight in the wind-wave conditions in the

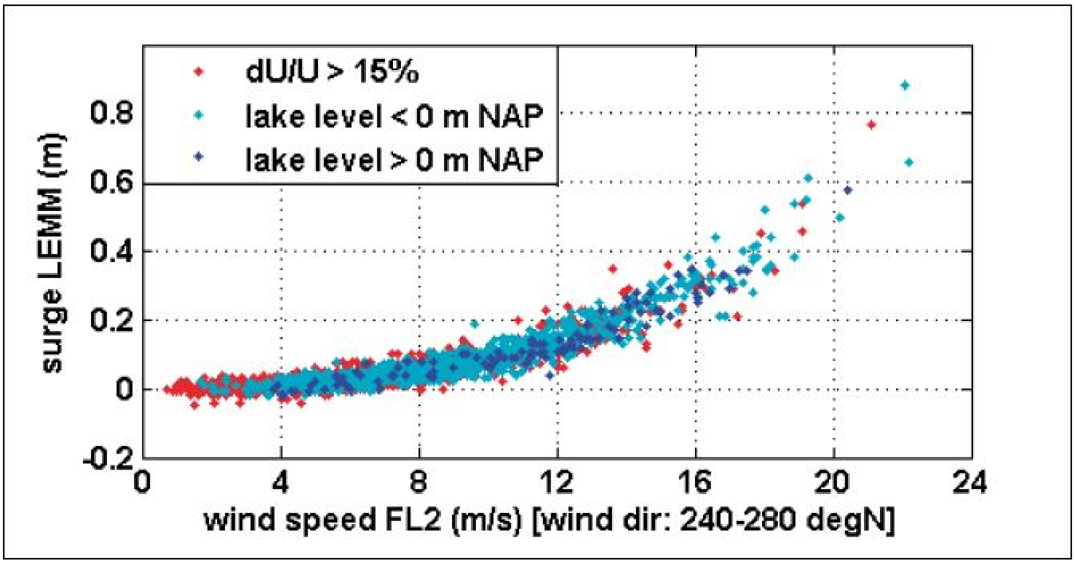


Figure I.2: Wind set up near station FL2 and measured wind speeds for the period 1997-2007 [13]

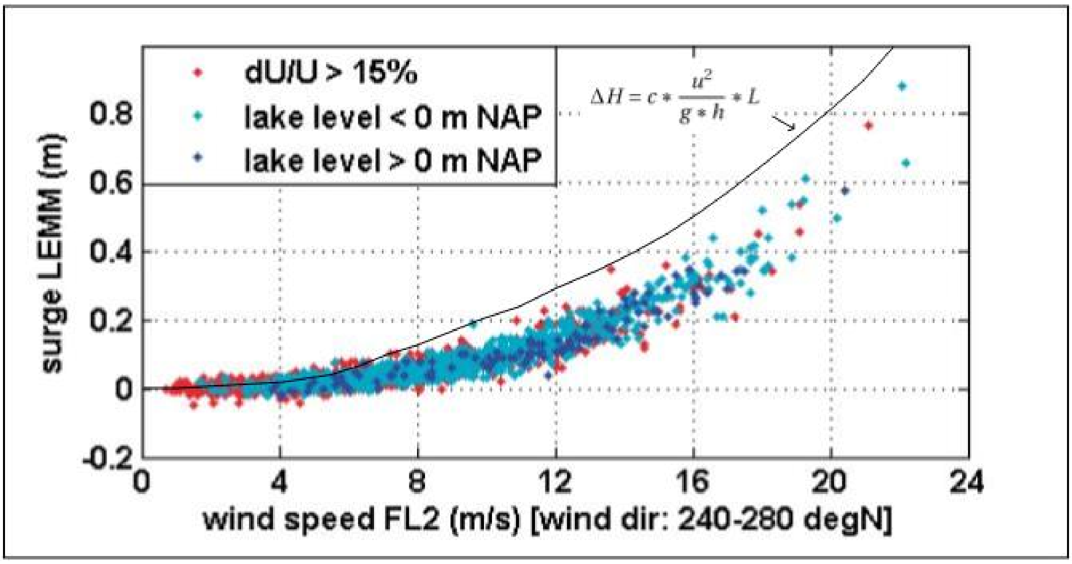


Figure I.3: Wind set up near station FL2 and measured wind speeds for the period 1997-2007 together with the results of equation (I.1) [13]

IJsselmeer, but unfortunately not a single storm took place in this measuring period. Therefore, no extreme wind speeds were found. Because of this, the conditions during very unlikely storms are still unknown.

I.2. NON STATIONARY BEHAVIOR

Up to now, this thesis assumed that the calculated wind set up will follow from a steady state of wind conditions. In reality wind speeds can fluctuate much over time, wind directions can change, or wind speeds can drop down very abruptly. Part of this fluctuation is solved by using hourly averaged wind speeds only, in this way peak wind speeds that do not have a significant effect on the water set up are filtered out. Other non steady behavior is not accounted for.

Rijkswaterstaat showed that the unsteady behavior of the water levels in the IJsselmeer are also of importance [13]. Rapidly changing wind directions or wind speeds can results in complex waves and currents coming from multiple directions. For instance, if the wind directions changes from Northwest to Southwest the increased water level near the Noordoostpolder (due to the Northwestern wind) will be surged up to Lemmer (due to the Southwestern wind). This causes more extreme water set up than water set up due to the effects of Southwestern wind only. Also locally, waves can diffract or break, leading to a complex system of currents and local water levels.

Another important aspect to be considered is the response time of water set up due to wind speeds. According to Rijkswaterstaat [13] it is hardly possible to find the response time in the IJsselmeer. This is because it is very unlikely that quick wind changes during high wind speeds occur during the time the measurements are taken place. Rijkswaterstaat assumes that the response time is more or less equal to the travel time of a long wave in the IJsselmeer which is in the order of 1-2 hours (see also the last two paragraphs on page 53).

Sudden changes in wind speed and wind direction can also lead to resonance oscillations. Rijkswaterstaat also showed that resonance in the IJsselmeer can lead to resonant peak values even larger than water set up due to wind in a steady situation [13]. However, this is highly depended on the location around the IJsselmeer. Of all the measurement locations, station FL2 experiences one of the largest resonance amplitudes. The measurements also show that on a time scale of 0.5-3 hours, the response of IJsselmeer water to wind has generally an unsteady character.

I.3. CONCLUSIONS

The data showed that equation (I.1) can be used, but it must be noted that it gives an overestimation of the measured data. Party, this is caused by the assumption that the fetch will be ideal (maximum length in a certain direction) for high water set up. By fitting the measured data it is found that the surge level is proportional to $u^{2.2}$. This means that probably the overestimation in water level set up during more extreme wind speeds than found in the period of 1997-2007 is not extreme. However, to safely conclude this hypothesis more data (and especially data for wind speeds in the order of 26 to 29 m/s) are needed.

It can be assumed that the response time of water set up due to wind is more or less equal to the travel time of a long wave in the IJsselmeer. As the expected travel time of the water flow in through the breach also used this assumption (see section 4.1), both delays cancel each other out.

Due to unsteady response of the water level to wind, wind set up can be up to three times as large as wind set up during steady wind conditions. This is caused by resonance phenomena in the IJsselmeer and are very location and situation dependent. For time scales more than 3 hours resonance has not much effect and a (quasi-)steady system can be assumed. Although this factor three is very unlikely and the whole breaching process is assumed to last at least a couple of hours, the effects of unsteady wind conditions on the IJsselmeer system are very much relevant. Therefore it is highly recommended that these effects are investigated to a larger extant.

J
Influence of the IJsselmeer water level on the amount of discharge through the breach

In this appendix, a sensitivity analysis is carried out to see if the IJsselmeer water level has any significant influence on the discharge through the breach. For this analysis the Matlab model is run twice for each scenario. The first run is the original run (as presented in section 4.5 and the second run is exactly the same but now the water level in the IJsselmeer is 2 meters lower than in the first run. The difference of two meters is assumed to be the approximate wind set down at the inside of the Afsluitdijk once the Afsluitdijk will breach.

Figure J.1, J.2, J.3, and J.4 present the results of the sensitivity analysis for every scenario. The red line represents the original Matlab script, the blue line represents the adjusted Matlab script (i.e. all IJsselmeer water levels are downgraded with 2 meters). Only at around the jump in scenario 1 and around the frequencies of 1/100,000 minor differences are found. These minor differences are minimized if a higher number of Monte Carlo runs is executed. Therefore it is safe to assume these differences are caused by insecurities in the model rather than the difference in IJsselmeer water level.

For all the other frequencies, the two lines are more or less at the same place. This indicates that whether or not wind set down is taken into the model, the results stay the same. This means it can be concluded the discharge through the Afsluitdijk is hardly effected by the water level on the inside of the breach. This also means that the Matlab script used in chapter 4 does not have to take the amount of wind set down during an extreme storm into account. No (cor)relation between wind set down and peak water level in the Waddenzee during an extreme storm is needed in the model.

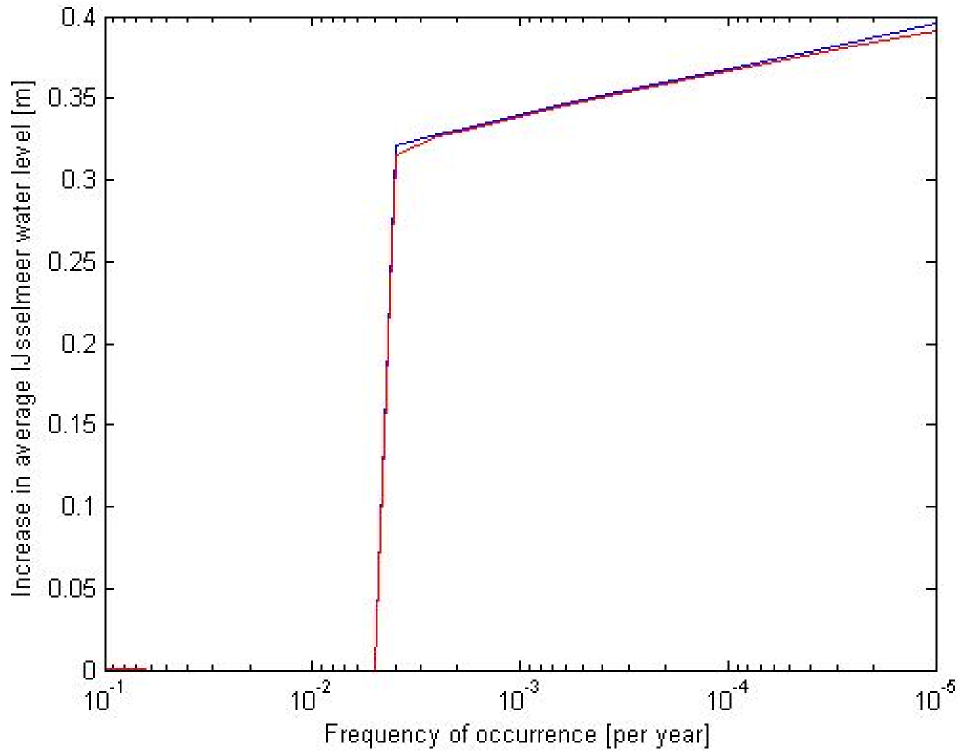


Figure J.1: Sensitivity analysis with scenario 1

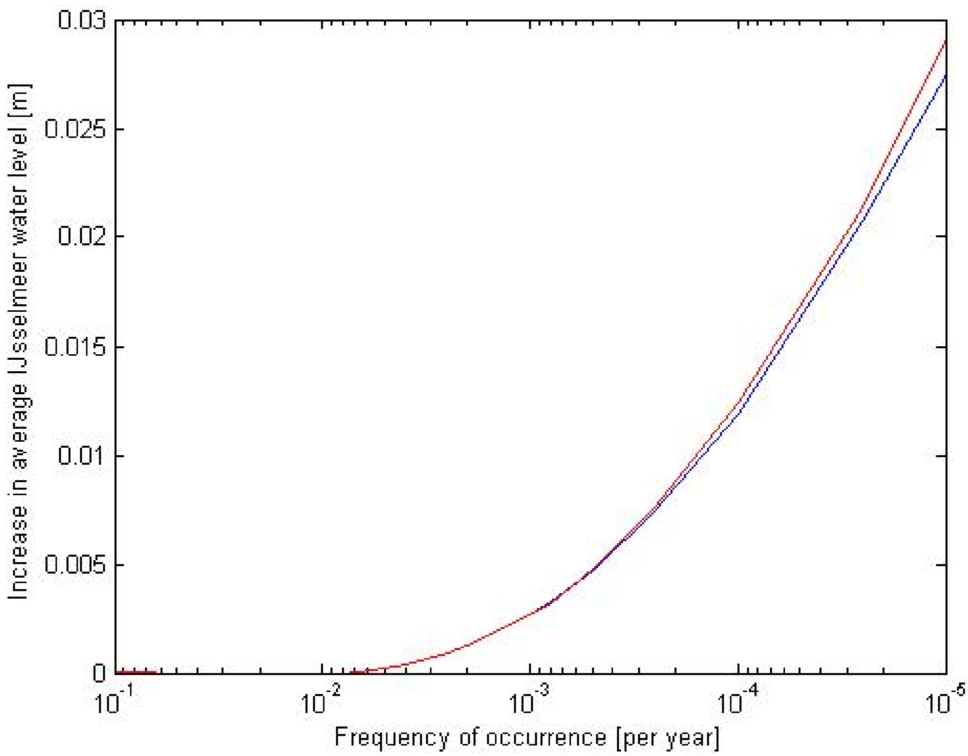


Figure J.2: Sensitivity analysis with scenario 2

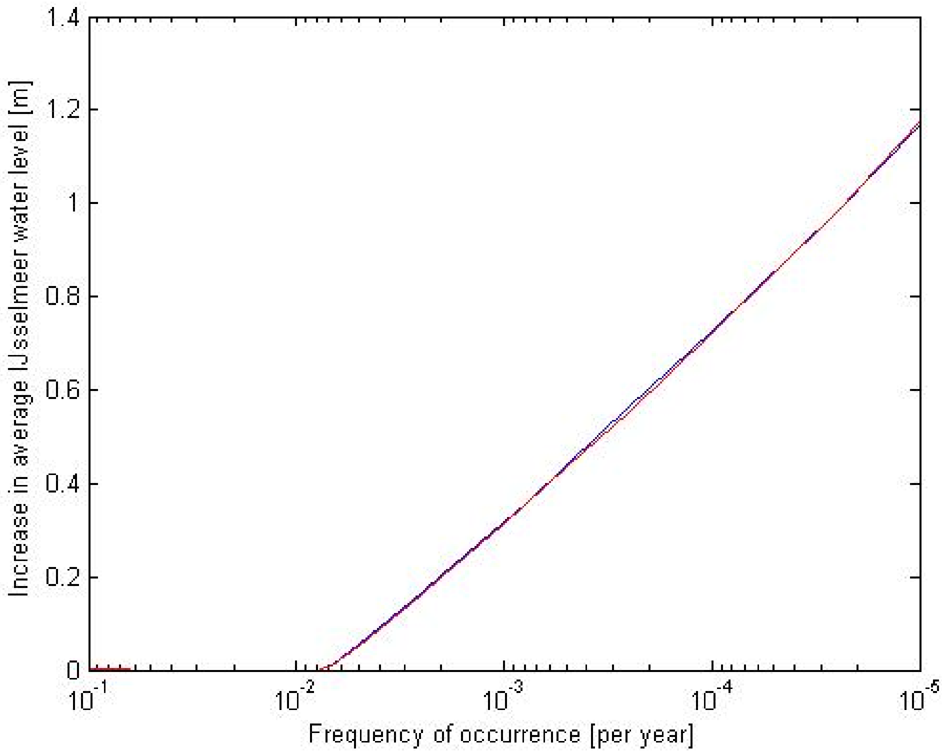


Figure J.3: Sensitivity analysis with scenario 3

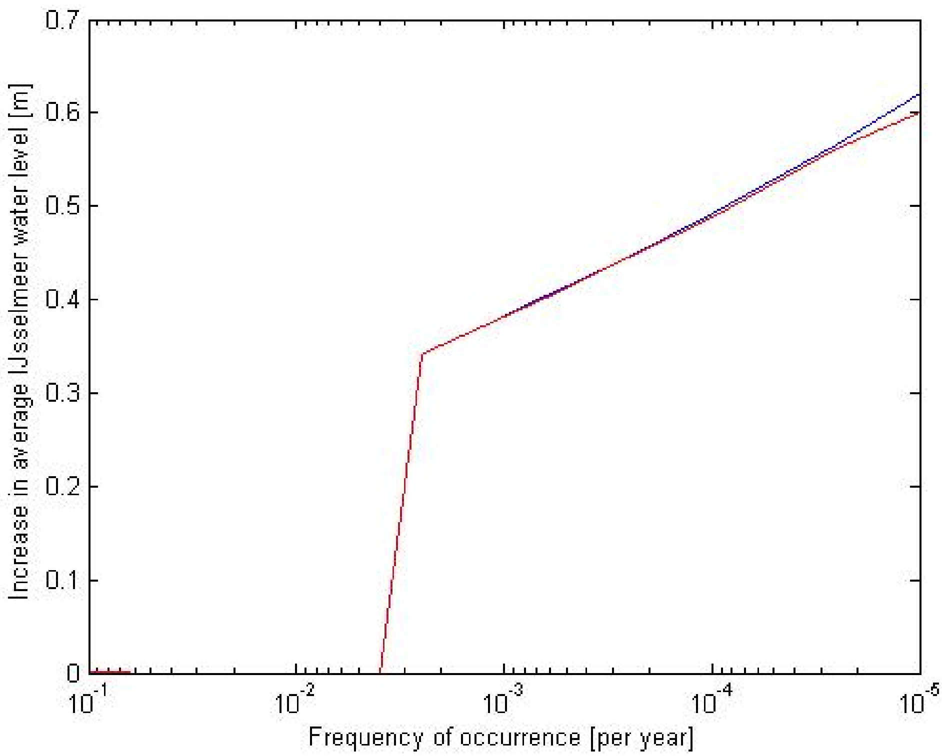


Figure J.4: Sensitivity analysis with scenario 4

K

Results Hydra-M model

Below the results of the Hydra-M model are given. These results are used in section 4.2.1 to find the probability density function for the IJsselmeer water level.

```
** Hydra-M voor het toetsen van waterkeringen versie  1.4 **

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Rijkswaterstaat : RIZA   (afd. WRV)                blz.  :       1
Model           : Hydra-M (versie  1.4)
Gebruiker       : akkp
Datum           : 13:10:04 Tuesday, August 19, 2014
----------------------------------------------------------------------

Faalmechanisme : waterstand
++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++


Weergave ingevoerde gegevens
****************************
Invoerbestand             :  C:\PROGRA~2\Hydra-M\userdata\sommen\01A Wieringermeerdijk Noord@Pr_1op4@WS_000_04000_xxxxx.IN
Uitvoerbestand            :  C:\PROGRA~2\Hydra-M\userdata\sommen\01A Wieringermeerdijk Noord@Pr_1op4@WS_000_04000_xxxxx.EXT
Locations                 :  C:\PROGRA~2\Hydra-M\data\data_ym\hydra2001loc_ysm.txt
Randvoorwaarden           :  C:\PROGRA~2\Hydra-M\data\data_ym\hydrvw_r.ym
MeerpeilStatistiek1       :  C:\PROGRA~2\Hydra-M\data\data_ym\of_mptop.dym
MeerpeilStatistiek2       :  C:\PROGRA~2\Hydra-M\data\data_ym\of_mpdag.dym
WindStatistiekJaar        :  C:\PROGRA~2\Hydra-M\data\windstat\p_wijaar.dat
WindStatistiekDag         :  C:\PROGRA~2\Hydra-M\data\windstat\p_widag.dat
WindStatistiekSec         :  C:\PROGRA~2\Hydra-M\data\windstat\p_wrsec.dat


Gebied                    :  IJsselmeer
Locatie                   :  01A Wieringermeerdij
Profielcode               :  Pr_1op4
X-coördinaat              :       133008  (m)
Y-coördinaat              :       548201  (m)
Kruinhoogte               :         4.65  (m+NAP)
Dijknormaal               :          230  (graden tov noord)

 *******  Tabel overschrijdingsfrequenties  *******

       Terugkeertijd          Waterstand
          (jaren)           (m. tov. NAP)
       +++++++++++++    +++++++++++++++++++++++
           10.000              0.379
           25.000              0.446
           50.000              0.500
          100.000              0.565
          250.000              0.655
          500.000              0.727
         1000.000              0.803
         1250.000              0.830
         2000.000              0.885
         4000.000              0.971
        10000.000              1.091

======================================================================
♀

++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++
Rijkswaterstaat : RIZA   (afd. WRV)                blz.  :       2
Model           : Hydra-M (versie  1.4)
Gebruiker       : akkp
Datum           : 13:10:04 Tuesday, August 19, 2014
----------------------------------------------------------------------

Faalmechanisme : waterstand
++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++++


  ***** Tabel illustratiepunten per windrichting *****

 Frequentie waarvoor het illustratiepunt is bepaald: 1/  4000 (jaar)
 Waterstand bij deze frequentie:   0.97 (m+NAP)

 Gegevens per windrichting waarvoor de waterstand
 optreedt:


 r         | u         | meerpeil | waterst  | golfh     | piekp     | golfr     | ov. freq
 (graden)  | (m/s)     | (m+NAP)  | (m+NAP)  | (m)       | (s)       | (graden)  | (%)
----------+----------+----------+----------+----------+----------+----------+----------
     0.0  |    4.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |     1.3
    30.0  |    5.00  |    0.96  |    0.97  |  ----    |  ----    |  ----    |     1.4
    60.0  |    0.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |    15.7
    90.0  |    0.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |    16.6
   120.0  |    0.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |    12.1
   150.0  |    0.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |    16.6
   180.0  |    0.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |    23.5
   210.0  |    4.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |     4.1
   240.0  |    3.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |     3.2
   270.0  |    3.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |     2.4
   300.0  |    3.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |     1.6
   330.0  |    6.00  |    0.97  |    0.97  |  ----    |  ----    |  ----    |     1.4
----------+----------+----------+----------+----------+----------+----------+----------
 som      |          |          |          |          |          |          |   100.0

 Illustratiepunt (combinatie windrichting-windsnelheid-
 meerpeil met grootste frequentie van optreden):
```

r (graden)	u (m/s)	meerpeil (m+NAP)	waterst (m+NAP)	golfh (m)	piekp (s)	golfr (graden)	ov. freq (%)
180.0	0.00	0.97	0.97	----	----	----	23.5

Betekenis gegevens:
- r   = de windrichting (graden t.o.v. Noord)
- u   = de windsnelheid in het illustratiepunt (m/s)
- meerpeil = het meerpeil in het illustratiepunt (m+NAP)
- waterst = de waterstand in het illustratiepunt (m+NAP)
- golfh   = de significante golfhoogte in het illustratiepunt (m)
- piekp   = de golf(piek)periode in het illustratiepunt (s)
- golfr   = de golfrichting in het illustratiepunt in nautische conventie zoals voor de windrichting (graden)
- ov. freq = bijdrage van de windrichting aan de overschrijdingsfrequentie

De toeslag voor slingeringen is niet in de berekeningen verwerkt. Zie hiervoor het Voorschrift Toetsen op Veiligheid of het Randvoorwaardenboek.

L

Results of the Matlab simulations

In section 4.5 of the main report, the results of the Matlab simulations were presented. In this appendix a detailed overview is given. The detailed results of the Matlab simulation are given in figure L.1, L.2, L.3, L.4 and their corresponding tables L.1, L.2, L.3, and L.4. The figures (starting on next page) present the increase in average IJsselmeer water level because of the possibility of a breach in the Afsluitdijk. It shows the increase in water level after the last hour of the storm. The corresponding tables show the increase in water level for their corresponding return period. Also, for each scenario a short analysis is written.

L.1. RESULTS SCENARIO 1

Figure L.1 and table L.1 show the results for scenario 1. The jump in the figure is the effect of the failure of the sluices. More extreme storms than the 1/250 storm do not have much influence on the final increase in IJsselmeer average water level at the end of the storm. Failure of the sluices cause a 33 cm increase in water level. An outside water level of 5.48 meter (for a 1/100,000 storm) instead of 4.18 meter (for a 1/250 storm) only increases the average IJsselmeer water level with 6 extra cm. This means scenario 1 is dominated by whether it fails or not. If it fails the consequences are immediately extreme.

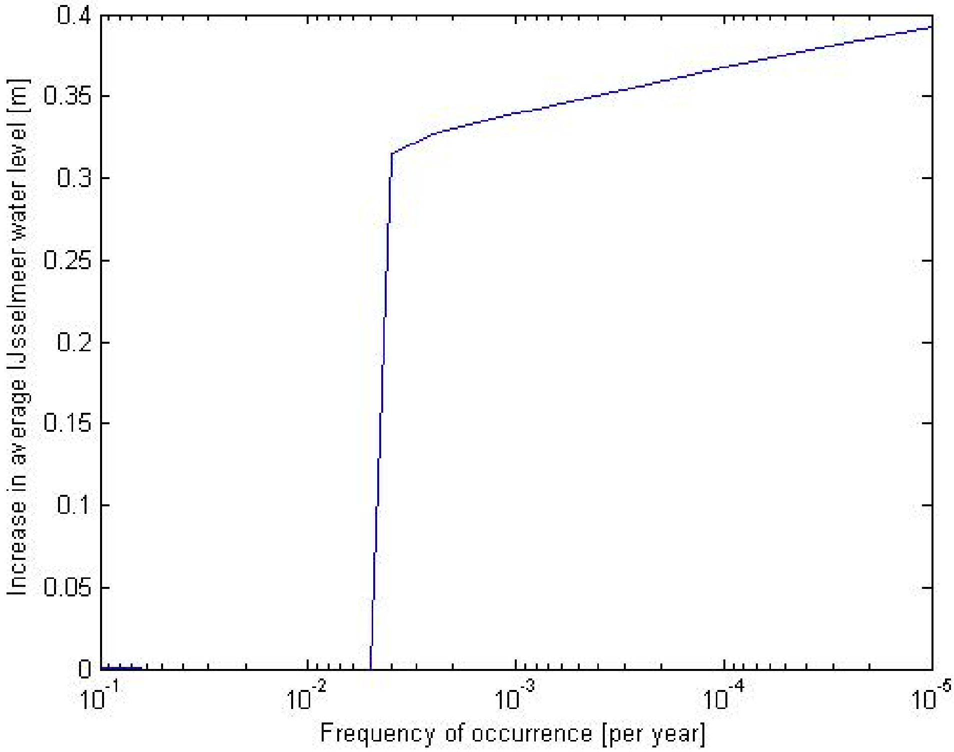


Figure L.1: Results of scenario 1

Return period [years]	Water level increase [m]	Return period [years]	Water level increase [m]
100	0	1,000	0.34
125	0	1,250	0.34
150	0	2,000	0.35
200	0	4,000	0.36
250	0	10,000	0.37
400	0.33	40,000	0.38
500	0.33	100,000	0.39

Table L.1: Increase in average water level in the IJsselmeer with its corresponding frequencies for scenario 1

L.2. RESULTS SCENARIO 2

The results of the Matlab-model for scenario 2 are given in figure L.2 and table L.2. Where at first sight the increase in average IJsselmeer water level looks to explode, the actual values are not that bad. In the very unlikely event of a 1/100,000 year storm the water level in the IJsselmeer only increases with 3 cm. This is such a low value there is no need to reckon with the consequences of failure of the Afsluitdijk for the safety of the dikes around the IJsselmeer in this scenario.

This low increase of water level is mainly caused by the boulder clay layer staying intact. Because of this layer, the depth of the breach is limited to 2m +NAP. Only in the first hours of the storm water will flow in. Soon after the peak Waddenzee water level, the water levels will be low enough to cause only minimal to no head difference over the reduced crest level of the Afsluitdijk.

It can be concluded that if the boulder clay layer will stay fully intact during (all) extreme storms, the influence of a breach in the Afsluitdijk on the IJsselmeer is negligible.

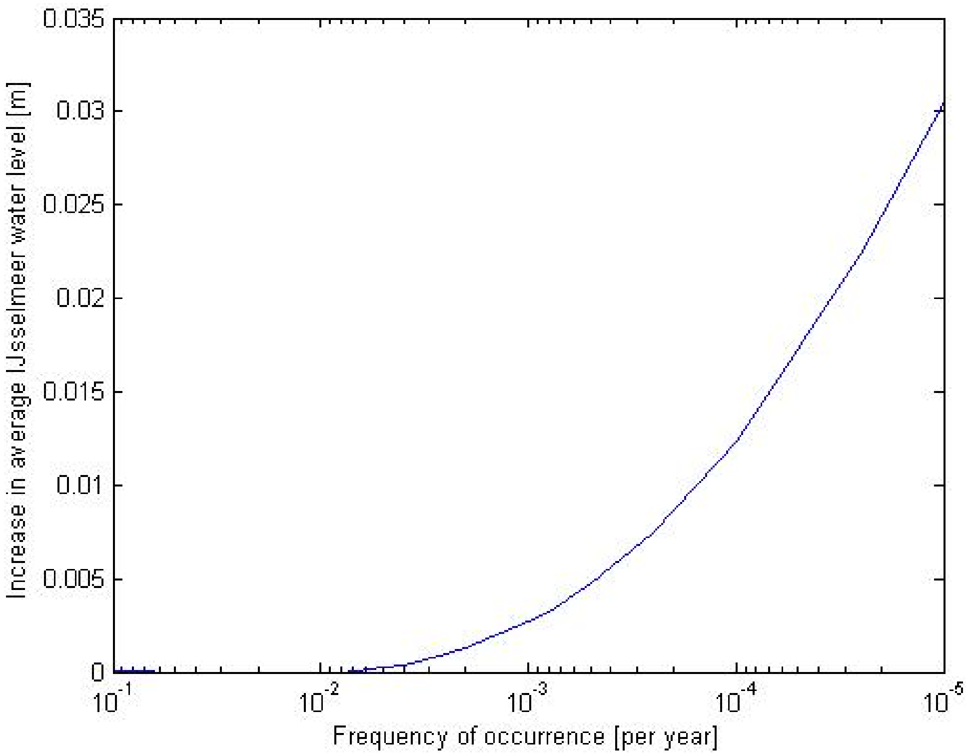


Figure L.2: Results of scenario 2

Return period [years]	Water level increase [m]	Return period [years]	Water level increase [m]
100	0	1,000	0.00
125	0	1,250	0.00
150	0	2,000	0.00
200	0.00	4,000	0.01
250	0.00	10,000	0.01
400	0.00	40,000	0.02
500	0.00	100,000	0.03

Table L.2: Increase in average water level in the IJsselmeer with its corresponding frequencies for scenario 2

L.3. RESULTS SCENARIO 3

Scenario 3 gives the most significant results. The combination of the large width of the breach (1300 meter at a 1/10,000 storm) together with a complete erosion of the boulder clay layer leads to high water level increases. At low return periods (150-400 years) the water level increase is in the order of centimetres (see figure L.3) which is acceptable. At higher return periods (i.e. lower frequencies) the results become problematic. With a frequency of once in 10,000 year the average water level will increase with approximately 70 centimetres (see table L.3) which is quite significant for an average depth of the IJsselmeer of approximately 5.5 meters.

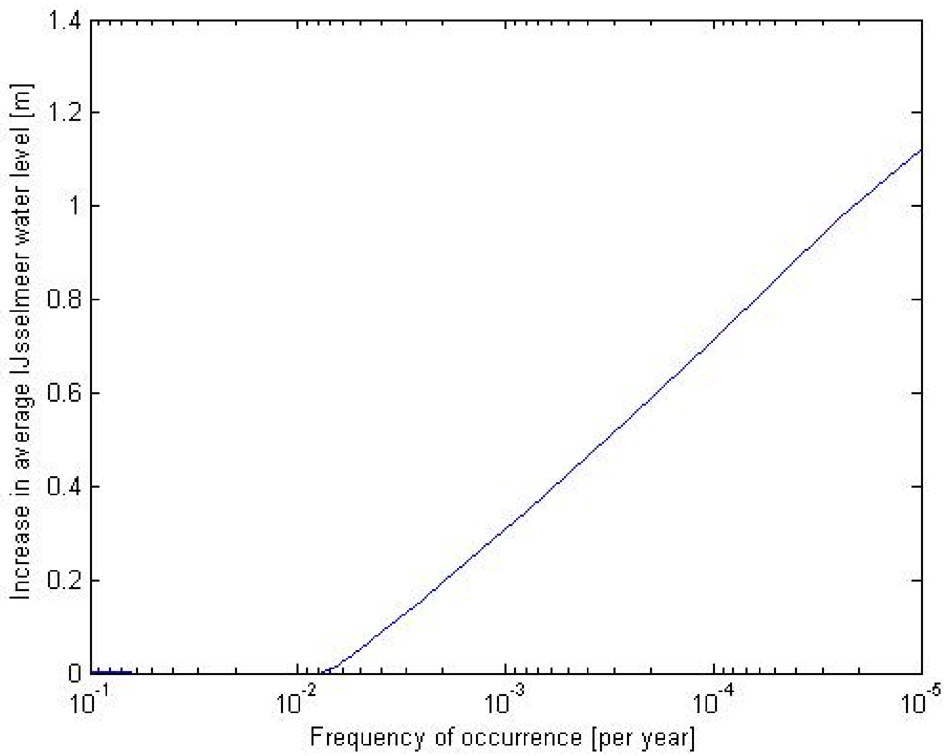


Figure L.3: Results of scenario 3

Return period [years]	Water level increase [m]	Return period [years]	Water level increase [m]
100	0	1,000	0.31
125	0	1,250	0.35
150	0.01	2,000	0.43
200	0.05	4,000	0.55
250	0.09	10,000	0.71*
400	0.16	40,000	0.97
500	0.19	100,000	1.15

Table L.3: Increase in average water level in the IJsselmeer with its corresponding frequencies for scenario 3

*: The 0.71 meter increase of average water level during a 1/10,000 year storm is much more than the 0.33 meter increase found in Visser's thesis [28]. For this 0.33 meter, the same scenario and 1/10,000 year storm (i.e. outside water level of 5.02 m +NAP) is used. This difference can be explained by the following two reasons. First one is the time taken for the breach to develop. Based on the assumptions of Deltares [20], this thesis uses a linear development of the breach for 12 hours after the peak of the storm. Visser assumes the same linear development but assumes the process takes 16 hours. Second difference is the description of the outside water level. Visser assumes the outside water level is lower than the breach depth as soon as the

depth of the breach is fully developed (i.e. 16 hours). This thesis assumes a linear decrease in outside water level which reaches 0 m +NAP after 30 hours. If these two differences are changed in the Matlab model used in this chapter, the results are (also) a couple of decimetres lower than 0.71 meter.

L.4. RESULTS SCENARIO 4

Figure L.4 and table L.4 present the results for scenario 4. The figure clearly shows scenario 4 is a combination of scenario 1 and 3 (or 2). The jump around the failure frequency is found as can also be seen in the results of scenario 1. Different to scenario 1 is the development of the line at lower frequencies. After the jump, the line follows the same characteristics as scenario 3 only less steep as a lower width and smaller depth for the levee breach are assumed in this scenario.

At the return period of 10,000 years the water level increase is approximately half a meter which makes the consequences of this scenario worth investigating in the next chapter(s). Also the relatively high water level increases during the lower return periods (~ 400 years) are also interesting and might have big consequences later on.

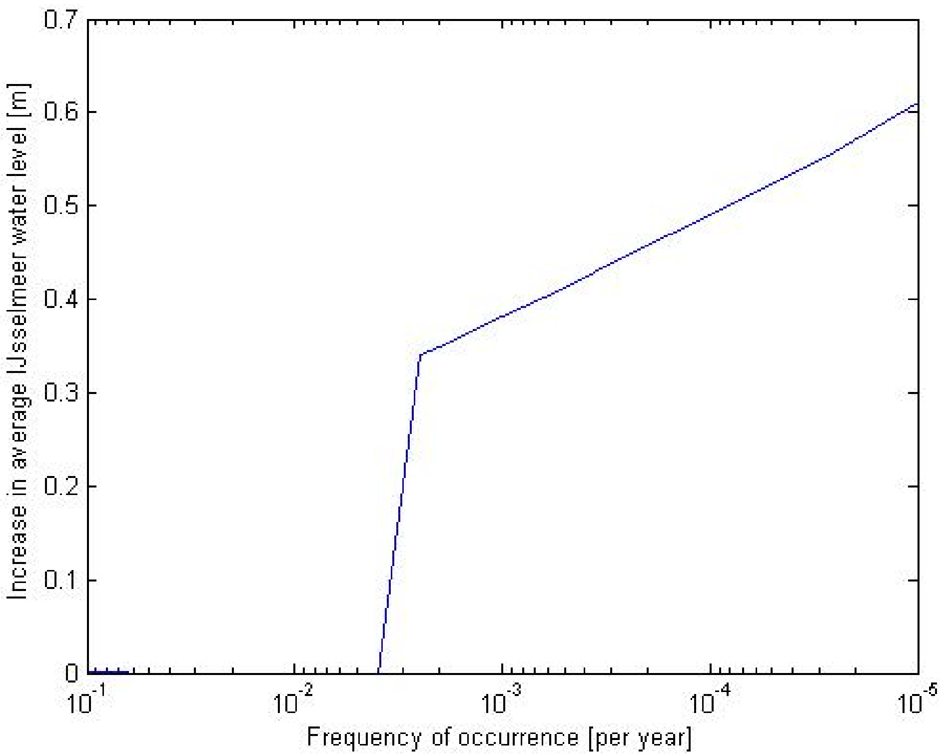


Figure L.4: Results of scenario 4

Return period [years]	Water level increase [m]	Return period [years]	Water level increase [m]
100	0	1,000	0.38
125	0	1,250	0.39
150	0	2,000	0.41
200	0	4,000	0.45
250	0	10,000	0.49
400	0.34	40,000	0.56
500	0.35	100,000	0.61

Table L.4: Increase in average water level in the IJsselmeer with its corresponding frequencies for scenario 4

M

Sensitivity of the assumptions on the development of the storm

In section 3.7 and in appendix E the assumed development of the Waddenzee water level during the storm was described. For this relation the measurements of the water level of the storm in 1953 were used, as described in the documentation of the storm of 1953 [22].

The used assumptions and simplifications might have an impact on the final conclusions of this thesis. In this appendix the calculations of this thesis are done once more with different assumptions in the description of the storm in the Waddenzee and the timing of the breach during that storm. By keeping the assumptions for all other phenomena the same, the sensitivity of the used assumptions regarding the development of the storm is found.

M.1. Description of the water level

In this thesis the water level during the storm is simplified as a triangle (see figure E.4). The basis of the triangle lies at NAP level and the peak at the peak water level corresponding with the frequency of the storm (e.g. the 1/10,000 per year storm gives a peak water level of NAP + 5.12 m). From NAP level before the storm to the peak level of the storm a linear development is assumed which takes 20 hours. After the peak the storm, the water level will decrease to NAP again. This process is assumed to take 30 hours and will also be linear. This development is shown in figure M.1 as the blue line. The schematization of this development is based on the measurements of the storm in 1953.

In 'Leidraad Zee- en Meerdijken' by TAW [66] a different development of the water level is described. This guideline (Dutch: leidraad) is also used for the hydra models (see page 39 for a description of the hydra models)[1]. It uses a combination of a simplification of the storm set up and a simplification of the average tide at the location. Together they form the description of the water level during the storm (see the yellow line in figure M.1).

The simplification of the tide is taken as the average amplitude of the tide and is timed in such a way the peak water level of the tide coincides with the peak of the storm (see the grey line in figure M.1). The simplification of the storm set up is less straightforward and looks much the same as the simplifications used in this thesis. The guidelines state that the total storm duration (for the Waddenzee) should be taken as 45 hours. The peak of the storm set up lies in the middle of the storm (i.e. 22.5 hours). Two hours before and after the peak of the storm the storm set up is (only) 10 cm lower than the peak value of the storm set up. Before these marks, the storm set up increase linearly and after it decreases linearly. The peak of the storm set up has a height of the peak of the storm (i.e. NAP + 5.12 m) minus the peak amplitude of the tide.

The differences between the two simplifications are only minor (see the blue and the yellow line in figure M.1). The main difference is that in the simplification of TAW the effects of tide are taken into account where

[1] This description is also used in the Deltares report [20]

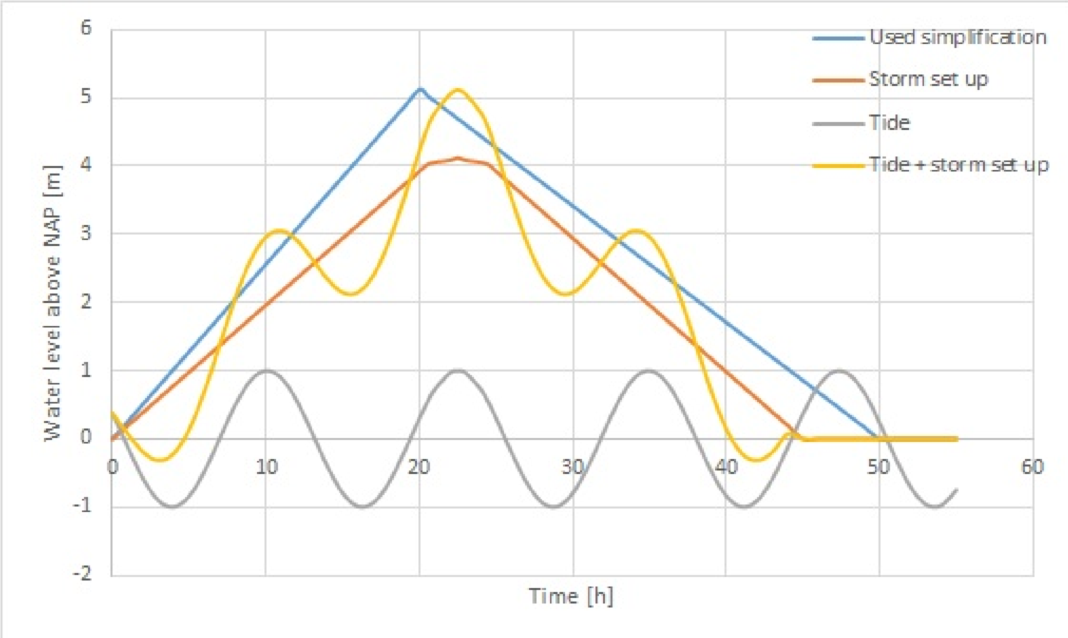


Figure M.1: Assumed water level development during the storm (in blue) and the water level development according to [66] (in yellow)

in this thesis this is effect is simplified by conservatively taking the the maximum tide level everywhere during the storm. Judging on figure M.1 it looks like the assumed development of the water level during the storm is taken conservatively in this thesis when compared to the guidelines of TAW. Only at the peaks of the tide the TAW simplifications slightly exceed the used assumptions. During every other hour the blue line is the 'worst case scenario'. Whether this is correct will be discussed in the upcoming sections.

M.2. ASSUMPTIONS REGARDING THE BREACH TIME

For the conclusions in the end of section 5.1 another assumption might be very crucial. In this thesis it is assumed that the breach starts to develop at the peak water level of the storm regardless of the magnitude of the storm. This approach is reasonable if the frequency of the storm is more or less equal to the probability of failure of the Afsluitdijk (e.g. a storm with frequency of 1/140 per year for a breach at the levee). However, this thesis uses the same assumption for every other storm. For the 1/10,000 per year storm, the breach will mostly likely start sooner than the peak as the water levels, crucial for the Afsluitdijk, are also at lower values than the peak values of the storm. This is a different (and more realistic) assumption than was used in this thesis.

The sensitivity of this assumption regarding the timing of the breach during the storm will also be investigated. This is done by taking this effect also into account in next section.

M.3. DEVELOPMENT OF INCREASE OF WATER LEVEL DURING THE STORM

To look whether or not the used assumptions regarding the Waddenzee water level description and the timing of the breach have a big impact on the conclusions on the impact of failure of the Afsluitdijk on the IJsselmeer dikes during the same storm, the same approach as in section 5.1 is done again. Only this time the water level description according to the guidelines is used and the breach time is taken at the time the water level of failure is reached (and not the peak of the storm).

Again, equation (5.1) is used. The average water level increase due to discharge through the breach is calculated with the Matlab script and for the wind set up in the local IJsselmeer water level the same assumptions are used. The two changed assumptions are changed in the Matlab script and the results in figure M.2 and figure M.3 are found.

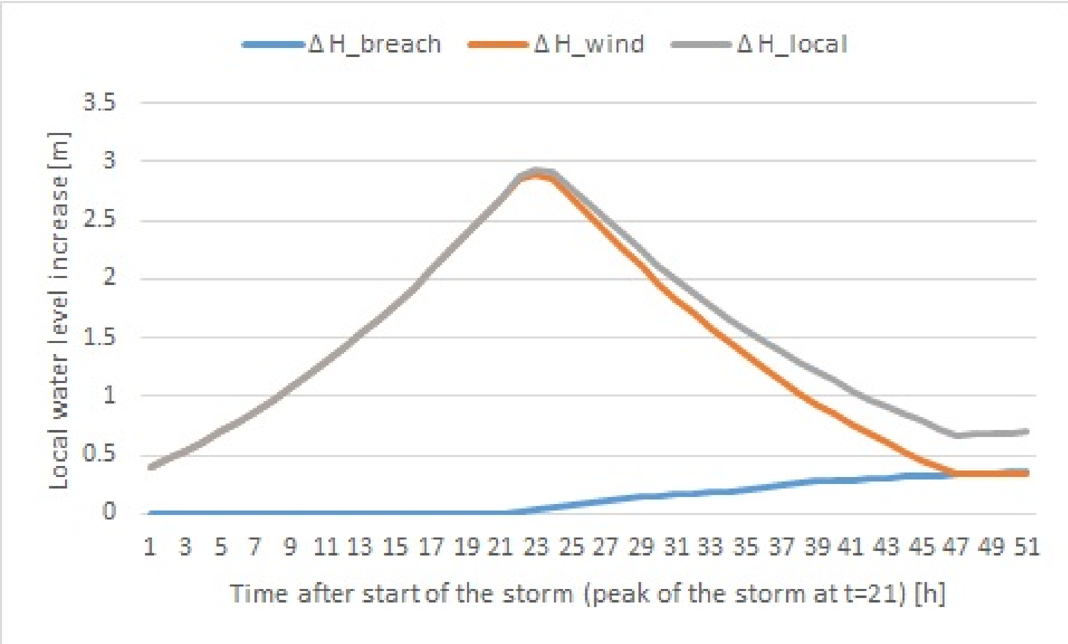


Figure M.2: Water level increases due to failure of the Afsluitdijk and wind set up for scenario 1 and an 1/10,000 per year storm. Same figure as figure 5.2 but with other assumptions.

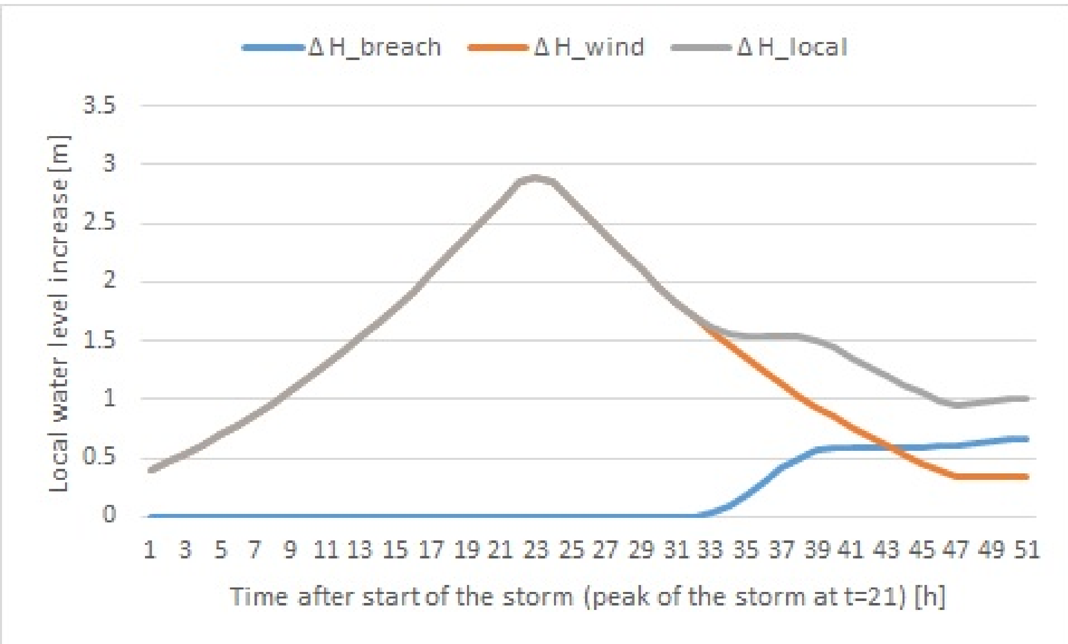


Figure M.3: Water level increases due to failure of the Afsluitdijk and wind set up for scenario 3 and an 1/10,000 per year storm. Same figure as figure 5.3 but with other assumptions.

M.4. CONCLUSION ON THE SENSITIVITY OF THE USED ASSUMPTIONS

It can be seen that figures M.2 and M.3 are very similar to figures 5.2 and 5.3. However, the differences caused by the two changed assumptions are also very noticeable. At scenario 3, the influences of tide can be seen by the 'shocking' behavior of the increase in water level due to the breach. As the outside water level fluctuates, the discharge through the breach does as well. The magnitude of the second high tide (approximately 12.5 hours after the peak of the storm) mainly determines the amount of discharge through the breach. For scenario 1 the change in water level description is not easily recognized. Although there are little 'bumps', this is not very noteworthy [2]. Also, the values found in this sensitivity analysis are a little lower than the values found in the thesis (section 5.1.3). This confirms the statement in the end of section M.1 that the taken water level description in this thesis is a conservative assumption.

The other changed assumption (timing of the breach) shows little change in the outcome of the calculations. Looking at the figures only the early breach can't be noticed at scenario 3 (because the breach in the levee needs time to develop) and it is only slightly noticeable at scenario 1 (the orange line starts to deviate a little from the grey line just before the peak). This indicates that the water inflow through the breach is influencing the local water level at the maximum wind speeds (peak of the storm) and thus increases the local governing loads.

However, the increase is only minor at the peak of the storm even with a complete blow out of all of the sluices. It is found that if taking failure of the Afsluitdijk before the peak also into account (for reasons discussed in section M.2) the starting time of the breach will lie only 2 to 3 hours before the peak of the storm. This results in a water level increase of 3.5 cm for scenario 1 and no increase for scenario 3. These values are negligible. Especially when taking the other assumptions (no travel time of Waddenzee water to the other side of the IJsselmeer, no inertia in the wind - water set up relation, etc.) into consideration.

Finalizing this appendix, it can be concluded that also with the water level description of the guidelines of TAW and the added assumption that the breach will start during the build up of the storm the conclusions in the end of section 5.1.3 still hold.

[2] This difference between scenario 1 and 3 can be explained by the underflow and overflow formula used for both scenarios (see equation (4.7) and (4.13)). Overflow (scenario 3) discharge is proportional to $h^{(3/2)}$ where underflow discharge only to $h^{(1/2)}$

N

Statistics for hydraulic conditions in PC-Ring

On the next page, in figure N.1, the original data file of location N170 Westermeerdijk is given (N170 is one of the four investigated locations on the Westermeerdijk, see figure 5.10). The values of the local water level are increased manually to implement the effects of a breach. For more information on this implementation see section 5.2.2. Note: Posting the whole file in this appendix will grow this appendix to 10 pages, therefore only wind direction 0.00 degrees (i.e. North) is presented. The values for the other 11 wind directions are presented in the same way in the input file (of course, the values for significant wave height, wave period, wave direction and local water level differ).

Lokatie: 168462.000 526787.000 N170 Westermeerdijk

Windrichting [graden]: 0.00

Meerpeil [m+NAP]	Windsnelheid [m/s]	Sig. golf-hoogte [m]	Gem. golf-periode [s]	Golfrichting [graden]	Waterstand [m+NAP]
-0.4	14	0.85	4.512	288	-0.27
-0.1	14	0.86	4.588	289	0.02
0.4	14	0.87	4.688	291	0.51
1	14	0.88	4.787	292	1.1
1.8	14	0.9	4.9	293	1.89
-0.4	19	1.15	5.088	288	-0.13
-0.1	19	1.16	5.175	289	0.15
0.4	19	1.18	5.3	291	0.63
1	19	1.21	5.425	292	1.21
1.8	19	1.25	5.575	293	1.98
-0.4	22	1.31	5.35	287	-0.02
-0.1	22	1.33	5.45	288	0.26
0.4	22	1.36	5.6	290	0.73
1	22	1.4	5.75	291	1.3
1.8	22	1.46	5.925	292	2.06
-0.4	25	1.45	5.563	287	0.11
-0.1	25	1.48	5.675	288	0.39
0.4	25	1.52	5.85	289	0.85
1	25	1.59	6.025	291	1.41
1.8	25	1.66	6.212	292	2.16
-0.4	28	1.58	5.738	286	0.26
-0.1	28	1.61	5.863	287	0.54
0.4	28	1.67	6.05	289	0.99
1	28	1.75	6.237	290	1.54
1.8	28	1.85	6.463	291	2.28
-0.4	31	1.72	5.875	285	0.43
-0.1	31	1.76	6.012	286	0.71
0.4	31	1.82	6.212	288	1.16
1	31	1.91	6.413	288	1.69
1.8	31	2.02	6.662	290	2.41
-0.4	34	1.85	6.012	285	0.61
-0.1	34	1.9	6.15	286	0.88
0.4	34	1.97	6.35	287	1.33
1	34	2.06	6.563	288	1.86
1.8	34	2.17	6.825	289	2.56
-0.4	38	2.03	6.175	284	0.85
-0.1	38	2.09	6.313	285	1.14
0.4	38	2.17	6.525	286	1.59
1	38	2.27	6.75	287	2.11
1.8	38	2.38	7.025	287	2.79
-0.4	42	2.21	6.325	283	1.1
-0.1	42	2.27	6.463	284	1.4
0.4	42	2.37	6.688	285	1.86
1	42	2.47	6.925	286	2.38
1.8	42	2.6	7.2	287	3.05

Figure N.1: Statistics for hydrualic conditions in PC-Ring

O
STATISTICS OF WIND DIRECTION AND WIND SPEEDS IN PC-RING

Below, the first part of the input file for the wind direction and wind speeds used in PC-Ring is presented. For adjusting the wind statistics the values of a_w, b_w, and c_w are changed at point number 25 (see also section 5.2.3).

```
  20
: zeespiegelrijzing
 0.07
: aantal getijden Q_Lobith, Q_Maas, Jaar
 16 16  352
: debiet Lobith
: werklijn met 3 takken a1, b1, a2, b2, a3, b3, wn
  1620.70   5893.30   1517.78  5964.63  1316.43  6612.61 18000000. 18000.
: overschrijdingsduurlijn
  -7.15D-13  5.8725D-8  -0.0016213  18.821256
: debiet Lith
: werklijn a1, b1, s, a2, b2, a3, b3, a4, b4
   327.7   1315.1  1.0   354.0   1315.1   288.9   1569.7   342.1  1190.3
: overschrijdingsduurlijn
  -8.387D-11  1.126D-6  -5.493D-3  13.75
: Schiphol ZONDER Volkerfactor en Deelen
: Eerste regel Algemene gegevens en Schiphol, tweede regel Deelen
:   wr              P            m_d      p_c      alpha    sigma    A_h   B_h   rho_w a_w    b_w    c_w     m_w
 ' NOORD'   0.0   0.0471387197 1.620769 0.064508 0.783795 0.086598 0.873 0.236 0.356 0.0010 0.2347 -0.5771 0.67
                  0.017510165  1.620769 0.064508 0.783795 0.086598 0.000 1.000 0.000 0.0207 0.0000 -0.1000 1.00
 'N.N.O.'  22.5   0.0452411865 1.453671 0.079793 0.501801 0.001982 0.000 1.000 0.000 0.0196 0.1654 -1.4610 1.00
                  0.025311198  1.453671 0.079793 0.501801 0.001982 0.000 1.000 0.000 0.0187 0.1804 -1.3655 1.00
 '  N.O.'  45.0   0.0557275542 1.435749 0.078015 0.498317 0.001856 0.000 1.000 0.000 0.0289 0.0000 -1.2479 1.00
                  0.04681869   1.435749 0.078015 0.498317 0.001856 0.000 1.000 0.000 0.0283 0.0397 -1.0570 1.00
 'O.N.O.'  67.5   0.0644162589 1.428922 0.072644 0.497982 0.001843 0.000 1.000 0.000 0.0289 0.0000 -1.2821 1.00
                  0.060971725  1.428922 0.072644 0.497982 0.001843 0.000 1.000 0.000 0.0275 0.0660 -1.2346 1.00
 '  OOST'  90.0   0.0575252172 1.432811 0.078260 0.498078 0.001847 0.000 1.000 0.000 0.0290 0.0272 -1.0583 1.00
                  0.065807554  1.432811 0.078260 0.498078 0.001847 0.000 1.000 0.000 0.0320 0.0000 -0.6885 1.00
 'O.Z.O.' 112.5   0.0414461200 1.461826 0.082783 0.497404 0.001823 0.000 1.000 0.000 0.0280 0.1013 -1.1597 1.00
                  0.062642483  1.461826 0.082783 0.497404 0.001823 0.000 1.000 0.000 0.0330 0.0359 -0.6322 1.00
 '  Z.O.' 135.0   0.0444422251 1.470346 0.071444 0.497319 0.001819 0.000 1.000 0.000 0.0293 0.0651 -1.1684 1.00
                  0.062302866  1.470346 0.071444 0.497319 0.001819 0.000 1.000 0.000 0.0276 0.1668 -1.2966 1.00
 'Z.Z.O.' 157.5   0.0582243084 1.452588 0.064981 0.496364 0.001785 0.000 1.000 0.000 0.0197 0.1547 -1.4637 1.00
                  0.058559276  1.452588 0.064981 0.496364 0.001785 0.000 1.000 0.000 0.0173 0.2340 -1.6356 1.00
 '  ZUID' 180.0   0.0745031459 1.436479 0.059367 0.495626 0.001760 0.000 1.000 0.000 0.0224 0.0000 -0.8438 1.00
                  0.073262724  1.436479 0.059367 0.495626 0.001760 0.000 1.000 0.000 0.0196 0.1031 -1.3106 1.00
 'Z.Z.W.' 202.5   0.0906821133 1.167933 0.675147 0.505930 0.002140 0.000 1.000 0.000 0.0110 0.1475 -1.6730 1.00
                  0.11573901   1.167933 0.675147 0.505930 0.002140 0.000 1.000 0.000 0.0077 0.2487 -1.7949 1.00
 '  Z.W.' 225.0   0.0959752322 1.338305 0.782153 0.842447 0.058747 1.227 0.122 0.506 0.0007 0.2931 -1.6875 1.00
                  0.13213117   1.338305 0.782153 0.842447 0.058747 0.000 1.000 0.000 0.0044 0.3058 -2.2349 1.00
 'W.Z.W.' 247.5   0.0908818536 1.300599 1.377435 0.831132 0.075904 1.230 0.169 0.605 0.0013 0.2756 -1.4733 1.00
                  0.11149887   1.300599 1.377435 0.831132 0.075904 0.000 1.000 0.000 0.0038 0.2877 -2.1512 1.00
 '  WEST' 270.0   0.0759013283 1.447817 0.820360 0.787524 0.081759 1.224 0.228 0.477 0.0010 0.1854 -0.8484 0.67
                  0.063694123  1.447817 0.820360 0.787524 0.081759 0.000 1.000 0.000 0.0028 0.2779 -1.9698 1.00
 'W.N.W.' 292.5   0.0575252172 1.426047 0.916124 0.776521 0.090158 1.195 0.262 0.613 0.0027 0.1740 -0.6280 0.67
                  0.043220315  1.426047 0.916124 0.776521 0.090158 0.000 1.000 0.000 0.0029 0.2664 -1.5942 1.00
 '  N.W.' 315.0   0.0508339159 1.260168 0.896914 0.736404 0.090069 0.887 0.326 0.768 0.0042 0.1810 -0.2651 0.67
                  0.035232688  1.260168 0.896914 0.736404 0.090069 0.000 1.000 0.000 0.0026 0.2923 -1.5005 1.00
 'N.N.W.' 337.5   0.0495356037 2.121971 0.021788 0.752875 0.089100 0.904 0.292 0.677 0.0039 0.2184 -0.4279 0.67
                  0.025297145  2.121971 0.021788 0.752875 0.089100 0.000 1.000 0.000 0.0068 0.2377 -1.1050 1.00
```

Figure O.1: Statistics of wind direction and wind speeds in PC-Ring part 1/3

```
  23
: meerpeilstijging
 0.00
: aantal getijden Meerpeil, IJssel, Lobith, Jaar
   10   10   352
: Schiphol ZONDER Volkerfactor voor Merengebied
:    wr            P             a_w    b_w    c_w
 ' NOORD'    0.0   0.0471387197  0.0199 0.0776 -0.8213
 'N.N.O.'   22.5   0.0452411865  0.0196 0.1654 -1.4610
 '  N.O.'   45.0   0.0557275542  0.0289 0.0000 -1.2479
 'O.N.O.'   67.5   0.0644162589  0.0289 0.0000 -1.2821
 '  OOST'   90.0   0.0575252172  0.0290 0.0272 -1.0583
 'O.Z.O.'  112.5   0.0414461200  0.0280 0.1013 -1.1599
 '  Z.O.'  135.0   0.0444422251  0.0293 0.0652 -1.1685
 'Z.Z.O.'  157.5   0.0582243084  0.0197 0.1547 -1.4638
 '  ZUID'  180.0   0.0745031459  0.0224 0.0000 -0.8438
 'Z.Z.W.'  202.5   0.0906821133  0.0110 0.1475 -1.6730
 '  Z.W.'  225.0   0.0959752322  0.0083 0.1711 -1.9799
 'W.Z.W.'  247.5   0.0908818536  0.0067 0.1970 -2.0424
 '  WEST'  270.0   0.0759013283  0.0052 0.2235 -2.0877
 'W.N.W.'  292.5   0.0575252172  0.0055 0.2244 -1.9588
 '  N.W.'  315.0   0.0508339159  0.0052 0.2613 -2.0077
 'N.N.W.'  337.5   0.0495356037  0.0096 0.2162 -1.6373
  24
: meerpeilstijging
 0.00
: aantal getijden Meerpeil, Vecht, Lobith, Jaar
   10   10   352
: Schiphol ZONDER Volkerfactor voor Merengebied
:    wr            P             a_w    b_w    c_w
 ' NOORD'    0.0   0.0471387197  0.0199 0.0776 -0.8213
 'N.N.O.'   22.5   0.0452411865  0.0196 0.1654 -1.4610
 '  N.O.'   45.0   0.0557275542  0.0289 0.0000 -1.2479
 'O.N.O.'   67.5   0.0644162589  0.0289 0.0000 -1.2821
 '  OOST'   90.0   0.0575252172  0.0290 0.0272 -1.0583
 'O.Z.O.'  112.5   0.0414461200  0.0280 0.1013 -1.1599
 '  Z.O.'  135.0   0.0444422251  0.0293 0.0652 -1.1685
 'Z.Z.O.'  157.5   0.0582243084  0.0197 0.1547 -1.4638
 '  ZUID'  180.0   0.0745031459  0.0224 0.0000 -0.8438
 'Z.Z.W.'  202.5   0.0906821133  0.0110 0.1475 -1.6730
 '  Z.W.'  225.0   0.0959752322  0.0083 0.1711 -1.9799
 'W.Z.W.'  247.5   0.0908818536  0.0067 0.1970 -2.0424
 '  WEST'  270.0   0.0759013283  0.0052 0.2235 -2.0877
 'W.N.W.'  292.5   0.0575252172  0.0055 0.2244 -1.9588
 '  N.W.'  315.0   0.0508339159  0.0052 0.2613 -2.0077
 'N.N.W.'  337.5   0.0495356037  0.0096 0.2162 -1.6373
```

Figure O.2: Statistics of wind direction and wind speeds in PC-Ring part 2/3

```
  25
: meerpeilstijging
 0.00
: aantal getijden Meerpeil, Jaar
  15 352
: Schiphol ZONDER Volkerfactor voor Merengebied
:    wr            P            a_w          b_w          c_w
 ' NOORD'    0.0   0.045293565  0.0212831    0.0024599   -0.8315342
 '    30'   30.0   0.068137623  0.0274440    0.0000000   -0.9124936
 '    60'   60.0   0.081192237  0.0269930    0.0000000   -0.9982341
 '  OOST'   90.0   0.076606806  0.0283086    0.0000000   -0.7812025
 '   120'  120.0   0.069471116  0.0405595   -0.2095188    0.0148060
 '   150'  150.0   0.079910919  0.0334403   -0.0957586   -0.6279250
 '  ZUID'  180.0   0.109481504  0.0212150   -0.0112443   -0.7794127
 '   210'  210.0   0.136398335  0.0132637    0.0195042   -0.6829987
 '   240'  240.0   0.135235319  0.0078872    0.1168630   -1.0026627
 '  WEST'  270.0   0.101951458  0.0075699    0.0926526   -0.9223612
 '   300'  300.0   0.059326259  0.0082307    0.0806982   -0.7676162
 '   330'  330.0   0.036991859  0.0116444    0.0987639   -1.5070786
 'stop'
  26
: meerpeilstijging
 0.00
: aantal getijden Meerpeil, Jaar
  15  352
: Schiphol ZONDER Volkerfactor voor Merengebied
:    wr            P            a_w          b_w          c_w
 ' NOORD'    0.0   0.045293565  0.0212831    0.0024599   -0.8315342
 '    30'   30.0   0.068137623  0.0274440    0.0000000   -0.9124936
 '    60'   60.0   0.081192237  0.0269930    0.0000000   -0.9982341
 '  OOST'   90.0   0.076606806  0.0283086    0.0000000   -0.7812025
 '   120'  120.0   0.069471116  0.0405595   -0.2095188    0.0148060
 '   150'  150.0   0.079910919  0.0334403   -0.0957586   -0.6279250
 '  ZUID'  180.0   0.109481504  0.0212150   -0.0112443   -0.7794127
 '   210'  210.0   0.136398335  0.0132637    0.0195042   -0.6829987
 '   240'  240.0   0.135235319  0.0078872    0.1168630   -1.0026627
 '  WEST'  270.0   0.101951458  0.0075699    0.0926526   -0.9223612
 '   300'  300.0   0.059326259  0.0082307    0.0806982   -0.7676162
 '   330'  330.0   0.036991859  0.0116444    0.0987639   -1.5070786
 'stop'
```

Figure O.3: Statistics of wind direction and wind speeds in PC-Ring part 3/3

P

RESULTS PC-RING FOR THE PROBABILITY OF FAILURE

In this appendix the detailed results of the PC-Ring runs are given. PC-Ring is run twice, once for the case that the Afsluitdijk does not fail (section P.1) and once for the case that the Afsluitdijk does fail (section P.2). A summary of this appendix is given in the main report in section 5.4.

P.1. PROBABILITY OF FAILURE IF THE AFSLUITDIJK WILL NOT FAIL

The effects of a failure of the Afsluitdijk will be compared with the effects of no failure of the Afsluitdijk. As stated before, no failure of the Afsluitdijk is the current starting point for the calculations of failure probabilities of the IJsselmeer dikes. To find this probability of failure of dike ring area 7, PC-Ring is used. If done correctly the results will be the same as in the VNK2 report [14]. This provides a good validation of the results found with PC-Ring.

The total probability of failure of the current dike ring area provided **the Afsluitdijk will not fail** amounts **1/1,000 per year**. As the current norm lies on 1/4,000 per year the current water defence system does not provide enough safety for the Noordoostpolder. At the moment this system is being improved to provide more safety against flooding. In this thesis the current system is worked with and the results of the other PC-Ring calculations are compared with the current probability of failure of 1/1,000 per year.

The total probability of failure of 1/1,000 per year is calculated by calculating the probability of failure for each failure mechanism and for each dike ring element (see appendix B). Below, these results are given.

P.1.1. PROBABILITY OF FAILURE PER FAILURE MECHANISM

In table P.1 the probability of failure per failure mechanism is presented. As dikes and hydraulic structures have different failure mechanisms, these different kind of dike ring elements are split. It can be seen that failure mechanism overflow and over-topping (at the dike elements) is the mayor failure mechanism.

When looking at the combined probability of failure (the total sum), overflow and wave overtopping accounts for 73% of the total probability of failure. The other 27% is due to failure mechanism instability of outer slope protection (18%) and the failure mechanisms in the structure combined (9%). The failure mechanisms in table P.1 with a probability of failure smaller than 1/1,000,000 are negligible compared to the other failure mechanisms. Dune erosions does not have a probability of failure due to the fact that no dunes are present in dike ring area 7.

Type of water defence	Failure mechanism	Probability of failure [per year]
Dike	Overflow and wave overtopping	1/1,200
Dike	Sliding inner slope	<1/1,000,000
Dike	Piping and heave	<1/1,000,000
Dike	Instability of outer slope protection	1/4,700
Dunes	Dune erosion	-
Structure	Overflow and wave overtopping	<1/1,000,000
Structure	Failure of closing	1/25,000
Structure	Piping	1/15,000
Structure	Failure of construction	<1/1,000,000
Total probability of failure		1/1,000

Table P.1: Results of PC-Ring, probability of failure per failure mechanism (no failure Afsluitdijk).

P.1.2. PROBABILITY OF FAILURE PER DIKE RING ELEMENT

Table P.2 gives the probability of failure per dike ring element. For each calculated failure mechanism the return period is given as is the combination of all. The number/code of each element corresponds with the numbers in figure 5.10 (the elements are numbered from the Zwarte meer in the South west to Lemmer in the North following the water defence line).

Dike ring element	Overflow and wave overtopping	Sliding inner slope	Piping and heave	Instability outer slope	Combined
Z08.55300.55500	>1,000,000	-	-	-	>1,000,000
Z07.54600.55300	>1,000,000	-	-	-	>1,000,000
Z06.53100.54600	>1,000,000	-	-	-	>1,000,000
Z05.51800.53100	>1,000,000	-	-	-	>1,000,000
Z04.50100.51800	>1,000,000	-	-	-	>1,000,000
Z03.48800.50100	>1,000,000	-	-	-	>1,000,000
Z02.46300.48800	>1,000,000	-	-	-	>1,000,000
Z01.44000.46300	>1,000,000	-	-	-	>1,000,000
K11.43700.44000	>1,000,000	-	-	-	>1,000,000
K10.40800.43700	33,000	-	-	-	33,000
K09.39200.40800	66,000	-	-	-	66,000
K08.38400.39200	230,000	-	-	-	230,000
K07.37600.38400	22,000	-	-	-	11,000
K06.37400.37600	140,000	-	-	-	140,000
K05.36100.37400	330,000	-	-	>1,000,000	330,000
K04.34700.36100	69,000	-	-	-	69,000
K03.33000.34700	390,000	-	-	>1,000,000	390,000
K02.32100.33000	>1,000,000	-	-	-	>1,000,000
K01.31300.32100	520,000	>1,000,000	-	-	470,000
IJ21.29900.31300	9,700	-	-	-	9,700
IJ20.28200.29900	4,100	-	-	6,700	3,100
IJ19.26800.28200	4,300	-	-	-	4,300
IJ18.26300.26800	380,000	-	-	-	380,000
IJ17.25700.26300	50,000	-	-	-	50,000
IJ16.24500.25700	>1,000,000	-	-	-	>1,000,000
IJ15.24300.24500	>1,000,000	-	-	-	>1,000,000
IJ14.23000.24300	26,000	-	-	-	26,000
IJ13.21100.23000	5,800	-	-	9,800	4,100
IJ12.18400.21100	5,700	-	-	-	5,700
IJ11.14900.18400	4,200	-	-	17,000	4,000
IJ10.12900.14900	2,600	-	-	-	2,600
IJ09.09900.12900	1,600	-	-	17,000	1,600
IJ08.08600.09900	5,700	-	-	-	5,700
IJ07.07500.08600	14,000	-	-	-	14,000
IJ06.05100.07500	2,400	-	-	-	2,400
IJ05.04300.05100	30,000	-	-	13,000	12,000
IJ04.01600.04300	17,000	-	-	-	17,000
IJ03.01100.01600	340,000	-	-	-	340,000
IJ02.00600.01100	33,000	-	-	-	33,000
IJ01.00000.00600	170,000	>1,000,000	-	-	170,000

Table P.2: Results of PC-Ring, probability of failure per dike ring element (no failure Afsluitdijk).

In table P.2 most of the values for the probability of failure of the failure mechanisms sliding inner slope, piping and heave, and instability outer slope are not calculated. These values are not calculated because it can be safely expected these specific failure mechanisms have a probability of failure lower than 1/1,000,000 for these specific dike ring elements. The absence of all piping and heave probabilities of failure is remarkable. The foundation of the dikes of the Noordoostpolder is characterised by large cohesive layers that prevent the formation of piping. As this layer stops at a relatively large distance away from the dike, the creep length is also very long and thus very high water levels are needed in order to create piping wells. With such water levels the dike has most likely already failed due to overflow and wave overtopping. Therefore the probabilities of failure for piping and heave are left out of the calculation.

The few values that are calculated for the failure mechanisms sliding of the inner slope and instability of the outer slope are picked because they showed weaknesses in the results of the second and third assessment. In table P.2 it can be seen that only failure mechanism instability of outer slope give significant results. Although the highest probability of failure is found to be 1/6,700 per year this still has a significant influence on the combined probability of failure.

The same calculations can be done for the hydraulic structures. The results are given in table P.3.

Hydraulic structure	Overflow and wave overtopping	Failure of closing	Piping	Failure of construction	Combined
VNK.07.02.001	-	-	100,000	-	100,000
VNK.07.02.002	-	-	510,000	-	510,000
VNK.07.03.001	-	230,000	-	-	230,000
VNK.07.03.002	-	>1,000,000	-	>1,000,000	>1,000,000
VNK.07.03.003	-	>1,000,000	>1,000,000	-	>1,000,000
VNK.07.06.001	>1,000,000	27,000	26,000	>1,000,000	13,000
VNK.07.06.002	-	570,000	60,000	-	54,000

Table P.3: Results of PC-Ring, probability of failure per hydraulic structure (no failure Afsluitdijk).

P.2. PROBABILITY OF FAILURE IF THE AFSLUITDIJK WILL FAIL

A second run is done with PC-Ring to find the probability of failure of the dike ring area, but in this second run the Afsluitdijk has failed during a previous storm. For this previous storm (the first storm) any kind of magnitude can be assumed. However, the more extreme the first storm will be, the less likely a combination of the first and the second storm will be (see section 5.4.4 for the probability of the combination of the two storms). Secondly, higher frequencies will lead to minor increases in discharge at the cost of major increases in return period (also for scenarios 2 and 3). Therefore, the combination of a first storm with a frequency of 1/250 per year with the second storm will be used to investigate the effects of a second storm on the safety of the dikes of the Noordoostpolder.

As was shown in chapter 4, scenario 3 starts to be the dominating scenario for storms extremer than 1/1,000 and only at storms more extreme than 1/10,000 the discharges through the breach are significantly more (leading to differences of a few decimetres). In scenario 1 and 4, the storm blows out the sluices completely leading to gap of 336 meters in width. This width is much wider than the width of a breach in the levee at the end of a 1/250 year storm. Because of this, scenario 1 and 4 (both leading to the same values for a 1/250 storm) are used and not scenario 3 (or 2).

Note: It would be interesting to see the probability of failure of multiple magnitudes of the first storm (e.g. 1/250, 1/1,000, and 1/10,000) and every scenario (1, 2, 3, and 4). Unfortunately, calculation of these storms and scenarios requires a lot of time. For each combination of storm and scenario different discharges through the breach are found and thus different increases in water level. These increases in water level need to be adjusted for 30 locations around the Noordoostpolder, for 12 different wind directions, and 9 different wind speeds. Also the wind statistics should be corrected as the timing of the governing load differs for the combination of storm and scenario (which also need to be found first). All these computations require much time and will most likely lead to less interesting values than the combination of the 1/250 storm and scenario 1 or 4. Therefore, these other combinations are not executed for this thesis.

After implementing these starting points in the input files of PC-Ring, **the total probability of failure** of the current dike ring area **provided the Afsluitdijk has failed** during a previous 1/250 storm is found to be **1/450 per year**. Below, the results of second run are given.

P.2.1. PROBABILITY OF FAILURE PER FAILURE MECHANISM

In table P.4 the probability of failure per failure mechanism is presented. As dikes and hydraulic structures have different failure mechanisms these different kind of dike ring elements are split again. It can be seen that failure mechanism overflow and over-topping (at the dike elements) is still the mayor failure mechanism.

When looking at the combined probability of failure (the total sum), overflow and wave overtopping of dikes is still the most dominant failure mechanism with 48% of the total probability of failure (it was 73%). Instability of the outer slope protection is much higher (42%), which is mainly caused by dike ring element IJ20 (previously it was 18%). Failure of the hydraulic structures is hardly changed with 10% of the total probability of failure (previously 9%).

Overflow and wave overtopping at the dike elements is still the most crucial failure mechanism. The contribution of failure mechanism instability of the outer slope to the total probability of failure has increased much. This is mainly because of element IJ20. If this element is left out (for instance if the strength of the outer slope gets improved) overflow and wave overtopping will get back to its original dominance.

P.2.2. PROBABILITY OF FAILURE PER DIKE RING ELEMENT

The changes in the probability of failure per dike ring element are more interesting than the changes in the probability of failure per failure mechanism. As table P.5 shows, all dike ring elements have an increased probability of failure (i.e. reduced return period).

Type of water defence	Failure mechanism	Probability of failure [per year]
Dike	Overflow and wave overtopping	1/830
Dike	Sliding inner slope	<1/1,000,000
Dike	Piping and heave	<1/1,000,000
Dike	Instability of outer slope protection	1/930
Dunes	Dune erosion	-
Structure	Overflow and wave overtopping	1/230,000
Structure	Failure of closing	1/7,600
Structure	Piping	1/8,600
Structure	Failure of construction	<1/1,000,000
Total probability of failure		1/453 ≈ 1/450

Table P.4: Results of PC-Ring, probability of failure per failure mechanism (failure of the Afsluitdijk).

Dike ring element	Overflow and wave overtopping	Sliding inner slope	Piping and heave	Instability outer slope	Combined
Z08.55300.55500	>1,000,000	-	-	-	>1,000,000
Z07.54600.55300	>1,000,000	-	-	-	>1,000,000
Z06.53100.54600	>1,000,000	-	-	-	>1,000,000
Z05.51800.53100	>1,000,000	-	-	-	>1,000,000
Z04.50100.51800	>1,000,000	-	-	-	>1,000,000
Z03.48800.50100	>1,000,000	-	-	-	>1,000,000
Z02.46300.48800	>1,000,000	-	-	-	>1,000,000
Z01.44000.46300	>1,000,000	-	-	-	>1,000,000
K11.43700.44000	>1,000,000	-	-	-	>1,000,000
K10.40800.43700	15,000	-	-	-	15,000
K09.39200.40800	15,000	-	-	-	15,000
K08.38400.39200	32,000	-	-	-	32,000
K07.37600.38400	5,600	-	-	-	5,600
K06.37400.37600	16,000	-	-	-	16,000
K05.36100.37400	50,000	-	-	>1,000,000	50,000
K04.34700.36100	12,000	-	-	-	12,000
K03.33000.34700	46,000	-	-	>1,000,000	46,000
K02.32100.33000	140,000	-	-	-	140,000
K01.31300.32100	72,000	>1,000,000	-	-	71,000
IJ21.29900.31300	11,000	-	-	-	11,000
IJ20.28200.29900	3,300	-	-	1,100	900
IJ19.26800.28200	4,000	-	-	-	4,000
IJ18.26300.26800	110,000	-	-	-	110,000
IJ17.25700.26300	20,000	-	-	-	20,000
IJ16.24500.25700	540,000	-	-	-	540,000
IJ15.24300.24500	>1,000,000	-	-	-	>1,000,000
IJ14.23000.24300	22,000	-	-	-	22,000
IJ13.21100.23000	3,900	-	-	5,300	2,800
IJ12.18400.21100	3,500	-	-	-	3,500
IJ11.14900.18400	2,700	-	-	6,300	2,400
IJ10.12900.14900	1,900	-	-	-	1,900
IJ09.09900.12900	1,300	-	-	7,800	1,300
IJ08.08600.09900	5,100	-	-	-	5,100
IJ07.07500.08600	8,900	-	-	-	8,900
IJ06.05100.07500	1,900	-	-	-	1,900
IJ05.04300.05100	9,900	-	-	5,200	4,200
IJ04.01600.04300	9,900	-	-	-	9,900
IJ03.01100.01600	57,000	-	-	-	57,000
IJ02.00600.01100	8,200	-	-	-	8,200
IJ01.00000.00600	35,000	>1000000	-	-	35,000

Table P.5: Results of PC-Ring, probability of failure per dike ring element (failure of the Afsluitdijk).

The results for the structures are given in table P.6. All have an increased probability of failure. As the hydraulic structures are relatively safe elements in dike ring area 7, these changes are not of much importance. Only element VNK.07.06.001 is worth noticing as the new probability of failure gets close to the safety standard of the whole dike ring area (i.e. 1/4,000 per year). This element is called the Friese Sluis and is a sluice near Lemmer.

Hydraulic structure	Overflow and wave overtopping	Failure of closing	Piping	Failure of construction	Combined
VNK.07.02.001	-	-	67000	-	67,000
VNK.07.02.002	-	-	280000	-	280,000
VNK.07.03.001	-	73000	-	-	73,000
VNK.07.03.002	-	>1000000	-	>1000000	>1,000,000
VNK.07.03.003	-	>1000000	>1000000	-	>1,000,000
VNK.07.06.001	230000	8400	14000	>1000000	5,200
VNK.07.06.002	-	69000	36000	-	24,000

Table P.6: Results of PC-Ring, probability of failure per hydraulic structure (failure of the Afsluitdijk).

After the first run, table P.5 gave two strange values for the probability of failure of the failure mechanism instability of outer slope at element IJ20 and the probability of failure of the failure mechanism overflow and wave overtopping at element IJ03. Two unreliable probabilities of failure were found (return period of 89 for IJ20 and a return period of 1 for IJ03). The changes in these two values were much higher than the changes in all other values. As the two elements are on two completely different locations (IJ20 South of Urk and IJ03 near Lemmer) and are two different failure mechanisms it was assumed these abnormal values are caused by a computation error. The mathematical method for calculating these specific failure mechanisms at these locations was changed from the FORM method to the Monte Carlo method. This gave more reliable results for IJ03 and IJ20 (see table P.5 for the results).

Q

Reliability of the findings in this thesis

The final conclusions of this thesis should not be followed blindly as their are many assumption made throughout the whole research. Some of the assumptions could really have a big impact on the final conclusion where others have only minor effect. The effect of these assumptions on the estimates of certain values (e.g. the risks and the probabilities of failure) can be quite significant.

The numbers in this thesis are chosen as realistic as possible and as a consequence of the many assumptions, the calculated estimates give only an indication of the total influence of a breach. For instance, the 62,500 euro per day estimate of the risks of flooding for the Noordoostpolder given that the Afsluitdijk already has failed during a previous 1/250 per year storm (see section 5.7.3) should not be taken too seriously. The actual value of this risk could easily be a factor 10 lower or higher. This is because the many uncertainties that got into the research along the way.

The rest of this section gives a summary of the most important assumptions that have been done in the previous 3 chapters. This list of assumptions and lack of knowledge gives an indication on the total reliability of the conclusions of this thesis. For all the uncertainties the best assumptions were tried to be used, but unfortunately it is unavoidable that some assumptions are more realistic than others. For some of the assumptions the contribution to the total uncertainty is unknown. Therefore it is tried to deal with the uncertainties as best as possible (e.g. creating scenarios or choosing the most conservative way).

Q.1. Uncertainties in chapter 3

Chapter 3 focussed on defining the possibilities of a breach in the Afsluitdijk. To reduce the error caused by doing simplifications and making assumptions, 4 scenarios were developed. By using these 4 different scenarios the three most influencing certainties are dealt with.

The three main uncertainties of chapter 3

First uncertainty was whether a breach in the Afsluitdijk will take place at the levee or at the sluices. As both are likely to occur a division was made between scenario 1 and 2. Also scenario 4 was made in which both a breach in the levee and at the sluices was assumed.

Second uncertainty lies in the behaviour of the boulder clay in the levee of the Afsluitdijk. The boulder clay is likely to prevent much of the erosion during a breach. Unfortunately, its behaviour is very uncertain and the boulder clay could also completely be washed away. Therefore, scenario 3 was formed in which it was assumed the boulder clay will act as sand during a breach. This approach is a conservative approach as it is highly likely the boulder clay layer will have at least some retardant effect.

By making these different scenarios the uncertainty in the location of the breach is completely taken out of the way. In chapter 5, only the worst of the scenarios was used in the calculations thereby taking also a conservative approach. For the third main uncertainty in this chapter, the expected probability of failure of the Afsluitdijk, the uncertainty can not be taken away completely by using different scenarios. The actual probability of failure can lie anywhere between (roughly) 1/100 and 1/10,000 per year. To deal with this uncertainty rather low probability of failures where chosen, thereby making another conservative assumption. As was discussed in chapter 3 the current probability of failure of the Afsluitdijk will be most likely be lower than the assumed 1/140 per year for a levee breach and 1/250 for a breach at the sluices.

Q.1.1. OTHER UNCERTAINTIES IN CHAPTER 3

Unfortunately main other assumptions and simplifications were needed. For instance it is expected a levee breach will only develop to at most a depth of -0.4 m +NAP. It is assumed the berm under the sand layer (at -0.4 m +NAP) can not erode away during the storm, therefore the breach depth is limited to this level.

In the case of a breach at the sluices it is assumed all sluices will blow out together. This is not very realistic as they do not all have exactly the same strength and are not even on the same location! In the case of a breach in the levee it is assumed only one breach will occur. But in reality it can also happen that multiple breaches are formed leading to even wider breach than 1300 meters (in scenario 3). Both errors in the simplification are not accounted for in this thesis.

The expected widths of a levee breach at a 1/10,000 per year storm are retrieved from the results of the BRES model [28]. In this model the development of a breach is divided into 5 stages for which different relations are assumed. Although the model is validated with existing data, the model is still only an approach of reality.

For finding the breach dimensions of the levee during other storms than the 1/10,000 per year storm a relation is assumed. First of all, this relation is based on empirical formulas (introducing the first error). Second simplification that only stage IV contributes to the different widths of the breach also introduces an error. Third, the exponential function is fitted through only two points (which are both not fully reliable, see previous paragraph). This building up of errors makes that the final relation only gives a slight idea on how wide the breach will be during different storms.

Where for the three main uncertainties safe conservative approaches were chosen this is not always done for the other assumptions. Most of the time for these assumptions, the most realistic assumption is used. This might have led to an overestimate of the phenomena.

Q.2. UNCERTAINTIES IN CHAPTER 4

In chapter 4, a Matlab model was made to calculate the increase in water level through a breach. Apart from the breach scenarios (from chapter 3), other input was needed for this model.

First, a probability density function is approximated for the water levels in the Waddenzee and the IJsselmeer. For this, the Hydra models were used to find certain values for water levels and return periods. Subsequently, an extreme value distribution was fit through these found values. With the help of the least square method the right distributions were found. It could be that the wrong distribution was chosen, leading to aberrant results in the Matlab model. Also in the values found with the Hydra models uncertainties are found. The models extrapolates the water levels from a measured data set. Even with 100 years of data, predicting the water level that occurs only once every 10,000 years is very difficult and therefore not very precise.

The duration of high water during the storm is assumed to build up linearly to the peak in 20 hours and then linearly decrease back to 0 m +NAP in 30 hours. It is assumed this duration is the same for any kind of extreme storm. Also in this simplification the fluctuations in water level due to tide are not taken into account which leads to overestimates of the water level during low tide and little underestimates of the water level during high tide. For more on the effects of this assumption see appendix M.

Also the development and the duration of wind is simplified. It is assumed the highest wind speeds are found at the highest water levels and also build up linearly and decease linearly. According to figure E.1 in the appendix this seems like a reasonable approach but also here a simplification error is made.

The linear development of the breach during 12 hours after the peak is also an important assumption. It is found that changes in this assumption lead to significant other results in the water level increases. This can be seen if the results of chapter 4 are compared with Vissers results [28]. This difference is mostly explained by the assumption on how long the breach needs before it is fully developed (see also the explanation in appendix L on page 150).

For determining the discharge through the breach it is assumed the downstream water level does not influence the amount of discharge. Although results show that 10% of the time the IJsselmeer water level does influence the discharge through the breach in case of a levee breach, it is found that this simplification error is not significant (see appendix J).

In the two used discharge formula, a flow coefficient (for over flow) and a contraction coefficient are assumed (for under flow). These values were found in articles in literature. These articles also state that there is some spread in these values depending on local conditions. These factors influence the amount of discharge directly and therefore cause a direct uncertainty in the amount of discharge through the gap.

Also in the simulation method an uncertainty can be found. As a Monte Carlo approach is used the highest values of the water levels (thus during the more extreme storms) fluctuate during different runs. To minimize this fluctuation the number of runs was increased at least 100 times the highest required frequency (so in case 1/1,000,000 per year values are needed the number of Monte Carlo runs was 100,000,000).

Q.3. UNCERTAINTIES IN CHAPTER 5

Chapter 5 used the findings from chapter 3 and chapter 4. Consequently, errors made in these previous chapters are also found in the results of chapter 5. Apart from these continuation errors, chapter 5 also has a few assumptions that have an impact on the results of chapter 5 and thus on the final results of this thesis.

In this chapter the timing of the governing conditions is estimated by looking only at the effects of wind and the breach in the Afsluitdijk on the local water level. In this approach, the effect of wind on waves is neglected. As waves are a very important factor for determining the probability of failure of a water defence, ignoring the effects of waves leads to a misjudged estimate of the timing of the governing load[1]. Taking this effect into account leads to an earlier peak of the governing conditions than is taken in chapter 5. In chapter 5 it is assumed this effect is countered by the fact that water through the breach needs time to travel to the other side of the IJsselmeer, as this makes the peak of the governing load happen later than is assumed in chapter 5. Whether these two effects balance each other is unknown.

In section 5.1.2 it was assumed the wind statistics for the wind direction of 330 degrees are used as it is expected the Afsluitdijk will fail during the conditions of a storm with that direction. Figure 5.1 shows however, that for the wind directions of 270 and 300 degrees much higher winds speeds are found (or the probability for certain wind speeds is higher). A slight change in the wind direction might lead to higher wind speeds and thus to a change in the probability of failure of the Noordoostpolder. As the effect of wind on the local water conditions is found to be quite significant, the uncertainty in the assumption of the dominant wind directions is also significant.

Also in the correction of the wind statistics uncertainties can be found. First of all, the previously explained assumptions of the maximum wind speed, the linear decrease during the storm, and the timing of the governing load have their influence on the correction of the wind statistics. These three assumptions are combined to find the required amount of reduction in the wind speed. Secondly, this correction is implemented manually and the values of a_w, b_w, and c_w are fitted through the adjusted data. By fitting the Kr-function (see equation (5.3)) through these new data points an approximation error is made. Unfortunately, it is too hard to express how big this error is. It is wise to improve the implementation of this adjusted wind statistics in PC-Ring in further studies (see also section 7.2).

[1] Ignoring the effect of wind on waves is only done for the estimate of the timing of the governing load, not for determining the probability of failure!

After a first run with PC-Ring to define the probability of failure of the Noordoostpolder given that the Afsluitdijk will stay intact, it was found that the water defence elements corresponding to the water defence line protecting the Noordoostpolder from the Zwarte Meer have a probability of failure higher than 1/1,000,000 per year. Because of these findings the hydraulic loads in the Zwarte Meer weren't adjusted to higher local water levels due to the breach in the Afsluitdijk. Therefore, the consequences of failure of the Afsluitdijk are not calculated for that part of the dike ring area. Although it is expected this area is much less affected by a breach in the Afsluitdijk than the rest of the dike ring (due to the fact that the distance to the Afsluitdijk is much further), it could be this assumption is wrong and that, due to the breach in the Afsluitdijk, the hydraulic conditions in the Zwarte Meer will get significant.

Also in the model PC-Ring many uncertainties can be found. PC-Ring uses only 4 failure mechanisms for dikes. These are the 4 most important ones and the other mechanisms are neglected. Also in the enormous number of variables of the strength of the dikes (e.g. height of the dike, quality of the revetment, critical discharges, etc.) uncertainties can be found. As it is impossible to measure each of those variables at every meter water defence, simplifications are made and spatial correlations are assumed. This leads to uncertainties in the schematization of the water defence line.

Also in the estimated consequences uncertainties are found. The economic value of the region and the number of inhabitants are only estimates. Also the evacuation factor and probability (see table 5.3) are only rough expectations of reality. Therefore the final results of the risks of flooding expressed in euros per year and casualties per year is very uncertain.

Q.4. UNCERTAINTY IN THE MAIN CONCLUSIONS

Although these uncertainties presume to have a big influence on the numbers of this thesis. The sensitivity of the assumptions on the main conclusion is expected to be not that big. The main conclusion (i.e. That the effect of failure of the Afsluitdijk does not influence the risks of flooding) is based on very significant findings. For the increased risk during the first storm wind speed set up in the order of 3 meters (at a 1/10,000 per year storm) are found and the maximum local water set up is 'only' 1.15 meters (at a 1/100,000 per year storm in the worst of the assumed scenarios). This difference is large enough to safely state that the wind set up is more dominant than the increase of average water level due to a breach. For the risk of flooding during a second storm the total calculated risk is 112 times lower than the current risks of flooding (see section 5.4.4). By changing assumptions this factor 112 can change a lot, but it is very unlikely this factor drops below 1. Therefore it is also very unlikely the risks of flooding increase because of two storms. This means that it can safely be stated that the effect of failure of the Afsluitdijk does not influence the risks of flooding of the IJsselmeer dikes.

R

PERSPECTIVE OF THE CONCLUSIONS OF THIS THESIS

The conclusions of this thesis are in contradiction to the current standard of the Afsluitdijk and the studies done by Deltares and CPB [20] [19]. This thesis states that the assumed current probability of failure of the Afsluitdijk (approximately 1/250 per year) is high enough to protect the Noordoostpolder from floods from the North sea. The CPB report advises a probability of failure of 1/9,600 per year (which is close to the current statutory standard) to provide enough safety to protect the hinterlands from floods from the North sea. The difference between the CPB report and this thesis is quite significant. However, from the conclusions of this thesis it doesn't follow that increasing the strength of the Afsluitdijk is an useless execution or a waste of money. The main reason for this are given in chapter 6[1] and appendix Q[2], but also the difference between the approach of this thesis and the Deltares and CPB reports needs to be explained. The main reasons for the contradiction in the conclusions are discussed in this appendix.

R.1. CURRENT AND FUTURE SITUATION

First of all, this thesis assumes the current situation and doesn't look at the changes in the future. These changes can be quite significant. The derogation of the Afsluitdijk or the dikes of the Noordoostpolder over time is not included. It is highly unlikely (if no repairs are executed) that the current probability of failure of the Afsluitdijk will not grow in the next decades. Doors of the sluices will wear and toe protections can be eroded away during minor storms. These things can be helped by using maintenance, but the probable change in hydraulic conditions can not. Because of climate change it is likely that sea water levels will rise and extreme storm events will occur more often. CPB for instance, assumes the W+ climate scenario will occur which will lead to a sea water level rise of 85 cm in 2100 [19]. This will also have an effect on the water level in the IJsselmeer. This sea level rise drastically changes the protection against floods of the IJsselmeer system. Also the changes in the economic value and number of civilians will probably increase in the future. This aspect is also not accounted for in this thesis.

R.2. INERTIA EFFECTS OF THE WATER AND THE WIND

Deltares [20] concludes that an increase in water level during the extreme storm will mainly be caused by overtopping of water over the Afsluitdijk and failure of the sluices. The water levels 4 hours after the peak of the storm are calculated and the same wind speeds as at the peak of the storm is used to define the increase in probability of failure (called P2 in the report). This is a very conservative assumption as the wind speeds will most likely be lower after the peak of the storm. Also the travel time of Waddenzee water to the other side of the IJsselmeer is assumed to be instantaneous. These two conservative assumptions lead to an increase of probability of failure of the Noordoostpolder. This also contributes to the advised standard in the CPB report [19]. In this thesis it was assumed that wind speeds will decrease after the peak of the storm (based on mea-

[1] Here, the other functions of the Afsluitdijk are discussed.

[2] Here, a quantitative analysis about the uncertainties in this research is presented.

surements of wind speeds during the storm of 1953 (see figure E.1)) and that it will take quite some time for the Waddenzee water to influence the local water level at the Noordoostpolder. With these assumptions the probability of failure doesn't increase.

R.3. OTHER AREAS AROUND THE IJSSELMEER

In chapter 5 it was argued that the impact of a breach in the Afsluitdijk will probably have the biggest consequences for the probability of failure of the dikes of the Noordoostpolder. This, because the Afsluitdijk will most likely fail under wind conditions coming from the North or North West. It was assumed that if failure of the Afsluitdijk has any effect on the safety of the dikes around the IJsselmeer, it will most likely have the most effect on the Noordoostpolder (see section 5.3). According to the calculations of the CPB report, the probability of failure of the South West area of Friesland is influenced the most if the Afsluitdijk breaches (probability of failure of 1/17,000 per year if the Afsluitdijk stays intact and 1/1,300 per year if the Afsluitdijk breaches). Unfortunately, the reports of Deltares and CPB don't explain this increase in the probability of failure or why this change differs so much from the other areas around the IJsselmeer. It is recommended to investigate this change in probability of failure for both the approach in thesis and the approach in the literature (see point 4 in section 7.2).

R.4. FINAL WORDS

This appendix shows the three main differences between the approaches of the CPB and Deltares reports and this thesis. Some of the assumptions should have been included in this thesis (e.g. investigation of the increase in probability of failure of the dikes of Friesland (section R.3)) where other assumptions that are used in this thesis should have been included in the other studies (e.g. the unrealistic assumption that wind speeds will still be maximum 4 hours after a breach in the Afsluitdijk (section R.2)). This thesis fuels the debate on the required probability of failure of the Afsluitdijk, but there are still many things that need to be investigated to come to a satisfying optimal safety level.

Although the conclusions of this thesis can be seen as a statement that the current standard of safety of the Afsluitdijk is too high, this appendix (together with chapter 6 and appendix Q) shows that this can't be stated solely based on the findings of this thesis. What this thesis does show however is that, despite the currently low quality of the Afsluitdijk and the dikes of the Noordoostpolder, the protective ability of the whole IJsselmeer system against flooding is still very high.

Therefore, the conclusions of this thesis should not be seen as a contradiction to the current plans to improve the Afsluitdijk, but as a reassurance of the quality of the whole system. Even if an extreme storm will hit the Afsluitdijk tomorrow, it is highly unlikely that the consequences of a breach in the Afsluitdijk will turn into a national disaster. This thesis showed that there are many factors influencing the impact of a breach in the Afsluitdijk of which many have a positive effect on the safety against flooding of the hinterland.