Uncertainties in Future Dike Design

Master Thesis Report

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HKV LIJN IN WATER

Challenge the future

Uncertainties in Future Dike Design

Master Thesis Report

by

Guido Anthony Evers

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The cover image is an edited photo, the original comes from [Geomatics Park, 2013]

Preface

In front of you is my master thesis which will complete my time as a student at the Delft University of Technology. Over the last couple of years, I developed a special interest for the possibilities of probabilistic design in hydraulic engineering. Consequently, the subject of my thesis is "uncertainty handling in future dike design". An exciting, relevant and inherently uncertain subject in my opinion.

The purpose of the thesis is to re-evaluate several key components of dike designing at this moment in time and in the near future. In so, the reader is explained about the things we know (which are considered certainties), and maybe moreover, about things we do not know (the ensuing uncertainties).

I have observed and experienced that handling of uncertainty is often based on expert judgement. The challenge in this is not so much about reducing the amount of uncertainties, but moreover about estimating the significance of the recognized uncertainties. Since this is a hard measure to quantify, statistical analysis can help in certain cases. The effects of uncertainties can be a great order, therefore, a researcher has to think carefully about how uncertainties can be quantified if he or she ever wants to convince colleagues about the necessity of inclusion.

The first months of this research I experienced as quite turbulent. Several times I had to readjust and reformulate my research goals because the previously formulated were in-reachable or already carried out. In many instances I as looking to tackle problems which did not have a steppingstone. This was moreover noticeable when using existing models. Learning and mastering models takes a lot of time and when you notice that the answers you seek force you look beyond a model's domain, then you know you are on the frontiers of research. Stepping into a field of 'uncertainty'. It is at these times that you need to clear your mind, evaluate your findings and focus at the problem at hand.

Although, a master thesis can be considered as the test a student as to complete on its own, I was fortunate to discuss these new frontier with others. My research is carried out and supported by HKV Lijn in Water. In my experience, a company where good ideas by own initiative and a high level of detail are hailed. At several pivotal moments, Jan Stijnen helped me clear my mind, so that I could focus at the problem at hand. My gratitude goes out to the chair of my assessment committee Matthijs Kok for helping me find an appropriate thesis subject and committee members Oswaldo Morales Napoles and Henk Jan Verhagen for providing useful feedback. I would also like to thank Chris Geerse, Karolina Wojciechowska, Gerbert Pleijter and Matthijs Duits who were always standby to spar about various brainteasers. Furthermore, my family supported my throughout the last months for which I am very grateful. In particular my mother and Kim who have been my biggest supporters. Thank you all.

Guido Anthony Evers
Delft, June 2015

Summary

With the submission of *the Deltaprogramma 2015* a decisive choice was made in the way we defend ourselves against flooding. The introduction of a flooding probability as the new safety standard impacts the way water defences are assessed and the way they are designed. Because the new safety standard has not yet been taken into formal effect, the Design Instrument 2014 (Ontwerpinstrumentarium 2014) has been released as a means to design new flood defenses in the transition period.

The method specified by the *Ontwerpinstrumentarium 2014* functions as a guideline for a dike designer. The extreme events for which a dike is designed or assessed greatly exceed the situation under daily circumstances. In this thesis, a dike location along the coast of Flevoland and a location along the Waal are investigated. The exceedance probabilities that correspond with these extreme circumstances are in the range of 1/187,500 and 1/62,500 per year respectively. In practice, this could mean a dike heightening of 1 and 2 metres. Looking in more detail at the described in the *Ontwerpinstrumentarium 2014*, questions can be asked about the underlying assumptions. This study addresses (some of) these questions.

The first question occurs when looking into the distribution of the various failure mechanisms to the overall failure probability of a dike section. Depending on the local situation, one or more of the failure mechanisms is more important than the others. The weighting factors in the Ontwerpinstrumentarium 2014 are derived for an "average" dike. By distributing the weight factors of the contributing failure mechanisms corresponding to the local situation, more efficient dike design may well be possible. In the case of the coast of Flevoland and the dike along the Waal river, a reduction up to 0.7 m and 0.3 m respectively can be realised in the required crest height.

One of the ways to handle uncertainties in a the design or assessment of dikes is by using probabilistic models. With the aid of such models, epistemic and inherent uncertainties are translated into random variables, each with their own distribution and corresponding parameters. To fully incorporate the uncertainties into a model, every uncertainty should be modelled as a random variable. However, a probabilistic model becomes more computationally intensive with each addition of a random variable. Therefore, it is useful to identify the parameters that are most influential and thus limit the number of implemented random variables. Another option would be to find a different way to take these uncertainties into account, but without adding all of the random variables.

In this research, the composition of the currently used robustness surcharge in dike design was evaluated to gain insight about the most important sources of uncertainty. The results show that the incorporation of uncertainties is in line with the current robustness surcharge. Upon further investigation, it is observed that the statistical uncertainty in the wind is one of the most influential factors. A solid background documentation of this uncertainty could not be found, therefore it was decided to investigate this specific uncertainty further, and to see how it contributes to the overall design of a dike.

A statistical wind uncertainty model was built on the premise that uncertainty increases for more extreme return periods, whereas previous models assume constant uncertainty for all return periods. The model shows that it is possible to change the order of integration of random variables. Thus, the uncertainty can be integrated a priori (i.e. outside the existing probabilistic models). In so, only the input of a model is changed and not the model itself.

Using this new model, hydraulic boundary conditions and minimum required crest levels are computed for extreme return periods. For lake systems it was found that the current robustness surcharge is insufficient. The normative water level increases by almost 1 metre, wave lengths increase by 10% and wave heights as much as 16%. For upper-river systems the effects are negligible, because the impact of the wind is much less under design conditions in this area.

From this research three main findings can be deduced. Firstly, the proposed methods of the *Ontwerpinstrumentarium 2014* do not lead to a cost-optimal dike design. Secondly, addition of statistical wind uncertainty leads to a significant increase of hydraulic boundary conditions for lake systems. Consequently, when taking epistemic uncertainties into account in dike design, the current robustness surcharge for lake systems need refinement. Lastly, the prior integration of any random variable has many computational benefits, and it is therefore recommended to incorporate in future models. The validity of the prior integration should still be carefully verified in all other water systems though, especially when correlations between random variables play a role.

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GLOSSARY

Assessment	Dikes are assessed for compliance with the safety standard.
Contribution factor	A parameter assigned for a specific failure mechanism which presents a measure of significance.
Crest height	The height of a dike.
Delta Program	A program imitated to prepare the Netherlands for flood safety and water availability
Dike ring	System of flood defences.
Dike section	A stretch of dike which is considered constant in properties and loading.
Dike trajectory	Usually consists out of several sections. The length is tailored to the expected damage in case of a breach.
Epistemic uncertainty	Uncertainty which comes from the use of models and statistical data.
Failure mechanism	Various mechanisms of dike failure are identified, such that it can be addressed separately.
Inherent uncertainty	Uncertainty which comes from natural variability.
Primary flood defence	A flood defence which is part of dike ring or lies in front of a dike ring.
River discharge	The water which flows in a river.
Robustness surcharge	An additive or multiplication factor for loading conditions which can be proccesed to take account for uncertainties.
Safety Standard	A set standard to which a dike design has to comply.

Acronyms

BC	Boundary Conditions
CI	Confidence Interval
FLORIS	Flood Risk in the Netherlands
GPD	Generalized Pareto distribution
HBN	Required crest height due to overtopping
io	Integrated Out
IPCC	Intergovernmental Panel on Climate Change
KNMI	Koninklijk Nederlands Meteorologisch Instituut
LIR	Local Individual Risk
MHW	Mean High Water
MKBA	Maatschappijke Kosten Baten analyse
NAP	Normaal Amsterdams Peil
OI2014	Ontwerp Instrumentarium 2014
RW	Rijkoort Weibull
SLR	Sea Level Rise
SSR	Sum of Squared Residuals
TAW	Technische Adviescommissie voor de Waterkeringen
UNEP	United Nations Environment Programme
VTV	Voorschrift Toetsen op Veiligheid primaire waterkeringen
WMO	World Meteorological Organization
WTI	Wettelijk Toetsinstrumentarium

List of Symbols

A	[m^2]	Area
α	[rad]	Slope angle
b_0	[m+NAP]	Average bottom level
β	[-]	Reliability index
C	[-]	Constant for wind set-up
d_0	[m]	Average water depth
Δh	[m]	Wind set-up
F	[m]	Fetch length
F_{water}	[N]	Force induced by water
F_{wind}	[N]	Force induced by wind
g	[m/s^2]	Gravitational acceleration
γ	[-]	Shape parameter
γ_b	[-]	Berm influence
γ_{beta}	[-]	Wave incident influence
γ_f	[-]	Slope roughness influence
γ_v	[-]	Vertical wall influence
h	[m]	Water depth
h_{crest}	[m+NAP]	Crest height of dike
h_{design}	[m]	Design height
h_{loc}	[m]	Local water depth
H_s	[m]	Significant wave height
i	[-]	Inflation
L	[-]	Length
L	[m]	Length
N	[-]	Length effect factor
m_0	[m+NAP]	Average lake level
μ	[-]	Location parameter
μ_w	[m/s]	Mean of w(gamma)
N	[-]	Length effect factor
n	[-]	Amount of samples
N	[-]	Amount of 12 hours intervals in winter months
ω	[-]	Failure mechanism contribution factor
P	[-]	probability
$P_{required}$	[-]	Required probability
P_{design}	[-]	Design probability
Q	[m^3/s]	Discharge
r	[degrees]	Direction
ρ_a	[kg/m^3]	Density for air
ρ_w	[kg/m^3]	Density for water
s	[-]	Wave steepness
σ	[-]	Scalar parameter
σ^2	[-]	standard deviation
σ_w	[m/s]	Standard deviation of w(gamma)
T	[s]	Time
$T_{m1,0}$	[s]	Spectral wave period
T_s	[s]	Signifiant wave period
u	[m/s]	Flow velocity
ξ	[-]	Breaker parameter

1 INTRODUCTION

The Delta Program project has initiated a new era of flood safety assessment with the introduction of the new safety standard. The program is characterized by combining state of the art insights about flood safety to develop a safe environment in flood prone areas for current and future activity. This proactive stance towards protection against flooding enforces a re-evaluational of what we know, and maybe moreover, what we do not know. The document in front of you is looking into unclear and uncertain matters concerning the evolving dike design and assessment methods in this moment in time.

In September 2014, two sweeping chances were introduced. One being a new safety standard for primary flood defenses of the Netherlands, the other being the inherent dike trajectory approach the new safety standard brings. The proposed change in safety standard is from a exceedance frequency to a flooding probability. The former safety standard was first introduced after the Great Flood Disaster of 1953. Some refinements of the safety standard have taken place in recent decades, however a more elaborate and arguable more up to date safety standard seemed applicable with changes in land use and a growing population. Also, technical developments have led to more possibilities in dike design and flood modeling. A major difference in the new flooding probability approach is that loss of life and economical damage is better accounted for in the determination of the new safety standard.

Another fundamental difference between former and future approach will be the change from a dike ring approach to a dike trajectory approach. This choice is made since new insights about flooding patterns have led to the believe that not every dike breach will cause the same damage of an area. A more tailored approach is therefore introduced with the trajectory approach. In this approach, a dike trajectory of several kilometers will be evaluated and be given its own appropriate safety standard. In so, weaker dike sections can be identified more easily and addressed accordingly.

The interpretation of the implementation of the new approach has proven to be rather difficult to grasp for policy makers and dike designers alike. For this reason the "Ontwerp Instrumentarium 2014" (OI2014) has been released to make the new approach more tangible. This documents facilitates a set of design rules which can be addressed in the interim period of safety standards. It is projected that the legal assessment guidelines of 2017 (WTI2017) will feature new assessment rules and models.

The OI2014 features two main topics which will also be of special interest in this study. For one is the contribution factor for various failure mechanism ω (dutch: faalkansruimtefactor) and the updated robustness surcharges which explicitly account for uncertainty. Although, the OI2014 tries to explain in detail what these features stand for, these two topics still raise many questions. For example, where do the specific factors for the contribution factor come from? Or, where does such a robustness surcharge origin from? As an adequate engineer these are sound questions. From a scientific perspective, one should not blindly follow design rules, especially if they are still in development. And from a practical point of view, the designation of certain factors in the OI2014 are highly influential on a dike's design.

By asking these questions, it can quickly be deduced that many design rules are influenced with certain amount of uncertainty and are therefore debatable. The designation of the robustness surcharge especially relies heavily on expert judgment. From literature research in the interest of this study, it also shows that the robustness surcharge as proposed in the OI2014 was never intended to be put in a framework wherein extreme return periods are looked into.This study takes a look closer to the establishment of the robustness surcharges and what the implications are when extreme return periods are concerned.

As a results the following topics are addressed in this study; how large is an error one makes using estimation methods, what do subjective choices entail for a design and how can uncertainty be accounted for in future modeling? Equally important is which uncertainties are to be addressed and which are of lesser importance. To this end, an alternative implementation of statistical wind uncertainty is investigated.

Of special interest throughout the report is how uncertainties affect the flooding probabilities of a dike. When determining failure probabilities probabilistic models are typically used to incorporate uncertainties, and to give a better representation of nature. However, the validity of some of the random variables is a concern. Solid research towards the nature of these uncertainties is critical when trying to determine a proper description of the random variables in terms of distributions and parameters. Sometimes the parameters that are part of these distributions are also considered uncertain (and have their own distribution and parameters). While this may be mathematically sound, one cannot help but get the feeling that we are considering the uncertainty of an uncertainty. Finally, there are the physical models that are used to model nature. Though several uncertainties are incorporated in the methods to determine flooding probabilities, and assess the state of our current flood defenses, there are many more that have not been included.

This report will investigates the underpinnings of various elements within the OI2014 and current handling of uncertainties. Also, an attempt is done to contribute in the uncertainty handling for future modeling. The scope of the research will be further explained in the next chapter wherein research questions and the structure of the report are displayed.

2
Aim of study

The main topic of this study is what the impact of uncertainty is on the assesment and design of a dike. As can already be duducted from the introduction of this study, uncertain factors can be identified in every level of detail during a dike assesment/design. Three main topics of interest are distilled in this study. Togheter, a clear overview is retrieved which some might refer as hot topics from now on.

2.1. Problem description

Given the new safety standard based on flooding probability, the question is what the new guidelines entail for the assessment of the current flood defenses and future designs. Finding a way to properly handle and validate the current methods is essential. The OI2014 introduces a method in which the contributing factors ω (Dutch: faalkansruimte) for the various failure mechanisms causes confusion among designers. The distribution of the contributing factors is expected to be important for the ultimate normative assesment or design values. To this end, implications of the guidelines will be investigated.

In the last decade, a trend of uncertainty inclusion can be observerd. Inclusion in the WTI2017 is also advised by Nicolai et al. (2014). Guidelines for river dikes (Ministerie van Verkeer en Waterstaat, 2007) first introduced the inclusion of model uncertainty. Statistical uncertainty was soon added to the equation by Nicolai, Wojciechowska, Vrouwenvelder, and Steenbergen (2010). It is this latter document to which the robustness surcharge of the OI2014 is inferred. During the establishment of the robustness surcharge, extreme return periods as proposed in the OI2014 were not investigated. This study will put that framework into this new scope of extreme return periods.

For the WTI2017 the desire is expressed to consider options regarding the inclusion of model and statistical uncertainty (Nicolai et al., 2014). Inclusion is aimed in the to be realized probabilistic Hydra-Ring model. If such a statement is sought, then solid research towards the nature of these uncertainties is critical to ensure proper incorperation and designation. As follows from this study, statistical wind uncertainty has a significant impact in the robustness surcharge. By investigating this matter, smart and proper solutions are sought for the incorperation of uncertainty in a model.

This triggers an interesting debate: to which extent should uncertainties be included in these computations? What level of details should be used to properly use these uncertainties, without stacking uncertainty on top of uncertainty, and ending up with an enormous amount of conservatism. This last part is exactly what you don't want, because avoiding conservatism is the primary reason for using a probabilistic model in the first place.

2.2. RESEARCH QUESTION AND GOALS OF STUDY

This study will focus on the influence of uncertainties in the assessment and design of dikes for lake and upper-river systems. The aim of this research is to give an answer to the following research question:

What impact do uncertainties have in future dike designs?

To answer this question the following sub-questions are investigated:

1. With special attention to uncertainties, what are the consequences of a change in safety standard for the design of a dike?
2. Which uncertainties are of importance in the estimation of loading conditions?
3. In search improved uncertainty inclusion methods, how can the inclusion of statistical wind uncertainty be improved in future modelling?

By answering these questions, a contribution can be made to the effectiveness and efficiency of dike designs. This research will be relevant to civil engineers, spatial planners, policy makers and residents of flood-prone areas.

2.3. FRAMEWORK OF THE RESEARCH

In this thesis, several assumptions are made so that the objective is clear and the research domain is framed. Some assumptions have been chosen by the researcher judgement, whereas others originate from time or resource limitations. Several important assumptions are outlined below.

MAGNITUDE OF THE SAFETY STANDARD

The magnitude of the safety standard is not questioned. For the maintenance of flood safety in the Netherlands, dikes have been appointed a safety standards. The safety standard can be seen as a measure of reliability that a dike has to meet, as has been formalised by the Water Act. More information about the safety standard can be found in section 3.1.5.

The magnitude of the current safety standard originates from a risk analysis performed after the North Sea flood of 1953. Thereafter, several refinements have taken place to establish a appropriate safety level for all dike rings. The magnitude of the new safety standard is derived from a thorough cost benefit analysis, wherein potential economical damage and loss of life is better accounted for. Considering the costs of preventing a flood disaster, an optimal magnitude of flood probability is derived. In some specific cases, the appointment of safety standard's magnitude is also a political choice.

Risk analyses such as these are outside the scope of this research. Therefore, the magnitude of the safety standard is taken as proposed by Ministerie van Verkeer en Waterstaat (2010) and Rijkswaterstaat (2014b).

THE OVERTOPPING FAILURE MECHANISMS IS MAINLY ANALYSED

This thesis mainly looks into the impact of uncertainties on hydraulic boundary conditions. This is made tangible by translating the hydraulic boundary conditions to a required crest height of a dike. This can be done by elaborating the overtopping and overflow failure mechanism.

Obviously, a comprehensive dike design should take account of (preferably) all failure mechanisms. During the course of this research, opportunities arose in the former stated field. Therefore, other failure mechanisms such as piping, erosion or stability have not been further analysed.

USED DIKE DESIGN/ASSES MODELS

The Hydra-Zoet model and the PC-Ring model are used extensively throughout this research. The Hydra-Zoet model has a legal status in the Netherlands, in the sense that it is part of the statuary tools for the assessment of dikes. The PC-Ring model does not have a legal status. It is mostly used for research purposes. Both models are commonly used in the Dutch water sector and are made available for this research.

SMART DIKE PROFILE OPTIMISATION IS DISREGARDED

This thesis does not look into 'smart' solutions in dike designs. This refers to berms, slope roughness, etcetera. Several examples in this report use a standard geometric dike profile to simplify the elaboration and diminish the influences from not looked at parameters.

2.4. STRUCTURE OF REPORT

The structure of the report can be seen in figure 2.1 and will be explained accordingly. During the course of the project, research goals had to be adjusted several times since new opportunities arose after new insights were obtained. For this reason, a broad scope of topics are threated in this study. This has not be considered as a downside, on the contrary, it suffices in an overall overview of uncertainties which are inherently connected to one and other.

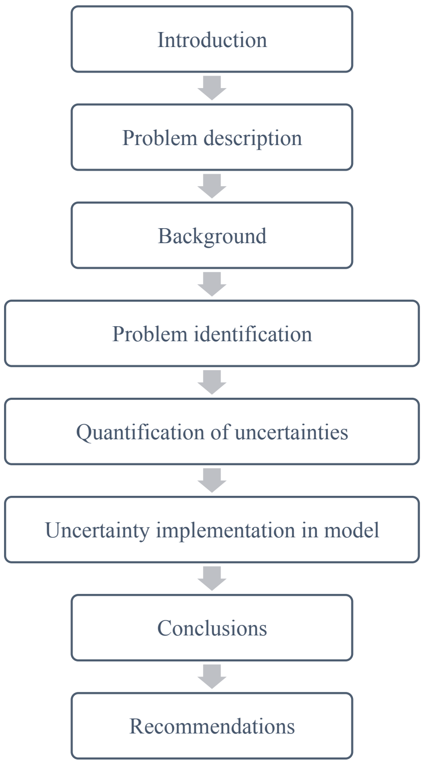


Figure 2.1: The structure of the report.

The introduction and the problem description set the scope of the study. More detailed information about current environment of dike design and assessment in the Netherlands is introduced in the "Background" chapter. Also, reoccurring topics such as the OI2014 and the establishment of the robustness surcharge are further treated in this chapter.

The analyses of this study can be distinguished in three main topics. Namely, problem identification, quantification of uncertainties and uncertainty implementation in a model. Together, these topics form the core of this report. Two practical cases will be considered to make these topics tangible. The locations considered are the coast of Flevoland and a dike location along the river Waal. These two locations are expected to be highly influenced by the new safety standard.

Designers and policy makers are faced with new design rules. This causes confusion in execution and brings debate of subsequent results. In "Problem identification", the current considered dike stretches will be compared with rough dike designs using the new OI2014 guidelines. In so, the implications of the OI2014 are made clear and problems can be identified. The analysis will be concluded with an extended view on the distribution of the contribution factor for failure mechanisms.

The next topic in this study addresses the robustness surcharge which is displayed in the OI2014. The robustness surcharges is used to explicitly account for uncertainties in models and statistical data. The attribution of surcharges is indirectly taken from a research carried out by Nicolai et al. (2010). During that research, a special version of PC-Ring is developed in which extra stochastic variables can be added for model and statistical uncertainty. This model is made available for this study. The aim of this analysis is to gain insight about uncertainty behavior for extreme return periods (as proposed by the new safety standard) by using the special version of PC-Ring. This leads to a greater understanding of the composition of the robustness surcharge. Also, most influential parameters are identified.

The last topic of the study's core is "Uncertainty implementation in a model". During the quantification analysis in the previous step, it becomes apparent that statistical wind uncertainty has a prominent impact on the robustness surcharge. Therefore, inclusion of statistical wind uncertainty is further investigated. The uncertainty will be modeled based on a set of Generalized Pareto Distributions (GPD). Furthermore, a smart implementation technique will be evaluated wherein an integrated version of the wind speed exceedance probability will be used. During the realization of this example, several statistical and numerical computation techniques will be used. The analysis concludes with the practical implementation of alternative statistical wind speed data in the Hydra-Zoet model. Overall, the analysis displays a reaffirmation of the mathematic theory about changing the order of integration of double integrals in a practical manner.

In the end a wrap of of all findings and overall implications will be gathered in the "Conclusions" chapter. Also, "Recommendations" will be formulated in follow up of this study.

3

BACKGROUND

The following chapter will provide background information about why and how the Dutch defend themselves against the sea and rivers, in section 3.1 and section 3.2. The role that uncertainties have in this are highlighted in section 3.3.

3.1. AN OUTLINE OF SAFETY AGAINST FLOODING IN THE NETHERLANDS

In the following section, an outline will be given about the interaction between the Dutch and water. After a short historical description, the framework about dike assessments and new developments in the safety against flooding are highlighted.

3.1.1. THE RELATION BETWEEN THE NETHERLANDS AND WATER

The Netherlands has a long history with water. In the past two centuries, the Dutch have sought many ways to inhabit an area formed by the sea and rivers. Before the controlled water system as we know it today was formed, artificial dwelling hills were build to serve as a safe haven during flooding. These "terpen" were first noted to be used in Friesland and Groningen during the Iron Age, approximately 500 years B.C.. Churches were usually build on these higher grounds. On lower grounds farms could be found.

Around the year 1,000 the embankment of the rivers and coastal area began to prevent frequent flooding. These dikes were far from reliable, breaches frequently occurred during storm surges. After several centuries, dike designs were improved and placement was carefully studied. In so, an enclosed areas by flood defences was created what is known to be a polder. The polders were drained using windmills. Large parts of the West-Netherlands have been drained in this way. One should realize that, initially, this area was not below sea level. Mining of peat and poldering has led to subsidence, a phenomena which can be seen in many parts of the Netherlands. Today, drainage of the polders still has to be done to prevent inundation, however, electrical pumping stations are used instead of windmills. Local water control boards are responsible for this. In figure 3.1 the development of water control over several centuries can be seen.

The North Sea Flood of 1953 caused a flood in large parts of South-Holland, Zealand and North-Brabant. It is the most costly flood in terms of damage and fatalities the Netherlands has ever experienced. In total 1800 people died, which showed how fragile the flood defense system was. The disaster has led to initiation of the Delta Works devised by Johan van Veen, a project which should prevent that a flooding like 1953 would never occur again. The project consisted out of a series of dikes, sluices and barriers, and in so reducing the total length of the Dutch coastline with 700 km. In this way, other parts of the Netherlands could remain untouched.

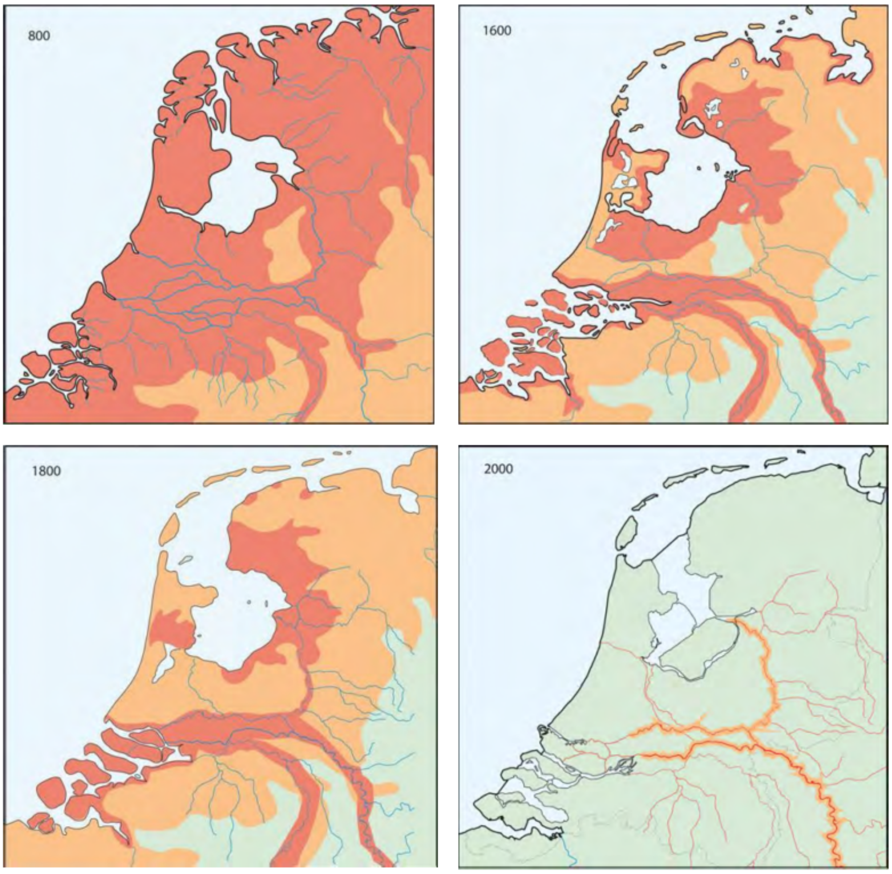


Figure 3.1: Change in Dutch control of water level. (Kok et al., 2008)

In the last decades a shift has taken place in the way flood defences are being designed. Whereas formerly flood defences where being build as a reactive measures when a flooding occurred, sophistication has led to a more elaborated approach in which anticipation has taken a leading role. A flood defence design has to be robust and effective for years to come, keeping in mind that a flood of a significantly larger magnitude could occur then ever experienced.

3.1.2. INFLOWING WATER: A THREAT FROM SEA AND RIVERS

On one hand, large parts of the Netherlands are situated below sea level. On the other, the rivers Rhine and Meuse cross the country before they debouch into the North Sea. Consequently, approximately 60% of the Netherlands is under threat of inflowing water.

During winter, storms on the North Sea cause storm surges at the Dutch coast. The first line of defence consists out of dunes and sea dikes. They have to cope with high water levels and severe wave attack. Effects of a storm surge at sea can be noticed far inland. This is because sea water flowing into the river system (delta river system) and by setting a high water level at sea which functions as a boundary condition for rivers to meet.

High river discharges from the Rhine and the Meuse, which originate from the Alphes and the Ardennes, also call for the attention in other parts of the country not adjacent to the sea. The Rhine has a large capture area in which precipitation and melting ice is aggregated during spring and summer months. High river discharges from the Meuse are usually triggered by rainfall in the Ardennes. Rarely solely melting ice or precipitation causes high water levels along the river, but usually a combination of both lead to exceptional cases. For instance, the drain capacity of the subsoil plays a role. When the subsoil is saturated due to persistent rainfall or is frozen due to a persistent cold period, a very unfavourable situation is created.

It is when storms at sea and high river disparages coincide that problematic situation occurs. With high storm surges at sea, and river high discharges with nowhere to go, the dikes which protect the hinterland from flooding are under a great loading. A tool developed by HKV Lijn in Water, shows the exposed area which is prone for inundation. Figure 3.2 shows an overview map of this data. What can be taken from this introduction is that various systems or mechanisms play an important role in the overall loading on a flood defence, these systems will be explained in section 3.2.1. In section 3.2.2 causes of dike failure are shortly elaborated.

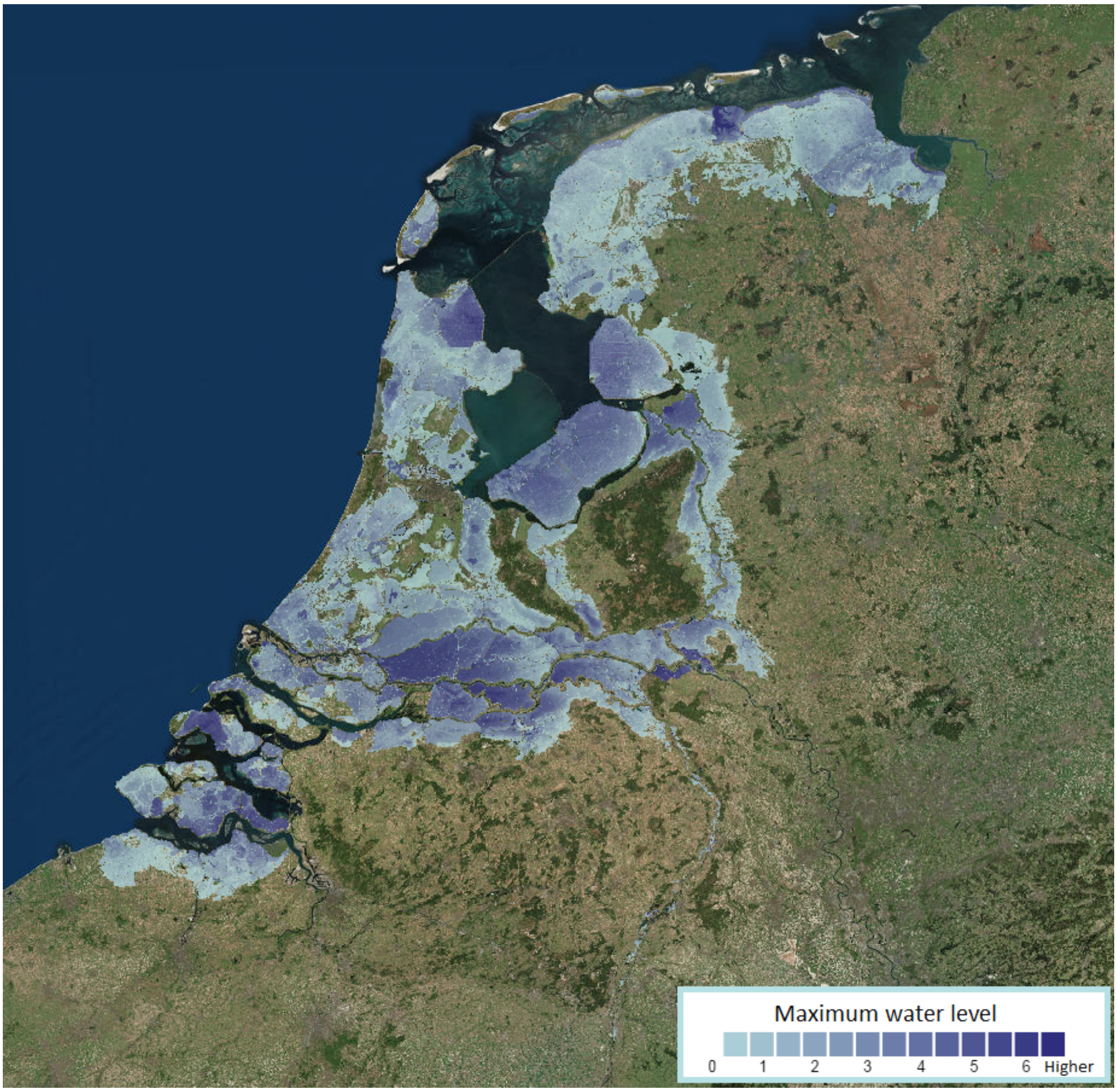


Figure 3.2: Visualization of flood prone areas. (HKV, 2014)

3.1.3. DUTCH FRAMEWORK ON FLOOD SAFETY IN RECENT YEARS

In the Water Act it is enshrined that Water boards and Rijkswaterstaat must asses primary flood defences on a periodic basis if they meet the legal safety requirements. This assessment has to be done every 5 years. The requirements are presented in the Statutory Test Instruments or "Wettelijk Toetsinstrumentarium" (WTI) in Dutch. Below the most recent developments are highlighted.

THIRD INSPECTION ROUND 2006-2011

The WTI2006 consists of the Safety Regulations for the Assessment of Primary Flood Defences 2006 (VTV2006), Hydraulic Boundary Conditions 2006 (HBC2006) and the accompanying Hydra-models.The assessment rules are collected in a report which we denote here as 'Safety Regulations for the Assessment of Primary Flood Defences (in Dutch: Voorschrift Toetsen op Veiligheid Primaire Waterkeringen). To obtain a clear picture of the current national situation on flood safety, the VTV2006 dictates how to test flood defences and by whom. An uniform standard is specified for the assessment of the quality of the flood defences, such as the dike ring approach. In addition, general guidelines were given on reporting to aid consistency.

HBC2006 gives hydraulic boundary conditions consisting of normative water levels and wave conditions during normative events, associated with their specific dike rings or connection flood defences in several water systems. The HBC have been established for the first time in 1996, in 2001 a first modification was carried into effect. The HBC are obtained with the following models:

1. Hydra-B for tidal river area
2. Hydra-VIJ for the Vecht-IJssel delta
3. Hydra-M and Hydra-Q for the IJssellake and Marker Lake
4. Hydra-R for upper river area.

$\}$ Hydra-Zoet

5. Hydra-K for the coast

$\}$ Hydra-Zout

CONCEPT FOR FOURTH INSPECTION ROUND FROM 2011-2017

From 2006 to 2011, a conceptual Statutory Test Instruments was proposed for the fourth inspection round of 2011-2017, its abbreviation is " WTI2011". Mainly building on the current system. This concept was proposed by the Ministry of Infrastructure & Environment, and was planned to be initiated early 2012, however the concept never came to realize. This due to recent policy developments. Namely, a new safety standard came in to play, therefore it was agreed that the fourth assessment is postponed until there is clarity on the updated safety standard and the third assessment round was prolonged.

As the WTI2011 was never officially adapted to a prevailing legal state, does not mean that all the work was a waste of time. The concept of WTI2011 is considered to be 'the basis' for further development in the context of the WTI2017 program. Firstly, the desire was called for a more uniform and probabilistic model. As explained by (Geerse et al., 2011): "This resulted in a single probabilistic model for the lower reaches of the Rhine, Meuse, Vecht and IJssel. Even better, the upper reaches of these rivers and Lake IJssel and Lake Marken fit into the general scheme of this model as well, meaning that all flood defences of type a of the fresh water systems are part of a single new probabilistic model, called Hydra-Zoet."
The coastal water system is modelled using Hydra-K. Aside of the fusion of models, also knowledge gained about the strength of and loads on flood defenses is better processed in the two models. To increase that insight, the Ministry of Infrastructure and Environment started the "Sterkte en Belastingen Waterkeringen (SBW)" program in 2008. A report written by Smale (2011) outlines the results of this programme. Failure mechanisms are regarded in a more comprehensive way, for instance the effect of piping.

NEW FOURTH INSPECTION ROUND 2017-2023

As the the Statutory Test Instruments of 2006 are still the prevailing methods to be used, it is said that the currently inspection round is an extended third round. Subsequently, the next inspection round is proposed to be the fourth in the series, and will be started in 2017. For this inspection round, updated methods are proposed. These will be combined in the WTI2017. The biggest difference being that the assessment has to be compliance with the new safety standard which is formulated as a flood probability per dike trajectory.

Also, all knowledge about dike failure mechanisms will be bundled in a probabilistic model called Hydra-Ring, which will be developed by several participants. This model will be able to handle flood probability, whereas previous models such as Hydra-Zoet and PC-ring only consider exceedance probabilities. Since the Directorate-General for Environment and Water has showed the preference to present the new WTI as soon as possible, the participants of this project have proposed to deliver the WTI in several phases. Concept of the WTI would be ready beginning of 2017, and the final WTI will be ready as of 2019.
To get acquainted with the new design and asses rules, a document has been released by Rijkswaterstaat (2014c) in which it explains how to handle a design and asses process using a flood probability. The contents of this will be elaborated in section 3.1.6.

3.1.4. PROGRESSIVE INSIGHTS ABOUT THE FLOOD RISK OF THE NETHERLANDS

After the 1953 North Sea flood, the Delta Commission was formed to give advise about what measures were necessary to prevent another flood like the one of 1953. In 1960 they presented a report, consisting of The Delta Works, a series of dams, sluices and storm surge barriers, and a scheme of dike rings which were given a certain required safety level. This safety level was based on the economic value of the to be protected area, and in that way an acceptable level of risk. After several refinements, the safety standards where later embedded in law in the Law on Flood Defences in the year 1993 (in Dutch: "Wet op de Waterkering"). The provinces North- and South-Holland have a safety standard of 1/10,000, many other low lying areas have the safety standard 1/4,000, with the exception of several areas along rivers where the safety standard ranges from 1/250 to 1/2,000. Water levels and wave conditions which occur during the specified safety standard are defined in the Hydraulic Boundaries Conditions guidelines.

In general, when risk is considered, the definition of risk implicates that a combination of probability and consequences are considered. In the case of assessment of flooding risk analysis, the consequences of a flood prone area are estimated by quantifying the loss of economical value (damage) and the amount of fatalities which might occur during a flooding. These insights are coupled to the probability of flooding, the expected occurrence of the event. In this way risk tries to present a realistic figure of exposed risk. The definition is given by equation (3.1). Research has been done to the current state of exposed risk due to flooding. This research has been conducted by Rijkswaterstaat and will be shortly explained below, since the research has put a groundwork for future development in the assessment of flood safety.

$$Risk = Probability * Consequence \tag{3.1}$$

THE FLOOD RISK IN THE NETHERLANDS PROJECT

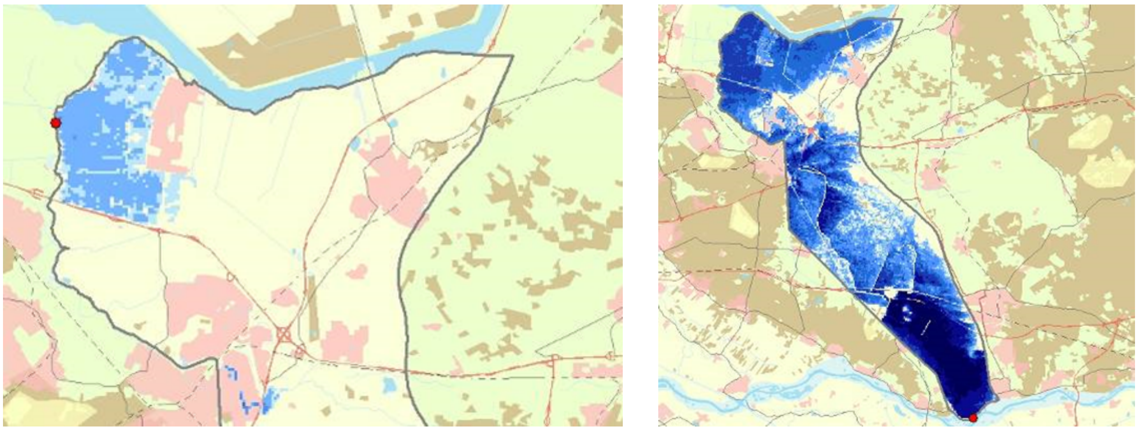
An other way of looking at safety standard is considering the amount of risk one is exposed to. In 2001 the Flood Risk in the Netherlands research (FLORIS)(Dutch: Veiligheid Nederland in Kaart (VNK)) is brought in to life to gain more insight about risk due to flooding in flood prone area's. Due to recent technological developments, an extensive location analysis as this is possible and therefore called for. The research quantifies the current level of exposed risk due to flooding in the Netherlands by estimating the exposed economical damage and fatalities. The research also gives insight about weak links in the flood defence system. In a later stage, the foundation of this research will function as a steppingstone to the new safety standard approach.

The method used is to come to such figures is particular interesting, since several scenarios of breaches in dike rings are considered. Due to natural variability in ground level, different locations of a breaching can cause completely different flood patterns. Subsequently, this will also lead to a differentiation in economical damage and fatalities. In figure 3.3 and table 3.1 this effect can be seen, a breach at the Eemdijk has different consequences then a breach at the Grebbedijk. Considering different failure mechanisms, the research also gains insight which failure mechanisms are more likely to cause failure in the scenarios considered.The both the probability of flooding and the consequences of such a flooding are calculated. The cumulative figure of probabilities of failure during all scenarios is the total probability of failure. Consequences are being elaborated using the HIS-SSM model. The model computes estimations of damage and fatalities.

What can be seen from this research is that, with the use of evolving knowledge en technology, the current state of the flood risk analysis can be presented in a more insightful manner. Subsequently, this leads to a better understanding of the Dutch water system and enhance way of communicability between engineers and policy makers.

	Economical damage	**Fatalaties**
Eemdijk	25 million	0
Grebbedijk	10 billion	100-950

Table 3.1: Difference in consequences Eemdijk versus Grebbedijk (Maaskant & ter Horst, 2014).



(a) eemdijk (b) grebbedijk

Figure 3.3: Difference in flood patterns Eemdijk versus Grebbedijk (Maaskant & ter Horst, 2014).

3.1.5. A CHANGE IN SAFETY STANDARD

In recent years two types of safety standards can be distinguished. Firstly, safety standards can be presented as an expected extreme water level. This water level represents a thinkable scenario in which various conditions can lead to an extreme water level. For example, a 1/100,000 year flood event with is related conditions, such as water level and wind speed. This type of designation emphasizes the magnitude of the hydraulic load. Secondly, flooding probability is the latest instalment of safety standards. In this definition, the area protected by the dike should be protected with a flooding probability assigned to that dike trajectory.

Due to changes in approach, policy, available knowledge and technology, the safety standard can represents different values. What can be taken from this short introduction is that the definition of a safety standard has great influence on the implication and communicability of such a figure. The following chapter will go more in to depth in the realization of such a safety standard.

EXCEEDANCE PROBABILITY

The safety standard reflects a, sometimes yet to encounter, flood event for which the flood defence should be able to withstand its load, mainly consisting of normative water levels, waves and wind. The categories appointed to dike rings are established from a risk analysis conducted by the Delta Commission after the flood of 1953, quantifying the consequences based on economical damage. For instance, the main part of South-Holland is enclosed by dike ring 14. This dike ring has been given a safety level based on probability of flooding with a return period of 1/10,000 per year. Flood defences then have to withstand water levels which might occur during a flood with a return period of 1/10,000. The design method is also known as the overloading approach for dike compartments. A transformation is made to come from dike compartment to a dike ring safety level. There is arbitrary provided that the failure of a flood defence will occur during 90% of the normative load associated with the safety standard. The safety standards appointed to each dike ring are formalized in the Water Act 2009. Currently, the normative water levels are taken up in the Hydraulic Boundary Conditions 2006 report (HBC2006), to which the Water Act 2009 refers.

As applied in the current safety standard, the HBC2006 gives hydraulic boundary conditions consisting of normative water levels and wave conditions during normative events, associated with their specific dike rings or connecting flood defences in several water systems. The normative water level came to be using a statistical analysis and a model of the water system observed. An extrapolation method is performed on historical data to 'predict' a water level or discharge for a yet to occur event. In this way, the water level of a ones-in-a-thousand year flood event can be predicted without having actual historical data of a thousand years. This gives a definition of the safety level as a return period of an extreme water level.

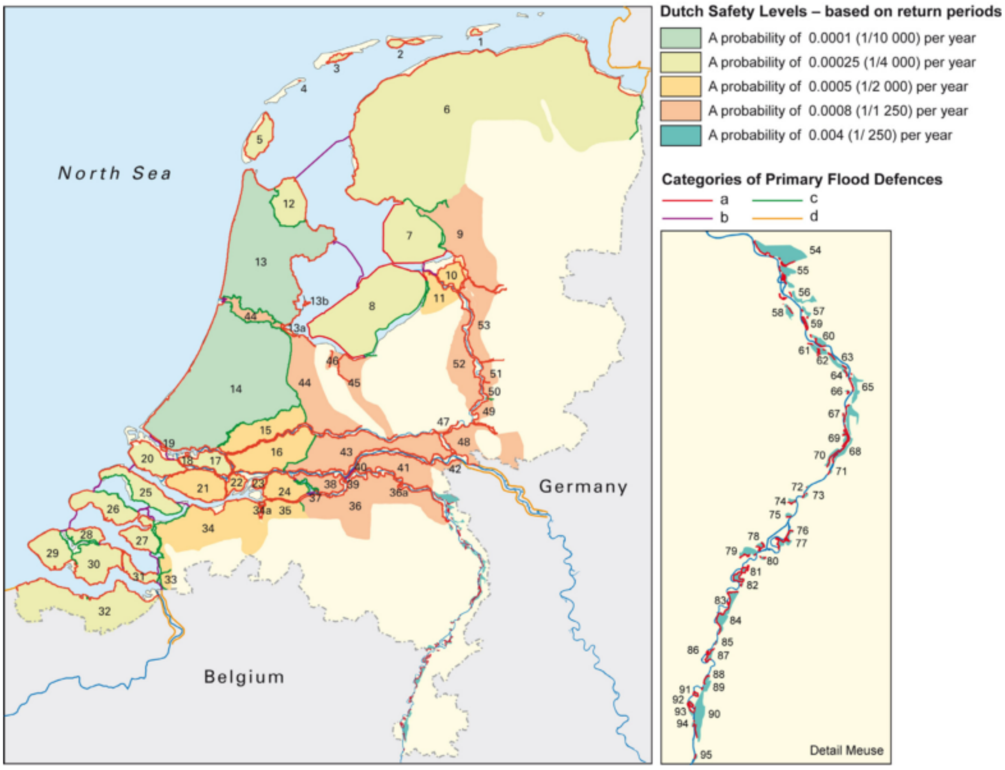


Figure 3.4: The dike rings in the Netherlands with corresponding safety standards according to the Water Act. Note that the panel at the right/below corner applies to the part of the river Meuse in the south-east of the Netherlands.(Geerse et al., 2011)

FLOOD PROBABILITY

A safety standard expressed as a flooding probability presents a figure which is based on risk analysis and expressed as a probability of flooding by dike trajectory. As in a risk approach safety standard, the consequences of a potential flood are taken into consideration and emphasize is made to the area protected by a flood defence. The economical value and number of potential fatalities play a role in the provision of prioritized flood defence improvements.

Due to technological developments in recent years, it is now possible to calculate the effect of different flood scenarios and new insights about failure mechanisms of dikes can be better taken account for. Therefore, the preference to switch to a flood probability approach for the assessment of flood risk safety is feasible. In the Netherlands the safety standard for flood defences is based on a risk approach expressed in an flood probability per dike track.The foundation of the risk analysis has been laid out by the VNK2 research. Dike trajectory designation on the other hand is a fundamental change in approach, whereas dike rings where previously considered. This approach is believed to be more efficient at prioritizing weak links in a system of flood defences. In the figure 3.5 below the new method of trajectories is visualized. Each trajectory is assigned a required safety standard. The assigned values are projected values.

As a starting point it is given that a basic level of safety should be provided for everyone behind the dike. To couple this to a number, the probability of death for an individual as a result of a flood may not be larger then 1 in 100.000 per year, provided that disaster management is in order. This last remark gives opportunities to reduce the probability of death due to evacuation, this can be seen as a link to implementation of multi-layered-safety. If the considered area shows characteristics of large amounts of inhabitants and potential casualties, significant economical damage or critical infrastructure of national importance, the safety level can be enhanced to a more appropriate level. In the end the principal of an economical sound investment is decisive.

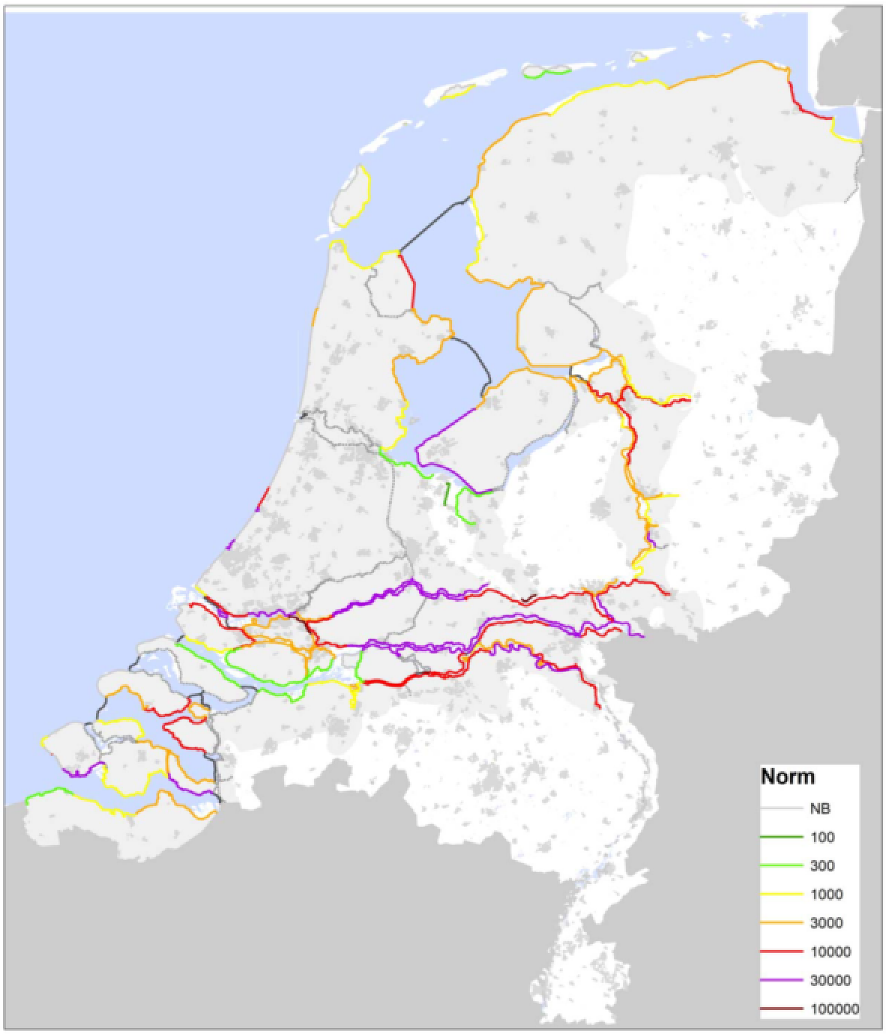


Figure 3.5: Projected safety standards assigned to trajectories (Nicolai et al., 2014)

The basic safety level of 1/100,000 for every individual, in Dutch "lokaal individueel risico" (LIR), is chosen as such to provide a sensible safety level to civilians. In comparison, the probability of death during a traffic accident is more probable, but the probability of death in the domain of external safety is less probable (namely 1/1,000,000). This is a questionable difference, however to substantiate the fact that the nature of risk is different. The domain of external safety is moreover focused on situation which stem from human activities, and therefore is considered more controllable by individuals. The probability of flooding is almost entirely out of reach of civilians.

3.1.6. ACQUAINTANCE TOWARDS NEW DESIGN RULES

In context of the Statutory Test Instruments 2017 (WTI2017) a preliminary design guide has been made available by *Rijkswaterstaat*. The new safety standards requires an updated dike design method. To this end, a new dike design and asses model is in development. Until this model is made public, designers can get acquainted with the new design method using preliminary guidelines by Rijkswaterstaat (2014c) wherein design rules are featured which can be handled using existing models. This guide will be shortly highlighted in section 3.1.6. Thereafter, a case study by ter Horst et al. (2014) of its implications is mentioned.

GUIDE FOR DIKE DESIGN USING A FLOOD PROBABILITY

The preliminary guide shows how to design dikes at the governance of a probability of flooding using the knowledge available at the date of publication (December, 2013). This means that the design rules are to be carried out with the Hydra-Zoet model which in principle cannot elaborate "flood propabilities" of dike trajectories. The working method of the preliminary guide will be shortly elaborated below:

Basically, the method explained in the guide introduces several operations which need to be applied to the loading condition prior to the elaboration of the strength of a dike. A couple of new parameters are introduced for the realization of the to reckon loading condition. Most importantly, the magnitude of the new safety standard and the contribution factor.

The flood probability of a certain dike trajectory can be seen as the joint failure probability resulting from the considered failure mechanisms. A result of the FLORIS research (Projectbureau VNK2, 2011) is the determination of a failure mechanism's significance for the investigated dikes. This has led to an overview about the contribution of each mechanism to the total failure probability. The factor is denoted as the "contribution factor" ω (Dutch: faalkansruimte) . In table 3.2 below the breakdown of the contribution factor for dikes can be seen. The sum of contribution is 1.0.

Type of failure mechanism	**Contribution factor ω**
Overflow & overtopping	0.24
Heave & piping	0.24
Macro stability	0.04
Revetments	0.10
Hydraulic structures	0.08
Other	0.30
Total	**1.00**

Table 3.2: Contribution factors resulting from a national average dike (Rijkswaterstaat, 2014c).

The guidelines state that the preliminary safety standard's magnitude $P_{standard}$ needs to be multiplied with a factor 2 to establish the design value P_{design} for which loading conditions need to be estimated. It is noted that this multiplication factor of 2 has been chosen as such that a dike will not fail at the end of its lifetime. A proper life cycle costing (LCC) analysis might indicate a more tuned design safety standard.

$$P_{design} = 2 \cdot P_{standard} \tag{3.2}$$

Since the current Hydra-Zoet model cannot compute "dike *trajectory* flooding probabilities", an operation needs to be carried out in which the safety standard is translated to a cross-section loading condition which can be elaborated in the Hydra-Zoet model. Every failure mechanism is addressed in a slightly different way, but all incorporate some sort of processing of the contribution factor and a factor which accounts for the spatial influence of a mechanism. Below, the methods for failure mechanisms overtopping, piping and macro stability are shortly highlighted:

- The minimal crest height of the dike is mainly determined by the failure mechanism overtopping. For a design to have a sufficiently high minimal crest height, the probability of exceedance of the normative overtopping discharge should be lower then the provided value of the failure probability requirement at section level. This relation is formulated as:

$$P_{required} = \frac{P_{design} \cdot \omega}{N} \tag{3.3}$$

  in which $P_{required}$ is the failure probability requirement, $P_{standard}$ is the probability of flooding at trajectory level. ω is a contribution factor for overtopping (0.24) and N is a value for the influence of the length-effect.

- The width of the dike is mainly governed by the influence of uplift and piping. The method of assessing this failure mechanism is similar to the approach used for the crest height. Meaning, the failure probability requirement should not be exceeded by the probability of exceedance of the normative effect of uplift and piping. The probability requirement at section level is formulated as:

$$P_{required} = \frac{P_{design} \cdot \omega}{\left(1 + \frac{a \cdot L_{trajectory}}{b}\right)} \tag{3.4}$$

- In OI2014 only modification to the current approach to macro stability are made. The assessment of macro stability can be investigated with analytical methods, like the Bishop method, or finite element models. If a semi-probabilistic approach is preferred, safety factors need to be applied to address the uncertainty of the subsoil conditions and water (over)pressure, model uncertainty, material uncertainty, failure due to high water and length-effect.

For ongoing projects which have not been designed using the previous stated design rules, an additional robustness surcharge is recommended in the hydraulic load. The supplement is given to be +0.20 meter, or in case of the Ketel Lake +0.50 meter.

EXPLORATION ON NEW DIKE DIMENSIONS WITH A NEW SAFETY STANDARD

An exploration study on the consequences of the new safety standard on dike dimensions has been carried out by ter Horst et al. (2014). This study shows the implication of the new safety standard and new piping formulas of Sellmeijer, modified by Förster, van den Ham, Calle, and Kruse (2012). The preliminary guide by Rijkswaterstaat (2014c) for the assessment of dike design using a probability of flooding is used for elaborating dike dimensions using the new safety standard. Influence of climate change is chosen to be in-line with the KNMI'06 W+ 2050 scenario.

Several locations across the Netherlands are considered. Among those locations were Eemshaven at sea, Nieuwegein along the Lek, Tiel along de Waal and Zwolle along de Vecht. Therefore, locations were chosen as such that different effects play a dominant rule. For instance, locations along rivers were the (sub)soil consist largely out of peat, effects of piping play a significant role. The dike location near Nieuwegein showed a required increase of 75 meters using the new piping formula, adding the new safety standard enhances the effect up to 250 meters, though no heightening of the dike was required. Whereas location at sea show no sensitivity towards piping. This can be explained since a peaty subsoil is not expected at the sandy coastline. Crest height does need to be addressed considering the new safety standard, were the difference between the current situation and required situation is about 1 meter. Some locations of primary flood defences showed

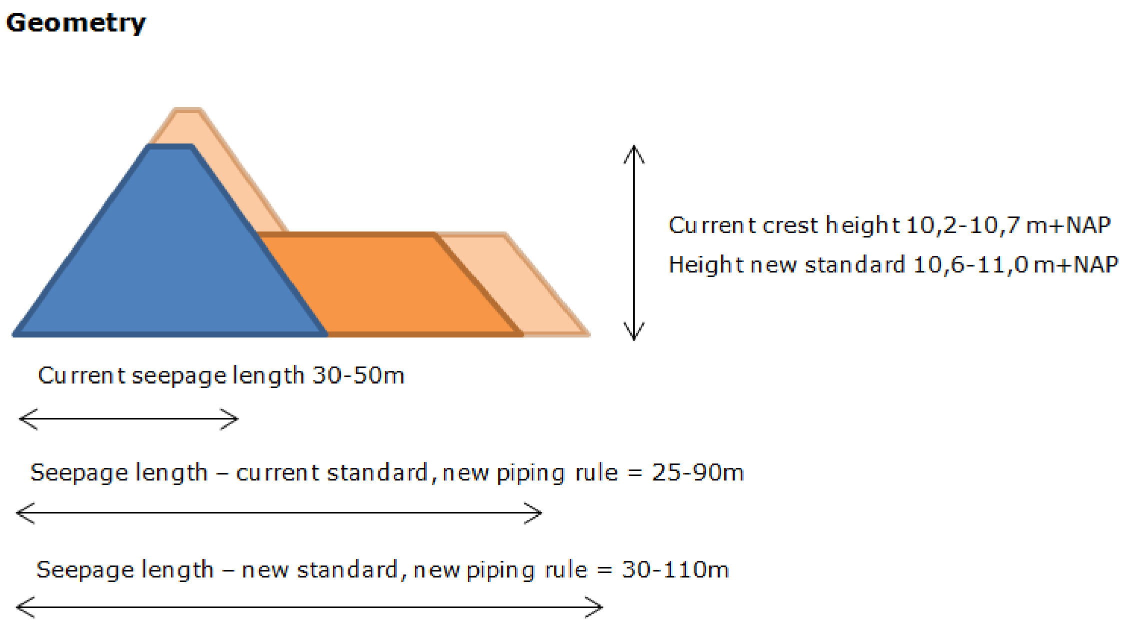


Figure 3.6: Estimated dike geometry of dike near Zutphen along the IJssel considering a new safety standard and new piping rule. (ter Horst et al., 2014)

a required improvement in both height and width, see figure 3.6, while other locations show little to no required improvement since they are already over-dimensioned.

As one of the first studies to address the tasks at hand considering the new safety standard, it shows that newly gained knowledge about piping in combination with newly established goals can have significant consequences for dike design. In some cases, dikes need to be reinforced to twice their current size.

3.2. APPROACH TO THE DUTCH WATER SYSTEM

In modelling for dike designs, it is useful to define certain boundaries for which a particular problem needs to be solved. Decades of research in behaviour of the sea and rivers have led to a division of water systems in the Netherlands. Likewise, various mechanisms of dike failure can be identified. Water systems will be explained in section 3.2.1. In section 3.2.2 causes of dike failure are shortly elaborated. The mathematical rules of these water systems and mechanisms are combined in dike design/asses models. The models of interested in this study are highlighted in section 3.2.3.

3.2.1. CLASSIFICATION OF WATER SYSTEMS

Looking at Dutch water system subdivisions can be made. This is because different natural effects play a dominant role for each system. As seen in the below table 3.3 a division is made. In this section, a short description of each area is given. This division is particularly useful during modelling of the system. In this way, only explicit care is taken into account of the dominant natural effects, reducing complexity and computational time of such a model.

	Water Systems				
Dominate Natural Effect	Upper River	Tidal River	Vecht-IJssel Delta	IJssel Lake	Coastal Zone
River Discharge	x	x	x		
Wind	x	x	x	x	x
Water Level at Sea		x			x
Waves at Sea					x
Water Level at Lake			x	x	
State of Barrier		x	x		

Table 3.3: Variables of influence on water systems in the Netherlands

The water system of eastern Netherlands is characterized by the river Rhine, Meuse and Vecht. No tidal effects play a role in water level or discharge. Therefore, the expected hydraulic boundary conditions (HBC) are governed by the discharge of the Rhine and Meuse. This area is called the upper river area.

The tidal river area covers the downstream branches of the Rhine and the Meuse. This is where water levels a experience a significant influence of both the storms at the North Sea and high discharges from the upstream rivers. Near the sea-mouth of a river, extreme water levels are influenced by storm surges combined with high wind speeds. This means dikes are threatened by wave action. On the other end, high river discharges can also give an increase in water level, increasing the load on dikes. With shorter fetches and relatively lower wind speeds, wind waves are of less importance in the upstream area. The interplay of storms at the North sea and high river discharges result in a highly dynamic system. Also of importance in this area is possible failure of the Maeslant Barrier.

Lakes as described here include the IJsselmeer, Ketelmeer, Vossemeer, Markermeer, Gooimeer, Eemmeer and the river Eem. Determination of normative hydraulic boundary conditions (HBC) are mainly given by a combination of the respective lake level and generated waves. The lake level can only be controlled to a certain extent, namely by draining water at low tide ('eb') and pump stations during hight tide ('vloed'). To anticipate on climate change, the Delta Decision IJsselmeer has presented plans to install more pumps in the Afsluitdijk to enhance the total drain capacity Rijkswaterstaat (2013). The growth of waves is driven by wind.

The normative hydraulic load on the Dutch coast and the Wadden Sea is mainly caused by the tide, wind set-up and wind waves. The Dutch coast is under influence of a semi-diurnal tide, which means two high waters and two low water each day. Wind set-up and wind waves depend on the wind speed and direction at the North Sea.

The IJssel-Vecht delta is the downstream area of the IJssel that connects the upper river area with the IJssel lake. Discharges from the the Vecht and water levels at the Ketel Lake influence the water levels in the IJssel-Vecht delta. High water levels at the Ketel Lake are usually a result of a high water level at the IJssel Lake, set-up caused by wind, or a combination of both. Also the Ramspol Flood Barrier plays a significant role in the dynamics of the system.

Figure 3.7: Different water systems in the Netherlands.(Geerse et al., 2011)

3.2.2. DIKE FAILURE MECHANISMS

The flood of 1953 gave an incentive to rethink the way flood defences are designed. In recent decades, methods have gradually been developed to counteract the mechanisms that cause failure to a dike. Initially, the designs were calibrated towards the prevention of the most obvious mechanism called 'overflow', which occurs when the crest height is lower then the water level. Waves colliding with dike slopes cause the water to 'run up' the slope, this can result in 'overtopping'. These two mechanisms cause inflowing water and erosion on the inner-side of the dike. This later effect can have devastating consequences, since erosion can cause instability of the soil body, and a breach in the flood defense system is realized.

Many dikes in the Netherlands have been designed to prevent these two failure mechanisms. In this way a required crest height is elaborated, and the dike should be constructed in such a way that it is stable. This be taken into account 'macro stability' of the inner- and outer-slope, and suitable revetments. However, many other failure mechanisms have been acknowledged to cause dike failure. For instance, a failure mechanism of significance is piping. The effect of this are shown by ter Horst et al. (2014), also see section 3.1.6. Below a short overview of import failure mechanisms is given:

- The first mechanism what comes into mind is **overflow** of a flood defence. When the normative water level exceeds the crest of a dike, overflow occurs. Overflow can cause serious erosion of the dike. Eventually, the dike can collapse. If the dike is resistant to to erosion, the question rises how much water can be absorbed by the area behind the dike. An insufficient crest height may be due to a underestimated normative water level, consolidation of the subsoil or the flood defence itself in case of a dike. An adequate estimation of the normative water level is key to counteract this failure mechanism.
- The stability of an entire slope is usually referred to **macro stability**. A soil embankments such as a dike should comply with the stability properties of the used soil and subsoil. A simplified approach in the stability analysis of embankments and slopes is the slip-circle approach, for which the Bishop-method is widely used. The load is schematized as the weight of the soil mass within the circle, the shear along this 'slip cirle' should counteract this induced force.
- Whenever a flood defence is attacked by waves, a certain amount of water can topple over. This mechanism is called **overtopping**. In case of a sloped flood defence, the 'run-up height' can give an indication on if the crest height is sufficient. A more elaborate approach might be to compute the overtopping discharge using overtopping formulas specified in the Eurotop Manual The EurOtop Team (2007). The overtopping discharge is commonly expressed as litres per second per meter [l/s/m].
- Forces imposed by water pressure and/or water flow can cause internal erosion in a dike. Cases referred to as **micro stability** concern eroding sand particles caused by seepage water that reaches the inner slope of a dike. A special case of internal erosion is the phenomena referred to as **piping**. The erosion will occur underneath a cohesive layer in the sub soil. Early research has been done by Bligh and Lane, concluding that increasing the seepage length (in general the width of a dike) will increase the stability of sand particles. More elaborated research has been done by Sellmeijer, his approach, known as the Sellmeijer formula, is now widely used.
- Other factors of influence on the dike's soundness can be settlements of the dike or subsoil, loads induced by drifting ice and other special cases such as collision other special cases. Area specific considerations must justify the need to take factors like these into account.

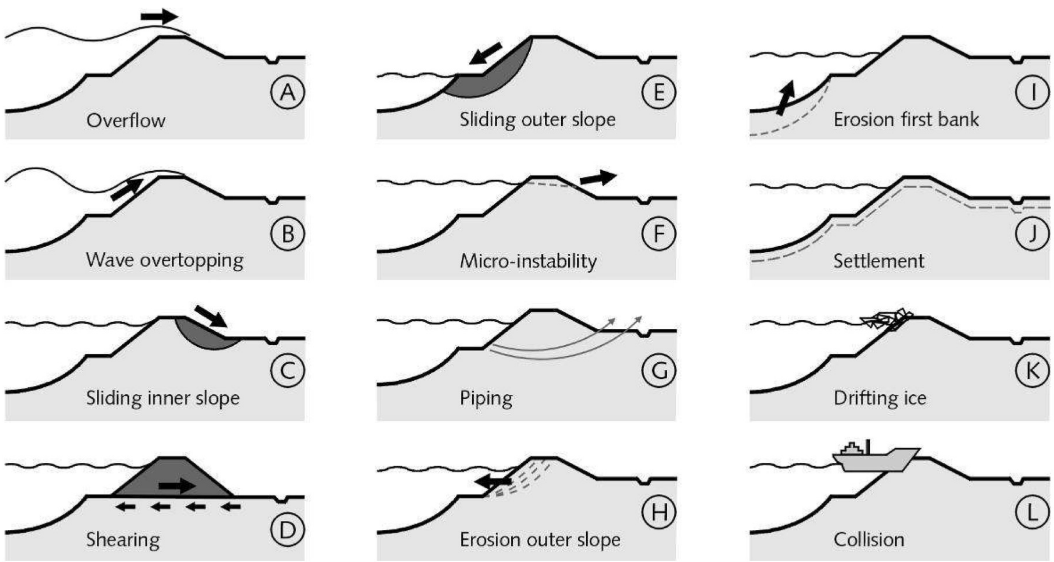


Figure 3.8: Failure modes defined by Weijers and Tonneijck (2009)

The previous list only present a limited number of failure mechanisms, although an effort is done to address to most important. Going into details of dike design, the stability of the revetments or grass cover can be considered. In the case of a multi-functional piece of land, when houses are build on top of the dike for instance, the designer or inspector should be extra cautious. What must be noted is that different failure mechanisms come in to play in their specific area of interest. For instance, sea dikes mainly have to cope with tidal induced water levels were high water levels last for approximately 6 hours, therefore high phreatic lines in the dike profile are less of an issue compared to river dikes. Whereas a high water level can be the case for longer period of time (order of days/weeks). A sudden drop in water level can cause over-pressures in the dike, causing push out of sediments or revetments. Another notable difference is the soil composition, sea dikes mainly consist of sand, more often then not a river dike has to cope with a subsoil of clay. This imposed the threat of piping and consolidation. Proper dike design relies on comprehensive understanding of the load the dike is exposed to and the strength of the dike.

3.2.3. DIKE DESIGN USING PROBABILISTIC MODELING

Combining the knowledge about dike loading and strength is done in dike design models. Two models will be highlighted below, since they will be thoroughly used in this report.

HYDRA-ZOET

The Hydra-Zoet model is probabilistic model which is used for the assessment of primary flood defences in the Netherlands. Normative water levels and wave conditions have been constituted in the Hydraulic Boundary Conditions, the latest formalized instalment of this is the HR2006. Many of these hydraulic boundary conditions have been established with the Hydra models. The assessment of flood defences is carried out every six years. The Hydra-Zoet model is a constituent of the Water Act which is formalized in 2009.

Two predecessors models of which the Hydra-Zoet is developed, are the Hydra-VIJ and Hydra-B model. The models were developed by Rijkswaterstaat Waterdienst and HKV Consultants in 1999 Geerse et al. (2011). The models can be used for water level and dike design computation for the Vecht-IJssel delta and the inter tidal rivers respectively. As development continued and technical possibilities expanded, the models were merged to a single model which covered all sweet-water system of the Netherlands. In the Hydra-Zoet model its framework, a lake system is modelled as a simplified delta model, and a upper-river system as a simplified inter tidal-river system.

The model accounts for several random variables. An overview can be seen below in table 3.4.

Table 3.4: Random variables for water systems (Geerse et al., 2011).

random variables	lake delta	lake area	tidal river	upper river
river discharge	x		x	x
lake level	x	x		
wind speed	x	x	x	x
wind direction	x	x	x	x
sea level			x	
barrier state	x		x	
predictions for barrier state			x	

Following the considered random variables, one can conclude that Hydra-Zoet solely is a probabilistic model in the elaboration of the loading. The 'strength' of the dike is computed in a deterministic manner. Currently, the upper-river systems is formally to be evaluated with the Hydra-R model, another predecessor of the Hydra-Zoet model. In the Hydra-R model no probabilistic calculations are incorporated.

To reduce computation time, not all calculations are carried out in the Hydra-Zoet model. Combination of 'basic variables' are evaluated with SOBEK or WAQUA, these models can compute water levels in a 1D- or 2D-roster respectively. Subsequently, a database of (all) possible water levels is implemented in the Hydra-Zoet model. This is combined with statistical data. In so, the Hydra-Zoet model relies on prior established data. Provided are the ease and accuracy of the hydraulic models, but also its inherent uncertainties.

Currently, the Hydra-Zoet model is capable of evaluating the run-up or overtopping discharge of a dike. In so other failure mechanisms, such as macro stability and piping, are neglected. Maybe these modules will be added in the future. According to Geerse et al. (2011) two notable projects were the Hydra-Zoet model has been used are The Safety Against Flooding in the 21st Century study and The Delta Program study.

PC-RING

The FLORIS project was initiated to gain more insight about the flood safety of the Netherlands. The project displays the current flooding probability of a specific area. To make this possible, the value of the land and the failure probability of dikes were estimated. For the computation of the failure probability a new model was developed, namely PC-Ring.

It must be stressed that the PC-Ring model is developed for research purposes and has no direct legal status. The key feature of the model is that it handles an extensive amount of load and strength properties in a probabilistic way. Many accounted for variables can be found in Steenbergen, Vrouwenvelder, and Koster (2007). The model uses several statistical (Steenbergen et al., 2007) and physical models Vrouwenvelder and Steenbergen (2003) to compute the failure probability of single dike section. Thereafter, an unification method for dike section failure probabilities is carried out to establish a failure portability for a dike stretch or dike ring of interest, the mathematical technique for the combination is revered to as 'oprollen' in Dutch and is explained in Vrouwenvelder and Steenbergen (2003).

The comprehensiveness of the model causes it to be relatively complex and computationally intensive. For slow varying time dependent loads, the model is simplified with the Ferry-Borges-Castanheta model (FBC-model). in the time domain the lake level and river discharge are discretized by blocks of a constant value (whereas the Hydra-Zoet uses a trapezium shape to define these relations).

A notable physical overtopping model which is implemented in the PC-Ring model is the CIRIA model. This models considers the resistance of the dike expressed by soil properties instead of an singular critical overtopping discharge in the failure elaboration of overtopping. The effect of this is that more overtopping is allowed as long as the dike's foundation is not compromised. This model is also used in the FLORIS project. Implications of this are elaborated in appendix A. The sensitivity analysis shows that the model is less conservative then the previously used critical overtopping discharge of 1-10 [l/s/m].

A shortcoming of the model is that it does not allow for the implementation of capping of river discharges. In recent years, a lot of debate about the possibility of extreme river discharge has taken place. No consensus has been established. It would be of great interest to investigate the effects of a physical maximum in a comprehensive model like PC-Ring.

Future model: Hydra-Ring

A currently in development dike design and assessment model is the Hydra-ring model. This is considered to be a fourth generation model. It is aimed to be a more streamlined model compared to earlier dike assessment/design models. The development project emphasizes in 'smart' modelling of loads and strengths and new approximation methods will be built into the program (Geerse et al., 2011).

3.3. Handling of uncertainties in dike design and dike assessment

The last section of the background chapter will provide information about uncertainty handling in dike designs. Types of uncertainties are outlined in section 3.3.1. An increasing uncertainty awareness in recent years can be recognized section 3.3.2, section 3.3.3 and section 3.3.4. Lastly, section 3.3.5 will look into the implications of climate change on dike designs.

3.3.1. Uncertainties in the design process of flood defenses

Admitting that men's knowledge does not fully comprehend all natural phenomenas at this moment, it can be concluded that certain uncertainties are invoked in the process of determination of a safety standard. The lack of addressing uncertainties in a proposed figure such as the safety standard for dikes, can result in significant consequences.

If for instance, a dike height is elaborated to such an extent that a designer believes that the dike will function as designed under normative conditions, initially one assumes the dike to be safe and functional. However, if during the design assumptions or mistakes are made, the elaborated dike height might not suffice anymore under normative conditions. Numerous reasons can be the cause of this, ranging from faulty statistical values of discharges/storm duration/set-up, non uniform soil conditions, negligence during construction, incorrectly assumed assumptions, etcetera. On the other hand, the same reasoning holds that uncertainty can lead to a more robust design then initially anticipated.

Stijnen et al. (2008) explains that two types of uncertainties can be distinguished. Inherent uncertainties related to fluctuations in nature and epistemic uncertainties which are the consequence due to a lack of knowledge. In the following paragraphs this division is further elaborated.

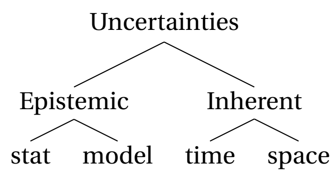


Figure 3.9: Uncertainty types, (Stijnen et al., 2008)

Inherent uncertainty

Inherent uncertainty is referred to as natural variability. This variability is due to fluctuations in time and space. This fluctuation can be found in discharges of rivers, sea water levels and wind speeds. Due to the natural fluctuations in precipitation it is not possible to predict the river discharge of the next month. This naturally present uncertainty cannot be reduced. However, the effect of it can be modeled by means of an assigned probability distribution.

Epistemic uncertainty

Epistemic uncertainty comprehends the uncertainties invoked by the lack of knowledge. This can be either a deliberate choice or not. Simplifying a model or neglecting certain factors can result in inaccurate results.

STATISTICAL UNCERTAINTY

Statistical uncertainties are brought into play when parameters need to be estimated. When only a limited (sometimes insufficient) amount of observations is available for the estimation of a certain parameter, an estimation is made with a large band of uncertainty. On the other hand, when many observations are readily available, a more precise estimation can be made.

MODEL UNCERTAINTY

Whereas statistical retrieved parameters give input to a model, this model also introduces a certain confidence interval. Although, models are becoming more sophisticated each day, they still give an approximation of reality. Complex hydrodynamic models such as SOBEK and WAQUA which are used for hydrodynamic modeling, simplify the physical process of the movement of water to a 1-dimensional or 2-dimensional process. Whereas in reality, water flows in three dimensions in highly variable river dimensions.

3.3.2. ROBUST ENGINEERING

For the design of dikes the following guides (Dutch: Leidraad) exist: "Leidraad Zee- en Meerdijken" (TAW, 1999) "Leidraad Rivieren" (Ministerie van Verkeer en Waterstaat, 2007). In the latter document, a notion of robust engineering is first mentioned. In this document it is stated that the designer should take relevant uncertainties into account to an extent that the design will suffice at the time of construction and at the end of the lifetime it is intended for. Several influential effects are identified, namely climate change, uncertainties and expandability. The aim is to be able to cope with the normative loading conditions at the end of the foreseen lifetime, even though unpredictable effects might occur. In this sense a carefully thought out design is asked which can cope with unforeseeable loading conditions or that is adaptive. From the this paragraph it can easily be understood that robust engineering is not a crystal clear process. To make the task more tangible for designers, addenda and memos have been released in the past decade in which the robustness surcharge is further explained and refined.

According to Ministerie van Verkeer en Waterstaat (2007) and TAW (2009) a robust design is not treated the same way at every location. This is understandable since in different water systems, different loading effects have a significant role. For instance, at sea and lakes the hydraulic boundary condition is heavily influenced by the occurring wind. In the upper-river system this is less of a concern. Consequently, the uncertainty in wind models is accounted for in the robustness factor for sea and lake dikes whereas this effect plays a lesser role in the robustness factor for river dikes.

Furthermore, it is added that robust engineering is not an entirely new aspect in the design of dikes since safety factors and material factors are already applied in the strength formulas of a dike. Concluding that a combination of robust engineering and safety factors will lead to a safe and maybe over-dimensioned design.

RIVER DIKES

In the "Leidraad Rivieren" (Ministerie van Verkeer en Waterstaat, 2007) report, it is said that the designer should consider robustness into the design of a dike. Robustness should be addressed while setting the boundary conditions of a design. In this stage of the design, uncertain effects should be identified and the extendibility should be considered.

The needed crest height of river dike is determined by either the normative water level (MHW) or due to overtopping. The first is considered to be the minimum crest height.

To guarantee access to a dike during high water, it is advised to incorporate a free-board into the design. The free-board is not meant to withstand hydraulic loading, but it can have a reducing effect on overtopping. It is advised to apply a minimum of +50 cm free-board in the design.

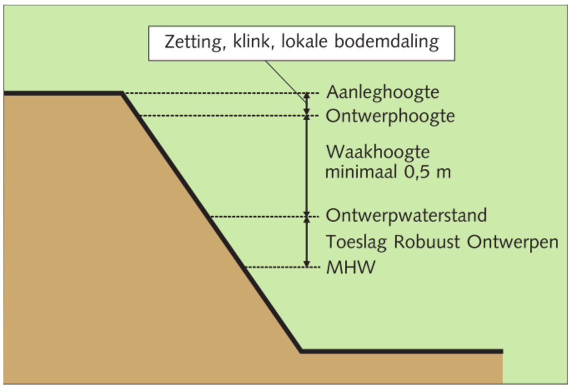
The robustness surcharge is in the order of +30 cm. This surcharge should take account for uncertainties in the hydraulic loading. One of uncertaintie which might contribute to this is the variability in the bifurcation at Pannerdensche Kop. This variability might introduce a higher river discharge then expected, and subsequently, hydraulic loading conditions can increase downstream. The design water level can be composed of the normative MHW at the end of the dike's lifetime plus a 30 cm robustness surcharge. The, so called, minimum required crest height is derived as;

$$h_{min,design} = 0.3m + MHW_{\text{end of lifetime}} + 0.5m \text{ freeboard } + \text{ subsidence} \tag{3.5}$$

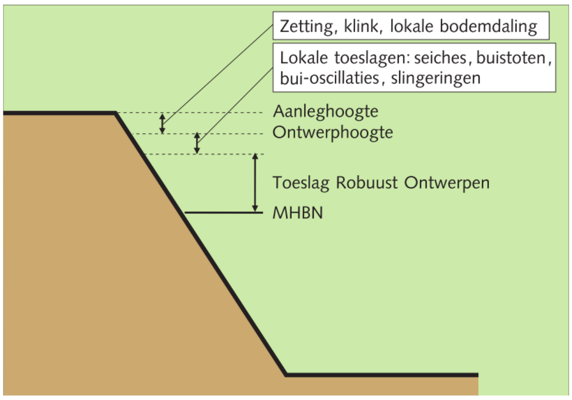
The robustness surcharge is also incorporated in the determination of the crest height due to overtopping. The required crest height at the end of the dike's lifetime following overtopping criteria is denoted as the MHBN. To this level, the +30 cm robustness surcharges is to be added. Locale surcharges such as effects due to seiches or storms should be appointed if applicable.

$$h_{design} = 0.3m + MHBN_{\text{end of lifetime}} + \text{ subsidence} \tag{3.6}$$

Since a dike is a soil body and is usually build on top of (sub)soil, the designer should be wary of subsidence. The expected subsidence should estimated and be incorporated in the dike design. The total height of the dike is therefore established as:



(a) Minimum required crest height



(b) Required crest height due to overtopping

Figure 3.10: Design of a river dike by Ministerie van Verkeer en Waterstaat (2007).

Following this guidelines of robust designing concerning the hydraulic loading conditions, it is suggested that when safety factors are used accordingly on the strength side of the dike, a sufficiently safe design has been established.

An interesting notion Ministerie van Verkeer en Waterstaat (2007) makes is this advised may be waived if a probabilistic analysis, in which all relevant uncertainties have been considered, is been carried out. This is issue will be highlighted in section 5.2.

Extendibility of a design should also carefully be thought out since improvement can lead to excessive costs. Improvements might be needed due to an increase in loading conditions or a change in safety standards. Because of this reason, a design which is hard to improve is not preferable. If a design is poorly extendible, a total overhaul would be needed in case of need improvement. The essence of a extendible design is that, in the unforeseeable future, a high gain for safety against flooding can be realised with a relative low investment.

SEA AND LAKE DIKES

The "Leidraad Zee- en Meerdijken" report by TAW (1999) gives guidelines for the design of lake or sea dike. At the year of release, robust engineering was not yet a topic explicitly highlighted in the guideline. This was later added by TAW (2009). The consideration in the elaboration of the required crest height that were accounted for by TAW (1999) are presented in this chapter.

The minimum free board consists out of the expected MHW at the end of the design's lifetime and several surcharges. The surcharges take account for local effects such as wind set-up, effects due to storms, seiches and subsidence. The only remark which has been made considering the minimum free board is that it needs to be at least 0.5 meters. This is to provide access to dike during high water.

The expected water level during the normative return period which is conveyed by a statistical extrapolation analysis and the estimation of sea- or lake level rise due to climate change.

Furthermore, it is stated that a critical overtopping discharge of 1 l/s/m should be reckoned with. The crest width should have a minimum of 2 meters, however a wider crest is preferable to ensure good ride ability for inspection vehicles. The outer slope angle is ought to be 1:4 for lake dikes and 1:5 for sea dikes. Inner slope are advised to be build with an angle of 1:4.

$$h_{min,design} = MHW_{\text{end of lifetime}} + waterlevelrise + 0.5m \text{ freeboard } + \text{local surcharges} + \text{ subsidence} \tag{3.7}$$

$$h_{design} = MHBN_{\text{end of lifetime}} + \text{local surcharges} + \text{ subsidence} \tag{3.8}$$

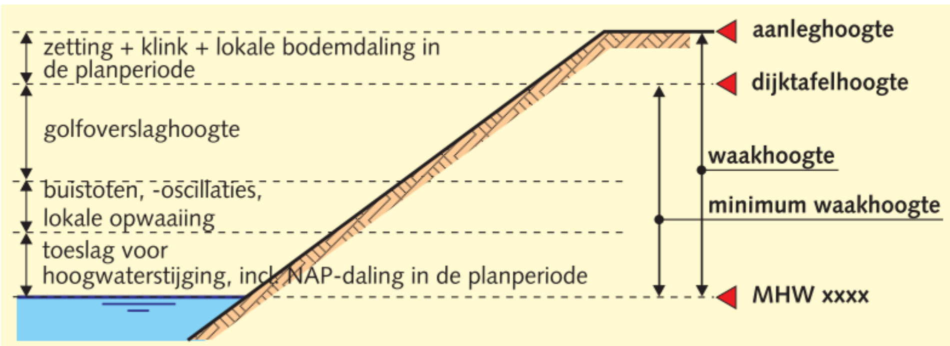


Figure 3.11: Robust engineering for sea and lake dikes (TAW, 1999).

ADDENDUM SEA AND RIVER DIKES

Soon after robust engineering was introduced in the "Leidraad rivieren", the "Addendum I bij de Leidraad Zee- en Meerdijken" was released which updated the guidelines for sea and lake dikes. This time a special chapter was dedicated explaining how robust engineering should be incorporated in the design of dikes in sea and lake systems.

The addendum uses as starting point that of the "Leidraad rivieren": 'A good robust design means that future developments and uncertainties should be taken into account, such that the execute design will remain function-able during its planning period, without the need of substantial and costly modification' To this is added that inherent uncertainties can cause an increase in loading conditions.

It is stated that robustness surcharges as proposed in the "Leidraad rivieren" are not to be simply copied in the updated guidelines for sea and lake dikes since other physical processes cause different uncertainties to play a significant role. For comparison TAW (2009) says that the uncertainty in MHW and normative wave conditions is far larger. Therefore, these were further mentioned as inherent model uncertainties.The inherent model uncertainties withheld the uncertainties associated with errors due to the in-comprehensiveness of the used physical models. A +20 cm surcharge should be applied to the MHW-level due to an increasing lake level during winter months and increased wind set-up. The equation (3.9) of Bottema led to believe that an increase of significant wave height (H_s) is in the order of +10% on the IJssel- and Marker Lake.

$$\Delta H_s = 0.45 * \Delta SLR \tag{3.9}$$

For the wave period (T_{m0}) associated with these conditions no readily available formula was derived. Therefore it was chosen to determine the wave period by using a preservation of wave steepness. This led to an increase of +10% for wave periods.

To the extent of climate change, it is expected that sea level rise (SLR) would occur, and therefore MHW and wave heights are to increase. However, this is not to be included in the robustness surcharge. Also, statistical uncertainties due to extrapolation to extreme conditions is not to be included. Another interesting notion in the addendum was to maintain the current lake level of the IJssel Lake till 2050, this can be realised by doubling the drainage capacity.

The consistency in sea/lake dike design and river dike design is that for both a free board of 0.5 meters is advised to provide access during normative conditions. Normative conditions are derived from statistical extrapolation analyses. And overtopping criteria are similar since the a critical overtopping discharge of 1 l/s/m is considered normative. Robustness in the design stems primarily from this conservative requirement. Also, it is stated that uncertainties are less of a significant factor when conservative hydraulic loading conditions are used and material factors and safety factors are properly accounted for in the dike design formulas.

CURRENT DEVELOPMENTS IN ROBUST ENGINEERING

As part of OI2014 by Rijkswaterstaat (2014c), robustness surcharges are proposed. The surcharges have been extended with the acknowledgment of statistical uncertainty. The extrapolation of the statistical analysis for water levels and river discharges tries to make an as accurately as possible estimation for the expected conditions for return periods which far exceed our available measurements. The more measurements are available, the better the estimation will be. For ordinary common conditions many measurements are available, however for extreme uncommon conditions only few are. This extreme instances however weight the most in determination of extrapolated cases.

Rijkswaterstaat (2014c) only shows a table of considered robustness surcharges, these are presented in table 3.6. For a detailed explanation about the realization of the robustness surcharges a reference is made to den Bieman and Smale (2014). They point out that the following uncertainties which are shown in table 3.5 can have a significant impact on the hydraulic loading on a dike. These are epistemic of nature, which include model uncertainties and statistical uncertainties. The consideration to further extend the robustness surcharges is in follow-up of the impact analysis about these uncertainties performed by Nicolai et al. (2010). Their report is further highlighted in section 3.3.3 on page 30. Although it is stated that these factors are (maybe) considered to be conservative, it still strongly advised to comply to these factors in current HWBP projects.

Table 3.5: Model and statistical uncertainties to be reckoned with by den Bieman and Smale (2014).

Uncertainty	**Water system:**				
	upper river	**tidal river**	**shore**	**lake**	**VIJ-delta**
Statistical uncertainty river discharge	✓	✓			✓
Statistical uncertainty wind speed	✓	✓	✓	✓	✓
Statistical uncertainty lake level				✓	✓
Statistical uncertainty sea level		✓	✓		
Model uncertainty local water depth	✓	✓			
Soil	✓	✓	✓	✓	✓
Fetch length	✓	✓			✓
Wave parameters	✓	✓	✓	✓	✓

Table 3.6: Robustness surcharges by Rijkswaterstaat (2014c).

Water system	**Parameter**	**Robustness**
Rivers	water level	0.30 m
Lakes (excluding Ketel Lake)	water level	0.40 m
	wave height	10%
	wave period	10%
Ketel Lake	water level	0.70 m
	wave height	10%
	wave period	10%
Tidal rivers	water level	0.40 m
	wave height	10%
	wave period	10%
Sea	water level	0.40 m
	wave height	10%
	wave period	10%

3.3.3. INFLUENCE OF EPISTEMIC UNCERTAINTIES

A report about the influence of epistemic uncertainties by Nicolai et al. (2011) has tried to quantify the amount of uncertainty which is invoked by the epistemic uncertainty. Overall, the researched pointed out that taking more uncertainties into account the expected water levels are underestimated by 0.1 to 0.5 meters. In the cases of Rotterdam and the IJssel-Vecht delta this figure can rise up to 0.8 meters. Figure 3.14 gives an overview of expected water level increase if epistemic uncertainties are accounted for. This increase is expected, considering more uncertainties are taken into account. More uncertainties usually translate in a higher expected values. The sea level and river discharge statistics and the uncertainty in the local water level appear to weigh heavily in the calculated estimated water levels. Also, wave conditions are influenced by the uncertainty in wind statistics.

The research recommends, if it is decided to calculate flooding probability or risk in a fully probabilistic manner, epistemic uncertainties should be taken into account. The possibility for an objective method for addressing epistemic uncertainties should be investigated, to not rely on expert subjective judgment. A second recommendation is to research a model in which all of the uncertainties also be taken into account on the resistance (strength of a dike) side, preferably in an integral manner. This model can function as a basis for an economical 'Cost-Benefit Optimization', such as shown by van Dantzig (1956). What can be taken from this research is that it is highly preferred to extent the current models used in flood risk safety assessment to gain more insight about the epistemic shortcomings and therefore points of attention.

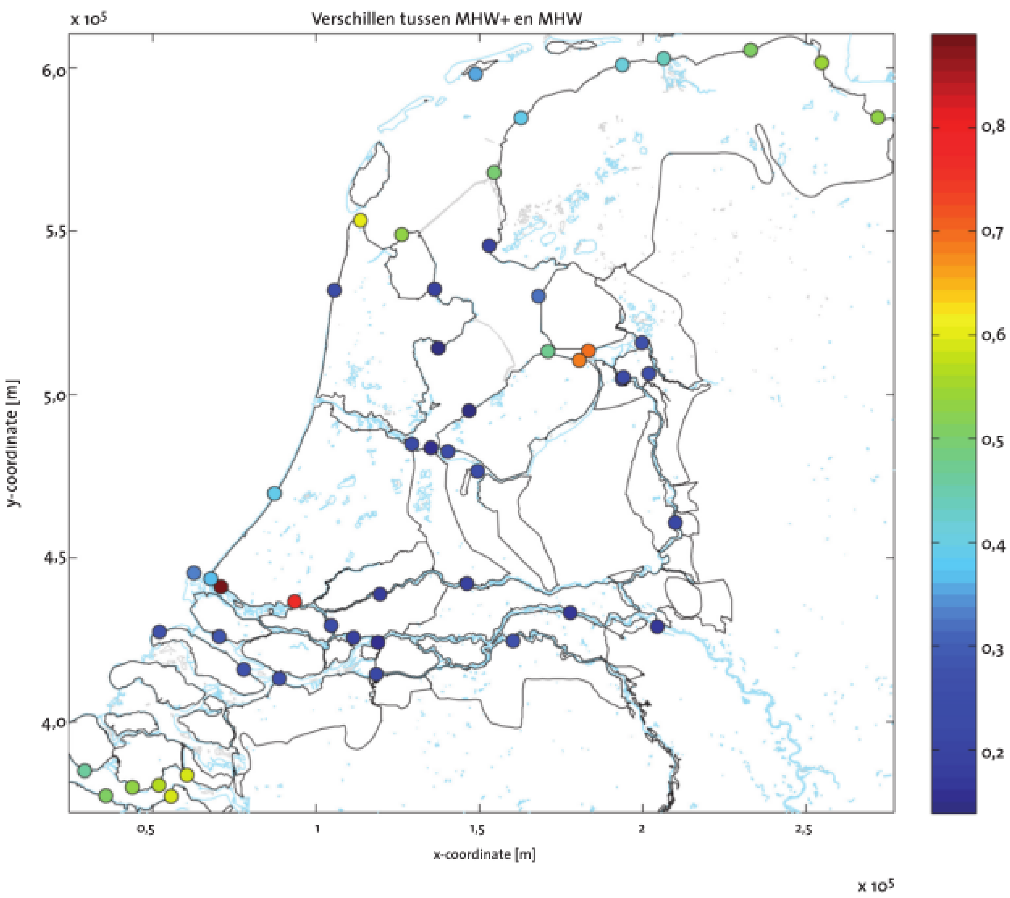


Figure 3.12: Expected water level increase when taking additional uncertainties into account. (Nicolai et al., 2011)

3.3.4. THE INCLUSION OF AN UNCERTAINTY IN A MODEL

Several ways of taking natural variability into account are shown by Geerse (2013). It is recommended to take account for the uncertainty of the discharge distribution of bifurcation of rivers and the waveform of the expected flood wave at Borgharen. One way of doing so is defining this parameters as a random variable during the input of a model.

In case of the bifurcation point near Pannerdensche Kop, Geerse (2013) proposes to model to influence of the uncertainty in the distribution of discharge to do a probabilistic calculation, when having a variable discharge distribution. As an example, the distribution is given a standard deviation of $230m^3/s$. A consequence of this, is that computational time is increased since more sessions have to be done. The stochastic was also introduced by Ogink (2006).

Another argument made was the modeling result of Delft3D at Pannerdensch Channel. This computations showed a reduced friction influence at the occurrence of low flood waves, resulting in a relatively higher discharge. This to emphasize the ambiguity of models. Three ways of implementing the uncertainty at bifurcation points are proposed. A fully probabilistic implementation at dike section- and dike ring-level, a implementation only on dike section level and a additional safety factor on ring level, or only adding safety factors on both levels. The first option is preferred, but is also hardest to implement, meaning current models need to be modified in detail. In all, it can be concluded that when uncertainties are considered to of have a effect on the normative load which cannot be neglected, implementation as such is difficult to comprehend and to apply.

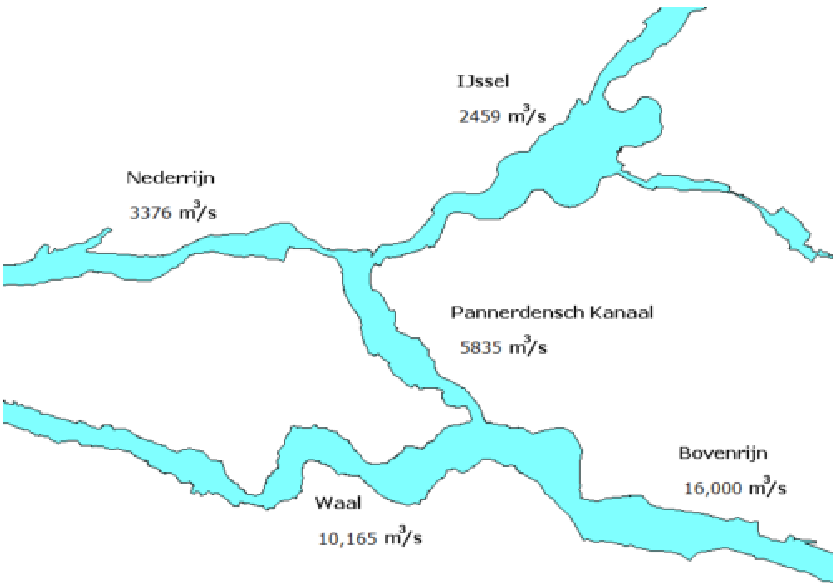


Figure 3.13: Bifurcation of Pannerdensche Kop as presented bij HBC2006. (Geerse, 2013)

Another study performed by Nicolai et al. (2014) also shows the preference to incorporate the effect of uncertainties in future modeling. On this account Nicolai et al. (2014) have identified uncertainties of interest, and showed their preference to include these uncertainties in the establishment of the currently in development Hydra-Ring model. Hydra-ring is a model which is aimed to be the dike assessment and design model as of 2017. It will also facilitate in the use of a flood probability. It is intended to be part of the legal assesment guidelines of 2017 (WTI2017).

The recommendation specifically states the incorporation of statistical uncertainties in the basic stochastic input values, such as river discharges, wind velocity and direction, water levels and wave parameters. as well as model uncertainties in the physical models WAQUA, IMPLIC, SWAN and Bretschneider. For instance, the uncertainty induced by the use of Bretschneider formulas for wave modeling, namely bottom simplifications, such as bottom roughness, river bifurcation, lateral flow, and fetch length, can be taken account for using a standard deviation in the local water depth. Effects of small lateral flows are considered to be neglect able using WAQUA, if desired large latereal flows, as for instance at the "Old IJssel", can be taken account for by adding a model uncertainty in the local water depth. One other interesting topic is the influence of capping of certain basic stochastic values considering to several climate change scenarios.

The study points out that to determine a representative total uncertainty for a certain area, expressed as a standard deviation [σ], all sub standard deviations should be combined to one. In this representative total uncertainty the assumption of statistically independent sources of uncertainties holds as showed in equation (3.10) for n sources of uncertainty:

$$\sigma_{rep,n} = \sqrt{\sigma_1^2 + \sigma_2^2 + \quad \dots \quad + \sigma_n^2} \tag{3.10}$$

In all, it can be seen that a switch from a deterministic approach to a probabilistic approach is quite clear. Implementing these advised measures is the next challenge. The report notes a consideration to either implement statistical uncertainties into the new Hydra-ring model or outside its framework. Implementation into the model will lead to a more uniform solution, although realization can be quite difficult. Model uncertainties can be addressed by improving the observed model, or by introducing probabilistic calculation methods.

As can be seen in the previous paragraphs, uncertainties can play a role in every level of a model. The significance of an uncertainty has to be properly judged when not addressing it. When it is decided that inclusion of an uncertainty is required, proper implementation is a challenge that arises. First, the uncertainty needs to be analyzed and understood, then quantified and implemented. This will mostly results in extra parameters in a model and/or more computations. Proper inclusion is equally of importance. Sometimes, the ideal inclusion is hard to realize when one considers currently used models. The expandability or modifiability can be limited in these models.

Several ways of inclusion of an uncertainty have been pointed out by Geerse (2013). As shown, a model can be extended by introducing probabilistic measures where this is not already being done. In turn, random variables that are described by a distribution are characteristic by its parameters, such as location, scale and shape, can be altered. In this fashion, the uncertainty of an input is quantified, and the effect it has on the output is addressed.

Uncertainty of a model is invoked by a series of simplifications is assumed when describing a natural process. As pointed out by Nicolai et al. (2011), inclusion of model uncertainty can result in a significance change in result.

As a last resort, when a result of a computation is ought to be inaccurate and improvement of such a computation is either complicated or not fully understood, a safety factors can be introduced. This method is widely applied in the construction sector where variation in quality of materials and execution is common, and the consequences are can be large.

3.3.5. INFLUENCE OF CLIMATE CHANGE

CLIMATE CHANGE

The Intergovernmental Panel on Climate Change (IPCC) is an international body for the assessment of climate change. Established by the United Nations Environment Programme (UNEP) and the World Meteorological Organization (WMO) in 1988 to provide the world with a clear scientific view on the current state of knowledge in climate change and its potential environmental and socio-economic impacts[1].

The ongoing IPCC study on climate change has released report in 2013 and 2014 which show estimates of significance to policy makers, especially concerning flood safety. The report of IPCC (2013) shows the estimated sea level rise of the coming century. They describe four scenarios (RCPs), ranging from slow to fast changes in sea level rise which are dependable on how much greenhouse gases are emitted. In the figure 3.14 below RCP2.6 and RCP8.5 are depicted. These two scenarios present to minimum and maximum expected

[1] More on this organization can be found at http://www.ipcc.ch/organization/organization.shtml

rate of change. Global mean sea level rise for 2081-2100 relative to 1986-2005 will likely be in the ranges of 0.26 meters to 0.55 meters for RCP2.6 and 0.45 meters to 0.82 meters for RCP8.5.

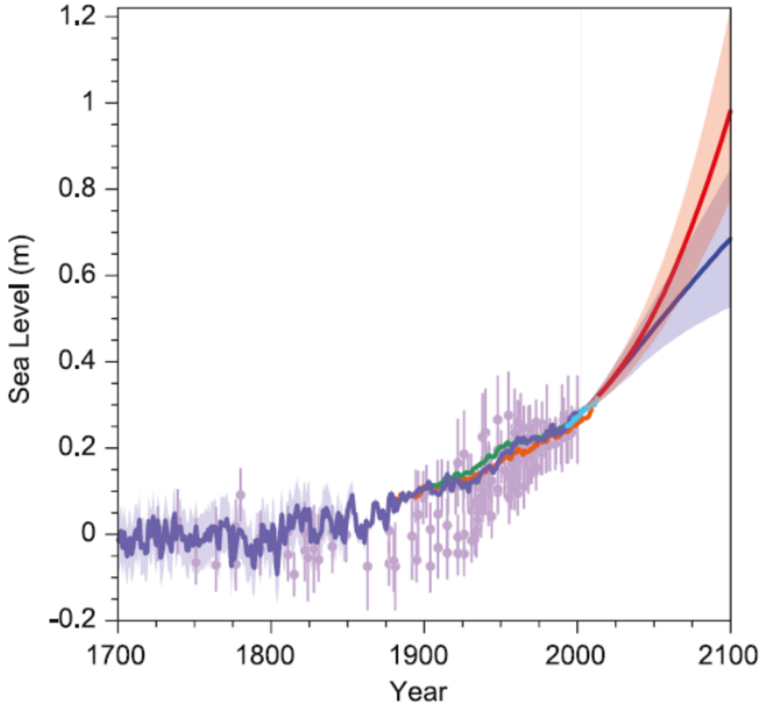


Figure 3.14: Expected global sea level rise, blue: RCP2.6 scenario, red: IPC8.5 scenario. (IPCC, 2013)

The most recent released report in 2014 continues to quantify the consequences of climate change. Implications of global warming and altering wind patterns are presented as key risk points. Expected are increased economic losses and people affected by flooding in river basins and coasts, driven by increasing urbanization, increasing sea levels, coastal erosion, and peak river discharges. Although the report points out that extreme precipitation, extreme temperatures and sea level rise are the main climate drivers in coming years, serious risk and need for potential adaptation is expected in the later half of the century.

Europe				
Key risk	**Adaptation issues & prospects**	**Climatic drivers**	**Timeframe**	**Risk & potential for adaptation**
				Very low / Medium / Very high
Increased economic losses and people affected by flooding in river basins and coasts, driven by increasing urbanization, increasing sea levels, coastal erosion, and peak river discharges (*high confidence*) [23.2-3, 23.7]	Adaptation can prevent most of the projected damages (*high confidence*). • Significant experience in hard flood-protection technologies and increasing experience with restoring wetlands • High costs for increasing flood protection • Potential barriers to implementation: demand for land in Europe and environmental and landscape concerns		Present Near term (2030–2040) Long term (2080–2100) 2°C 4°C	

Table 3.7: Table from (IPCC, 2014) showing implications of climate change for Europe

Whereas the IPCC draws attention and spreads awareness globally, the Koninklijk Nederlands Meteorologisch Instituut (KNMI) tries to deliver estimation about climate change in the Netherlands. Widely accepted effects of climate change to take account for are the KNMI'06 climate scenarios. The KNMI'06 climate scenarios reflect the changes in temperature , precipitation, wind and sea for a climatic period of 30 years . The scenarios for 2050 are therefore representative of the climate in the period between 2036 and 2065. Similar to the IPCC climate scenarios, four possible scenarios are defined, namely G G+ W and W+. However, there derivation is different. The KNMI has chosen two variables in which the future global climate can vary to wit change in global temperature and change in air flow patterns. G stands for moderate change in temperature (G=Gematigde) and W stands for rapid change in temperature (W=Warm). The "+" sign denotes if a change in air flow patterns is taken into account. A regularly used scenario in flood defense design is the KNMI'06

W+ 2050 scenario, since it has been enshrined in law to be used in the *Deltaprogramma*. In so taking into account the effects of climate change in a rapid increasing global temperature and changing air flow patterns representative to the period 2036-2065.

In follow-up to the IPCC'13 report, the KNMI has translated the global estimates to local conditions. Therefore, an update has taken place for the KNMI climate scenarios (KNMI, 2013). The IPCC calculations for greenhouse gas emissions, air pollution and the change of the land use form the basis for the KNMI'14 scenarios. Names of the scenarios have been slightly changed, since air flow pattern change is now denoted as L(ow) and H(igh). Expected effects by the report are an increase in temperature, an increase in extreme precipitation, increasing rate of sea level change and a small change in wind speed. notable differences to the KNMI'06 climate scenarios are an increase in temperature during winter, an increase of extreme precipitation intensity due to an increase of water vapor in the air and an overall increase of 5 centimeter in expected sea level rise (40 cm at 2050 and 100 cm at 2100) due to ice sheets on Greenland and Antarctica that are melting faster then previously expected. In figure 3.15 a comparisons is shown between the IPCC'13 climate scenarios and those of the KNMI'14. The figure shows that effects of climate change in the Netherlands are either more extreme or less severe, thus showing that the Dutch climate is highly sensitive to alterations.

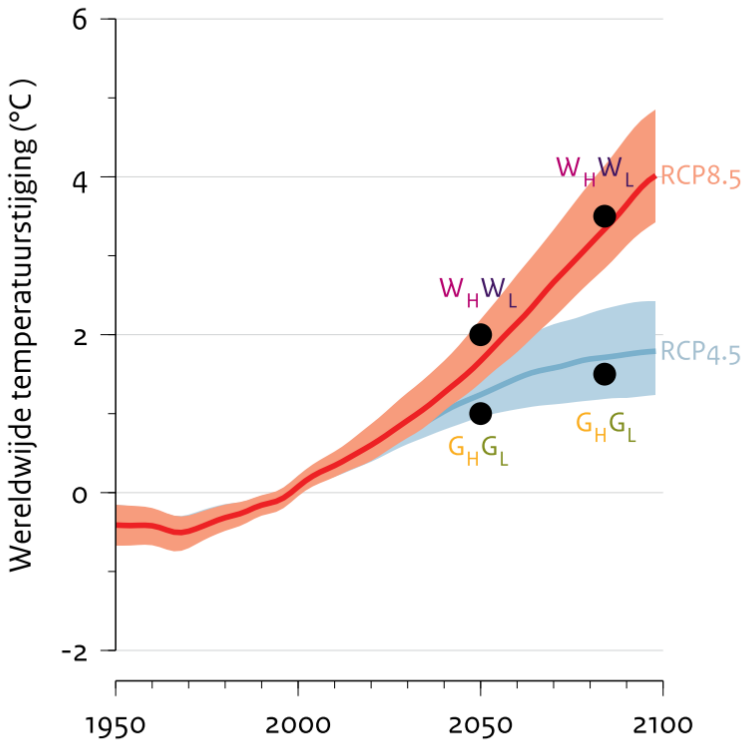


Figure 3.15: Comparison between KNMI'14 scenarios for the Netherlands and the IPCC report (KNMI, 2013)

TECHNICAL SUSTAINABILITY OF THE DUTCH WATER SYSTEM IN THE 21TH CENTURY

Looking into the future, Stijnen et al. (2014) have researched the sustainability of the Dutch polder system. Recent insights about climate change has raised the awareness of the necessity to protect the Dutch polders. Effects caused by climate change are expected to be sea level rise, an increase in river discharges and a change in precipitation patterns. Whereas, little more then a decade ago, the effects of climate change were heavily debated, nowadays the effects have been widely recognized by many as shown in the latest (IPCC, 2014) report. Therefore an increase of load on all flood defenses is expected. Also, due to urbanization in flood prone areas and economical growth, reassessment of flood protections strategy is called for.

The research points out that there seems to be no technical limitations in dike strengthening measures. Financially speaking, it is projected that the costs for the realization of sufficient dike reinforcements will be 1 billion per year for this century. A simplified relation is given for dike dimensions versus sea level rise, see figure 3.16. This approach shows a relationship between sea level rise and cost that is linear. In this study only technical en financial aspects have been accounted for. In reality, the process is more complex. For instance, spatial development and societal acceptance are significant contributors to a flood defense strategy.

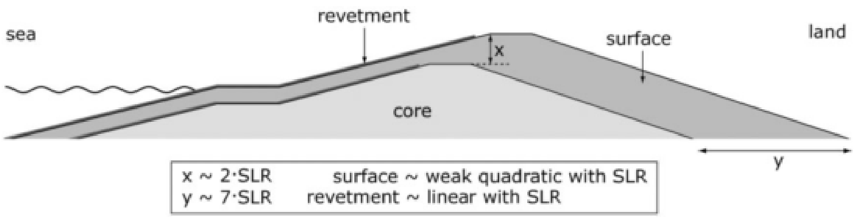


Figure 3.16: Relation between main dike parameters and increase in boundary conditions. (Stijnen et al., 2014)

UNCERTAIN VARIABLES IN AN OPTIMAL FLOOD DEFENSE DESIGN

Following the climate change debate of the last decade, Voortman and Vrijling (2004) showed the effect of an uncertain variable on a risk based design. Several scenarios concerning sea level rise were investigated. The basis for flood risk based optimization in is to balance to cost which needs to be invested in a flood defense and the attained reduction in flood probability. In the paper a momentary definition of risk-based cost-benefit analysis of flood defenses is used [van Gelder, 1999]. In this approach the optimal flooding probability is given by:

$$\min_{P_{flood}} C_{life}(P_{flood}) = I(P_{flood}) + \int_0^T P_{flood}\left(\frac{1+r_e+i}{1+r}\right)^t (c_b b_0 + c_d d_0)\,dt \tag{3.11}$$

Where:

I	=	Direct cost of flood defense
P_{flood}	=	Flooding probability
T	=	Reconstruction period
r_e	=	Rate of economic growth
i	=	Inflation
r	=	Interest rate
b_0	=	Lost benefits in case of flooding
d_0	=	Maximum possible damage
c_b	=	Damage factors lost benefits
c_d	=	Damage factor investments

In the analysis done by Voortman and Vrijling (2004), the effect of the sea level rise was researched using a method based on fragility curves. A fragility curves provided by the study show the probability of flooding for a given change in climate. Three strategies for construction of a dike considered were explicitly pointed out:

A A robust design which has to guarantee safety for a long time

B Suitable design for a short period, given fixed values for the design flooding probability and reinforcement period

C Suitable design for a short period, but optimized in terms of cost-benefit

The first option resulted in a initially safe design, however in the far future the design will lead to a highly unsafe situation. Also initial cost are enormous. Therefore, concerning a cost-benefit perspective this is not a preferred option. The latter to options show similarities in the fact that reinforcements to the flood defense are needed in the future. The researches points out that strategy C, with a optimized flood probability in terms of cost-benefit approach, shows the lowest life-cycle cost and life-cycle investment. The downside of this strategy is the increase of risk at certain moments in time. A graph of the strategy B and C is found in figure 3.17.

Given climate change and uncertainties to take into effect, and subsequently increasing the hydraulic load, will result in a higher price for flood defenses when it is desired to guarantee a certain safety standard in the future. A flood defense with a given geometry which can withstand loads as currently perceived might not suffice in the future with increasing loads. It must be noted that the expected cost of a flood defense in the future is difficult to estimate. The cost will be dependable on the materials used, quantiles, unit price, project cost, etc. These values are hard to predict. Developments in technology might have a strong influence on these factors. Subsequently, it is pointed out that the price of a flood defense will probably not follow inflation.

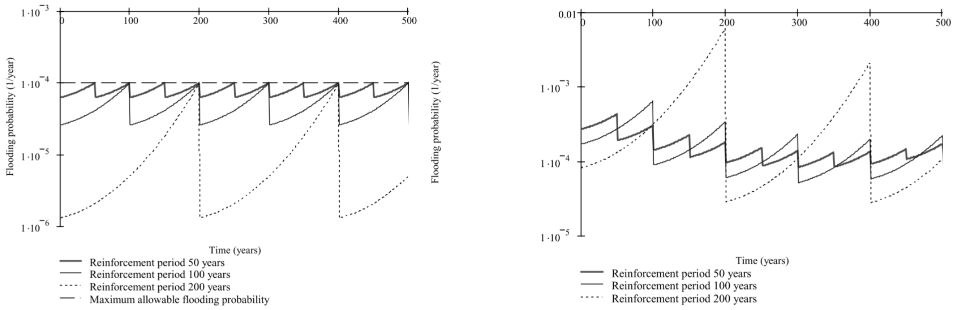


Figure 3.17: Left: Strategy B, right: strategy C (Voortman & Vrijling, 2004)

4

Problem exploration and identification

To facilitate the change in safety standard, the "Ontwerp Instrumentarium 2014" (OI2014) has been published to introduce dike designers and assessors to the new safety definition and dike trajectory approach. Still, the OI2014 raises more questions then than it answers. The guideline introduces new variables such as the contribution factor for various failure mechanisms, that need further clarification. The method explained in the guideline also enforces the designer to design for extreme return periods on section level which never have been encountered before.

In this chapter, the current considered dike stretches will be compared with rough dike designs using the new OI2014 guidelines. In so, the implications of the OI2014 are made clear and problems can be identified. The analysis will be concluded with an extended view on the distribution of the contribution factor for failure mechanisms.

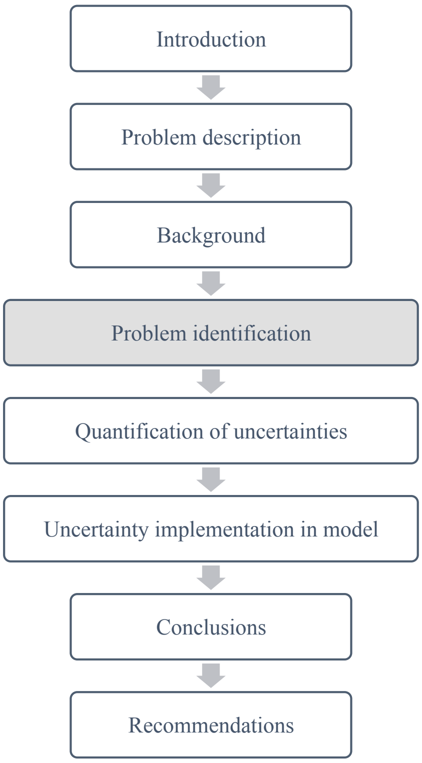


Figure 4.1: Structure of report, item "Problem identification".

4.1. CASE STUDY: FLEVOLAND

To accompany this research, a case study is carried out. The location of Flevoland has drawn the attention because this area is expected to be subdued to significant changes in terms of safety against flooding and a growing population. Exploratory research by VNK (2012) shows that current flood defences of Flevoland do not meet the future requirements. Also, with the city of Amsterdam expanding its boundaries to Almere and beyond, the population of Flevoland is predicted to grow with an explosive rate, as can be seen from figure E.2. This emphasizes the need for clarification.

4.1.1. LOCATION ANALYSIS OF DIKE TRAJECTORY 8-3

Dike ring 8 is identified by the artificial island of Flevoland. Shorelines which meet the IJssel Lake, Marker Lake and Ketel Lake have been appointed a category-a flood defence status, shorelines along the east side of Flevoland are of category-c.

CLASSIFICATION OF DIKE TRAJECTORY

The new trajectory approach divides the stretch of category-a dikes into several trajectories. Initially three stretches were defined, but as of Rijkswaterstaat (2014b) the stretch which meets the IJssel Lake and the Ketel Lake has been divided in two. This seemed like a logical thing to do since the hydraulic loading conditions for both stretches differs substantially. The four trajectories can be seen in figure 4.2. In this case study, dike trajectory 8_3 will be evaluated since it is expected by the FLORIS research (VNK, 2012) that these dikes are in need of improvement.

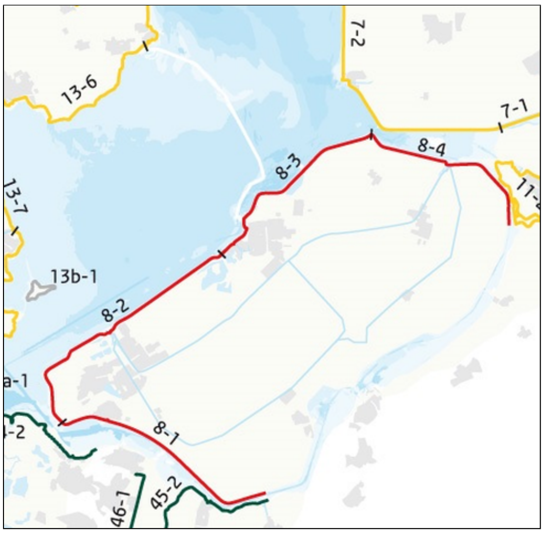


Figure 4.2: Dike trajectories of Flevoland (Rijkswaterstaat, 2014b).

What is noticeable is that the trajectory is not adapted to the water system the dike sections meet, but tailored to the consequences a breach might have for the hinterland. It is because of this that the 8_3 trajectory meets both the IJssel Lake and Marker Lake. A detailed description of this can be found in FLORIS research of Flevoland (VNK, 2012).

DIKES OF THE TRAJECTORY OF INTEREST

Dikes which are located in the proposed 8_3 trajectory are the "IJsselmeerdijk" and a part of the "Markermeerdijk". The dikes have been constructed in such a way that the failure mechanism of piping is hardly an issue. This is explained below.

The subsoil of Flevoland is made up out of sand, clay and peat. Deep lie the sand layers left behind by the Pleistocene epoch. During the Holocene epoch a peat layer was developed. This was due to a rise in sea level, and subsequently a rise in ground water level. The subsidence of this layer has led to a fairly impermeable layer of peat, and can be seen as an aquitard. The layers of sediments to be found in Flevoland are depicted in figure 4.3. From this figure can be seen that the layer of peat stretches the polder. Building in such conditions takes extra care, since such a layer is sensitive to subsidence. Below the layer of peat, a layer of sand can be found, which is permeable to water and therefore functions as an aquifer.

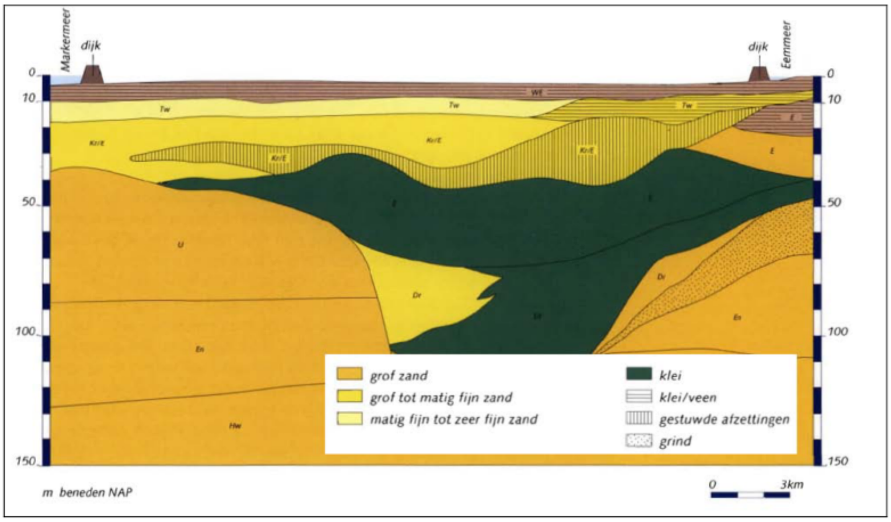


Figure 4.3: Dike ring 8 (VNK, 2012)

In current dike design along the coast of Flevoland, dikes have been constructed in such a way that the influence of the peat top layer are reduced to a minimum. The Holocene peat layer is excavated to such an extend that approximately 1 metre remains before reaching the Pleistocene sand layers. The 1 metre peat layer functions as a watertight layer between top and bottom sand layer. The advantage of a dike construction like this is that water pressures in the soil bodies is well known and effects of piping is hardly an issue due to a relative long seepage length.

SAFETY STANDARD OF FLEVOLAND

The dike ring approach Flevoland has appointed a 1/4,000 safety standard. The requirement states that the area can be protected against normative conditions which occur during a ones-in-four-thousand-year storm. All flood defences have been designed to cope with these conditions. The acting conditions have been enshrined by law and are presented in the HR2006 report.

The newly presented dike trajectories have been appointed with their own flooding probability requirement. The requirement for all four dike trajectories state a 1/30,000 year flood probability. This is the second highest safety stander, only special cases such as the Maeslant barrier have a higher standard of 1/100,000. The flooding probability is the result of a cost-benefit approach in which risk and fatalities are also considered. The expected positive economic prospect, population growth of Flevoland and relatively low investment costs have led to this high safety standard. Sometimes political choices also have an impact on the safety standard, since Maaskant and ter Horst (2014) show that a 1/10,000 year safety standard would also be suitable for the North-east part of Flevoland if one follows the elaborated values for the LIR and MKBA.

4.1.2. FAILURE PROBABILITY OF TRAJECTORY 8-3

In this chapter the trajectories 8_3 and 8_4 will be evaluated in which the flooding probability will be derived. For this it is assumed that the failure probability of a stretch of successive dike sections can be seen as the flooding probability of a dike trajectory. The trajectory extends from the "Vossemeerdijk" up to halfway through the "Oostvaardersdijk" where the internal Knardijk splits the country of Flevoland in two. This trajectory includes one hydraulic structure, namely the "inlaatduiker Ketelhaven VNK.08.03.001" situated in the "Keteldijk".

BOUNDARY CONDITIONS

TMR2006 DATABASE

The used PC-Ring database contains information about loading conditions and many random variables affiliated with inherent and model uncertainties. The loading conditions used are from the TMR2006 database. The TMR2006 database contains hydraulic loading conditions which can be used in the PC-Ring model. The TMR2006 originates from the HR2006 database which is used for Hydra models. Most notable difference is that the 16 wind direction database of HR2006 has been transformed to a 12 wind directions database for TMR2006. The transformation which has been done by using an interpolation method was needed to make the data suitable for the PC-Ring model.

FLORIS DATABASE

Furthermore, the PC-Ring database contains information about dike profile characteristics, parameters to take account for natural variability and several model parameters. Most of this characteristics come from the FLORIS research (VNK, 2012). Since this is the best and most comprehensive database currently available, this has been chosen. The lay out of dike sections can be seen

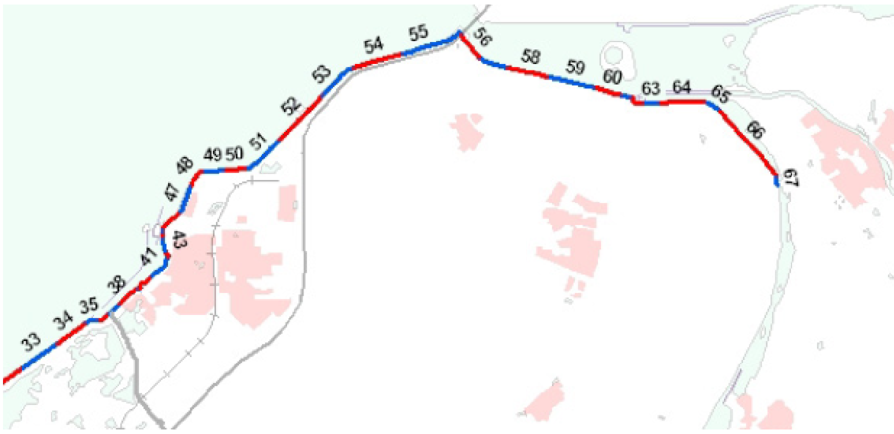


Figure 4.4: Dike sections of trajectory 8_3 (VNK, 2012)

RELEVANT FAILURE MECHANISMS

Following the FLORIS (VNK, 2012) research of Flevoland, not all failure mechanisms of flood defences are considered. Their analysis points out that only several failure mechanisms are of significance. This is because the location specific conditions do not allow for certain mechanisms to occur. By not computing failure mechanisms that are not considered to be of influence, the total computation time can be reduced. For all dike sections overflow and overtopping are accounted for. In several cases macro stability and damage to revetments are considered. The failure mechanisms are approached in a probabilistic manner, this means that the load and strength parameters are addressed as stochastic variables.

RESULTS FOR THE CURRENT FLOODING PROBABILITY OF NORTH-EAST FLEVOLAND

A FORM analysis shows that the combined trajectories of 8_3 and 8_4 have a flooding probability of 1/900 per year. This figure applies to the dike trajectories as if they were one.

In figure 4.5 a histogram is presented of the reliability indexes of each dike section. A lower reliability index corresponds with a higher probability of failure. What can be taken from this figure is that the current failure probability can vary for each dike section. Especially dike section 53 has a significant contribution to the

Table 4.1: Failure probability of the current dikes of Flevoland.

Trajectory	β	P_f	Return period
8_3 & 8_4	3.05	1.14E-04	900

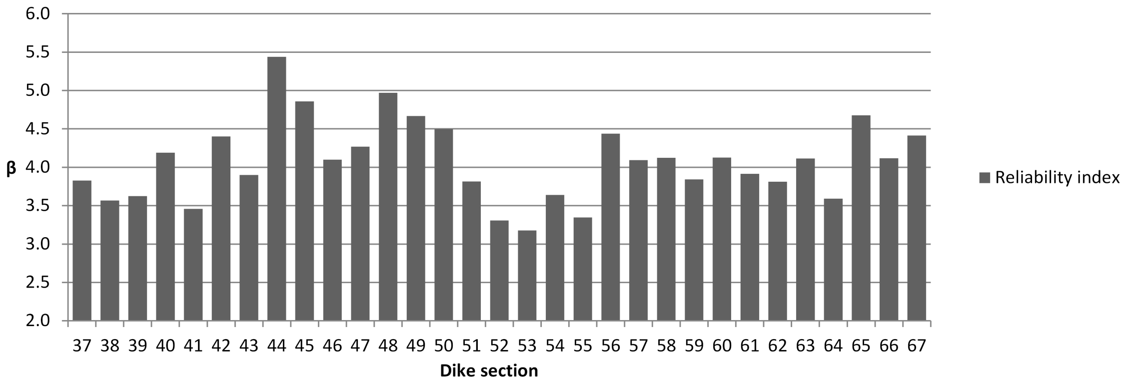


Figure 4.5: Failure probability of dike trajectory 8_3, lower β value = larger failure probability

overall flooding probability.

The untrained mind might observe a fairly constant dike height lengthwise, and therefore would expect a same level of failure probability for a dike stretch. However, this is hardly the case since many other factors can have an influence on the failure of a specific dike section. For example, the angle of incidence of the impacting waves can be different for each dike section due to the way a dike is situated. Also, the bathymetry of the IJssel Lake and Ketel Lake has its influence on the expected hydraulic load since waves height is limited by the water depth. Table 4.2 below shows the ten dike sections which show the highest probability of failure and, therefore, have the largest contribution to the overall failure probability.

Table 4.2: Dike section with the highest probability of failure

no.	Dike section	Return period	Dominant failure mechanism
53	IJS-03.22750.24750	1,300	overtopping/overflow
52	IJS-04.24750.27550	2,100	overtopping/overflow
55	IJS-01.17440.20400	2,400	overtopping/overflow and damage to revetment
41	O-03.01260.02470	3,700	overtopping/overflow
38	O-06.03450.04530	5,500	overtopping/overflow
64	V-04.05130.07300	6,100	overtopping/overflow
39	O-05.03310.03450	6,900	overtopping/overflow
54	IJS-02.20400.22750	7,300	overtopping/overflow
62	V-06.08020.08810	14,000	overtopping/overflow
51	IJS-05.27550.29550	15,000	overtopping/overflow

The computed flooding probability is the result of a combination of the failure mechanisms and dike sections considered. The contribution of each failure mechanism is shown in the table 4.3. Overtopping and overflow is clearly the most dominant failure mechanism. The figure for revetment damage is mainly governed by dike section 55, IJ-01.17440.2040, which shows a return period of failure of 4,400 years for this specific mechanism. Other failure mechanisms contribute very little to the overall failure probability due to their low probability.
One of the benefits of a FORM computation is that the influence factors can be extracted in the process. These influence factors can show the level of influence one parameter has on the result. In table 4.4 the ten

Table 4.3: Marginal contribution of failure mechanisms for trajectory 8_3 & 8_4

Mechanism	$\boldsymbol{\beta}$	$\boldsymbol{P_f}$
Overtopping Overflow	3.056	1.12E-03
Macro stability	5.055	2.16E-07
Revetment damage or erosion	3.502	2.31E-04
Failure of structure	7.357	9.48E-14

most influential parameters for overtopping for the whole trajectory can be seen. The influence of wind is undeniable. Both wind speed and wind direction have the largest influence on the elaborated flooding probability. Other factors of significance are the occurring IJssel Lake level and factors related to the definition of overtopping.

Table 4.4: Influence coefficients for the overtopping.

Parameter for overtopping	**Influence coefficient**
Wind speed Schiphol/Deelen	-0.759
Wind direction	-0.523
IJssel Lake level	-0.245
Factor Q_b f_b	0.193
Model factor overtopping discharge $m_q o$	-0.154
Model factor critical overtopping discharge $m_q c$	0.120
Factor Q_n f_n	0.075
Marker Lake level	-0.075
Storm duration t_s	-0.059
Roughness inner slope k	0.016

Comparing the computed flooding probability from section 4.1.4 with the new safety standard one comes to a staggering conclusion. Whereas in the next inspection round the 8_3 and 8_4 trajectory is intended to be assessed with the new requirement of a 1/30,000 years flooding probability, the current trajectory only suffices up to a flooding probability of 1/900 years. Several reinforcements proposed by VNK (2012) do have an improving effect, however the projected safety standard of 1/30,000 isn't accomplished by far.

4.1.3. VERIFICATION OF THE CURRENT "IJSSELMEERDIJK"

The conclusion of the previous section section 4.1.2 raises some confusing about the current dike. Since the dike has passed the last inspection round, one might assume that the dike is compliance with the (current) safety standard of an 1/4,000 exceedance probability. In this section the current dike will be assessed using the prevailing assessment model, Hydra-Zoet. The Hydra-Zoet model has been used to design and asses dikes in the Netherlands. The model allows to elaborate a crest height for a given return period.

Dike section 53 showed a high probability of failure in the former analysis, for this reason the associated dike location F260 will be evaluated.This location can be seen in figure 4.6. According to VNK (2012) the current crest height is 4.79 m+NAP.

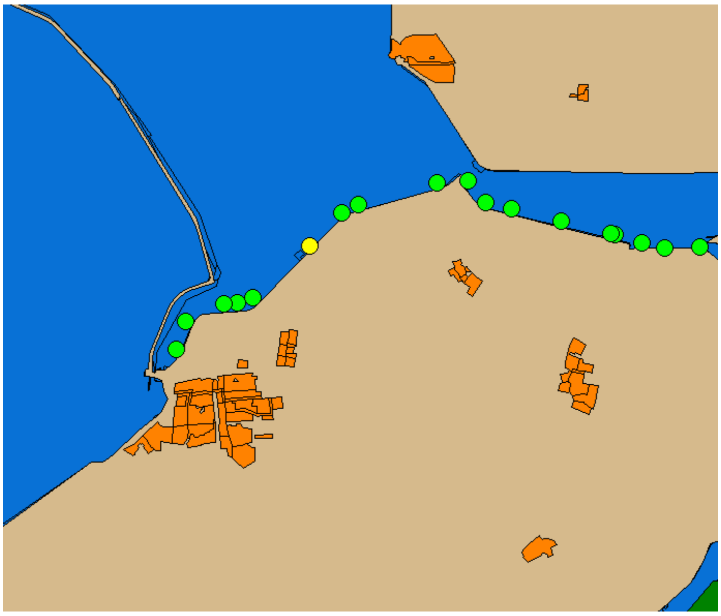


Figure 4.6: Hydra-Zoet F260 IJsselmeerdijk location

RESULTS OF THE HYDRA-ZOET MODEL

The following results have been computed using the Hydra-Zoet model. A HR2006 database can be implemented in the model. This database combines statistical information and physical information such that normative conditions can be computed. A critical overtopping discharge of 5 [l/s/m] has been set as normative following research by van der Meer (2012). The required crest height is visualized by figure 4.7 where it is plotted against its associated return period. A detailed presentation of the figures used to construct the plot are to be found in table 4.5.

Table 4.5: Required crest height due to overtopping for the "IJsselmeerdijk".

Return Period [1/yr]	**Required crest height [m+NAP]**
10	2.54
100	3.20
1,000	4.08
1,250	4.17
2,000	4.38
4,000	4.69
10,000	5.12
20,000	5.45

The figure 4.7 shows that the required crest height for 1/4,000 per year storm conditions should be 4.69 m+NAP. Thus, it is verified that the current "IJsselmeerdijk" has been in accordance with the prevailing safety

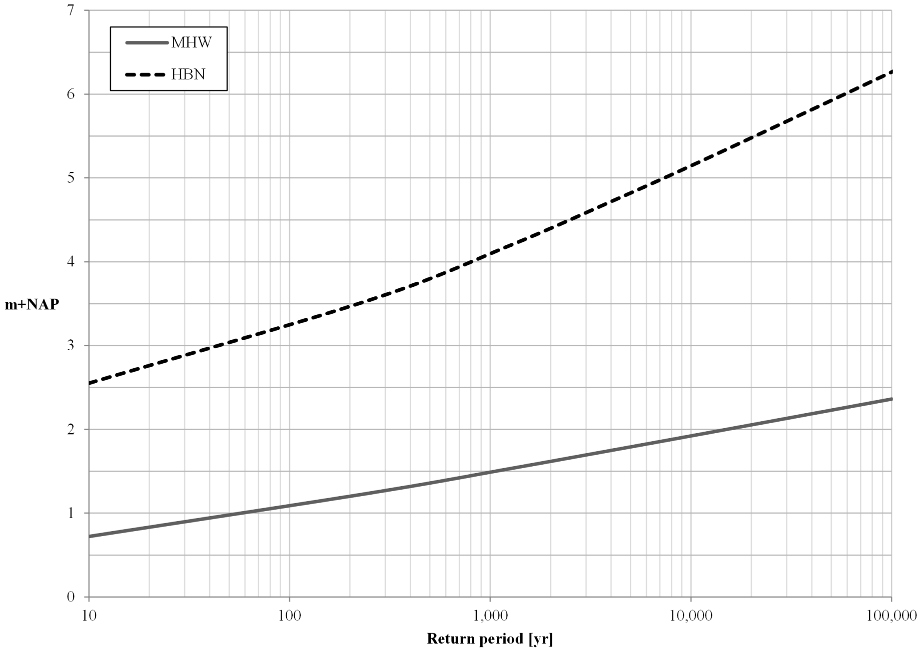


Figure 4.7: Hydra-Zoet F260 IJsselmeerdijk HR2006 WTI-modus

standard. This indicates that the newly proposed safety standard in the form of a flooding probability is not just an alternative representation of the required safety level, but also imposes a significant needed increment.

Another point of interest in figure 4.7 is that a gentle bend can be observed in the lower left region of the graph. A possible attribution of this effect can be the influence of the berm. As conditions get more severe, the influence of the berm reduces. This may give an incentive to reconsider the optimal dike profile during normative conditions.

4.1.4. CONSEQUENCES OF THE DESIGN RULES OF THE OI2014 FOR DIKE TRAJECTORY 8-3

In 2014 the document "Ontwerpinstrumentarium 2014" (OI2014) was released by Rijkswaterstaat (2014c). This document is intended to familiarize with the new safety standard of flooding probability. Transformation methods are given to apply the new safety standard in prevailing methods. As of now the prevailing model to be used for designing or assessing a dike is Hydra-Zoet. Implication as such will be discussed in section 4.1.4. Since Hydra-Zoet is not suited for the elaboration of a dike trajectory in section 4.1.5 the dike trajectory will be evaluated incorporating design choices of the OI2014.

In the following section an example will be shown of how the OI2014 and Hydra-Zoet would be applied for the shore line of Flevoland. A dike location of the "IJsselmeerdijk" is chosen for evaluation. Since FLORIS (VNK, 2012) states that overtopping is the only relevant failure mechanism for this dike location on this trajectory, a crest height will be elaborated using the design probability computed from the equation (4.1). Lake level increase due to climate change is not accounted for in this analysis. Also, the Rijkswaterstaat (2014b) notes that the lake level will be maintained up to 2050.

$$P_{required} = \frac{P_{standard} \cdot \omega}{N} \tag{4.1}$$

As the formula shows, several parameters need to be defined. These will be explained below:

FAILURE MECHANISM CONTRIBUTION FACTOR [ω]

Rijkswaterstaat (2014c, Appendix A) has given a table which provides contribution factors for failure mechanisms. For this example only overtopping is considered, since this is the only failure mechanism which is accounted for in the prevailing method of Hydra-Zoet. Therefore, the contribution factor is $\omega = 0.24$.

DESIGN PROBABILITY

The new safety standard defines a flood probability of $1/30,000$ per year for dike trajectory 8_3. This level denotes a 'average flood probability', a level that represents a value which is higher then the maximum allowed flood probability and lower then the design probability. The average flood probability is related to the average damage between two successive investments. From Rijkswaterstaat (2014c) follows that the design failure probability is twice the 'average flood probability'. The design failure probability needs to be met at the end for the design's lifetime.

$$P_{design} = 2 \cdot P_{standard} = 2 \cdot \frac{1}{30,000} = \frac{1}{15,000} [per\, year] \tag{4.2}$$

LENGTH EFFECT FACTOR [N]

According to Rijkswaterstaat (2014c, Appendix B) the length effect factor N should be 3 in for dike trajectory 8_3, the shoreline of Flevoland.

The table 4.6 shows a summary of the relevant parameters.

Table 4.6: Summary of used parameters

Parameter	Value
$P_{standard}$	1/30,000
P_{design}	1/15,000
ω	0.24
N	3

Now that all parameters have been defined of equation (4.1), a target failure probability can be computed to which the crest height needs to be designed:

$$P_{required} = \frac{P_{design} \cdot \omega}{N} = \frac{\frac{1}{15{,}000} \cdot 0.24}{3} = \frac{1}{187{,}500} \tag{4.3}$$

To be able to compute at extreme return periods, a special computation has to be done with the Hydra-Zoet model. The conditions associated with return periods like those of the 1/187,500 per year are not part of 'the physical database'. For these kind of computation the model is able to give an estimation by extrapolating the values which are within range of 'the physical database'. The extrapolated values will be shown and analysed in the next section.

RESULT

The figure below shows the results of the Hydra-Zoet model without a robustness surplus. These computation use the HR2006 database. Lake level rise due to climate change is not accounted for in this computation.

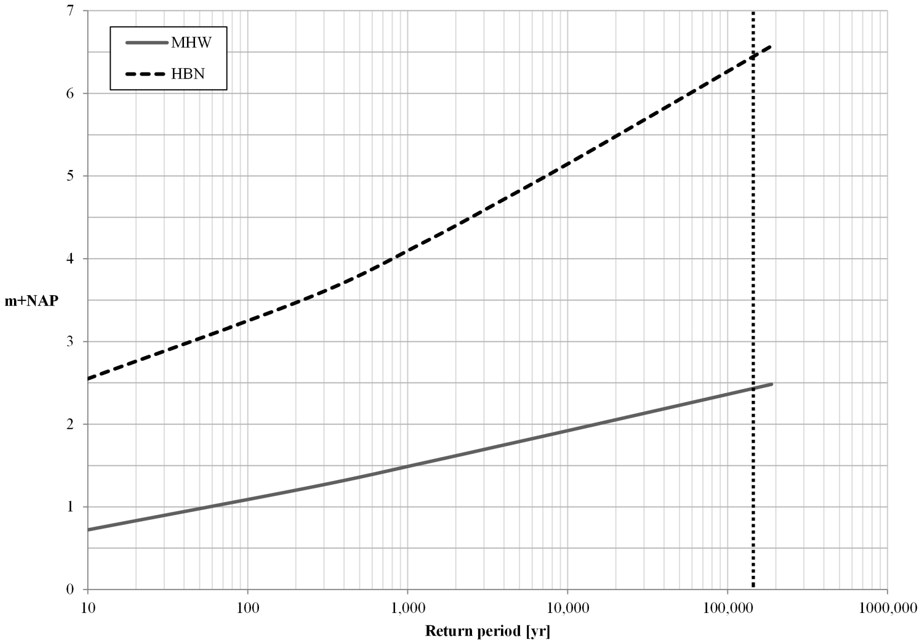


Figure 4.8: Hydra-Zoet F260 IJsselmeerdijk, water level versus required crest height

From figure 4.8 can be seen that the relationship between return period and hydraulic load is log-linear. Both hydraulic loading conditions (MHW and overtopping) are fairly straight lines threw the plot. Again, a bend can be seen in the overtopping loading which is probably due to the influence of the dike's berm. This figure shows clearly that the rate of change is far more significant for overtopping then for MHW. Therefore the effect it has on the crest height will be amplified when conditions go into the extreme. In the table 4.7 and overview can be found what the hydraulic loading conditions would be at a return period of 1/187,500. It is vastly concluded that for these conditions the current 4.8 metre high "IJsselmeerdijk" would not suffice.

Table 4.7: Hydra-Zoet HR2006

	Return period [yrs]	**Hydraulic load [m+NAP]**
Water level	187,500	2.48
Required crest height	187,500	6.57

4.1.5. FAILURE PROBABILITY OF TRAJECTORY 8-3 CONSIDERING DESIGN CONDITIONS ACCORDING TO THE OI2014

For the following example PC-Ring is used to elaborate a required crest height for the dike trajectory 8_3. As a requirement the reliability index is derived from the the design failure probability as proposed by OI2014 (Rijkswaterstaat, 2014a) for overtopping. As for now, the standard settings of PC-Ring are used. This means that the CIRIA model is considered for the determination of the required crest height.

$$P_f = \Phi(-\beta) \rightarrow \beta = -\Phi(P_f) \tag{4.4}$$

$$\beta = -\Phi(\frac{1}{187{,}500}) = 4.403 \tag{4.5}$$

In figure 4.9 the current and the required crest heights are presented for each individual dike section. Following the reasoning of Rijkswaterstaat (2014c) almost every dike section needs reinforcement to comply with the new design rules. Dike sections 51 to 55 are severely in need of reinforcement. Looking at their location, it can be understood that they are subject to a relativity large load due to a long fetch length which spans all across the IJssel Lake. Moving southwards along the dike trajectory, the orientation of the dike changes, fetches are reduced, and subsequently required crest heights are smaller due to a smaller hydraulic load.

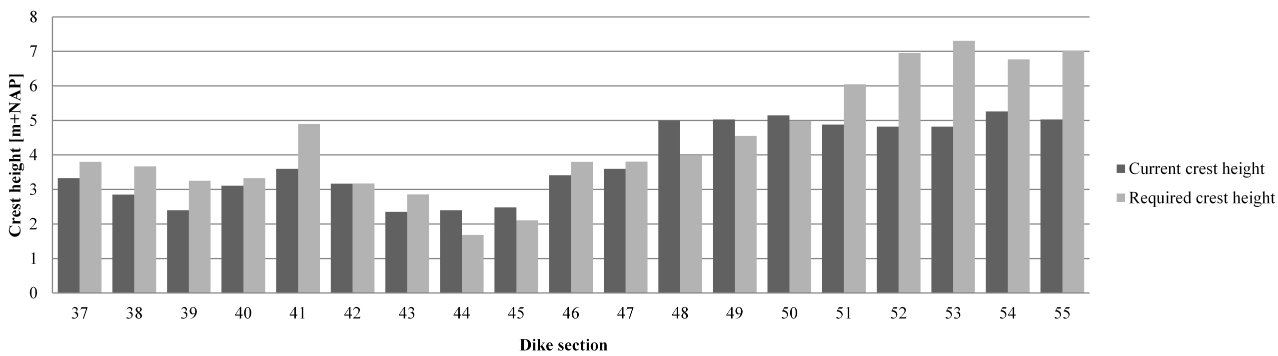


Figure 4.9: Required crest height following $P_{fd} = 1/187,000$ for overtopping per section

The table 4.8 shows the failure probability of the dike trajectory 8_3. When all relevant failure mechanisms according to FLORIS are evaluated during the computation, a failure probability of 1/4,211 years is computed. This figure is mainly driven due to the expected failure of dike section 55 where erosion plays an important role. Failure due to macro stability does not seem to have any significance along the whole trajectory. When the assumption is made that the erosion problem at dike section 55 will be addressed accordingly, we can take erosion and macro stability out of the equation. What is left is an computed failure probability solely dependable on overtopping. The attained figure is in the order of the desired failure probability of 1/30,000.

Table 4.8: Failure probability of dike trajectory 8_3

Considered mechanisms	β	Pf	cumulative Pf	Return period
Overtopping, macro stability and erosion	3.49	2.37E-04	2.63E-04	4,211
Overtopping	3.98	3.42E-05	3.42E-05	29,202

When one look at the achieved failure of the dike trajectory, it seems that the proposed OI2014 behaves quite well with its suggested parameters. However, a highly variable crest height as can be seen in figure 4.9 is highly unfavourable. Such a design for a trajectory looks peculiar and and will be politically difficult to justify. And as section 4.1.4 suggest, an optimisation of the profile may lead to lower needed crest heights.

The used parameter raise a lot of questions. In this example a contribution factor of 0.24 is used for overtopping, which implies the remaining 0.76 is to be used for other failure mechanisms. This seems harsh since overtopping is the only relevant failure mechanism according to VNK (2012) (though it seems unwise to completely neglect other failure mechanisms). Also, M. Duits, van Haaren, Havinga, and Vuik (2014) argue that a length effect factor $[N]$ of 3 seems too conservative considering the length of the trajectory, and might be reduced to 2. Alteration like these have a great influence on the target failure probability.

4.1.6. IMPACT OF AN ALTERNATIVE CONTRIBUTION FACTOR DISTRIBUTION AND LENGTH EFFECT FACTOR

The Rijkswaterstaat (2014a) states the contribution factor to be the maximum allowable failure probability as a percentages of the flood probability. In this way, the combination of failure probabilities of each corresponding failure mechanism will result in the overall failure probability. This overall failure probability.

The OI2014 presence a table with contribution factors which is based on a national average according to Rijkswaterstaat (2014a). In the specific case of the coast of Flevoland, applicability of this table can be called into question. On national average, 24% has been assigned for the piping mechanism, whereas the coast of Flevoland is hardly affected by any piping scenarios due to the construction of the dike. Namely, the dike has been constructed on top of a trench of sand as highlighted on page 39 and is explained by VNK (2012).

The FLORIS research also demonstrates that failure mechanism prominence can differ significantly throughout different water systems in the Netherlands. This is reasserted by the PC-Ring computation shown in table 4.3 on page 42 and table 4.12 on page 53.

SPECIFICATION OF FAILURE MECHANISM CONTRIBUTION FACTOR ω

Following the findings of VNK (2012), the PC-Ring computation on page 42 shows that three failure mechanisms are important for dike trajectory 8-3. Namely, overtopping & overflow, macro stability and revetment damage or erosion. Structure failure will be neglected in this example.

For the alternative take on the distribution of the contribution factors, other not apparent failure mechanisms will be dropped van the equation. The distribution will be changed scaled in such a way that the relative differences between failure mechanisms stays the same. The result of this can be seen in table 4.9.

Table 4.9: Alternative contribution factor ω along dike trajectory $8-3$.

Type	Failure mechanism	OI2014	presence	Alternative
Dike	overtopping	0.24	✓	0.63
	piping	0.24	-	0.00
	macro stability	0.04	✓	0.11
	erosion	0.10	✓	0.26
Structure	closure failure	0.04	-	0.00
	piping	0.02	-	0.00
	structural failure	0.02	-	0.00
Dune	erosion	0.00/0.10	-	0.00
	other	0.30/0.20	-	0.00
	total	**1.00**		**1.00**

The newly retained contribution factor distribution, subsequently, leads to a different design probability as is shown below in equation (4.6):

$$P_{required} = \frac{P_{design} \cdot \omega}{N} = \frac{\frac{1}{15{,}000} \cdot 0.63}{3} = \frac{1}{71{,}500} \tag{4.6}$$

By doing a Hydra-Zoet computation, the implications of this is a reduction in HBN-level of approximately 35 centimetres. This is visualised as the red arrow in figure 4.10, moving from 187,500 to 71,500 [1/yrs].

LENGTH EFFECT FACTOR N

As of the Rijkswaterstaat (2014b), a split has been introduced in trajectory for the North of Flevoland. This means that the former 8-3 trajectory has been divided into a new 8-3 and 8-4 trajectory. The reason for this was that the trajectory was relatively long, and moreover, the water loading was not uniform along the trajectory. Whereas the North-Western side is part of the IJssel Lake system, and the North-Eastern side is part of the Ketel Lake system. Considering these reasons, the choice is obvious.

With a reduction in trajectory length, the possibility arises to reduce the length effect factor N from 3 to 2. Justification of this needs to be further investigated. However, the implications of doing so alter the design probability accordingly:

$$P_{required} = \frac{P_{design} \cdot \omega}{N} = \frac{15,000 \cdot 0.63}{2} = 47,500 \tag{4.7}$$

Equation (4.14) shows that the design probability changes from 187,500 to 47,500 [1/yrs] when an alternative contribution factor and length effect factor are incorporated. This leads to a reduction of approximately 70 cm on the HBN. This is visualised as the blue arrow in figure 4.10.

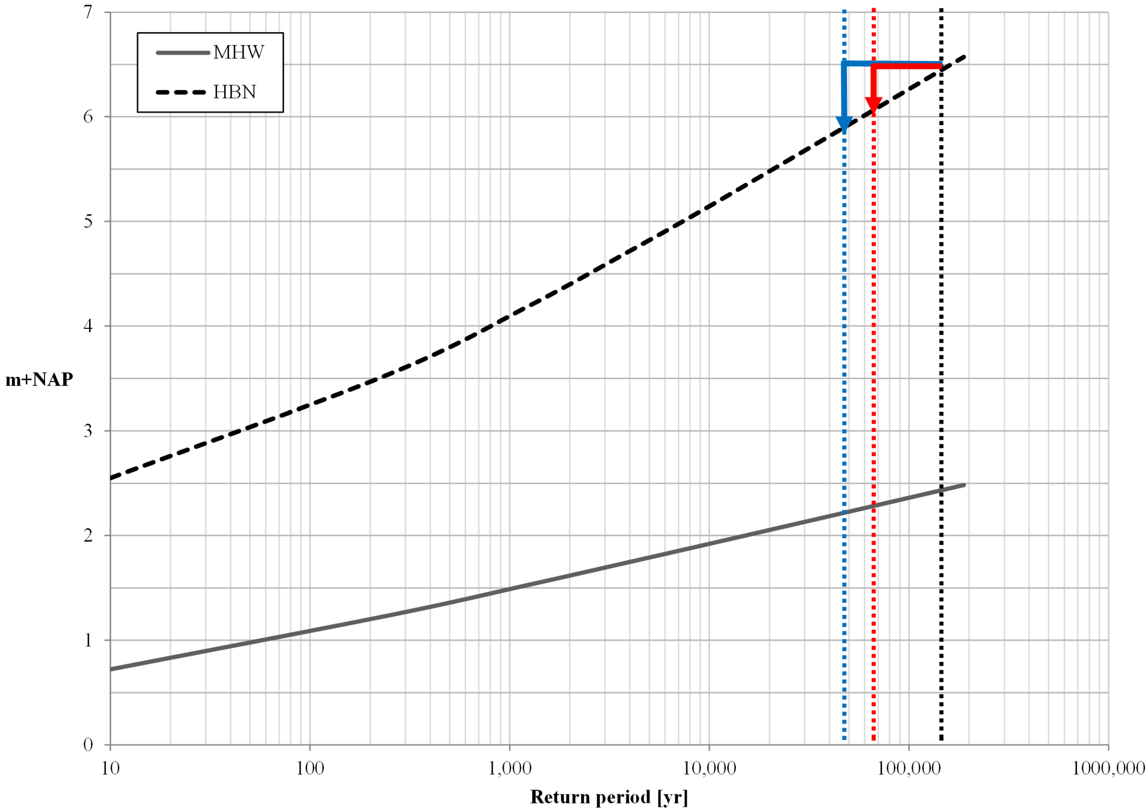


Figure 4.10: Result of alternative contribution factor. Return period red: 71,500 [yrs], Return period blue: 47,500 [yrs].

The decimal height for return periods larger then 10,000 [yrs] is 1,20 metres. This is relatively compared to other water systems. Therefore, the HBN-level is sensitive to parameter changes.

4.2. CASE STUDY: THE RIVER WAAL

Another dike stretch that will be studied is the dike trajectory 43-6 along the north side of the Waal. Because the Waal connects the harbour of Rotterdam with large parts of Europe, it is an important shipping industry route. Also, in the recent decade, deepening of 'kribben' are investigated to reduce water levels during high river discharges. This project is commonly referred to as the "Ruimte voor de Rivier" project.

4.2.1. LOCATION ANALYSIS OF DIKE TRAJECTORY 43-6.

The trajectory 43-6 is part of the formerly known dike ring 43. The area is maintained by the water board 'Rivierenland'. Relatively large cities Tiel, Zaltbommel and Gorinchem are located along the trajectory as well. The upstream Rhine river bifurcates into the "Pannerdensch Kanaal" and the Waal. It takes account for approximately 65% of the Rhine's river discharge, which comes down to about 15,000 m^3/s.

DIKE TRAJECTORY ANALYSIS

According to VNK (2014) the trajectory 43-6 is made up out of 47 dike sections. The cumulative length of all is about 80 kilometres. Because this is a relatively long stretch compared to other trajectories, it is investigated if the trajectory should be split into two. This might also be preferable since the part of Gorinchem-Waardenburg dike stretch is currently being investigated for improvement as part of the 'Hoogwaterbeschermingsprogramma' (HWBP) project.

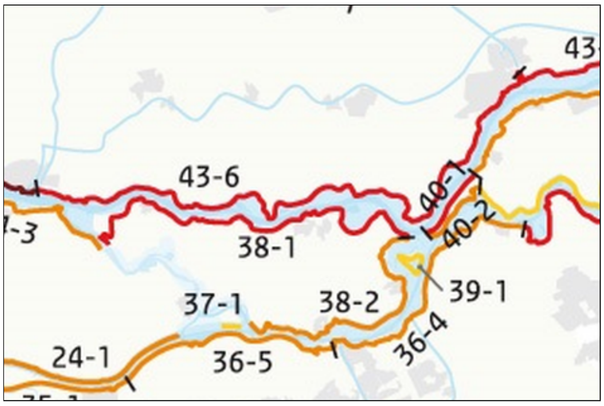


Figure 4.11: Trajectory 43-6 Gorinchem-Tiel (Rijkswaterstaat, 2014b).

Kribben are used to maintain a sufficiently deep channel for vessels to navigate. The kribben are part of a floodplain which can inundate during high river discharges which usually occur during winter when meltwater fills the river. The river dikes along the Waal have a slope of about 1-to-3. An aerial view of a river section can be seen in figure 4.12.

In general, the soil composition of river dikes is a combination of sand, peat and clay. Therefore, piping is a recurring issue. According to VNK (2014) piping and overtopping are the most important failure mechanisms to reckon with.

SAFETY STANDARD OF FLEVOLAND

The current safety standard for dike ring 43 is an exceedance probability of 1/1,250 per year. The requirement states that the area can be protected against normative loading conditions which occur during a one-in-twelve-hundred year storm. All flood defences have been designed to cope with these conditions. The acting conditions have been enshrined by law and are presented in the HR2006 report.

The proposed new safety standard is a flooding probability of 1/30,000 per year Rijkswaterstaat (2014b). The proposed flooding probability safety level is the result of a cost-benefit approach in which potential risk and fatalities are also considered.

Figure 4.12: Floodplains and dikes along the Waal during deepening of 'kribben' by Van Oord (2012).

4.2.2. FLOODING PROBABILITY OF DIKE TRAJECTORY 43-6

A FORM computation is carried out using the PC-Ring model. In this computation all relevant failure mechanisms, according to VNK (2014), are considered. This results in a current failure probability of the dike trajectory 43-6 of 1/100 per year. Comparing this figure with the new provisional safety standard of 1/30,000 per year, one might think this trajectory is due for improvement. Figure 4.13 shows a histogram with reliability indexes for each dike section in trajectory 43-6. Low reliability indexes indicate a vulnerable dike section.

Table 4.10: Failure probabilty of trajectory 43-6.

Trajectory	β	P_f	Return period
43-6	2.33	0.98E-04	100

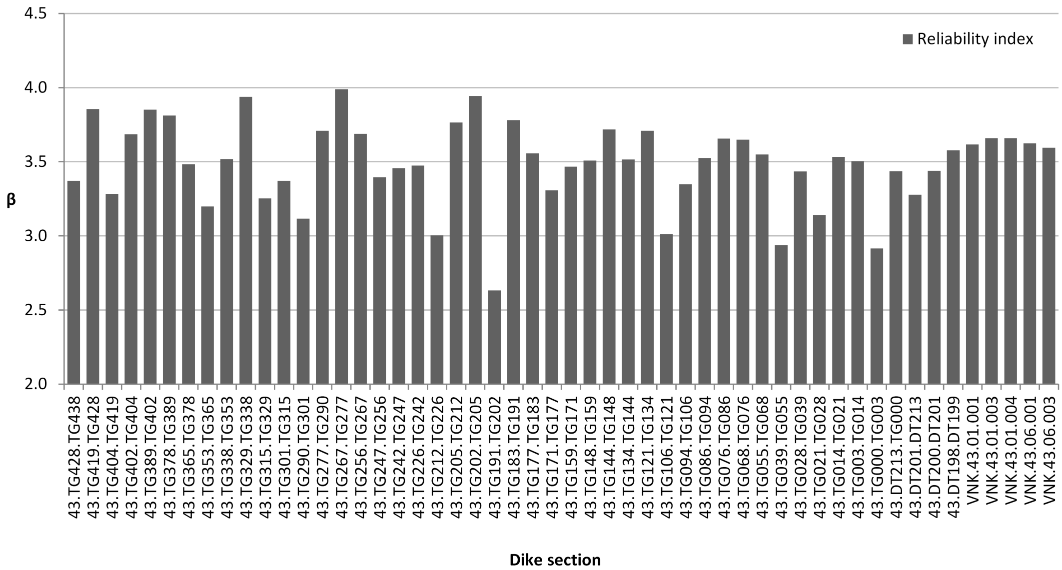


Figure 4.13: Reliability index of dike trajectory 43-6, lower β value means a larger failure probability

In the table 4.11 a list can be found of the ten most vulnerable dike sections. The observer can see that the failure mechanisms overtopping and piping are both frequent contributors.

Table 4.11: Dike section with the highest probability of failure along 43-6

Dike section	Return period	Failure mechanism	
43.TG191.TG202	240	Heave & piping	Overtopping & overflow
43.TG000.TG003	560	Heave & piping	Overtopping & overflow
43.TG039.TG055	600	Heave & piping	Overtopping & overflow
43.TG212.TG226	750	Heave & piping	Overtopping & overflow
43.TG106.TG121	770	Overtopping & overflow	
43.TG290.TG301	1,100	Heave & piping	Overtopping & overflow
43.TG021.TG028	1,200	Overtopping & overflow	Heave & piping
43.TG353.TG365	1,400	Macro-instability	Overtopping/overflow
43.TG315.TG329	1,700	Overtopping & overflow	
43.DT201.DT213	1,900	Overtopping & overflow	

The computed flooding probability is the result of a combination of the failure mechanisms and dike sections considered. The contribution of each failure mechanism is shown in the table 4.12. Overtopping, piping and macro stability are the dominant failure mechanisms. Though only one dike section out of the top 10 list should be addressed for macro instability, this failure mechanisms does have a significant contribution to the overall failure probability.

Table 4.12: Marginal contribution of failure mechanisms for trajectory 43-6

Mechanism	β	P_f
Overtopping and overflow	3.01	1.30E-03
Macro stability	2.59	4.78E-03
Heave and piping	2.53	5.71E-03
Revetment damage or erosion	4.03	2.81E-05
Structure overtopping and overflow	3.69	1.10E-04
Structure closure failure	3.94	4.12E-05
Structure failure	4.66	1.60E-06

Again, it must be noted that the presented figures in this analysis are the result of a computation in which only the relevant failure mechanisms are considered for each dike section according to the FLORIS research (VNK, 2014). This means that only these failure mechanisms can be specified for each dike section and are part of the computation. When a failure mechanisms is deemed not to be significant, then it is neglected in the computation. Ideally, all mechanisms should be accounted for all dike sections, however no readily available data exits for all dike sections.

4.2.3. VERIFICATION OF THE DIKE NEAR TIEL

Two dike locations will be evaluated with using Hydra-Zoet model. The Hydra-Zoet model is the currently statutory dike design model. Since dike sections 43.tg191.tg202 and 43.tg000.tg003 seem vulnerable, see table 4.11, these two dike locations will be analysed.

For river systems, the to be used Hydra-Zoet WTI mode performs a deterministic computation in which design wind speeds and local conditions are gather from a predefined HR2006 database. Hydra-Zoet uses design wind speeds which are based on the "Leidraad Bovenrivieren" TAW (1985). The values for the design wind speeds are shown in figure 4.14. Bretschneider formulas are used for the determiniation for wave conditions at the toe of the dike. Thereafter, a "*PC-Overslag module*" is used to compute run-up, overtopping discharge and a required crest height. For the analysis an overtopping discharge of 5 l/s/m is set as critical and geometric dike parameters are retained from the FLORIS research.

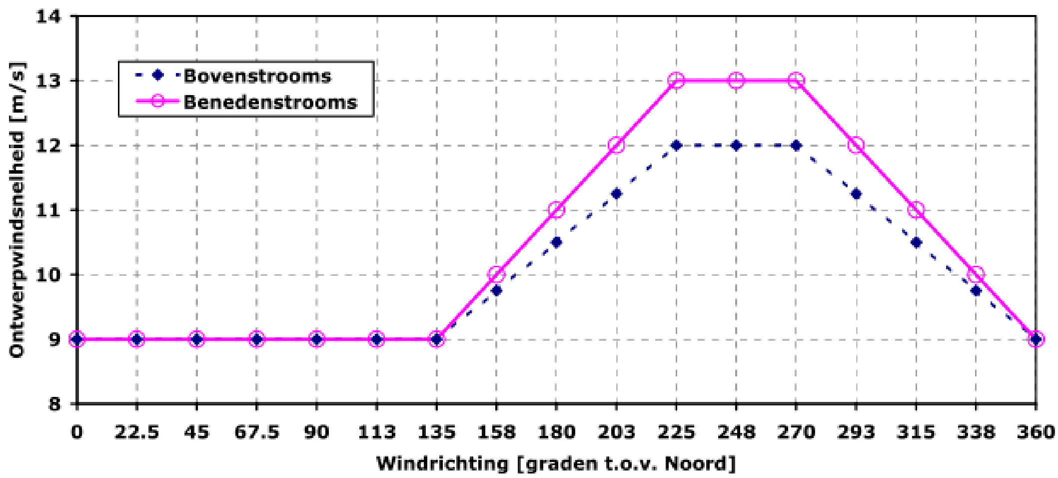


Figure 4.14: Design wind speed used in Hydra-Zoet WTI (M. T. Duits & Kuijper, 2012).

In table 4.13 the results can be seen of the Hydra-Zoet computation. The computed crest heights are almost in accordance with the current crest heights. It must be noted that the elaborated crest height is highly influenced by the user defined critical overtopping discharge and geometric profile of the dike. Furthermore, it can be seen that different wind directions seem to be normative for the two dike location. This is not surprising since the orientations of the dikes are different. This implies that different design wind speeds are used, see figure 4.14.

Table 4.13: Current crest height versus required crest height for trajectory 43-6.

Parameter	unit	43.tg191.tg202	43.tg000.tg003
Dike angle	[degrees]	168	112
Critical overtopping discharge	l/s/m	5.00	5.00
flow velocity	[m/s]	0.51	0.43
u_{wind}	[m/s]	13.00	9.00
r_{wind}	[degrees]	225	90
h	[m+NAP]	9.34	11.49
Hs	[m]	0.42	0.34
Tm-1,0	[s]	2.27	2.12
r	[degrees]	225	90
Exceedance probability requirement	[1/yr]	1/1,250	1/1,250
Required crest height	[m+NAP]	9.87	11.85
Current crest height	[m+NAP]	9.80	11.78

USING QH-RELATION TO GET MORE INSIGHT

To elaborate a dike profile which is in accordance with an other design exceedance probability then the 1/1,250 per year instance, an estimate has to be made for the loading conditions during other instances. To make this possible, a relation has to be defined for the river discharge [Q] and the conformable water level [h]. The Qh-relation depicted in figure 4.15 will be used. The horizontal axis shows the expected river discharge at Lobith where the Rhine enters the Netherlands, and on the vertical axis shows the expected water level at our specific location at Tiel. These are the water levels as they are implemented in Hydra-Zoet and are the result of a hydrodynamic modelling computation using the WAQUA model. 'QH relation provided by Hydra-Zoet test version, justification is needed.'

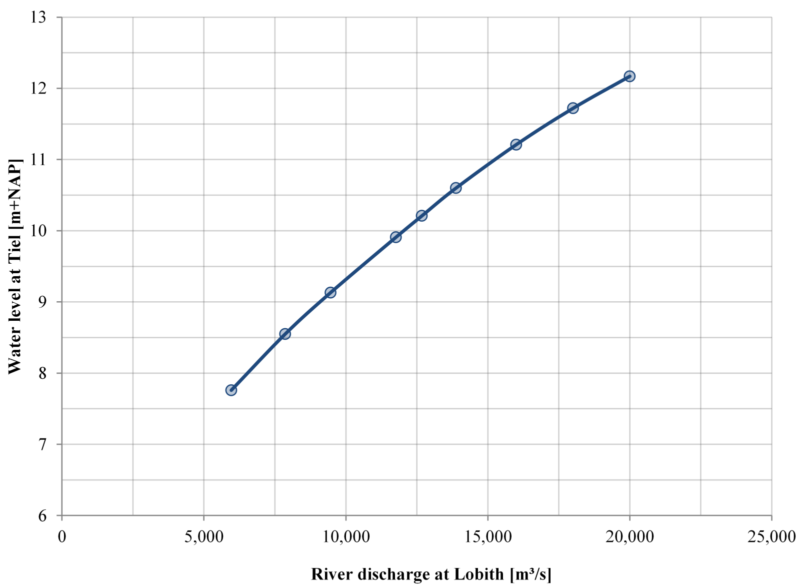


Figure 4.15: Qh-relation for the river discharge at Lobith and the water level at Tiel. Source: Hydra-Zoet test version "ZOEK Bron"

A lot of debate exists about the actual possibility of extreme river discharges as presented in figure 4.15. It is conceivable that river discharges higher then 18,000 m^3/s are physically impossible to occur. A potential dike breach along the Rhine in Germany may result in less severe loading conditions in the Netherlands. Therefore, capping of the Rhine river discharge results in a maximum loading scenario. This complex issue will not be further elaborated in this study.

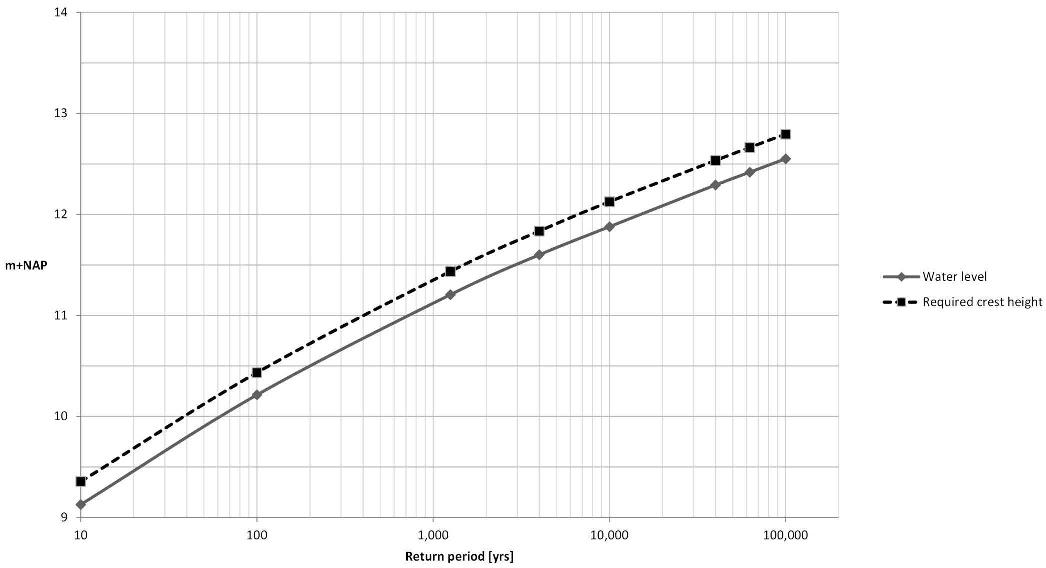


Figure 4.16: Water levels and required crest heights for dike 43.tg000.tg003 at Tiel.

Figure 4.16 shows that the required crest height will change accordingly with the expected water level. The decimal height (a multiplication of 10 in exceedance probability) slightly decreases when scenarios grow more extreme. No significant increase of wave conditions is apparent.

Table 4.14: Table of water levels and required crest heights for dike 43.tg000.tg003 at Tiel.

Return period [yrs]	**Water level** [m+NAP]	**Required crest height** [m+NAP]
10	9.13	9.36
100	10.21	10.43
1,250	11.21	11.44
4,000	11.60	11.84
10,000	11.88	12.13
40,000	12.29	12.54
62,500	12.42	12.66
100,000	12.55	12.80

Not accounted for elements in this analysis are changes in the Qh-relation because of climate change and errors made in the local water depth due to the use of hydraulic modelling. IPCC (2013) and KNMI (2013) both suggest that an increase of rainfall is to be expected in the coming century, a consequence of this can be an increasing river discharge. For this purpose a modified statistical data has been generated for the KNMI'06 2050W+ scenario. The consequence of this will be analysed in section 4.2.4.

4.2.4. CONSEQUENCES OF THE DESIGN RULES OF THE OI2014 FOR DIKE TRAJECTORY 43_6

Opposed to the shoreline of Flevoland where overtopping is believed to be the dominant and only relevant failure mechanism, the FLORIS research VNK (2014) indicates that for dike ring 43 several mechanisms can contribute to failure of a dike. This is largely due to a different composition of the subsoil and dike body. In the following example, the required crest height will be elaborated according to the design probability computed with equation (4.8) taken from the OI2014.

$$P_{required} = \frac{P_{standard} \cdot \omega}{N} \tag{4.8}$$

As the formula shows, several parameters need to be defined. These will be explained below:

FAILURE MECHANISM CONTRIBUTION FACTOR [ω]

The two main failure mechanisms are overtopping and piping. Rijkswaterstaat (2014c, Appendix A) has given a table which provides contribution factors for failure mechanisms. This study will investigate the impact of the OI2014 method on the dike's crest height due to overtopping requirements. Piping will not be further elaborated.

DESIGN PROBABILITY

The new safety standard defines a flood probability of $1/30,000$ per year for dike trajectory 43_6. This level denotes a 'average flood probability', a level that represents a value which is higher then the maximum allowed flood probability and lower then the design probability. The average flood probability is related to the average damage between two successive investments. From Rijkswaterstaat (2014c) follows that the design failure probability is twice the 'average flood probability'. The design failure probability needs to be met at the end for the design's lifetime.

$$P_{design} = 2 \cdot P_{standard} = 2 \cdot \frac{1}{30,000} = \frac{1}{15,000} [per\, year] \tag{4.9}$$

LENGTH EFFECT FACTOR [N]

According to Rijkswaterstaat (2014c, Appendix B) the length effect factor N should be 1 in for dike trajectory 43_6.

The parameters are summarized in table 4.15.

Table 4.15: Summary of used parameters for trajectory 43-6

Parameter	Value
$P_{standard}$	1/30,000
P_{design}	1/15,000
$\omega_{overtopping}$	0.24
ω_{piping}	0.24
N	1

Using the identified parameters of table 4.15 the design failure probability can be calculated. Following guidelines of Rijkswaterstaat (2014c), the designer has to comply to the required failure probability calculated in equation (4.14).

$$P_{required} = \frac{P_{standard} \cdot \omega}{N} = \frac{\frac{1}{15,000} \cdot 0.24}{1} = \frac{1}{62,500} \tag{4.10}$$

$$P_f = \Phi(-\beta) \rightarrow \beta = -\Phi(P_f) \tag{4.11}$$

$$\beta = -\Phi(\frac{1}{62,500}) = 4.159 \tag{4.12}$$

KNMI'06 2050W+ SCENARIO

Furthermore, it must be noted that the computations in this example are calculated using the KNMI'06 2050W+ scenario. The same scenario is part of the HR2006. The scenario specifies that the expected discharge in can rise in the near future. Considering the lifetime of a dike is assumed to be approximately 50 years, a modified maximum discharge is assumed of 1,700 m^3/s at a 1/1,250 per year probability, whereas in 2006 the maximum is believed to be 1,600 m^3/s. The effect of this can be seen in figure 4.17.

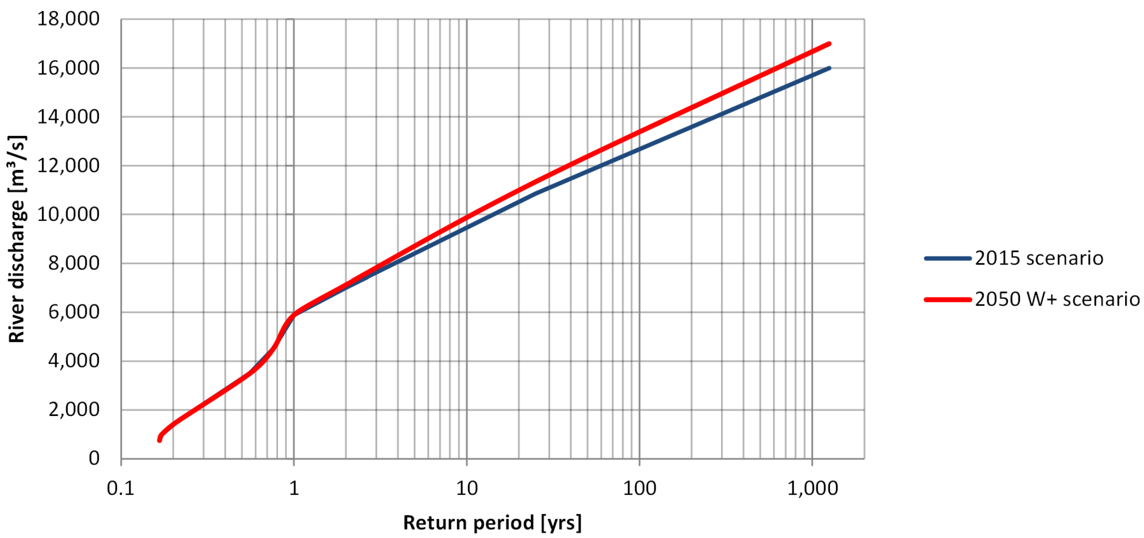


Figure 4.17: River discharge at Lobith, Source: Hydra-Zoet 1.6.3

RESULTS

CREST HEIGHT ELABORATED WITH HYDRA-ZOET

Figure 4.18 shows a frequency curve of the required crest height and the MHW level. The curve shows many similarities with figure 4.16. Only the MHW level has increased, which is due to the use of tge KNMI'06 2050+ scenario. Several aspects will be highlighted below.

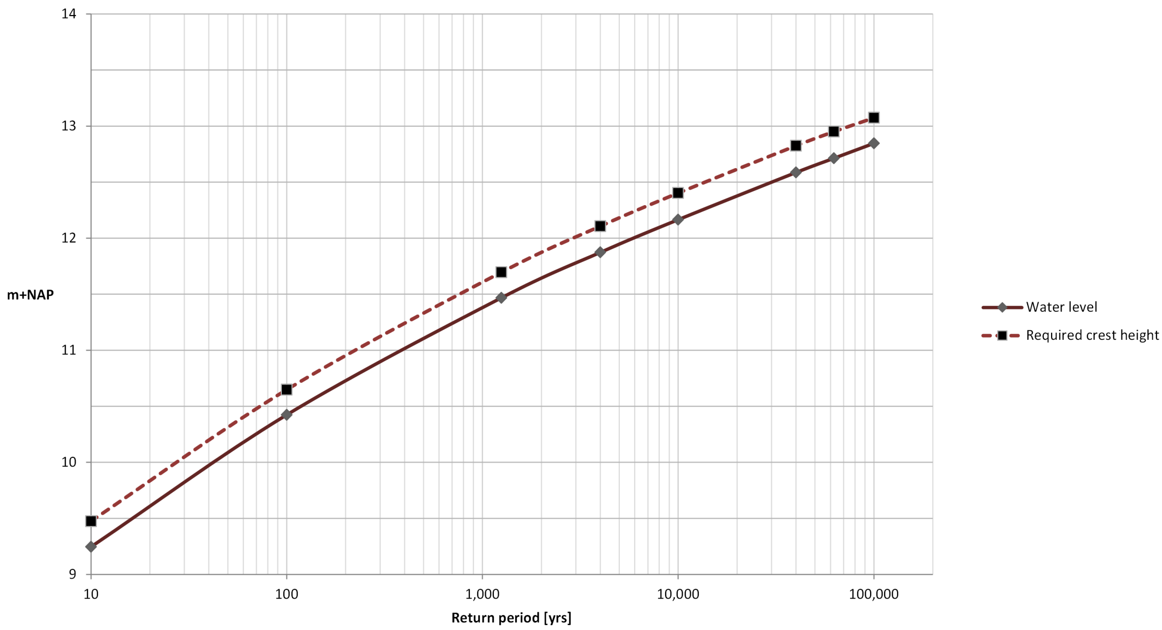


Figure 4.18: Water levels and required crest heights for dike 43.tg000.tg003 at Tiel. KNMI'06 2050W+ scenario

$$h_{crest}(P_{required}) = h_{crest}(\frac{1}{62{,}500}) = 12.95m + NAP \tag{4.13}$$

Since the currently in place dike has a crest height of 11.78 metres, the new design rules proposed by Rijkswaterstaat (2014a) indicate that the crest needs to be heightened by at least by 1.17 metres to comply with the new safety standard.

The results indicate that an increase of hydraulic loading is mainly due to an increase of river discharge. A higher river discharge demands a higher capacity of the river and a consequence of this is an increase of water level. This observation makes the specification of the Qh-relation a key element in the determination of the hydraulic loading.

The computed values for the expected wave conditions do not show great variation for the considered range of return periods. This suggests that extreme wind conditions do not change significantly, since wave conditions are elaborated using the Bretschneider wave growth formula and local river geometry. Looking closer into the expected wind conditions, it follow that for all instances the normative wind direction is East-North-East. Wind speeds are about 9 m/s and never exceed 10 m/s. An ever increasingly wind speed for higher return periods is prohibited by the effects of a conditional probability distribution of wind conditions and river discharge. This means that a less frequent higher wind speed will coincide with a lower river discharge, so that the overall probability stays as what is predefined. Then, the occurring hydraulic conditions are of a smaller significance then the combination computed in this example.

Table 4.16: Wave conditions at dike 43.tg000.tg003. The KNMI'06 2050W+ scenario is considered.

Return period	Wave height	Wave period	Wave direction	Wind speed
10	0.35	2.1	68	8.3
100	0.36	2.1	68	8.5
1,250	0.39	2.2	68	8.9
4,000	0.38	2.2	68	8.7
10,000	0.40	2.3	68	9.2
40,000	0.42	2.3	68	9.5
62,500	0.42	2.3	68	9.5
100,000	0.43	2.3	68	9.8

In figure 4.19 the difference is visualized between hydraulic loading conditions elaborated with current statistical data for river discharges and modified statistical data following the KNMI'06 2050+ scenario. In this scenario an increase of river discharge is processed in the statistical data. Because of this, an increase of 23 to 30 centimetre is expected and can be considered relatively significant. The modified statistical data only has an impact on the elaborated water level, since wave conditions do not seems to significantly enhanced because of this. Effects of enhanced wave conditions are discounted in the computed crest height, and are not apparent.

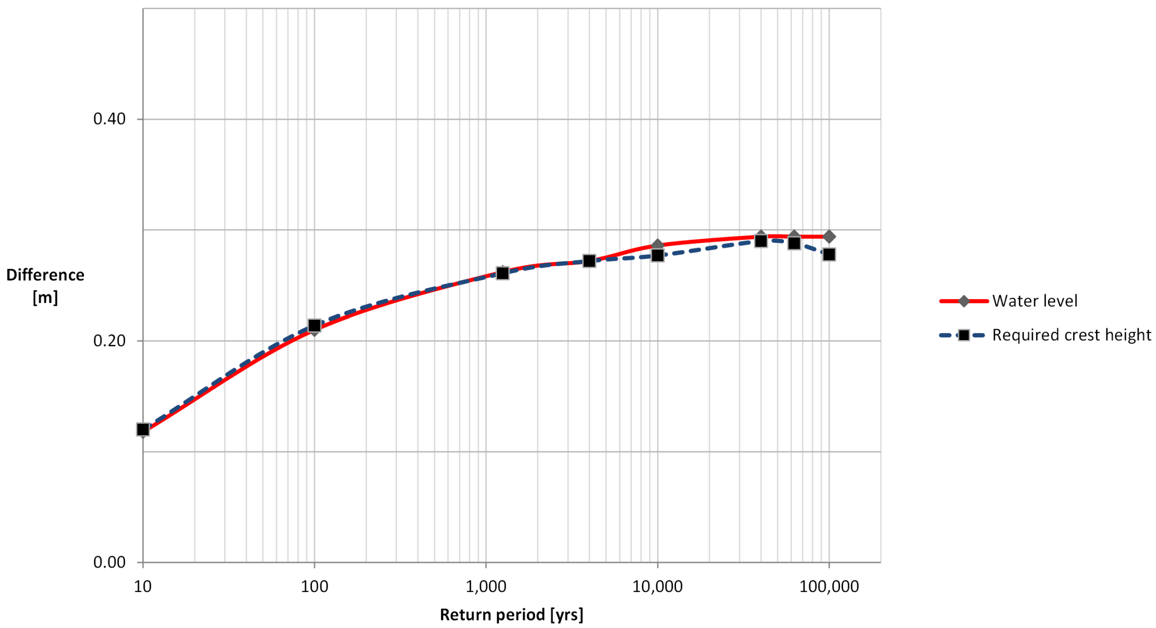


Figure 4.19: Water levels and required crest heights for dike 43.tg000.tg003 at Tiel. KNMI'06 2050 W+ scenario

4.2.5. FAILURE PROBABILITY OF TRAJECTORY 43-6 CONSIDERING DESIGN CONDITIONS ACCORDING TO THE OI2014

Since it is proposed to switch from a dike ring approach to a dike trajectory approach, the trajectory 43_6 (Tiel-Gorinchem) is analysed using the PC-Ring model. For each dike section a crest height will be elaborated in such a way that it complies with the design probability following from Rijkswaterstaat (2014a). The results of this can be seen in figure 4.20. The upper half of the figure shows the current crest height and the required crest height. The lower half shows the difference between the current crest height and the required crest height in metres.

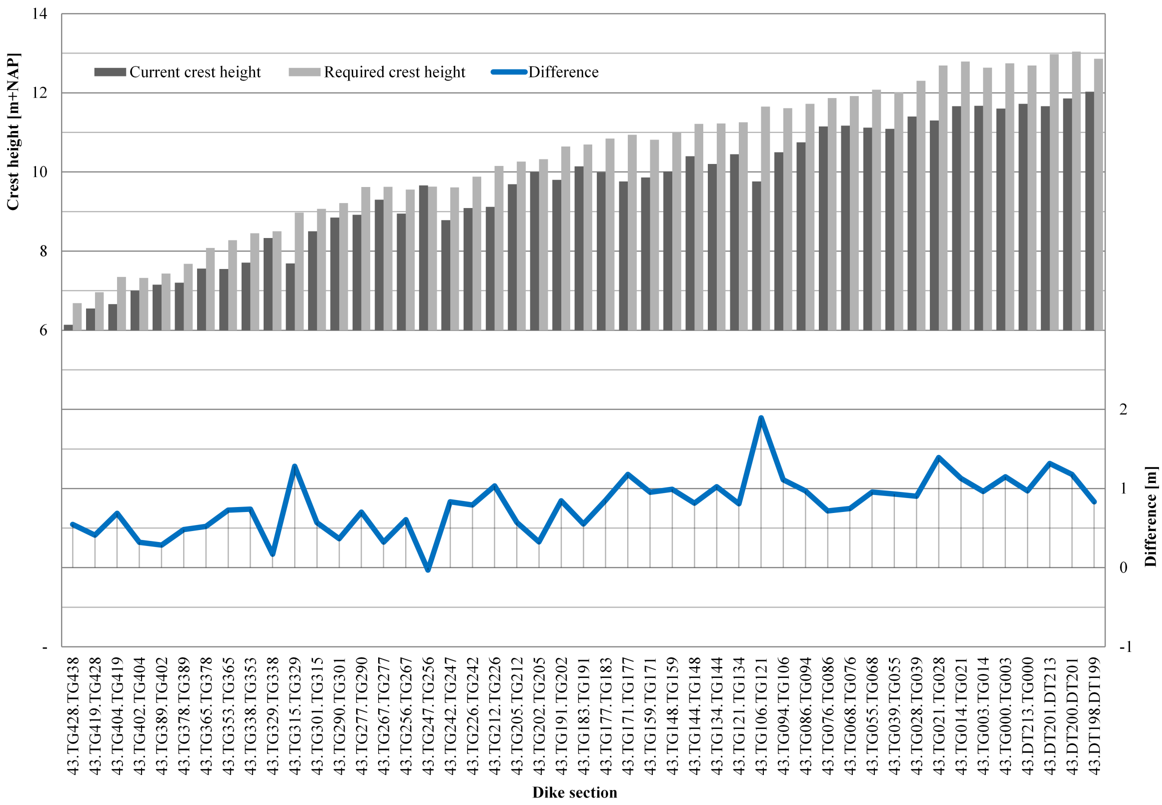


Figure 4.20: Required crest height following $P_{fd} = 1/62,500$ for overtopping per section

From left to right, Figure 4.20 shows an increase in crest height relative to NAP. This is to be expected since we move upstream of the river Waal, so ground level becomes higher relative to NAP. Furthermore, it can be seen that the current dike is expected to be due for dike improvements if the 1/30,000 flooding probability would be used as the new safety standard. The needed increase of the crest height is shown in the lower half of the figure. A overall increase of 1 metre is needed when design rules of Rijkswaterstaat (2014a) are used.

The failure probability of the current trajectory was computed in section 4.2.2, namely 1/100 yr. The failure probability of the designed trajectory with increased crest height is 1/31,000 yr. Do keep in mind that for this figure only overtopping is regarded.

Table 4.17: Failure probability of dike trajectory 43-6 using OI2014.

Considered mechanisms	β	Pf	Return period
Overtopping	3.99	3.21E-05	31,000

4.2.6. ALTERNATIVE CONTRIBUTION FACTOR DISTRIBUTION

Following the findings of VNK (2014), the PC-Ring computation on page 53 shows that three main failure mechanisms for dike trajectory 8-3 are overtopping & overflow, piping and macro stability. Structure failure and revetment damage will be neglected in this example.

For the alternative take on the distribution of the contribution factors, other not apparent failure mechanisms will be dropped van the equation. The distribution will be changed scaled in such a way that the relative differences between failure mechanisms stays the same. The result of this can be seen in table 4.18.

Table 4.18: Alternative contribution factor ω along dike trajectory 43 – 6.

Type	Failure mechanism	OI2014	presence	Alternative
Dike	overtopping	0.24	✓	0.46
	piping	0.24	✓	0.46
	macro stability	0.04	✓	0.08
	erosion	0.10	-	0.00
Structure	closure failure	0.04	-	0.00
	piping	0.02	-	0.00
	structural failure	0.02	-	0.00
Dune	erosion	0.00/0.10	-	0.00
	other	0.30/0.20	-	0.00
	total	**1.00**		**1.00**

The newly retained contribution factor distribution, subsequently, leads to a different design probability as is shown below in equation (4.14):

$$P_{required} = \frac{P_{standard} \cdot \omega}{N} = \frac{\frac{1}{15,000} \cdot 0.46}{1} = \frac{1}{36,000} \tag{4.14}$$

By doing a Hydra-Zoet computation, the implications of this is a reduction in HBN-level of approximately 20 centimetres. This is visualised as the red arrow in figure 4.21, moving from 62,500 to 36,000 [yrs].

The decimal height for return periods larger then 10,000 [yrs] is 0.67 metres. This is smaller then the Flevoland case. Subsequently, the HBN-level is less sensitive to parameter changes.

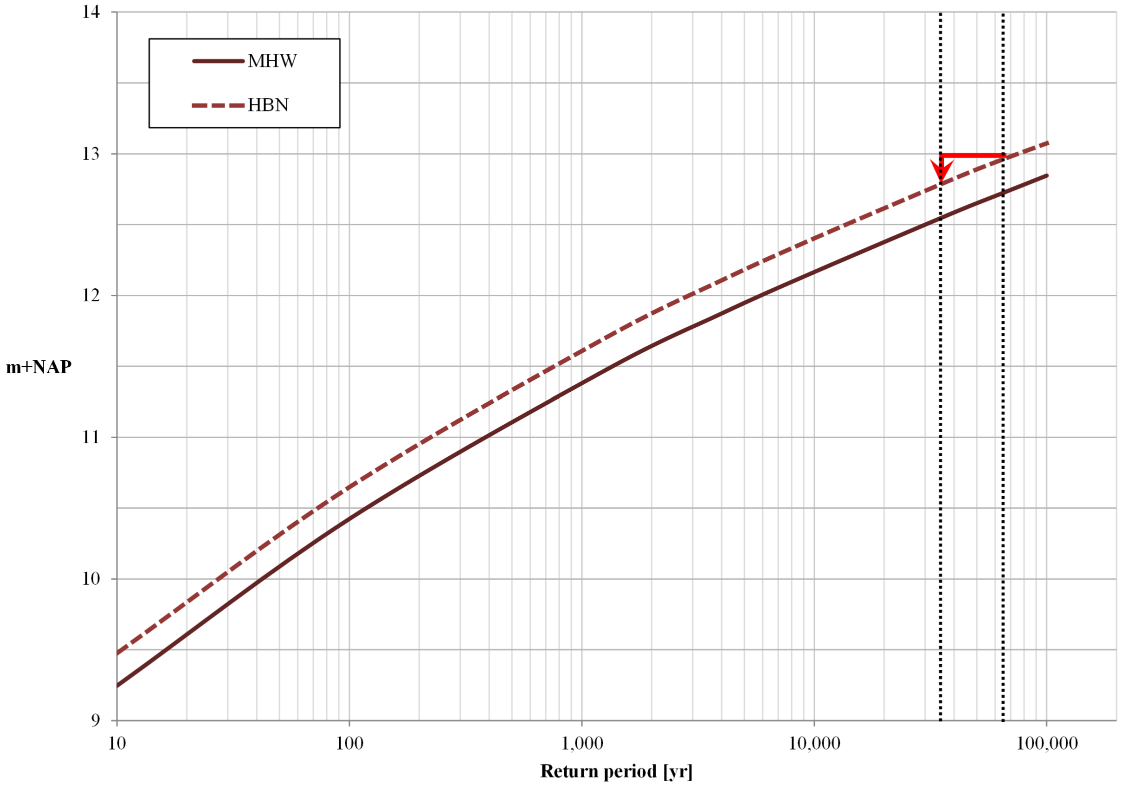


Figure 4.21: Alternative contribution factor. Red return period: 36,000 [yrs].

4.3. EVALUATION

The analyses in this chapter focus on the current dike with respect to new design rules invoked by the OI2014. In so, the current and desired flood probability is computed for dike trajectories 8-3 and 43-6. Each cases is concluded with an extended view on some of the used parameter during the design, and how these could be approached differently.

According to the new design rules of the OI2014, several dike heightening operations are needed for dike trajectory 8-3. Especially the Northern part of the "IJsselmeerdijk" is subdued to an increase of as much as 2 metres. The relatively large fetch length that the IJssel Lake facilitates is the cause for this sharp increase. An alternative take on the contribution factor and the length effect factor show an increase of approximately 1.3 metres (saving 70cm).

A failure probability computation for dike trajectory 43-6 shows a needed heightening of 1 metre for the entire length of the trajectory on average. In this elaboration, capping of the river discharge is not investigated (just as not done during the legal assessment). The greatest contributors to enhanced conditions are an increase of river discharge, which is mainly driven by the value of the standard, and the climate scenario. An alternative take on the contribution factor shows an increase of approximately 0.8 metres (saving 20cm).

Misinterpretation of the new guidelines of the OI2014 is easily conceivable when extreme return periods are considered at section level, please consult page 46 and page 57. However, by evaluating the method on trajectory level leads to the desirable flood probability in both cases considered in this study. On the other hand, in specific cases wherein a dike is only exposed to a select few of failure mechanisms, an alternative view on the distribution of the contribution factor ω can lead to a more optimized dike design. The influence increases with an increasing decimal height. Implications of tailored parameters need to be evaluated on trajectory level.

5

QUANTIFICATION OF UNCERTAINTIES

The next topic in this study addresses the robustness surcharge which is displayed in the OI2014. The robustness surcharges is used to explicitly account for uncertainties in models and statistical data. The attribution of surcharges is indirectly taken from a research carried out by Nicolai et al. (2010). During that research, a special version of PC-Ring is developed in which extra stochastic variables can be added for model and statistical uncertainty. This model is made available for this study. The aim of this analysis is to gain insight about uncertainty behavior for extreme return periods (as proposed by the new safety standard) by using the special version of PC-Ring. This leads to a greater understanding of the composition of the robustness surcharge. Also, most influential parameters are identified.

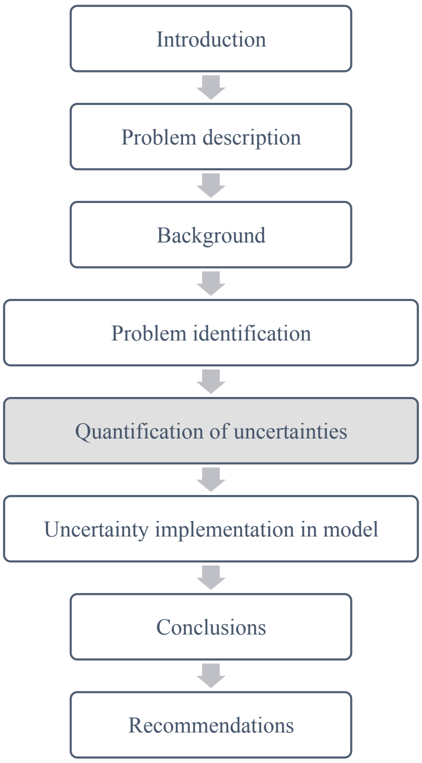


Figure 5.1: Structure of report, item "Quantification of uncertainties".

5.1. ROBUSTNESS SURCHARGE FROM THE OI2014

In 2009, TAW (2009) first noted that the addition of robustness factors should be considered in the establishment of hydraulic boundary conditions. The robustness factor should take account for the model uncertainty. Later in 2014, den Bieman and Smale (2014) proposed an updated description of the robustness factor, in this document they note that statistical uncertainty regarding the hydraulic boundary conditions can also have a significant influence on the expected load on the dike. Following these consideration, a computation will be performed in which the robustness factor will be applied as proposed by den Bieman and Smale (2014).

Table 5.1: Robustness factors.

System	Parameter	(TAW, 2009)	(den Bieman & Smale, 2014)
Lakes	Water level	+0.20 m	+0.40 m
	Wave height	+10%	+10%
	Wave period	+10%	+10%
Rivers	Water level	+0.30 m	+0.30 m

den Bieman and Smale (2014) give steps in how to apply the robustness into a dike design. The guidelines are to be used in the present commonly used Hydra models. Several steps to derive the normative loading conditions and a required crest height are explained:

1. The Hydra models are to be used to determine the design loading conditions for a chosen return period. This is a semi-probabilistic calculation in which the hydraulic load is obtained in a probabilistic fashion. Strength parameters are considered deterministic.
2. The robustness factors as defined in table 5.1 are to be applied to the obtained design conditions and loading conditions from the former step.
3. A single computation is to be performed in which the recalculated design conditions will be used as the input parameters for the determination of the required crest height.

5.1.1. CASE STUDY: FLEVOLAND

The analysis will be carried out for three locations. Two locations along the IJssel Lake, namely "F260 IJsselmeerdijk" and "F300 IJsselmeerdijk", and one location along the Marker Lake, "hm3.4 Oostvaardersdijk". Locations have been chosen as such that they cover different loading conditions. The locations can be seen in figure 5.2. Ideally, more locations might provide more insight. the Marker Lake database doesn't provide more locations to be analysed which are part of the dike trajectory 8_3.

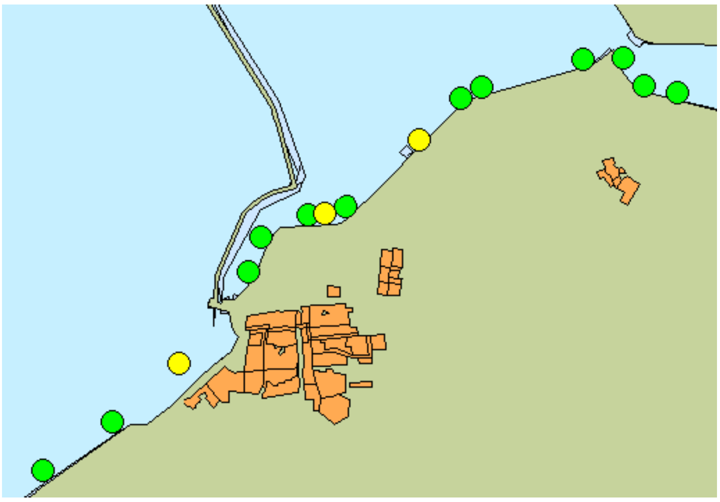


Figure 5.2: Three lake locations. From left to right: hm3.4,F300,F260.

Furthermore, the HR2006 database is going to be used. The critical overtopping discharge is chosen to be 5 [l/s/m], following van der Meer (2012) and appendix A. Geometric dike profile data is retained from the FLORIS research (VNK, 2012). An increase in loading conditions due to climate change or not accounted for in this analysis.

The following paragraph shows the results of dike locations "F260 IJsselmeerdijk". In table 5.2 the effects of the robustness factor can be seen. The increase in water depth, wave height and wave period has led to a required crest height of 5.91 m+NAP. The effect of incorporating robustness factors is an increase of the crest height of 1.22 m. This is significantly more then the water level increase of 0.40 m. Therefore the effective increase in required crest height due to overtopping is 1.22-0.40 = 0.82 m. It must be noted that optimization of the dike profile may lead to a lower required crest height.

Table 5.2: Conventional versus added robustness. Elaborated with: Hydra-Zoet 1.6.3 Deltamodel

Parameter	unit	conventional	added robustness
m	[m+NAP]	-0.16	-0.16
u	[m/s]	31.7	31.7
h	[m+NAP]	1.71	2.11
Hs	[m]	2.27	2.50
Tm-1,0	[s]	5.90	6.49
r	[degrees]	323	323
Exceedance probability requirement	[1/yr]	1/4,000	1/4,000
Required crest height	[m+NAP]	4.69	5.91

Since robustness factors have not been laid down by law, this observation does not immediately mean that the IJsselmeerdijk is rejected if it would be assessed at this moment. However, if consensus about the importance of robustness and uncertainties is reached, then research is needed to come up with a proper attribution method. The addition of robustness factors as a 'rough surplus' seems hardly sophisticated. Double-inclusion might also be a problem since 'safety factors' are also considered in the elaboration of the strength of the dike.

As seen on the previous page, the following two dike sections have been analysed in the same fashion. Detailed information about the normative conditions can be found in appendix B. The required crest height and MHW level due to robust designing are summarized in table 5.3.

Table 5.3: Summary of required crest height due to robustness designing

Dike location	Return period [yrs]	MHW [m+NAP]	Robust MHW [m+NAP]	hcrest [m+NAP]	Robust hcrest [m+NAP]
F260 IJsselmeerdijk	4,000	1.71	2.11	4.69	5.91
	187,500	2.46	2.86	6.54	7.94
F300 IJsselmeerdijk	4,000	1.71	2.11	3.81	4.85
	187,500	2.39	2.79	5.45	6.74
hm3.4 Oostvaardersdijk	4,000	1.18	1.58	4.73	5.95
	187,500	1.77	2.17	6.14	7.59

As proposed by the robustness guideline, an increase in MHW level is 40 centimetres in each of the cases. The effective influence on crest height due to overtopping is 64-82 centimetres for the 1/4,000 years cases and 89-105 centimetres for the 1/187,500 cases. A notable observation is that the current Oostvaardersdijk's crest height of 2.85 [m+NAP] and therefore does not meet the 1/4,000 years design conditions, which is the safety standard at this moment. The calculated crest height is heavily influenced by the attribution of the critical overtopping discharge. Though, the allowable critical overtopping discharge has been chosen more lenient then formerly used, namely 5 [l/s/m] instead of 1 [l/s/m].

A possible explanation of this observation might be that the geometric dike profile is not optimized for the considered extreme design conditions. Whereas the MHW and fetch length is smaller on the Marker Lake then on the IJssel Lake, the required crest heights for the "Oostvaardersdijk" still exceeds any of the IJssel Lake cases. Comparing the dike profiles of the "Oostvaardersdijk" and the "IJsselmeerdijk", it shows that the slope angles of the "Oostvaardersdijk" are steeper then those of the "IJsselmeerdijk".

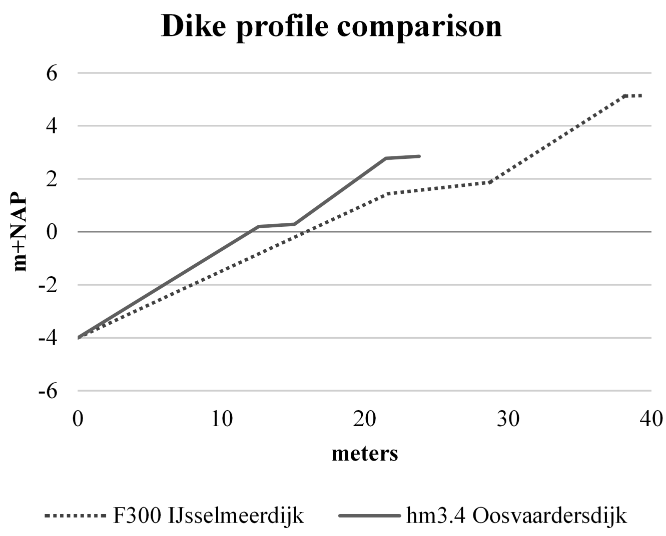


Figure 5.3: Dike profile comparison between F300 and hm3.4 (VNK, 2012)

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In all, it can be concluded that the implication of the robustness guidelines by den Bieman and Smale (2014) are substantial. The consequence the guidelines have on the required crest height seem to be more of a higher order then the factors for MHW initially suggest. This is due to an increase of 10% in wave conditions. This leads to a difference of a meter or more in required crest height. Optimization of the dike profile may lead to lower crest heights, but reinforcement are inevitable if robust designing is chosen to be compulsory.

5.1.2. CASE STUDY: THE RIVER WAAL

The effects of robustness in a river system will be elaborated in the following section. For this, a location along the Waal will be chosen. Following observations from table 4.11 and readily available data, the dike TG000.TG0003 near Tiel will be further analysed. The dike location lies within trajectory 43-6 and can be seen in figure 5.4.

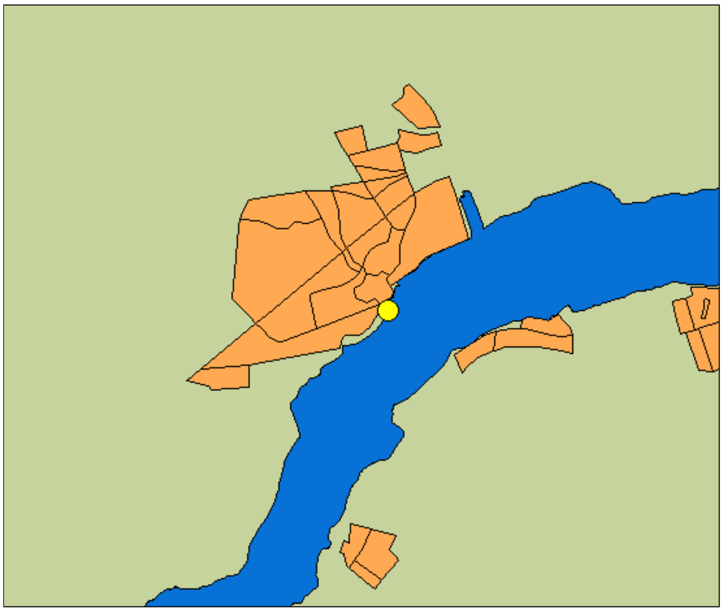

Figure 5.4: Location of dike tg.000.001 near Tiel. Source: Hydra-Zoet 1.6.3.

The modified database in which a Qh-relation is incorporated will be used. The database is in accordance with the HR2006, but gives the opportunity to elaborate cases for other then the one set in the HR2006. Again, an overtopping discharge of 5 [l/s/m] is considered to be normative. Geometric dike profiles are retained from the FLORIS research VNK (2014). Changes induced by climate change are covered to the extent of the use of a the KNMI'06 2050W+ scenario. This means that a river discharge of 17,000 [m^3/s] at Lobith is considered for a case with an 1/1,250 [yrs] exceedance probability.

Table 5.4: Conventional versus added robustness for a dike near Tiel. Elaborated with: Hydra-Zoet 1.6.3

Parameter	unit	conventional	added robustness
u	[m/s]	8.9	8.9
h	[m+NAP]	11.29	11.59
Hs	[m]	0.39	0.39
Tm-1,0	[s]	2.2	2.2
r	[degrees]	68	68
Exceedance probability requirement	[1/yr]	1/1,250	1/1,250
Required crest height	[m+NAP]	11.70	12.00
u	[m/s]	9.5	9.5
h	[m+NAP]	12.50	12.80
Hs	[m]	0.42	0.42
Tm-1,0	[s]	2.3	2.3
r	[degrees]	68	68
Exceedance probability requirement	[1/yr]	1/62,200	1/62,200
Required crest height	[m+NAP]	12.93	13.23

In the computations a surplus of 30 centimetre is added to the expected water level at the toe of the dike. Consequently, this additional load has its impact on the required crest height of the dike. In both cases the crest height is computed to be 30 cm higher. This is the same as the predefined increase of the water level. Therefore, it is suggested that the increase in crest height is only due to an increase water level. Wave conditions are presumed to remain identical, and increased loading effects due to waves are not apparent in the presented crest heights.

In this example the introduction of robustness additions do not seem to have an significant impact on the required crest height of a dike. The significance are of the same order as the considerations about climate scenario, namely 20-30 centimetre. Implications of the to reckon with safety standard shows a far greater influence (which is more the 1 meter). Although, capping of the maximum river discharge may lead to less severe normative conditions which comply with the safety standard. In so, a lower crest height may be required as is implicated in this analysis.
Increased wave conditions are deemed not to occur and are not incorporated in this analysis. Wave conditions can be elaborated with the Bretschneider formulas, in which wind speed and water depth play an important role. Increased wave conditions are conceivable if one considers an increase in water level, like proposed by the robustness surcharge. Also, an effect of climate change can be an increase of wind speeds. Modified statistical wind data might give insight in this matter. The challenge in this is what an appropriate variation would be.

5.2. IMPACT OF ADDING ADDITIONAL UNCERTAINTIES IN HYDRAULIC LOADING CONDITIONS

In the following chapter an analysis is performed in which the robustness surcharge as proposed in den Bieman and Smale (2014) will be verified to its accuracy. The robustness surcharges stems from the the uncertainty in physical models and statistical analysis of basic stochastic variables. The resulting robustness factors are the result of an analysis which only considers several locations. From this an expert judged median is chosen. Of interest will be how this figure compares to a whole stretch of dike using the same methodology. Analogously, the marginal contribution of statistical and model uncertainty is observed. The former robustness factor as proposed by TAW (2009) considers model uncertainties to have an influence of +20 cm in lake systems. den Bieman and Smale (2014) believe that the combined influence of model and statistical uncertainties is +40 cm. This presumable 50/50 contribution will also be analysed for extreme return periods since Rijkswaterstaat (2014a) has given (preliminary) design rules in which extreme cases have a more prominent role.

5.2.1. METHODOLOGY

Following literature by (Nicolai et al., 2010), significant uncertainties will be quantified. Addressing all uncertainties would be labour intensive and would be philosophically considered impossible. However, addressing the most important should provide reasonable indications whether uncertain factors play a significant role or not. The new uncertainties will be combined with progressive model parameters such as a more lenient critical overtopping discharge. In this sense, an up to date framework is established. To include these specifications into a model, a special version of PC-Ring will be used which allows the inclusion of the treated parameters in this analysis as random variables.

In the analysis random variables are gradually added in the elaboration of MHW and required crest height following from a overtopping computation. Four steps will be performed. First, a base computation will carried out to which other computations can be compared. This base computation is comparable to the conventional approach of dike failure design using the PC-Ring model. The boundary conditions are highlighted in section 5.2.2. Various influential uncertainties have been pointed out by Nicolai et al. (2010). In the seconds computation additional model uncertainties will be introduced. The additional random variables will address the error in local water depth due to hydrodynamic models, the error in wave conditions as a result of the wave models. A third computation will evaluate the impact of additional statistical uncertainties. Parameters are incorporated to address the uncertainty in river discharges, local water depth and wind conditions. In the fourth and last computation, additional model and statistical uncertainties, following from the former steps, are both incorporated to see their combined effect on MHW and the required crest height.

5.2.2. ASSUMPTIONS

BOUNDARY CONDITIONS

To make this research in-line with previous research by VNK (2012) and Nicolai et al. (2010) those reports will be used as the starting point of this research. Also the same modelling techniques will be applied to safe significant time instead of making a whole new model. Keeping progressive research in mind, several assumptions will be made which can differ from previous research. Data will be based on the best available and acceptable assumptions following recent research. Below, several important aspects are highlighted.

TMR2006 DATABASE

The used PC-Ring database contains information about loading conditions and many random variables affiliated with inherent and model uncertainties. The loading conditions used are from the TMR2006 database. The TMR2006 database contains hydraulic loading conditions which can be used in the PC-Ring model. The TMR2006 originates from the HR2006 database which is used for Hydra models. Most notable difference is that the 16 wind direction database of HR2006 has been transformed to a 12 wind directions database for TMR2006. The transformation which has been done by using an interpolation method was needed to make the data suitable for the PC-Ring model.

FLORIS DATABASE

Furthermore, the PC-Ring database contains information about dike profile characteristics, parameters to take account for natural variability and several model parameters. Most of this characteristics come from the FLORIS research (Projectbureau VNK2, 2011). Since this is the best and most comprehensive database currently available, this has been chosen.

ADDRESSING MISSING COMPONENTS OF DATABASE

Since no PC-Ring database exists for the cases considered in this analysis, a specific database needs to be developed. The TMR2006 and FLORIS database form the base of this newly developed database. Accordingly all locations parameters have been used from a version 19 database and have been included in the special version 17 database which can be used in the PC-Ring-EO model. All extra parameters which are newly added to the database due to the inclusion of new uncertainties for the new locations have been initially mirrored to the existing parameters for other locations. Then validity is checked using the Nicolai et al. (2010) analysis and other literature treated in this chapter. If this database is requested, please contact the author of this report.

GEOMETRIC DIKE PROFILE

A notable modification to the crest height variation has been applied. The FLORIS research has used a standard deviation of $\sigma = 0.025$ for its crest height, whereas in this analysis it chosen to be increased to $\sigma = 0.1$. The same value is used by Nicolai et al. (2010) in there analysis. The altimetry of the Netherlands is managed by the "Actueel Hoogtebestand Nederland 2" project (AHN-2). In this project laser altimetry (LIDAR) is used to measure the surface. The accuracy of LIDAR is influenced by a systematic error of 5 cm and a stochastic error of 5 cm according to Swart, Flos, and Zomer (2007). Therefore, the standard deviation proposed by FLORIS seems a bit optimistic.

CRITICAL OVERTOPPING DISCHARGE

An overtopping discharge of 5 l/m/s is chosen to be critical in this analysis. This differs from the analysis of Nicolai et al. (2010) in which a overtopping discharge of 1 l/m/s is used. van der Meer (2012) argues that a overtopping discharge of 1 l/s/m is a conservative value. This figure is also analysed in appendix A. Following van der Meer (2012) and appendix A, it can even be argued that a overtopping discharge of 10 l/s/m is a possible option although not sufficient in all cases. These considerations have lead to believe that an overtopping discharge of 5 l/s/m is a good and safe estimate for the critical overtopping discharge, and was therefore proposed in Rijkswaterstaat (2014c).

A special version of PC-Ring is used to be able to incorporate statistical uncertainties in the elaboration. A customized database has been made in which the following random variables are added to the elaboration:

INCLUDED ADDITIONAL UNCERTAINTIES

ERROR IN LOCAL WATER DEPTH

Physical hydrodynamic models estimate the expected local water depth during predefined conditions. This gives the advantage to estimate water depths at locations where no statistical analysis can be performed due to the lack of a specific measurement gauge. Therefore water depths along a stretch of river or a lake area can be computed. In the Netherlands computations are made using the SOBEK, HISWA and WAQUA model. Although many research has focused on the dynamics of water bodies, hydrodynamic models which have been developed in recent years can only comprehend this behaviour to a certain extend. This is due a lack of knowledge or an expert judged choice to simplify the model to decrease its computational time. This is the case in many computational models.

The use of the WAQUA model in lake areas impose the following uncertainties according to Nicolai et al. (2010). First, at high wind speeds the water roughness can have an significant effect on the local water depth. Secondly, the WAQUA model assumes that an uniform wind field occurs at the same time at every location. Thirdly, WAQUA is a 2D-model, which neglects 3D such as lateral flow. The following table of standard deviation are proposed by Nicolai et al. (2010).

Table 5.5: Random variables for error in local water depth

	type	mean	deviation
Lakes	Gaussian	0	0.15
River Waal	Gaussian	0	0.16

ERROR IN WAVE CONDITIONS

Wave models are used to estimate wave height, period and direction. In the PC-Ring model, the Bretschneider model is used for upper river systems. This is an 1-dimensional wave growth model. By using the Bretschneider model during a crest height computation, Beckers (2009) estimates that an error of 0.1 to 0.3 meters. This notion of accuracy is translated to a random variable for wave height and wave length which is $LN[1,0.15]$.

For lake systems, the SWAN model is used for the estimation of wave conditions. This is a 2-dimensional model. A benefit of this model is that it can cope with shallow-waters, diffraction and refraction. Beckers (2009) estimates that the error in the computed required crest height is in the order of decimetres. In turn, this is processed as the random variable for wave conditions as $N[1,0.10]$.

An assumption made in the PC-Ring model is that the wave direction is identical to the wind direction. In reality, this might not be the case. A deviation of 10 degrees is processed in the PC-RIng model.

Table 5.6: Random variables for wave conditions.

	type	mean	deviation	unit
Bretschneider: Hs,Ts	Log	1	0.15	-
SWAN: Hs, Ts	Gaussian	1	0.10	-
Wave direction	Gaussian	0	10	degrees

STATISTICAL UNCERTAINTY IN RIVER DISCHARGE

From statistical analysis, a probability density function (PDF) can be derived for the rivers such as the Rhine and Meuse. A work line represents an expected water level with a corresponding river discharge. Measurements of the rivers discharge and water level roughly exists of the past 100 years. Using statistical extrapolation methods, an estimation can be made of the expected hydraulic loading conditions of a less frequent and more extreme instance, one we have not encountered yet. In this statistical analysis, the more extreme instances weigh more then the moderate instances. Unfortunately, this data is scares since they do not occur frequently. Therefore it is expected that the estimation is prone to uncertainty. For this a 95% band of probability has been formulated. The uncertainty in the work line will be addressed by a multiplication factor which corresponds with relative variability. "See the table below (to be included)."

Table 5.7: Random variables for river discharge.

	type	mean	deviation
River discharge at Lobith	Gaussian	1	0.08

STATISTICAL UNCERTAINTY IN LAKE LEVEL

First, two definition will be explained: Frequency line; is the exceedance frequency of lake level per year. Quotient line; is the average time of a flood wave.

The PC-Ring model uses statistical lake level data which first has been introduced in the Hydra-M model (Hydra-M is a predecessor of the Hydra-Zoet model). The statistical data cover a dataset of the lake levels for the period of 1976 -1995. Only winter months are considered since the most severe conditions occur during these months. The quotient line is derived using a exponential For the higher range of the water level frequency line, the yearly maxima have been analysed using a statistical averaging method which comprise out of four extreme value distributions:

1. Pearson-type 3
2. Gumbel
3. Gaussian
4. Raleigh

In the PC-Ring model the wind is being modelled by rectangular blocks (FBC-model). The width of each block is derived from the quotient line, and the height of the block is derived from the frequency line.

Considering these steps, two sources of uncertainty are introduced: the statistical analysis for the frequency/quotient line and the FBC-model. Also, the underestimation of the lake level has been pointed out by Geerse (2006). Though, policy related choices have led to maintain the use of the old statistics which have been enforced in the HR2006.

Table 5.8: Random variable for the lake level.

	type	mean	deviation
Lake level uncertainty	Gaussian	0	0.1

STATISTICAL UNCERTAINTY IN WIND CONDITIONS

In the Netherlands, wind data from 12 different measurement stations is gathered across the country. This wind data is statistically analysed using the Rijkoort Weibull model (RW model). The RW model consists of several steps in which probability distribution are estimated, up-scaling of data occurs and smoothing techniques are applied. Using the model, an as close as possible estimation is made to reality. However, one can argue the accuracy of the model since a precise representation of the wind's behaviour is difficult to achieve and several objective decisions are at the foundation of the model. At this stage of the research, the parameters proposed by Nicolai et al. (2010) for statistical wind uncertainty are being used. This means that a multiplication parameter of $N[1;0.085]$ is introduced for all wind stations.

Table 5.9: Random variable for wind speed.

	type	mean	deviation
Wind speed uncertainty	Gaussian	0	0.085

5.2.3. RESULTS

INFLUENCE AT A CONVENTIONAL RETURN PERIOD

A FORM computation has been performed using the special version of the PC-Ring model. The four steps as explained in section 5.2.1 have been carried out. In this sense, uncertainties are gradually added to the computation. The results of the MWH level and the required crest height due to overtopping can be seen in figure 5.5 and figure 5.6. It must be noted that about every computation showed difficulty in convergence of the FORM analysis. However, some clear observations can be extracted from these figures.
The first thing to notice is that the MHW is fairly constant along each each dike stretch. At the IJssel Lake, the MHW level is expected to be about half a metre higher then at the Marker Lake. Looking into the differences between the four bars per dike section, it shows that the presumable assumption of a 50/50 contribution of additional model and addition statistical uncertainties is not valid. In every case, the addition of statistical uncertainties shows a far greater attribution to expected MHW level. Also, the combined effect of model and statistical uncertainties is not a straightforward summation of the added value of bars 2 and 3. This might be due to effect of correlation between certain parameters. However, a Monte Carlo simulation might provide more accurate results.

Comparing the results of no additional uncertainties versus additional model and statistical uncertainties (bars 1 and 4) it follows that the robustness factor of Rijkswaterstaat (2014a) is fairly accurate. Most cases show an increase of 30 cm in the MHW level. Considering the robustness factor of Rijkswaterstaat (2014a) is elaborated by looking at only a handful of test cases, this is a convenient observation for whom has been using this robustness factor in their design.

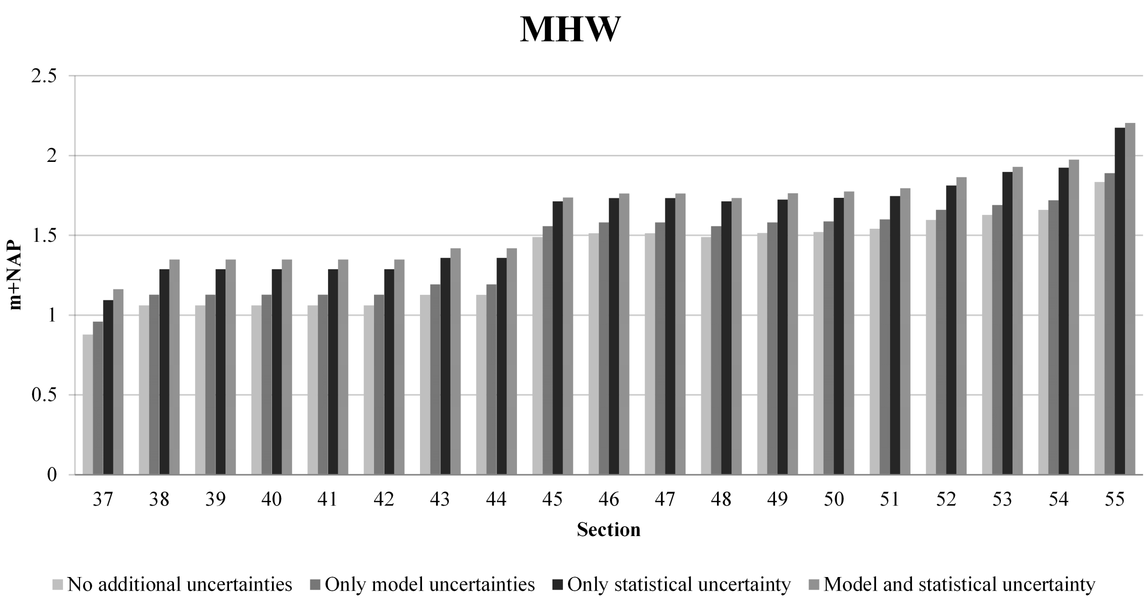

Figure 5.5: The ifluence of model and statistical uncertainty on the MHW for dike trajectory 8-3.

Results of the overtopping computation are show in figure 5.6. Convergence problems seem to be more apparent in this case, especially at sections 44 and 45 where the computation was unstable that such an extent that no result could be computed. Looking into sections 44 and 45 specifically will show that these sections represent a sluice, applicability of the model for this special structure should be sought out. Furthermore what can be seen from the figure is that dike sections 38,40,41,52,53,54 and 55 show a significant higher required crest height then the other sections. This is mainly due their position which allows for a relatively greater fetch. The dominant wind direction in various from 270-300 degrees at the Marker Lake, to 300-330 degrees at the IJssel Lake. For most of this sections this means that the dominant wind direction is almost perpendicular to the dike. This is highly unfavourable against wave attack. Other dike sections are more sheltered, so wave attack would be less. However, it can be concluded that the effect of wave attack is significant, since MHW level is 1-2 metres as seen in figure 5.5.

These are all expected observations. The introduction of more uncertainty only emphasizes this effect. Compared to the MHW computation of the previous page, there seems to be a more evenly attributed effect of model uncertainty and statistical uncertainty. Looking at the figure, a roughly estimated 40/60 distribution can be obtained. The additional statistical uncertainty is the dominant factor.

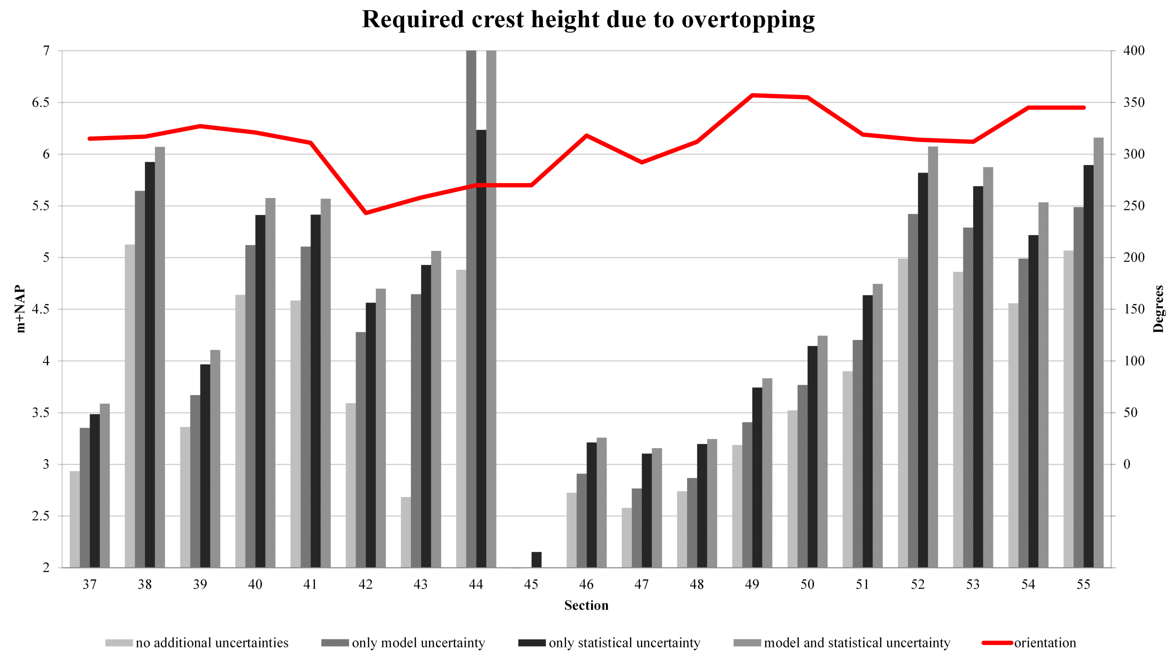

Figure 5.6: The ifluence of model and statistical uncertainty on the HBN for dike trajectory 8-3.

To analyse the attribution of additional model and statistical uncertainty in more detail, the following paragraph will focus on the relative influence of the additional uncertainties.

The following figure 5.7 shows the relative influence of the added uncertainties in the MHW level. All results have been normalized to the first standard computation, as shown in equation (5.1). The *reference* being the computation with no additional model or statistical uncertainties, and *i* being any of the computation with added uncertainties.

$$\text{relative influence} = \frac{MHW_i}{MHW_{reference}} \tag{5.1}$$

The relative influence of additional uncertainties is greater at the Marker Lake opposed to the IJssel Lake. The chosen additional statistical random variables (green line) have a greater contribution to an enhanced MHW level then the additional model random variables (red line). Their combined influence at the Marker Lake is about 1.28, at the IJssel Lake this is 1.18. This gives an indication that the robustness factor might be given a more specified factor for each of the lakes since the difference is quite clear. Overall, you can conclude that the robustness factor for MHW for lakes is in the order of 1.25 times the reference MHW level without uncertainties.

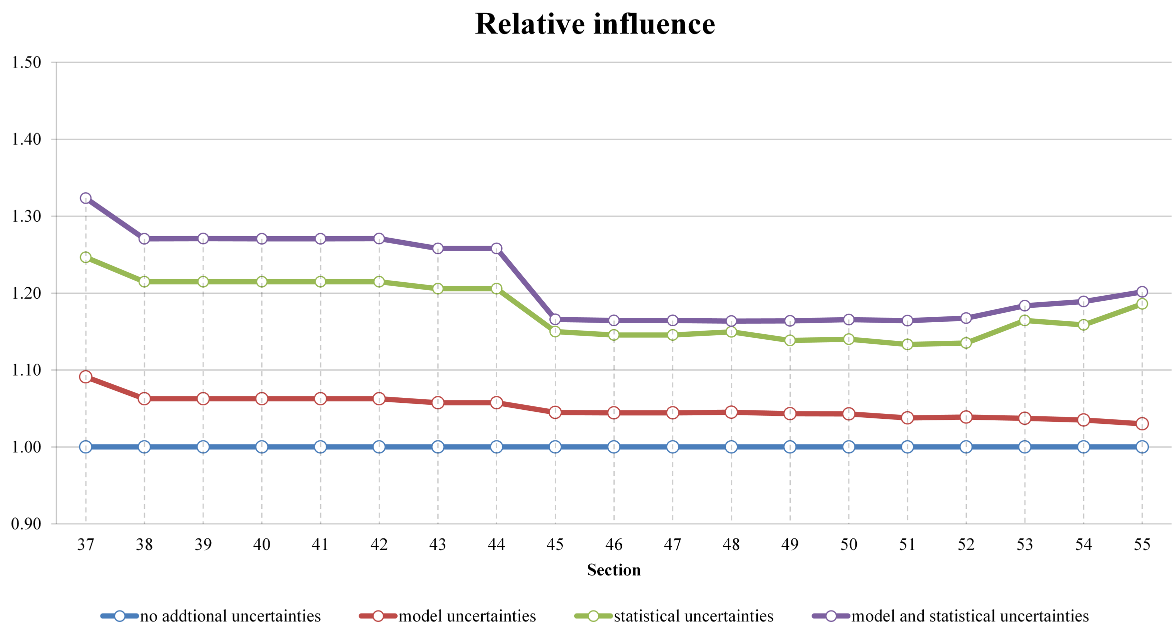

Figure 5.7: Relative influence for MHW, return period = 4,000

After taking notice of the convergence problem when performing a FORM computation, a directional sampling Monte Carlo (DS) computation is carried out. Directional sampling is a Monte Carlo method in which a large amount of samples is being generated to obtain a numerical solution. These technique is further explained in Vrouwenvelder and Steenbergen (2003, Chapter 4). In figure 5.8 the differences between the FORM and DS are visualized. The graph presents the difference of the computed DS value against the calculated FORM value. Although convergence was not realized in many of the MHW computations, the offset seems to be small, only a matter of a few centimetres. Only dike sections 51-55 show a clear offset. Therefore, it can be concluded that a FORM analysis would be a sufficient estimation when return period of only a several thousand years are considered.

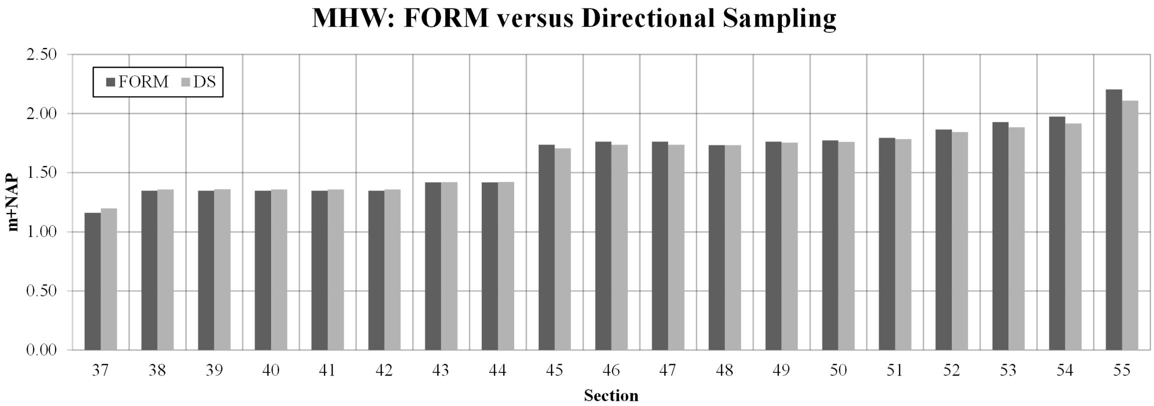

Figure 5.8: MHW computation using FORM and directional sampling.

However, this is not the case for the overtopping calculation in which the required crest height is derived. This can be seen in figure 5.9. Dike sections located along the Marker Lake show an offset of about +50 cm. This effect is even more severe along the IJssel Lake, whereas differences can go as high as +1.74 cm (the sluice sections 44 and 45 are not considered since the FORM analysis seem to be inconsistent).

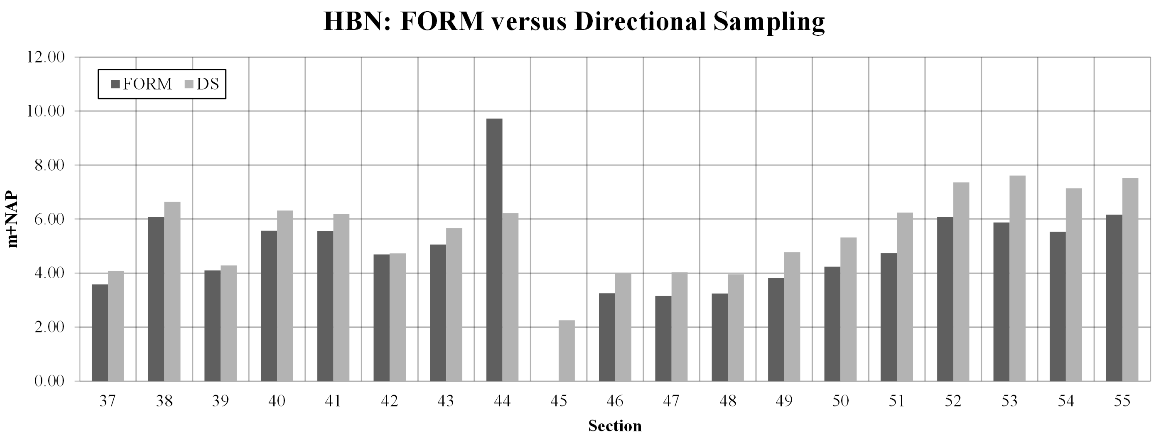

Figure 5.9: HBN computation using FORM and directional sampling.

In the following section the influence of additional uncertainties will be highlighted during extreme return period. The convergence problems as explained on this page are more of an issue when looking at extreme return periods. Therefore, directional sampling computations have been executed in the next analysis.

INFLUENCE AT AN EXTREME RETURN PERIOD

In the following section the validity of the robustness surcharge as proposed by den Bieman and Smale (2014) will be sought out for extreme return periods. MHW and the required crest height due to overtopping will be evaluated. A topic of interest will be the whether the contribution of additional model and additional statistical uncertainties will be the same as compared to the findings of the previous section where a return period of 4,000 years is considered. Extreme return periods are used in this analysis since the design rules of den Bieman and Smale (2014) lead to do so. As shown in section 4.1.5, imposing a heavy requirement at section level can lead to a desired failure probability of the dike trajectory.

The special version of the PC-Ring model will again be used to compute the desired analysis. This time, convergence problems impose a non-negligible role during a FORM analysis. For this reason, all computation have been done using the directional sampling method. It is chosen to use 10,000 samples for the analysis. This leads to a sufficiently stable results. A list of parameters is presented below.

Table 5.10: List of parameters

parameter	value
return period	187,500
β	4.403
N	3
$\omega_{overtopping}$	0.24
n samples	10,000

The figure 5.10 shows the results of a MHW level computation which accompanies the projected overtopping conditions. Again a clear distinction between the Marker Lake and the IJssel Lake is easily recognized. The average gain on the Marker Lake is 37 centimetres, on the IJssel Lake this is 43 cm. For the 8_3 trajectory, the average is 40 centimetres. Surprisingly, the proposed +40 cm robustness surcharge on MHW level by den Bieman and Smale (2014) is still in reasonable range. However, it must be noted that now the surcharge only provides for the chosen accounted for uncertainties. There is no room for other non-accounted for uncertainties.

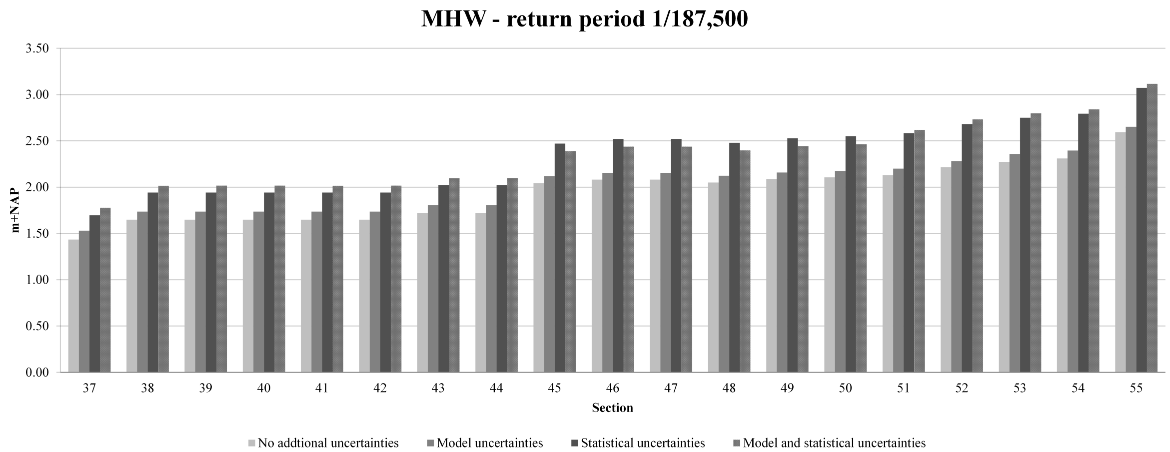

Figure 5.10: Influence of additional uncertainty on MHW-levels for dike trajectory 8_3 for 1/187,500 yrs conditions.

The relative influence shows that the contribution of the additional model and statistical uncertainties combined is about the same for both the Marker Lake and IJssel Lake, namely a factor of 1.22. This differs from the relative influence at smaller return periods from figure 5.7 whereas the influence is larger on the Marker Lake. Looking at the marginal contributions, it shows that the statistical uncertainties cause a greater influence then the model uncertainties.

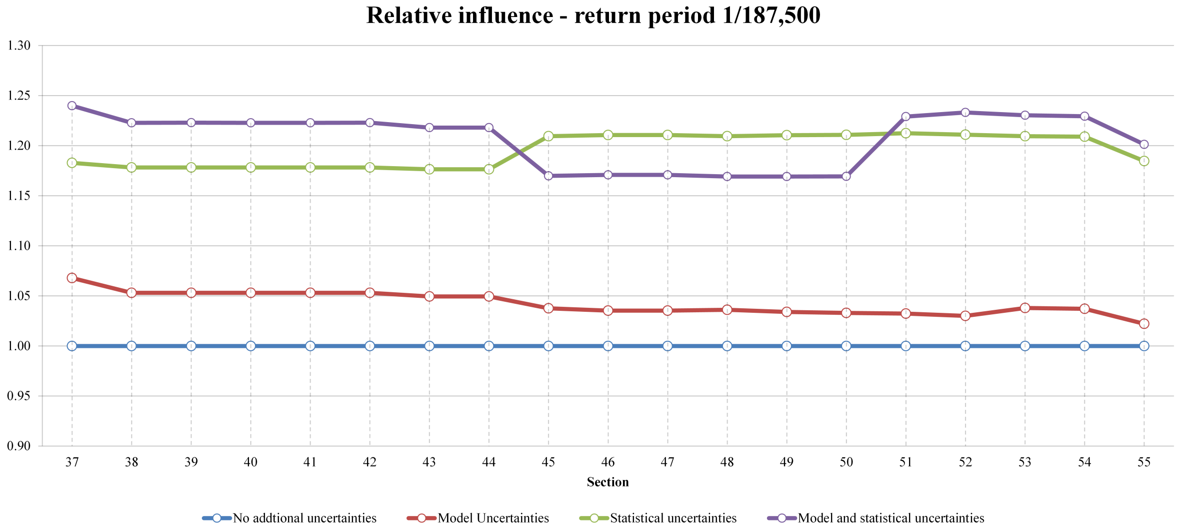

Figure 5.11: Relative influence for MHW, return period = 187,500 using Directional sampling.

A remarkable observation can be deduced from figure 5.10 and figure 5.11. At dike sections 45 to 51, the additional statistical influence actually surpasses the combined instant wherein additional statistical and model influence is accounted for. This is counter-intuitive, one would suspect the MHW level to rise with the inclusion of more uncertainties.

This effect is further analysed for dike section 50 where this effect is clearly evident. Alpha values retained during the numerical computation can give an indication which random variable is causing this effect. Two computations are shown in table 5.11. The two left columns show the alpha values and the design points of the random variables for the computation with solely additional statistical uncertainties, the two right columns show the results of the combined computation.

Table 5.11: Dike section 50 [8005006], dominant wind direction = 300° degrees, return period 187,500 years

Random variable	Additional statistical		Additional model and statistical	
	α-value	design point	α-value	design point
Error in local water depth			-0.1675	0.12
Wind speed	-0.8668	39.61	-0.8616	37.61
Lake level	-0.3083	0.09	-0.4026	0.06
Wind speed uncertainty	-0.3859	1.16	-0.2564	1.11
Lake level uncertainty	-0.0680	0.03	0.0406	-0.02
MHW		2.46		2.57

From table 5.11 can be observed that the design value for lake level uncertainty is positive in the additional statistical computation and is negative in the additional model and statical computation. This indicates that this random variable has a reducing effect on the MHW level in the second latter computation. Excluding this random variable would results in the expected assumption wherein the expected MHW level increases with the inclusion of more uncertainties.

Why the lake level uncertainty causes the MHW to drop when lake level model uncertainty is added in the computation remains unknown. Further investigation on how this random variable is included in the PC-Ring model is needed to find out what is causes this effect. Since the computational core of the PC-Ring model is not readily accessible this will not be further analysed at this moment.

Another interesting remark is that the lake level in both instances is about 0.0 meter, this indicates that the enhanced water level conditions, depicted in figure 5.10, is mostly due to wind set-up.

SENSITIVITY ANALYSIS OF ADDITIONAL STATISTICAL UNCERTAINTIES

Figure 5.11 shows that the added statistical uncertainties have a significant impact on the calculated MHW level. The alpha-values of table 5.11 suggest that wind speed is one of the main contributors to this. In the following section the significance of the additional statistical uncertainty will be analysed. For this, the former analyses in which only statistical uncertainties are added will be decomposed.

The additional statistical uncertainties for the IJssel Lake area are the statistical uncertainty for lake level and wind speed. These are incorporated in the model as shown in table 5.12. A computation will be performed using PC-Ring in which solely one additional statistical uncertainty will be added. In so, the influence of each uncertainty can be evaluated.

Table 5.12: List of additional statistical uncertainties

Stochastic	Value
Wind speed uncertainty	N[0;0.085]
Lake level uncertainty	N[0;0.10]

Below in table 5.13 the results are presented of the analysis in which the PC-Ring model is used to gradually add statistical uncertainty. The figures show that the influence of added wind uncertainty has a dominant influence on the MHW-level.

Table 5.13: Result of MHW computation

Dike section	50 MHW [m+NAP]	53 MHW [m+NAP]
No additional uncertainties	2.13	2.27
Only additional lake level uncertainty	2.14	2.31
Only additional wind speed uncertainty	2.53	2.80
additional lake level and wind speed uncertainty	2.55	2.76

In table 5.14 the results are presented from the overtopping computation. In this computation both additional uncertainties have an significant impact on the hydraulic loading condition. It is expected that the formerly explained computational difficulties, highlighted on the previous page, influence the result to some extent. However, increasing hydraulic condition due to the impact of additional wind uncertainty is undeniable.

Table 5.14: Results of overtopping computation

	hloc	Hs	Ts	Crest h	Wind dir.
Dike section 53;8005009					
No additional uncertainties	2.23	2.66	7.39	8.48	300
Only additional lake level uncertainty	2.26	2.60	7.35	8.18	300
Only additional wind speed uncertainty	2.16	2.64	7.35	8.23	300
Additional lake level and wind speed uncertainty	1.81	2.39	7.02	9.50	300
Dike section 50;8005006					
No additional uncertainties	1.99	2.02	6.47	6.07	300
Only additional lake level uncertainty	2.22	2.3	6.37	6.64	300
Only additional wind speed uncertainty	2.38	2.45	6.57	7.40	300
Additional lake level and wind speed uncertainty	1.75	2.08	6.20	6.55	300

5.2.4. EVALUATION & DISCUSSION

EVALUATION

In the previous section a more detailed approach has been carried out in respect to the robustness surcharge as proposed by den Bieman and Smale (2014). Following literature from Rijkswaterstaat (2014a) and Nicolai et al. (2010), the robustness surcharge is decomposed into additional model and statistical elements. Two cases have been considered for the dike trajectory 8_3 in which conventional normative conditions and extreme normative conditions are analysed.

From the analysis can be seen that the proposed robustness surcharge by den Bieman and Smale (2014) is fairly accurate for the MHW level, even for normative conditions of extreme return periods. This, however, can only be justified if one assumes the considered uncertainties to be the only normative uncertainties. Assuming that they are, then the robustness surcharge is slightly conservative for conditions with a return periods of several thousands years. The needed robustness would be on average +28cm as opposed to +40cm. At extreme return periods the robustness charge is almost on par with the needed surcharge following from the decomposed approach. In the analysis a needed +43cm of robustness is obtained. In both cases the offset to the robustness surcharge of +40cm is only in the order of centimetres.

The relative influence of the combined additional model and statistical uncertainties show that an multiplication factor might be more applicable instead of an additive factor. Although a multiplication factor for a water level is purely emperical of nature, in all cases a multiplication factor of 1.2 seems very suitable. The multiplication factor holds for conventional and extreme return periods scenarios, therefore a linear dependant relation can be observed. This might not be the case for even more extreme return periods, however that can be considered an irrelevant time span. Also, the factor is deduced for the coastline of Flevoland, the influence might be different at other (lake) location.

The stochastic parameter for wind uncertainty, N[0;0.0085] is acquired from an analysis by (de Deugd, 2007). However, when consulting this document a proper description is lacking. Since this stochastic parameter has a prominent role in the realization in the robustness surcharge by den Bieman and Smale (2014), a proper substantiation is required.

A technical notion: the FORM computation underestimates in several cases. However, error seems to be systematic, therefore R-Factor remains constant.

DISCUSSION

The performed analysis show great reassertion of the proposed robustness surcharge by den Bieman and Smale (2014). However, this is hardly a coincidence since the same influential factors are analysed. It does show that, although, only a few locations were considered in the initial determination of the robustness surcharge, the system (and model) does not show a large variability which might lead to disproportional results.

This analysis only looks into the uncertainties which are deemed to have the most significant effect on hydraulic load conditions. The derived figure therefore only covers those included. As can be seen from the analysis in which random variables are gradually added in the computation, the outcome is severely influenced by the subjectively chosen parameters. Other additional uncertainties might also have an effect on the hydraulic load. Therefore, it is sensible to acknowledge the fact that the real value can be of a higher value then the one derived. Then again, one has to determine if it is preferable to address this kind of robustness factor on top of a robustness factor.

During the process of the analysis, several problems have occurred. Some being of a computational nature, others related to the human mindset whether a modelled result is acceptable or not. In the case where the retained result do not coincide with a certain expectancy, it becomes complicated to give an accurate answer. Hypothetically, three solutions remain: the user would be right to reject the model's outcome, the user would be wrong to reject the model's outcome, or both the expectancy and the model's outcome are wrong. Considering the shortcomings of the user and the model, it would be unwise to solely depend on one, whereas the one and only truth is certainly neither of them. In that sense it would be logical to define a range of possibility. But as this is done, the possibility on a possibility is addressed.

6 Inclusion of wind uncertainty in a model

In the previous analysis it has become evident that the uncertainty associated with statical wind data is a great contributor to enhanced loading conditions. This observation could be made during the decomposition of additional uncertainties. Leading up to a new WTI of 2017, the desire is expressed for the inclusion of model and statistical uncertainties. It then follows that the attribution of such an influential parameter needs to be sound and transparent. For this purpose it will be sought out how wind uncertainty can be included in a model, and what the significance is of inclusion as loading conditions will be elaborated.

The uncertainty will be modelled based on a set of Generalized Pareto Distributions (GPD). Furthermore, a smart implementation technique will be evaluated wherein an integrated version of the wind speed exceedance probability will be used. The analysis concludes with the practical implementation of alternative statistical wind speed data in the Hydra-Zoet model.

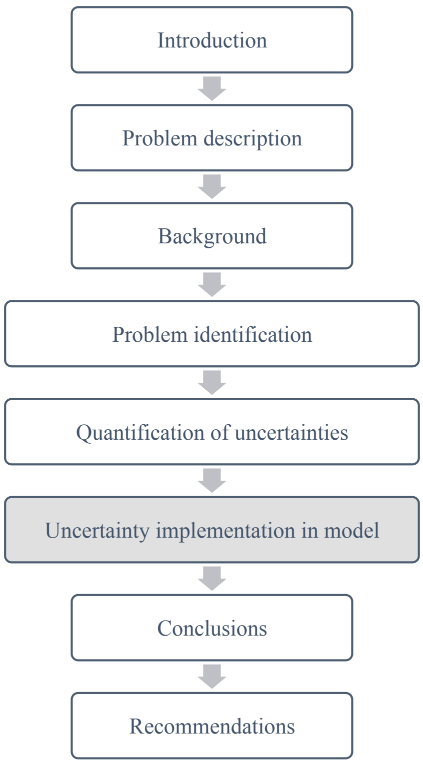

Figure 6.1: Structure of report, item "Uncertainty implementation".

6.1. BACKGROUND

UNCERTAINTIES AMONG WIND CONDITIONS

Several uncertainties in the determination and use of design wind conditions can be identified. The uncertainties can be distinguished by the classification explained on page 24. To recapitulate, these are inherent uncertainties and epistemic uncertainties (Stijnen et al., 2008).

Because wind is a natural dynamic process, it is impossible to predict wind in time and space. These uncertainties are moreover referred to as the aleatory uncertainties or inherent uncertainties.

The uncertainties of the other category, the epistemic, are attributable to the lack of knowledge. This can in turn be divided into statistical and model uncertainty. In many cases, a flood defence needs to be designed in such a way that it can cope with a scenarios that have never been encountered before. For the determination of the loading conditions for such a scenario, a statistical analysis can be carried out. To this end, the Rijkoort-Weibull model (RW-model) is used in many common dike design models to elaborate normative wind conditions..

In the RW-model, wind data from 12 different Dutch measurement stations is gathered. Several statistical techniques are performed to generate the exceedance probability of extreme wind speed scenarios. Some of these techniques are the estimation of probability distributions, up-scaling and smoothing techniques. In many of these techniques an error is introduced. A number of shortcomings associated with these techniques are investigated by Smits (2001). Smits also states that systematic deviations exist between the measurement stations.

Alternative extreme wind models have been investigated by Smits (2001) and the Hydra-project (Stijnen et al., 2008). Unfortunately, none of the alternative models performed consistently well for all test cases. Therefore, the RW-model is still used in the current Hydra models and PC-ring model.

The above mentioned models for the processing of wind data are referred to as statistical uncertainty of wind. Subsequently, the generated wind speed data of such an analysis is used in dike design and assessment models. In these models, several simplifications are made with respect to the wind conditions. In this sense, model uncertainty is introduced. For instance, wind is usually presumed to have a uniform wind field and to be of a constant value (the average value) over a certain amount of time. Dike design models such as PC-Ring and Hydra-Zoet also partition the wind into several wind directions to simplify the computation. Namely, the PC-Ring uses 14 wind directions whereas Hydra-Zoet uses 16. In reality, wind does not restrict itself to a certain wind direction, time or wind speed value.

INCLUSION OF UNCERTAINTY

As has been extensively dealt with in the previous chapter, Nicolai et al. (2010) have introduced a method in which statistical and model uncertainties are addressed. For wind speed in particular, the observed value of wind speed exceedance probability from the RW-model is manipulated by a multiplication factor of $N[1; 0.0085]$. Unfortunately, a description of this random variable is lacking in literature. The theoretical framework of the analysis by Nicolai et al. (2010) is clear and logical, but it can be concluded that the PC-Ring model is not a suitable model for the handling of additional random variables which are not included in the original model.

Continuous development in means of uncertainty inclusion has led to a research by Geerse and Wojciechowska (2014) wherein modelling of water level uncertainty is further elaborated. In this analysis, statistical water level data of several coastal and lake measurement stations are analysed and enhanced with uncertainty. The goal of the research is to define 95% confidence intervals for several water level measurement stations across the Netherlands.

The relation between exceedance probability and water level is expressed by a function which can give a good fit to statistical data. Documented are exponential, generalized Pareto and Weibull distributions. Uncertainty is modelled as an varying shape parameter. The parameter is assumed to be normally distributed and therefore characterized by its mean and standard deviation. For coastal measurement stations, an initial boot strap method is applied to derive a suitable standard deviation for Hoek van Holland, whereafter a transformation is applied for other stations. For lake measurement stations, a standard deviation is established based on expert judgement. By integrating with respect to the normally distributed scaler parameter, an alternative expression for water level including uncertainty can be derived.

The report succeeds to establish 95% confidence intervals for each measurement station. The integrated expressions for water level including uncertainties show an increase in expected water levels. For coastal stations this is 1.1 to 1.3 metres at a yearly frequency of 10^{-5}. For lake stations this is 0.35 metres at the same frequency. The report stresses that the results are highly influenced by subjective choices.

The framework of the analysis by Geerse and Wojciechowska (2014) is very suitable for the purposes of the forthcoming analysis. Therefore, this will be used as the base for this study. Details will be explained in the next section.

6.2. PROPOSED MODEL

The previous section shows that there is a tendency to include wind uncertainty in future modelling and designs, however a well thought out or transparent application is still lacking. It is because of this reason that in the following chapter the emphasize will be put on how to include statistical wind uncertainty in a model.

The implementation attempts to comply with several principles. The first will be an increasing uncertainty for less frequent cases. This principle is chosen since the accuracy of generated wind speeds for infrequent cases can be argued as prone for errors during the statistical processing or errors during measurements. An error in measurements can have an effect on a derived relation, whereas statistical processing errors are inherent to statistical analysis such as the RW-model. Considering these two causes, a deviation of generated infrequent wind conditions is very likely to be true.

A suitable function is needed which fits the statistical wind data and allows for easy modification so that 'the increasing uncertainty' principle can be modelled. A function that complies with these requirements is the generalized pareto distribution (GPD) function, modified to the location-scale family. The function is expressed below, wherein u is wind speed in [m/s]:

$$g(x,\mu,\sigma,\gamma) = \begin{cases} \dfrac{1}{\sigma}\left(1+\gamma\dfrac{u-\mu}{\sigma}\right)^{-\frac{1}{\gamma}-1} & \text{for} \quad \gamma \neq 0 \qquad (6.1) \\ \dfrac{1}{\sigma}\exp\left(-\dfrac{u-\mu}{\sigma}\right) & \text{for} \quad \gamma = 0 \qquad (6.2) \end{cases}$$

$$P(x,\mu,\sigma,\gamma) = \begin{cases} 1-\left(1+\gamma\dfrac{x-\mu}{\sigma}\right)^{-1/\gamma} & \text{for} \quad \gamma \neq 0 \qquad (6.3) \\ 1-exp\left(-\dfrac{x-\mu}{\sigma}\right) & \text{for} \quad \gamma = 0 \qquad (6.4) \end{cases}$$

The statistical data of wind shows exponential properties, as will be seen shortly. The GPD function allows for such a description. If the location parameter μ and the scale parameter σ are allowed to be deterministic, then the a stochastic shape parameter γ can be introduced to model uncertainty. For example, a normally distributed stochastic with $w(\gamma) = N[\mu_w;\sigma_w^2]$ and deterministic values μ_0 and σ_0. A shape parameter γ larger then zero will result in a heavy-tailed distribution, whereas a shape parameter smaller then zero results in a short-tailed distribution.

The implications will be evaluated for a yearly frequency of 0.00001. This extreme infrequent case is chosen because the Rijkswaterstaat (2014c) enforce designers to reckon with these kind of cases.

BOUNDARY CONDITIONS OF MODEL

STATISTICAL WIND DATA

This model will step in between the statistical wind data retained from the RW-model and the elaboration of dike design. The designation of appropriated loading conditions is fundamentally key for the design of a dike, since it determines the physical properties and subsequently the cost. The validation of the RW-model will not be further highlighted. As stated in section 6.1, the RW-model is still broadly used, and is arguably the most consistent and readily available statistical wind data there is for this analysis. Therefore, the statistical wind data of the RW-model will be used as input for the model.

As in the Hydra-Zoet model or PC-ring model, the wind speed probability of the RW-model are presented in a table rather then a formula. An interpolation or curve fitting can be applied to these data points.

To simplify the model, solely one wind direction will be elaborated. Tables of the RW-model are divided into several wind directions as can be seen in figure 6.2. The considered station will be that of Schiphol airport. The wind direction probability is denoted by $P(r)$.

STANDARD DEVIATION OF WIND SPEED

For the modelling of the statistical wind uncertainty, an estimation is made for the deviation of wind speed. A report by Meermans (1999) states that a previous study carried out by Wieringa & Rijkoord from 1983 has estimated the statistical uncertainty of wind to be $\sigma = 2m/s$ at a 0.01 per year frequency. Subsequently, Meermans (1999) believes that the uncertainty will increase up to $\sigma = 3m/s$ for a 0.001 per year frequency. In so, the uncertainty increases by 1.5 for a ten times lesser frequency.

As the forthcoming model moreover tries to emphasize the way of implementing uncertainty, an estimate will be done for the amount of uncertainty of a wind speed with a frequency of 0.0001 per year. This estimate will be done using literature and expert judgement. Meermans (1999) proposes a normally distributed multiplication factor for wind speed with a deviation of $\sigma = 3m/s$ for an 0.001 per year frequency. His approach would lead to a $\sigma = 4.5m/s$ at a frequency of 0.0001 per year. This seems like a rather large value. In consultation with Chris Geerse (HKV Lijn in Water), it was decided to model the statistical wind uncertainty as $4 * \sigma = 10 - 14m/s$ at an 0.0001 per year frequency. This definition will suffice for the sensitivity analysis at hand.

AMOUNT OF 12-HOURS INTERVALS IN WINTER HALF YEAR

The retained RW-model wind speed exceedance probability tables from Hydra-Zoet 1.6.3 are tuned to a 12-hours interval. To convert this to a yearly frequency the figures need to be multiplied by the amount of 12-hours intervals in a winter half year. It will be assumed that a winter half year consists out of $N = 2 * 180 = 360$ intervals.

INTERESTING NOTION IN ANALYSIS

The exceedance frequency of a certain wave height $F(H^* > H)$ integrated with respect to 'uncertainty' γ is expressed as equation (6.5). The equation shows a double integration. Conventionally, one could first integrate with respect to u and then with respect to γ. This can be denoted as the integration order $dud\gamma$. If one would approach the equation as a iterated integral, the order of integration can be changed. Subsequently, one can first integrate with respect to γ. The integration order changes to $d\gamma du$.

In the following elaboration an subscript of io is used to emphasize the fact that parameter γ has been integrated, and thus an integrated function influenced by uncertainty is derived.

There are no difficult limits in the corresponding integrations with respect of u and γ. The iterated integral simply iterates one-variable integration two times. Therefore, the following change in integration order can be made:

The first option is to integrate in the order of $dud\gamma$:

$$F_{io}(H^* > H) = N \cdot P(r) \int \int_{u_0}^{\infty} w(\gamma) g(u|r,\gamma) dud\gamma \tag{6.5}$$

$$= N \cdot P(r) \int w(\gamma) \int_{u_0}^{\infty} g(u|r,\gamma) dud\gamma \tag{6.6}$$

$$= N \cdot P(r) \int w(\gamma) P(U > u|r,\gamma) d\gamma \tag{6.7}$$

$$= N \cdot P(r) \cdot P_{io}(U > u|r) \tag{6.8}$$

Second option is to integrate in the order of $d\gamma du$:

$$F_{io}(H^* > H) = N \cdot P(r) \int \int_{u_0}^{\infty} w(\gamma) g(u|r,\gamma) dud\gamma \tag{6.9}$$

$$= N \cdot P(r) \int_{u_0}^{\infty} \int w(\gamma) g(u|r,\gamma) d\gamma du \tag{6.10}$$

$$= N \cdot P(r) \int_{u_0}^{\infty} g_{io}(u|r) du \tag{6.11}$$

$$= N \cdot P(r) \cdot P_{io}(U > u|r) \tag{6.12}$$

In terms of numerical modelling, this would imply that a single modified expression for wind speed probability including uncertainty g_{io} can be identified by first integrating with respect to γ instead of having to integrate γ for every integration with respect to u. In doing so, the process can be streamlined and the computation time reduced. Validity of this statement will be sought in the forthcoming analysis.

6.3. ELABORATION OF ANALYSIS

6.3.1. CURVE FITTING FOR BASE DISTRIBUTION

For the model, a function must be retained which describes the relation between wind speed and the probability of occurrence.

CONSIDERED WIND DIRECTION

Statistical wind data is provided by the RW-Model and is retained from the Hydra-Zoet 1.6.3 model. This means that the commonly used wind data from Schiphol is considered. The data is divided in 16 wind directions. Each wind direction has its own probability of occurrence $P(r)$. Since hydraulic structures are generally designed for the most severe conditions, the wind direction which has the highest probability for high wind speeds will be elaborated. In figure 6.2, a modified table retained form Hydra-Zoet 1.6.3 can be seen which shows the exceedance frequency of wind speeds. A colour scale has been applied to visualize the most probable high wind speeds.

U>u	NNO	NO	ONO	O	OZO	ZO	ZZO	Z	ZZW	ZW	WZW	W	WNW	NW	NNW	N
0	4.52E-02	5.57E-02	6.44E-02	5.75E-02	4.14E-02	4.44E-02	5.82E-02	7.45E-02	9.07E-02	9.60E-02	9.09E-02	7.59E-02	5.75E-02	5.08E-02	4.95E-02	4.71E-02
1	4.51E-02	5.56E-02	6.44E-02	5.75E-02	4.14E-02	4.44E-02	5.82E-02	7.45E-02	9.07E-02	9.60E-02	9.07E-02	7.57E-02	5.75E-02	5.08E-02	4.94E-02	4.55E-02
2	4.43E-02	5.49E-02	6.41E-02	5.68E-02	4.08E-02	4.38E-02	5.76E-02	7.35E-02	9.01E-02	9.55E-02	9.00E-02	7.54E-02	5.71E-02	5.02E-02	4.86E-02	4.47E-02
3	4.14E-02	5.29E-02	6.22E-02	5.35E-02	3.75E-02	4.15E-02	5.50E-02	7.06E-02	8.81E-02	9.34E-02	8.78E-02	7.37E-02	5.50E-02	4.84E-02	4.61E-02	4.04E-02
4	3.52E-02	4.85E-02	5.72E-02	4.70E-02	3.12E-02	3.46E-02	4.86E-02	6.39E-02	8.38E-02	8.85E-02	8.33E-02	7.03E-02	5.21E-02	4.47E-02	4.10E-02	3.27E-02
5	2.86E-02	4.22E-02	4.95E-02	3.80E-02	2.30E-02	2.63E-02	3.87E-02	5.34E-02	7.45E-02	8.08E-02	7.85E-02	6.60E-02	4.76E-02	4.00E-02	3.48E-02	2.61E-02
6	2.11E-02	3.46E-02	4.13E-02	2.96E-02	1.58E-02	1.84E-02	2.99E-02	4.31E-02	6.49E-02	7.28E-02	7.20E-02	5.96E-02	4.34E-02	3.55E-02	2.97E-02	1.98E-02
7	1.52E-02	2.66E-02	3.30E-02	2.07E-02	9.49E-03	1.20E-02	2.12E-02	3.13E-02	5.46E-02	6.36E-02	6.38E-02	5.34E-02	3.84E-02	2.94E-02	2.37E-02	1.48E-02
8	9.82E-03	1.92E-02	2.45E-02	1.29E-02	4.77E-03	6.67E-03	1.40E-02	2.17E-02	4.37E-02	5.56E-02	5.64E-02	4.56E-02	3.22E-02	2.39E-02	1.78E-02	1.01E-02
9	6.79E-03	1.31E-02	1.75E-02	7.42E-03	2.22E-03	3.59E-03	8.33E-03	1.57E-02	3.38E-02	4.67E-02	4.79E-02	3.78E-02	2.66E-02	1.91E-02	1.17E-02	6.69E-03
10	4.71E-03	8.47E-03	1.15E-02	4.44E-03	8.12E-04	1.73E-03	4.45E-03	1.03E-02	2.60E-02	3.93E-02	4.09E-02	3.16E-02	2.09E-02	1.47E-02	8.47E-03	4.76E-03
11	2.69E-03	4.49E-03	6.35E-03	2.72E-03	4.56E-04	7.33E-04	2.28E-03	6.79E-03	1.78E-02	3.15E-02	3.23E-02	2.56E-02	1.65E-02	1.10E-02	5.35E-03	3.53E-03
12	1.43E-03	2.49E-03	3.10E-03	1.45E-03	3.61E-04	4.62E-04	1.08E-03	4.33E-03	1.21E-02	2.50E-02	2.60E-02	2.05E-02	1.25E-02	7.73E-03	3.67E-03	2.41E-03
13	7.65E-04	1.00E-03	8.70E-04	6.27E-04	2.49E-04	3.08E-04	7.57E-04	2.44E-03	8.61E-03	1.77E-02	1.93E-02	1.51E-02	8.23E-03	5.29E-03	2.26E-03	1.36E-03
14	3.36E-04	4.15E-04	4.89E-04	3.34E-04	1.87E-04	2.40E-04	3.78E-04	1.48E-03	5.36E-03	1.31E-02	1.34E-02	1.01E-02	5.38E-03	3.04E-03	1.69E-03	4.52E-04
15	1.68E-04	2.10E-04	2.52E-04	1.63E-04	9.12E-05	1.09E-04	2.56E-04	9.09E-04	3.59E-03	9.79E-03	9.02E-03	6.52E-03	3.56E-03	2.34E-03	1.10E-03	2.49E-04
16	7.87E-05	9.86E-05	1.20E-04	7.36E-05	3.72E-05	4.43E-05	1.30E-04	5.19E-04	2.15E-03	6.25E-03	6.12E-03	4.56E-03	2.49E-03	1.45E-03	6.24E-04	1.32E-04
17	3.52E-05	4.30E-05	5.27E-05	3.03E-05	1.38E-05	1.64E-05	6.35E-05	2.76E-04	1.30E-03	4.22E-03	4.31E-03	3.06E-03	1.96E-03	8.74E-04	3.63E-04	6.74E-05
18	1.49E-05	1.73E-05	2.14E-05	1.14E-05	4.64E-06	5.60E-06	2.93E-05	1.39E-04	7.42E-04	2.69E-03	2.57E-03	1.87E-03	1.18E-03	4.68E-04	2.21E-04	3.31E-05
19	6.11E-06	6.52E-06	7.99E-06	3.92E-06	1.43E-06	1.76E-06	1.30E-05	6.54E-05	4.70E-04	1.73E-03	1.55E-03	1.09E-03	6.85E-04	3.14E-04	1.30E-04	1.58E-05
20	2.42E-06	2.27E-06	2.78E-06	1.24E-06	4.02E-07	5.02E-07	5.49E-06	2.92E-05	2.87E-04	9.60E-04	9.09E-04	7.59E-04	3.88E-04	2.07E-04	7.43E-05	7.26E-06
21	9.32E-07	7.41E-07	8.95E-07	3.57E-07	1.04E-07	1.33E-07	2.21E-06	1.22E-05	1.69E-04	6.91E-04	4.15E-04	4.69E-04	2.62E-04	1.34E-04	4.10E-05	3.21E-06
22	3.50E-07	2.25E-07	2.68E-07	9.43E-08	2.47E-08	3.21E-08	8.50E-07	4.83E-06	9.61E-05	3.67E-04	2.68E-04	3.13E-04	1.74E-04	8.59E-05	2.23E-05	1.37E-06
23	1.28E-07	6.30E-08	7.41E-08	2.31E-08	5.43E-09	7.16E-09	3.10E-07	1.80E-06	5.30E-05	2.63E-04	1.68E-04	2.05E-04	1.13E-04	5.44E-05	1.18E-05	5.56E-07
24	4.57E-08	1.66E-08	1.91E-08	5.19E-09	1.12E-09	1.47E-09	1.07E-07	6.27E-07	2.84E-05	6.61E-05	1.04E-04	1.31E-04	7.25E-05	3.40E-05	6.14E-06	2.18E-07
25	1.60E-08	4.04E-09	4.54E-09	1.08E-09	2.16E-10	2.77E-10	3.47E-08	2.06E-07	1.47E-05	3.82E-05	6.30E-05	8.35E-05	4.58E-05	2.10E-05	3.14E-06	8.20E-08
26	5.47E-09	9.08E-10	9.98E-10	2.08E-10	3.85E-11	4.76E-11	1.06E-08	6.33E-08	7.42E-06	2.17E-05	3.74E-05	5.18E-05	2.84E-05	1.28E-05	1.57E-06	2.95E-08
27	1.81E-09	1.91E-10	2.05E-10	3.75E-11	6.63E-12	7.33E-12	3.04E-09	1.82E-08	3.63E-06	1.22E-05	2.19E-05	3.19E-05	1.73E-05	7.68E-06	7.68E-07	1.01E-08
28	5.79E-10	3.74E-11	3.95E-11	7.02E-12	1.17E-12	1.08E-12	8.15E-10	4.88E-09	1.73E-06	6.67E-06	1.26E-05	1.94E-05	1.04E-05	4.54E-06	3.66E-07	3.34E-09
29	1.80E-10	6.69E-12	6.70E-12	0.00E+00	0.00E+00	0.00E+00	2.05E-10	1.23E-09	7.99E-07	3.61E-06	7.16E-06	1.16E-05	6.16E-06	2.65E-06	1.71E-07	1.05E-09
30	5.38E-11	5.85E-13	2.47E-13	0.00E+00	0.00E+00	0.00E+00	4.81E-11	2.88E-10	3.59E-07	1.93E-06	4.01E-06	6.91E-06	3.61E-06	1.52E-06	7.78E-08	3.17E-10
31	1.52E-11	4.36E-14	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.05E-11	6.30E-11	1.56E-07	1.01E-06	2.20E-06	4.04E-06	2.07E-06	8.59E-07	3.45E-08	9.10E-11
32	4.33E-12	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	2.07E-12	1.24E-11	6.59E-08	5.19E-07	1.19E-06	2.35E-06	1.17E-06	4.76E-07	1.50E-08	2.60E-11
33	5.29E-13	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	2.57E-13	1.54E-12	2.71E-08	2.65E-07	6.38E-07	1.34E-06	6.50E-07	2.60E-07	6.29E-09	3.17E-12
34	6.65E-15	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.08E-08	1.32E-07	3.36E-07	7.59E-07	3.57E-07	1.40E-07	2.59E-09	3.98E-14
35	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	4.15E-09	6.51E-08	1.75E-07	4.27E-07	1.93E-07	7.42E-08	1.04E-09	0.00E+00
36	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.55E-09	3.16E-08	9.02E-08	2.35E-07	1.03E-07	3.85E-08	4.05E-10	0.00E+00
37	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.61E-10	1.50E-08	4.58E-08	1.29E-07	5.41E-08	1.97E-08	1.54E-10	0.00E+00
38	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.98E-10	7.03E-09	2.31E-08	6.94E-08	2.80E-08	9.91E-09	5.60E-11	0.00E+00
39	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	6.63E-11	3.22E-09	1.15E-08	3.69E-08	1.43E-08	4.92E-09	2.03E-11	0.00E+00
40	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	2.15E-11	1.46E-09	5.63E-09	1.94E-08	7.19E-09	2.39E-09	0.00E+00	0.00E+00
41	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	7.19E-12	6.44E-10	2.74E-09	1.00E-08	3.58E-09	1.14E-09	0.00E+00	0.00E+00
42	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	3.67E-12	2.80E-10	1.32E-09	5.11E-09	1.75E-09	5.44E-10	0.00E+00	0.00E+00

Figure 6.2: Wind speed exceedance frequency [1/year]. Modified table retained from Hydra-Zoet 1.6.3.

The shoreline of Flevoland can be attacked by waves ranging from North-East to South-West (moving anti-clockwise). Other wind directions would cause offshore developing wind waves and can therefore be neglected. Out of the considered wind directions, the most probable high wind speeds are to be expected from the West. For this reason, western wind will be used in the following analysis.

BASE DISTRIBUTION FOR WIND-PROBABILITY RELATION

As explained in section 6.2, a base distribution will be used to describe the relation between wind speed and exceedance probability. The table in figure 6.2 provides a data set on which a curve fitting technique can be performed.

A simplified Generalized Pareto distribution (GPD) function has been fitted to the wind data. Simplified because γ is chosen to be equal to zero. In doing so the function is equal to an exponential function as shown in equation 6.4.

A fit for the whole range of wind speeds, 0 to 42 [m/s], does not give a good fit. However, the goal of the fitting is to estimate the probability at high wind speed values in particular. Subsequently, low wind speed values are neglected. Using a threshold value of 27 [m/s], a curve fitting with a smaller error can be realised. This means wind data from 28 to 42 [m/s] is used in the curve fitting. The y-axis is logarithmically scaled in figure 6.3.

The computations in this chapter are performed using Python. For the curve fitting technique shown below, the least squares method has been applied. Figure 6.3 is the result of the curve fitting method. The location parameter [μ] and location parameter [σ] are estimated to be $\mu = 11.80$ and $\sigma = 1.96$. A measure for the goodness of fit is the sum of squared residuals (SSR). As expressed in equation (6.13), wherein X is the true value and $\bar{X}$ is the estimated value. In this case this the $SSR = 2.53e-11$ for the fitted data, which is considered to be an acceptable error.

$$SSR = \sum_{i=1}^{n} \hat{e}_i = \sum_{i=1}^{n} X_i - \bar{X}_i \tag{6.13}$$

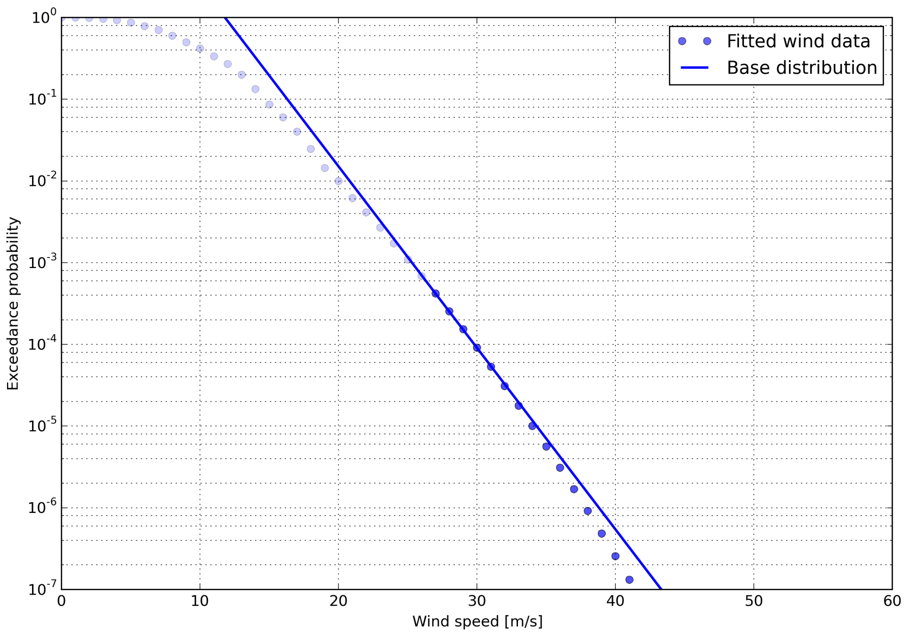

Figure 6.3: Exponential fit to statistical wind data.

From figure 6.3 can be seen that the GPD fit slightly overestimates the probability of the wind speed in the very low frequency range. The area of interest in this analysis is between the 10^5 and 10^6 exceedance probability (in a 12-hour interval). The error in this range is of the order 0.5 to 1.0 [m/s] (when visually observed). A GPD function wherein $\gamma \neq 0$ might improve the approximation. For the time being, the established relation is considered sufficient.

6.3.2. MODELLING OF UNCERTAINTY

As stated in section 6.2, it is strived to model uncertainty in such a way that the influence of uncertainty becomes more significant when higher wind speeds are considered. The framework by Geerse and Wojciechowska (2014) will be applied. The expression of the base distribution derived in the previous section will be manipulated in such a way that a deviation can be realised. To do this the scale parameter $[\gamma]$ will be varied.

The GPD function from equation 6.3 is characterized by 3 parameters, namely the location parameter $[\mu]$, the scale parameter $[\sigma]$ and the shape parameter $[\gamma]$. If the location and scale parameter are chosen to be equal to those of the previously retained base distribution of section 6.3.1, then this gives the opportunity to model the uncertainty as a varying shape parameter. This parameter is assumed to be normally distributed and is characterized by its mean and deviation. By varying the shape parameter γ the tail of the distribution can be influenced. For instance:$\gamma > 0$ will result in a heavy tail, whereas a value of $\gamma < 0$ will result in a short tail.

$$\mu_{0.GPD} = \mu_{base} = 11.80 \tag{6.14}$$

$$\sigma_{0.GPD} = \sigma_{base} = 1.96 \tag{6.15}$$

$$w(\gamma) = N[\mu_w, \sigma_w^2] \tag{6.16}$$

A designation of (un)certainty influence can be the 95% confidence interval, an interval for which can be said that it is 95% certain that the value for $P(U > u)$ will be within this interval for an arbitrary instance. In this analysis, the parameters μ_w and σ_w of $w(\gamma)$ are tuned to comply with the 95% confidence interval of $4 * \sigma = 10 - 14 m/s$. To accomplish this, a Monte Carlo simulation is performed. Consider 10.000 samples of $w(\gamma) = N[\mu_w, \sigma_w]$ which will be used to generate a 10.000 lines based on a GPD function. This will result in a bundle of a 10.000 possible relations between wind speed and exceedance probability, see figure 6.4.

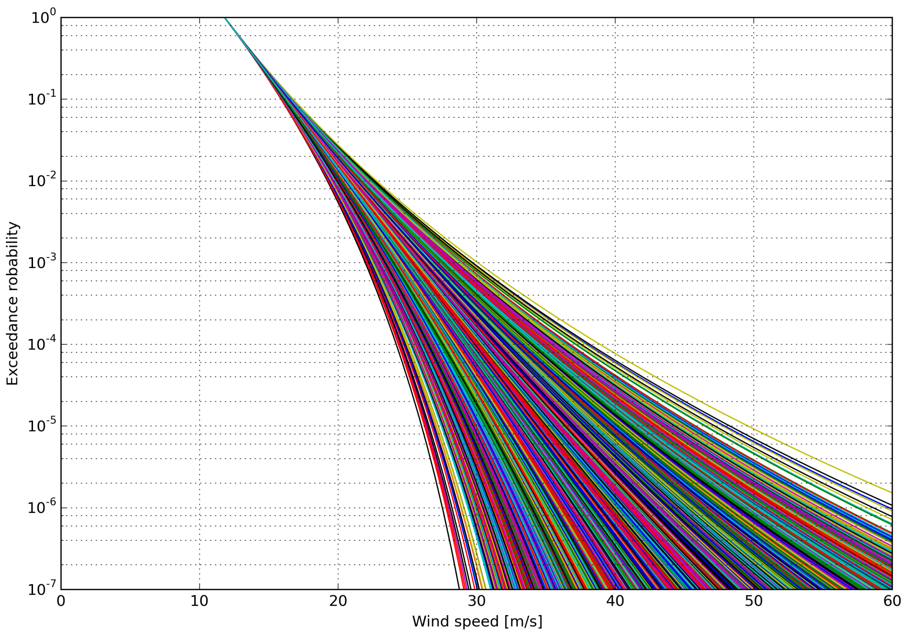

Figure 6.4: 10.000 samples of γ produce 1000 GDP lines.

In the following figure 6.5 and figure 6.6, a black horizontal bar visualises the 95% confidence interval at a yearly exceedance frequency of 0.0001. The y-axis, however, presents the exceedance probability of the maximum wind speed in a 12 hour interval. This can be converted to a yearly exceedance frequency: The amount of 12 hours blocks is roughly $2*180 = 360$ per winter half year and the probability of western wind is about 8%.

$$F(U > u) = N \cdot P(r) \cdot P(U > u) \tag{6.17}$$

$$0.0001 = 0.08 \cdot 360 \cdot P(U > u) \tag{6.18}$$

$$P_{bar}(U > u) = \frac{0.0001}{0.08 * 360} = 3.66e-06 \tag{6.19}$$

The 0.0001 yearly exceedance frequencies of each GPD realisation is further analysed, since it has to comply with two conditions. First, the mean of the 0.0001 per year quantile should be equal to exponential base distribution. Second, the observed 95% confidence intervals should coincide with the predetermined range, namely 10, 12 or 14 [m/s]. The established values can be found in table 6.1. Notable is that a mean of zero suited best in all estimation try-outs. Any other value for the mean μ_w causes a shift from the base distribution.

Table 6.1: Table of retained parameters μ_w and σ_w.

95% C.I.	μ_w	σ_w^2
10 [m/s]	0	0.0217
12 [m/s]	0	0.0265
14 [m/s]	0	0.0310

The visualised red lines in figure 6.5 present percentile intervals. The percentiles found in table 6.2 correspond with each consecutive red line. To further explain what these intervals represent please consider the following example: A 1,000 samples for $[\gamma]$ will produce 1,000 values for $P(U > u)$ at a given probability quantile, as can been seen in figure 6.4. For the 2.5 percentile, it means that 975 other retained values for $[\gamma]$ lead to a larger wind speed value then the considered $[\gamma]$ at the a probability percentile of 2.5. To obtain these values, the equations for the exponential base distribution function and the GPD function are rewritten for wind speed $[u]$:

$$u_{p.exp} = \mu - \sigma \cdot log\left(F(U > u)\right) \tag{6.20}$$

$$u_{p.GPD} = \left[F(U > u)^{1/(-1/\gamma)} - 1\right] \cdot \frac{\sigma}{\gamma} + \mu \tag{6.21}$$

Table 6.2: Percentiles for confidence intervals

2.5	5	10	20	30	40	50	60	70	80	95	97.5

The two outer red lines in figure 6.5 present the 2.5% and 97.5% percentiles. Subsequently, the 95% confidence interval can be retained from these two lines as the interval in between. In figure 6.5 and figure 6.6 the results are visualised for CI=10 [m/s] and CI=14 [m/s] respectively. Whereas the deviation from the base distribution seems quite symmetrical in the CI=10 [m/s] case, asymmetrical properties are more apparent in the CI=14 [m/s] case. This has to due with inherent properties of the GPD function.

Now that the wind uncertainty has been modelled, the question will be what effect this will have on wave conditions and subsequently loading on a dike. In the next section two different ways of inclusion will be explained.

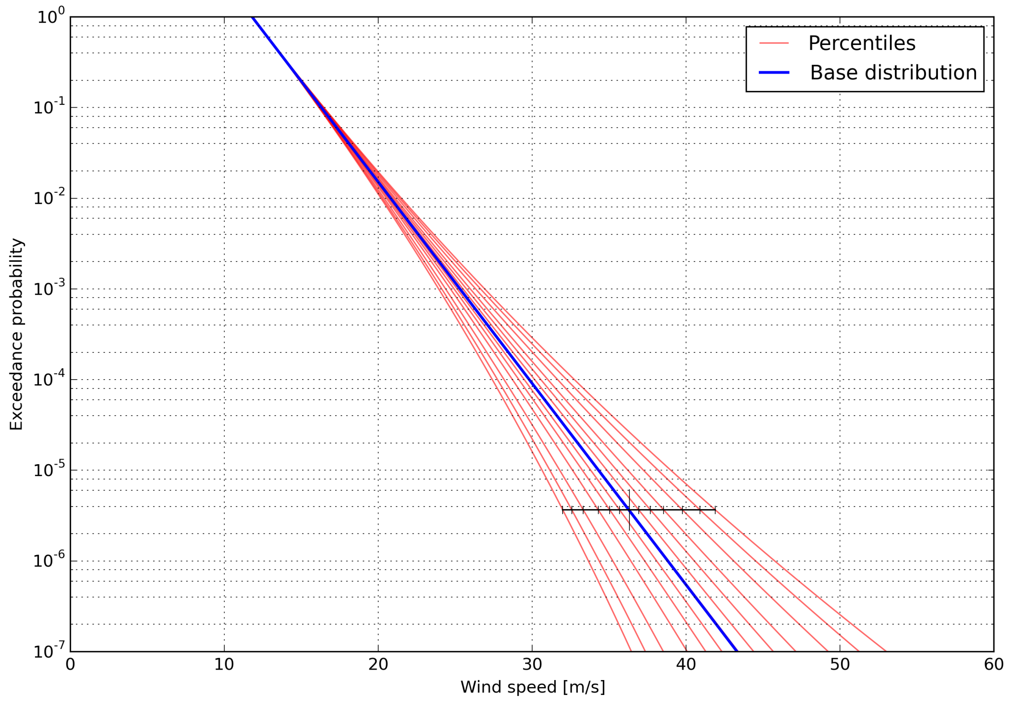

Figure 6.5: Visualisation of percentiles for a 95% confidence interval = 10 [m/s]

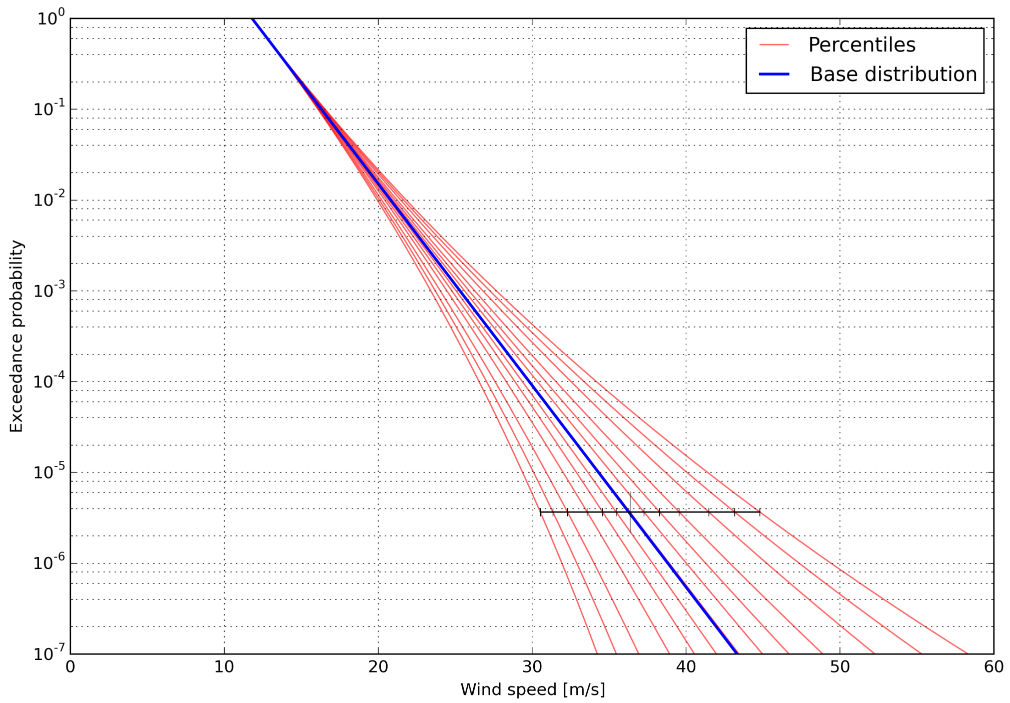

Figure 6.6: Visualisation of percentiles for a 95% confidence interval = 14 [m/s]

OPTION 1:CONVENTIONAL INTEGRATION METHOD

Reducing all possible expressions for $F(H^* > H|\gamma)$ to a single expression $F_{io}(H^* > H)$ can be realised by integrating with respect to γ. As explained in section 6.2 two option remain for the integration order.

Integration order $du d\gamma$ will be elaborated in this section. Mathematically, this can be written as:

$$
\begin{aligned}
F_{io}(H^* > H) &= N \cdot P(r) \int \int_{u_0}^{\infty} w(\gamma) g(u|r,\gamma) du d\gamma && (6.22)\\
&= N \cdot P(r) \int w(\gamma) \int_{u_0}^{\infty} g(u|r,\gamma) du d\gamma && (6.23)\\
&= N \cdot P(r) \int w(\gamma) P(U > u|r,\gamma) d\gamma && (6.24)\\
&= N \cdot P(r) \cdot P_{io}(U > u|r) && (6.25)
\end{aligned}
$$

with:

$$w(\gamma) = N[\mu_w, \sigma_w^2] \tag{6.26}$$

In the following section, the influence of statistical wind speed uncertainty on the significant wave height will be elaborated. This will be solved numerically. A list of assumptions is shown below. Considerations about these assumptions will be explained in section 6.3.3.

- A 95% confidence interval of 14 [m/s] is elaborated,
- wind set-up is incorporated,
- bottom level is -4.0 [m+NAP],
- lake level is -0.25 [m+NAP],
- Bretschneider formulas for wind wave development,
- and a fetch length of 25,000 metres is used.

The task is solved as follows. For each expression of $P(U > u|\gamma)$, as visualised in figure 6.6, a relation $F(H^* > H|\gamma)$ is realised. This can be done since the wave growth development by Bretschneider formulas are known and the exceedance frequency of wind speed can be retained from the analysis. Assuming fetch length and water depth to be deterministic, one can easily produce a graph of $F(H^* > H|\gamma)$.

The integration with respect to γ for all expressions of $F(H^* > H|\gamma)$ will be performed using the trapezium rule for $w(\gamma)$. The range of γ is expected to be within the interval of $\gamma = -0.1$ and $\gamma = +0.1$. The step size of γ is chosen to be 0.0002 (n=1000). Hence the probability of γ_i is:

$$P_{\gamma_i} = \frac{w(\gamma_{i+1}) + w(\gamma_i)}{2} \cdot (\gamma_{i+1} - \gamma_i) \tag{6.27}$$

Thereafter, P_{γ_i} is used as a weight factor for $F(H^* > H|r,\gamma)$. The wind direction parameter $[r]$ is omitted below.

$$
\begin{aligned}
F_{io}(H^* > H) = \int w(\gamma) F(H^* > H|\gamma) d\gamma &= \sum_{i=1}^{n} F(H^* > H|\gamma_i) P_{\gamma_i} && (6.28)\\
&= \sum_{i=1}^{n} F(H^* > H|\gamma_i) \frac{w(\gamma_i) + w(\gamma_{i+1})}{2} (\gamma_{i+1} - \gamma_i) && (6.29)
\end{aligned}
$$

The considered range for wind speed is limited by the interpolation range of the $U_p - U_{10}$ transformation. This differs for each fetch length.

The result of the exercise is shown in figure 6.7. The figure clearly shows an enhanced significant wave height due to the inclusion of uncertainty. A thorough analysis of the implications of this will follow in section 6.3.3. In the next section integration order $d\gamma du$ is elaborated.

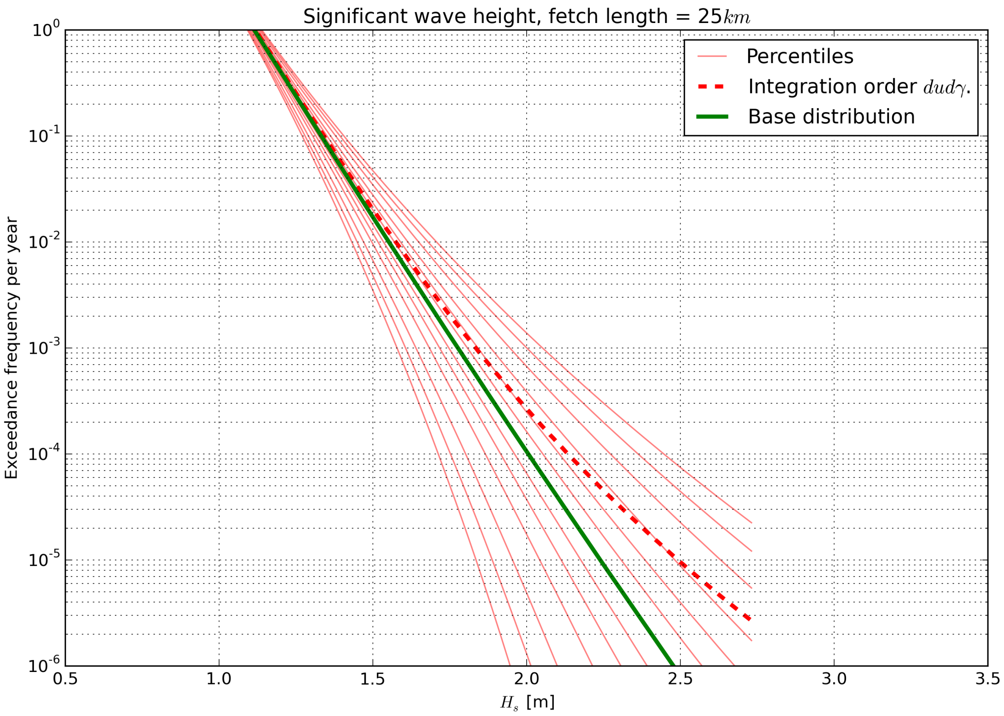

Figure 6.7: integrateduncertainty

OPTION 2: PRIOR INTEGRATION OF UNCERTAINTY

In the next approach the integration order $d\gamma du$ will be elaborated. The benefit in doing so is that the integration of γ will give a prior established enhanced wind speed expression $P_{io}(U > u)$. The singular expression simplifies the wave condition elaboration process.

$$F_{io}(U > u|r) = N \cdot P(r) \int_{u_0}^{\infty} \int w(\gamma) g(u|r,\gamma) d\gamma du \tag{6.30}$$

$$= N \cdot P(r) \int_{u_0}^{\infty} g_{io}(u|r) du \tag{6.31}$$

$$= N \cdot P(r) \cdot P_{io}(U > u|r) \tag{6.32}$$

with:

$$P_{io}(U > u|r) = \int w(\gamma) P(U > u|r) d\gamma \tag{6.33}$$

Again, the integration with respect to γ will be solved numerically using the trapezium rule. The computation of $P_{io}(U > u|r,\gamma)$ will consist out of two steps. First the probability of γ_i is computed (similar to option 1). Hence:

$$P_{\gamma_i} = \frac{w(\gamma_{i+1}) + w(\gamma_i)}{2} \cdot (\gamma_{i+1} - \gamma_i) \tag{6.34}$$

In the second step, P_{γ_i} is used as a weight factor for $P(U > u|r,\gamma)$. The wind direction parameter $[r]$ is omitted below.

$$P_{io}(U > u) = \int_{u_0}^{\infty} \int w(\gamma) g(u|\gamma) d\gamma du = \sum_{i=1}^{n} P(U > u|\gamma_i) P_{\gamma_i} \tag{6.35}$$

$$= \sum_{i=1}^{n} P(U > u|\gamma_i) \frac{w(\gamma_i) + w(\gamma_{i+1})}{2} (\gamma_{i+1} - \gamma_i) \tag{6.36}$$

To further clarify this operation, please consider figure 6.6. The figure shows the relation of the exceedance probability of wind speed. The tail of the expression can be influenced by the shape parameter γ. Since the parameter γ is designated to be normally distributed, a higher deviation from the mean has a lower probability of occurrence. Subsequently, the red lines visualise the percentiles. This can be reduced to a 'weighted averaged' single expression $P_{io}(U > u)$ by a numerical integration. The possible expressions of $P(U > u|\gamma_i)$ are multiplied by the probability of occurrence of that certain γ_i.

By performing this numerical integration, $P(U > u|\gamma)$ simplifies to a single expression $P_{io}(U > u)$. The result of this can be seen in figure 6.8. The figure shows that up to $u = 30$ [m/s] there is a minor influence of uncertainty. From 30 [m/s] onwards, the influence of uncertainty becomes more significant. At a yearly frequency of 10^{-5}, the increase can be as much as 3 [m/s] when this is compared to the base distribution.

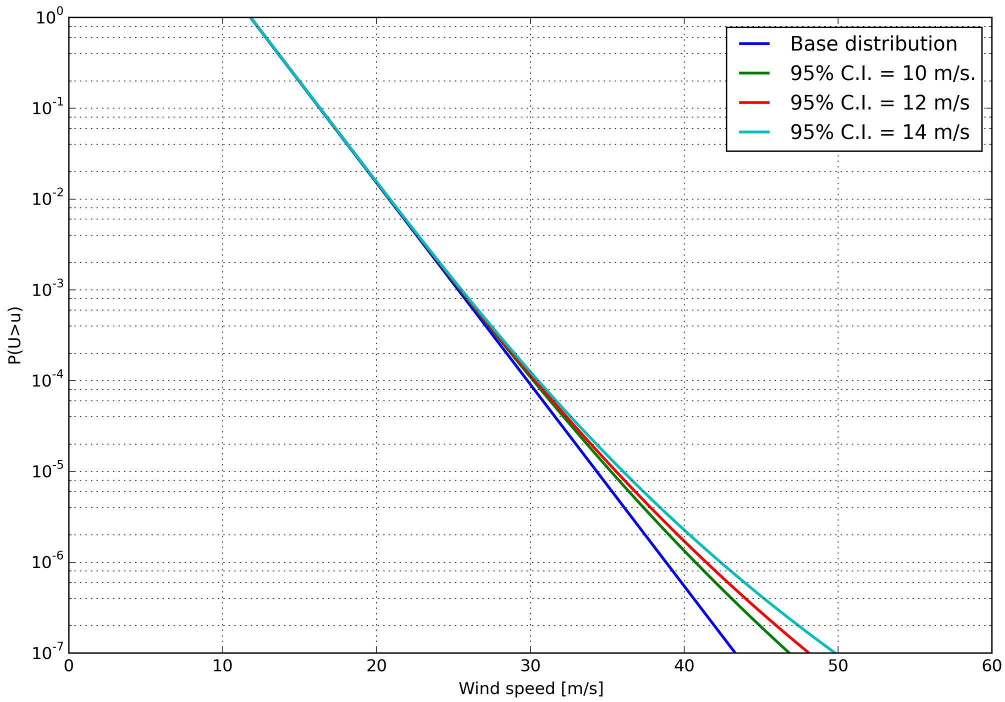

Figure 6.8: Simplification of uncertainty inclusion by integrating γ.

The expression for $P_{io}(U > u|4\sigma = 14m/s)$, the turquoise line in figure 6.8, will be used for the elaboration of the significant wave height. The same assumptions are applied as in option 1. Figure 6.9 shows the comparison between the two options.

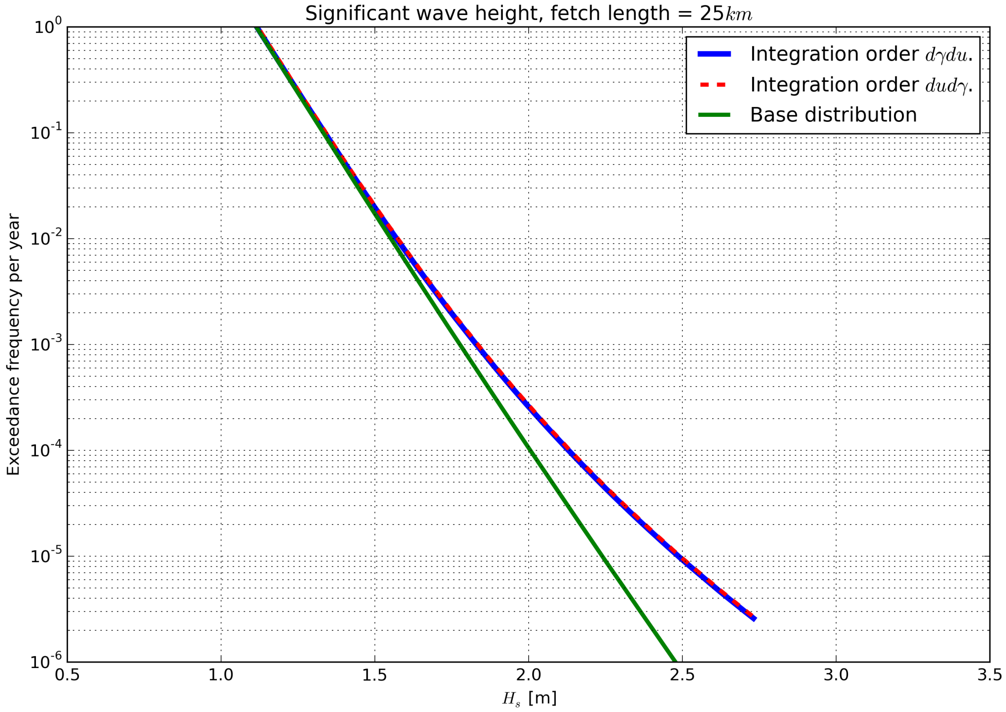

Figure 6.9: Geintegreerde versie heeft een $4 \cdot \sigma = 14[m/s]$

The exercise shows that the mathematical elaboration of section 6.2 is indeed correct. It was stated that the integration order does not matter for the result of $F_{io}(H^* > H)$. The figure 6.9 shows that both options for the integration order are almost identical. In so, this statement is practically reaffirmed.

The next section will focus on the impact of enhanced wind speeds due to inclusion of statistical uncertainty. Since this section shows that the integration order is not of importance for the result, the integration order $d\gamma du$ will be further elaborated.

6.3.3. EFFECT ON HYDRAULIC LOADING CONDITIONS

Normative loading conditions need to be set when desiging a dike. If it is either a deterministic or a probabilistic calculation, an average or mean of the expected loading is needed. Uncertain effects, as visualised in figure 6.6, impose the possibility of enhanced normative wind speeds. The wind data of figure 6.6 is further elaborated using commonly used wind set-up and wind wave development formulas.

WIND SET-UP

Wind stresses can cause an increase of water level in a basin. When the wind is pushed towards the shore and has no where to go, it can only move upwards. The increase in water level Δh follows from a force equilibrium between wind induced stresses and hydrostatic pressure. The formulas for wind set-up are based on a force equilibrium. An elaboration of this can be found on 107.

$$\Delta h = \frac{CFu_{10}^2}{d_0} \qquad \text{for} \quad d/F > 0.001 \tag{6.37}$$

$$\Delta h = \sqrt{2CFu_{10}^2 + d_0^2} - d_0 \qquad \text{for} \quad d/F < 0.001 \tag{6.38}$$

with:

$$d_0 = m_0 - h_{bottom} \tag{6.39}$$

Parameters:

C	= Constant $0.35 \cdot 10^{-6}$ [-]
F	= Fetch length [m]
u_{10}	= Wind speed at h=10 metre [m/s]
d_0	= Water depth [m]
m_0	= Average IJssel Lake level = -0.25 [m+NAP]
h_{bottom}	= Average IJssel Lake bottom level = -4.00 [m+NAP]

The IJssel Lake facilitates for relatively large fetches, three fetch lengths will bee highlighted in this example. The different fetches will be 5 kilometre, 10 kilometre and 25 kilometre. The average water depth of the IJssel Lake is assumed to be 3.75 metres. Considering this relatively shallow waters and long fetch lengths, the commonly used equation (6.37) is less suitable since Δh^2 can not be neglected in the elaboration of the simplified formula. The equation (6.38) will be used in this analysis. The equations differ most in the less frequent cases, where equation (6.38) provides smaller (and arguably more realistic) values for the expected wind set up.

Although equation (6.37) is broadly used in the Netherlands, the accuracy of the formula is highly debatable. The constant C=$0.35 \cdot 10^{-6}$ is easily recognized as a possible influence of uncertainty. No real consensus exists about this empirical established parameter. Feij (2015) conducted a research which stresses the observation that the inaccuracy of the wind set-up formula does not only stem for the constant C, but the formula as a whole. For the sake of this study, the 'conventional' equation (6.37) and equation (6.38) are used for the elaboration. This is choice is made to minimize the amount of deviations when compared to the Hydra-Zoet model.

In figure 6.10 three bundles of lines can be distinguished. A single bundle shows the expected exceedance frequency of wind set-up of varying uncertainty definition. Obviously, the fetches are increasing from left to right.

A roster of wind speeds is defined from 0 to 60 [m/s] with a step size of 0.01 [m/s]. For each wind speed the the appropriate water depth is computed. The computed increase in water level is plotted against its related wind speed exceedance frequency.

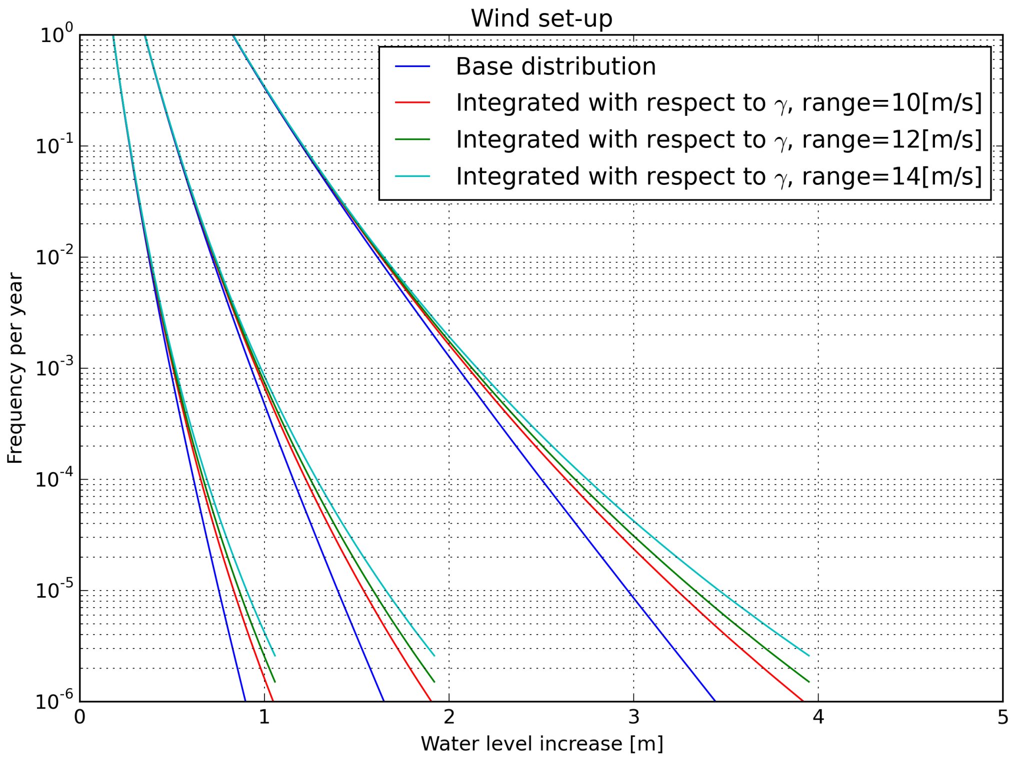

Figure 6.10: Influence of uncertainty on wind set-up for three different fetch lengths.

For a frequency of 0.00001 per year the influence of uncertainty is quite apparent. For the 95% confidence intervals considered, an increase of approximately 9% to 20% is computed. The percentage increase does seem to decrease for extreme fetches. The results are summarized in table 6.3.

Table 6.3: Percentage change of water level due to uncertainty at a frequency of 0.00001 per year.

Fetch length	95% CI: 10 [m/s]	95% CI: 12 [m/s]	95% CI: 14 [m/s]
5 km	+10%	+15%	+20%
10 km	+9%	+14%	+19%
25 km	+9%	+14%	+17%

The increase of water level due to wind set up allows for the development of wind waves. This will be elaborated in the next section.

DEVELOPMENT OF WIND WAVES

As seen in the previous section, an increase in wind speed needs to be taken into account when the uncertainties are is integrated over γ. Similarly, an increase in wave conditions is expected. The Bretschneider formulas will be used in this section to compute the effects of enhanced wind conditions due to the inclusion of statistical wind uncertainty.

BRETSCHNEIDER WAVE GROWTH FORMULAS

For this the Bretschneider formula is used. The input parameters for the Bretschneider formula are fetch length, water depth and wind speed. It is assumed that the fetch length and water depth are deterministic values.

$$H_s = \frac{0.283 u_{10}^2 v_1}{g} \tanh\left(\frac{0.0125}{v_1}\left(\frac{gF}{u_{10}^2}\right)^{0.42}\right) \tag{6.40}$$

$$T_s = \frac{2.4\pi u_{10} v_2}{g} \tanh\left(\frac{0.0077}{v_2}\left(\frac{gF}{u_{10}^2}\right)^{0.25}\right) \tag{6.41}$$

with:

$$v_1 = \tanh\left(0.530\left(\frac{gd}{u_{10}^2}\right)^{0.25}\right) \tag{6.42}$$

$$v_2 = \tanh\left(0.833\left(\frac{gd}{u_{10}^2}\right)^{0.375}\right) \tag{6.43}$$

Wherein:

H_s = Wave height [m]
T_s = Wave period [s]
F = Fetch length [m]
u_{10} = Wind speed at h=10 metre [m/s]
d = Water depth [m]
g = Gravitational constant 9.81 $[m/s^2]$

Initially, the IJssel Lake does not allow for the development of large wave heights due to its shallow waters. The depth limitation for the development of wind waves is given by equation (6.44). However, the extreme cases which are considered in this example cause a vast increase in water depth due to wind set up. Subsequently, the limitation on the maximum wave height is also increased.

$$\frac{H_s}{d} \approx 0.4 \tag{6.44}$$

The local water depth can be written as:

$$d = m_0 - h_{bottom} + \Delta h \tag{6.45}$$

wherein:

m_0 = Average IJssel Lake level = -0.25 [m+NAP]
h_{bottom} = Average IJssel Lake bottom level = -4.00 [m+NAP]
Δh = Wind set up

A disclaimer about the denoted 0.4 is well in place. The figure is far from applicable to all wave loading cases. The variation of the parameter can be roughly estimated to be 0.3-0.5 (Groeneweg & Dingemans, 2003).

A roster of wind speeds is defined from 0 to 60 [m/s] with a step size of 0.01 [m/s]. The related local water depth and wind speed is used to compute the significant wave height. Thereafter, the significant wave height is plotted against it related wind speed exceedance frequency.

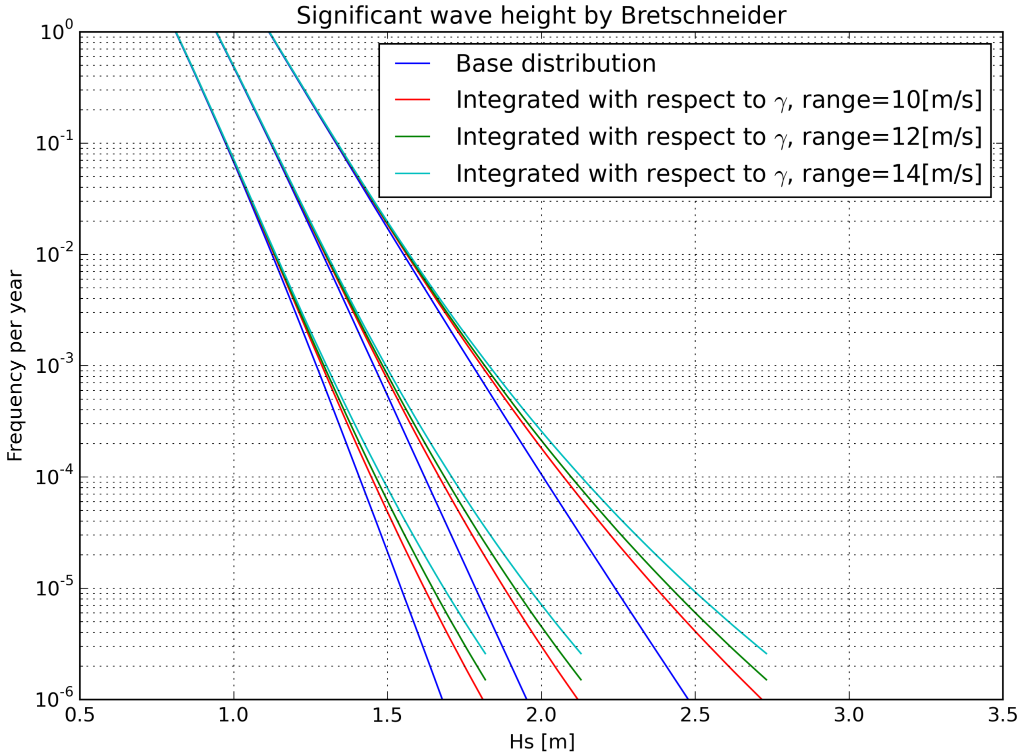

Figure 6.11: Influence of uncertainty on the significant wave height for three different fetch lengths.

The introduction of wind speed uncertainty leads to an increase of wave height. For a frequency of 0.00001 per year, the waves will increase by approximately 5% to 11%. The results are summarized in table 6.4. The significance of uncertainty seems to increase for larger 95% confidence intervals. The significance is less for more frequent cases.

Table 6.4: Percentage change of significant wave height due to uncertainty at a frequency of 0.00001 per year.

Fetch length	95% CI: 10 [m/s]	95% CI: 12 [m/s]	95% CI: 14 [m/s]
5 km	+5%	+7%	+9%
10 km	+6%	+7%	+10%
25 km	+6%	+8%	+11%

Analogues to the elaboration of the significant wave height, the significant wave period is computed. The result can be seen in figure 6.12 and is summarized in table 6.6.

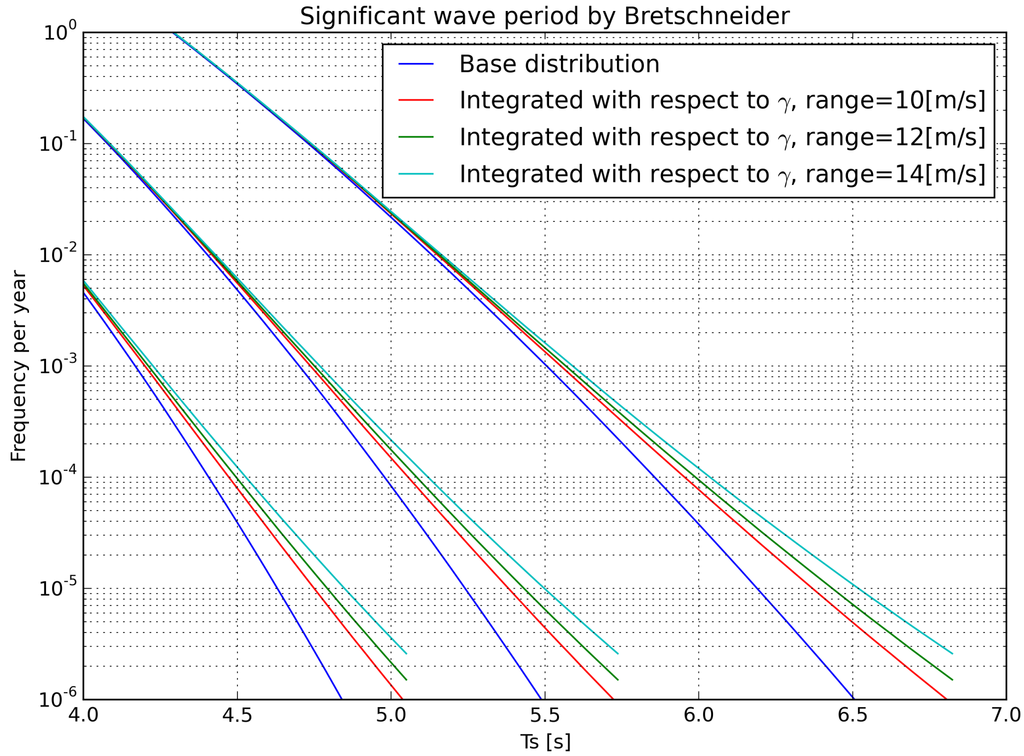

Figure 6.12: Influence of uncertainty on the significant wave period for three different fetch lengths

The figure shows that the significant wave period is easily influenced by wind speed. The significant wave period increases rapidly for lesser frequent cases (in which implies higher wind speeds). The introduction of uncertainty imposes an increase in significant wave period of about 3% to 5%. In that sense, it is concluded that the influence on wave period is relatively lower then that it is on wave height.

Table 6.5: Percentage change of significant wave period due to uncertainty at a frequency of 0.00001 per year.

Fetch length	95% CI: 10 [m/s]	95% CI: 12 [m/s]	95% CI: 14 [m/s]
5 km	+3%	+3%	+5%
10 km	+3%	+4%	+5%
25 km	+3%	+4%	+5%

WAVE RUN-UP

In the end, it all comes down to what the loading will be on the dike. To make the influence of added uncertainty more tangible the run-up is computed in this section. For this purpose, a standard 1-to-3 dike profile is elaborated using the equation (6.50) taken from J. W. van der Meer (1996). And as explained by J. W. van der Meer: "Wave run-up is often indicated by Ru2%. This is the run-up level, vertically measured with respect to the (adjusted) still water level (SWL), which is exceeded by two per cent of the incoming waves." This can be expressed as:

$$\frac{R_{u2\%}}{H_s} = 1.6\gamma_b\gamma_f\gamma_\beta\xi_{m-1,0} \qquad \text{with a maximum of} \quad 3.2\gamma_f\gamma_\beta \tag{6.46}$$

with:

$$\xi = \frac{tan\alpha}{\sqrt{s}} \tag{6.47}$$

$$s = \frac{H_s}{L} \tag{6.48}$$

$R_{u2\%}$ = Run-up level [m]
H_s = Significant wave height [m]
L = Wave length
$\xi_{m-1,0}$ = Breaker parameter [-]
s = Wave steepness [-]
α = Slope angle [radians]
γ_b = Reduction factor for berm influence [-]
γ_f = Reduction factor for slope roughness [-]
γ_β = Reduction factor for for oblique wave attack [-]

The wave steepness is elaborated using the local wave height H_s and local wave period $T_{m-1,0}$ using the deep water wave length L_0 formula as expressed in equation (6.49).

$$L_0 = \frac{gT^2_{m-1,0}}{2\pi} \tag{6.49}$$

Under the assumption that all γ-reduction-factors are neglected, the formula for wave run-up can be rewritten as:

$$R_{u2\%} = \frac{1.6 \cdot H_s \cdot \tan(\alpha)}{\sqrt{H_s/L}} \tag{6.50}$$

A standard profile is chosen since in this analysis the main point of interest is the influence of uncertainty. In this sense, the results will not be affected by other aspects like the influence of a berm or oblique wave attack. These aspects, however, are of great importance for an optimal dike design.

The H_s and T_s from the previous section are used in the elaboration of the 2% run-up height. The result of this can be seen in figure 6.13.

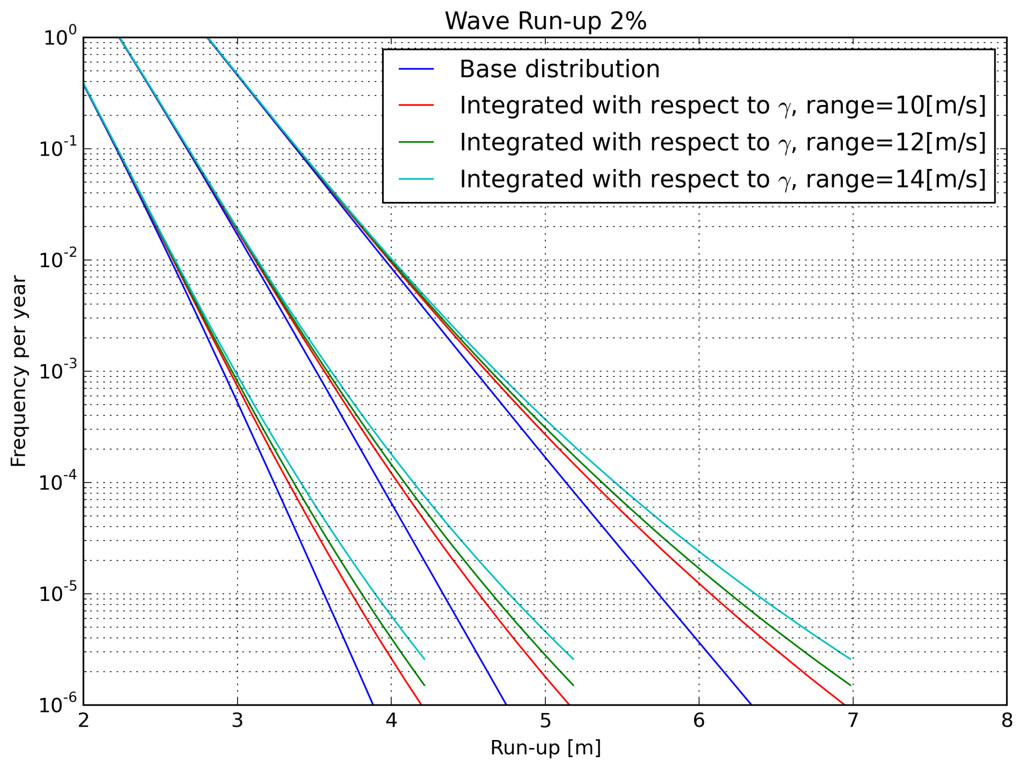

Figure 6.13: Influence of uncertainty on the run-up for three different fetch lengths

The increase in 2% run-up height is 5% to 11% for the considered 95% confidence intervals. If one assumes that an increase of the 2% wave run-up height can be translated to a needed crest height of dike, then a dike's crest level needs to be adjusted. This relates to an increase of 20 cm to 35 cm for fetches of 5 km and 40 cm to 70 cm for a 25 km fetch. It must be noted that these values are far from optimised since a standard 1-to-3 profile is reckoned in a most unfavourable scenario of a very large fetch length. However, the percentage chances do indicate that heavier loading conditions are possible when statistical wind uncertainty is included in the design of a dike.

Table 6.6: Percentage change of wave run-up due to uncertainty at a frequency of 0.00001 per year.

Fetch length	95% CI: 10 [m/s]	95% CI: 12 [m/s]	95% CI: 14 [m/s]
5 km	+5%	+7%	+9%
10 km	+5%	+8%	+10%
25 km	+6%	+8%	+11%

REQUIRED CREST HEIGHT DUE TO OVERTOPPING

In the Netherlands, dike are designed and assessed for the amount of water which can topple over the dike. This failure mechanisms is referred to as overtopping. The average water which topples over a is called the average overtopping discharge. This usually is expressed as the amount of litres per second per meter.

When water topples over the crest, it runs over the inner slope of the dike. Firstly, this causes an increasing water level of the hinterland. Secondly, and more importantly, the water running down the inner slope causes the inner slope to erode. The consequence of this is that the dike loses its firmness and can collapse. To counteract this failure mechanisms, a maximum allowable overtopping discharge can be defined, namely the critical overtopping discharge. In this example, the critical overtopping discharge is chosen to be 5 [l/s/m].

The deterministic overtopping formula for breaking waves is taken from The EurOtop Team (2007) and presented below in equation (6.51).

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{tan(\alpha)}}\gamma_b \cdot \xi_{m-1,0} \cdot exp\left(-4.3\frac{R_C}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_{b,f,\beta,v}}\right) \tag{6.51}$$

with

$$s = \frac{2\pi H_{m0}}{g \cdot T^2} \tag{6.52}$$

$$\xi_{m-1,0} = \frac{tan(\alpha)}{\sqrt{s}} \tag{6.53}$$

q = Average overtopping discharge $m^3/s/m$
g = Gravitational acceleration constant $[m/s^2]$
H_{m0} = Wave height [m]
α = Slope angle [radians]
R_C = Relative freeboard [m]
$\xi_{m-1,0}$ = Breaker parameter [-]
γ_b = Reduction factor for berm influence [-]
γ_f = Reduction factor for slope roughness [-]
γ_v = Reduction factor for vertical wall on dike [-]
γ_β = Reduction factor for for oblique wave attack [-]

The overtopping manual by equation (6.51) states that an alternative equation can be used in case of a probabilistic computation. In this equation the constant -4.3 is -4.75 and is assumed to be stochastic. It is normally distributed with an deviation of 0.5. Since this example does not address the model's uncertainty, the deterministic variant is chosen.

A standard dike profile with of a slope of 1-to-3 is used in the computation. All reduction factors γ due to berm influence, slope roughness, a vertical wall or oblique waves are neglected. Then equation (6.51) can be rewritten to equation (6.54). The crest height is defined by equation (6.55) [m+NAP].

$$R_C = \frac{\xi_{m-1,0} \cdot H_{m0}}{-4.3} \cdot log\left(\frac{q}{\sqrt{g \cdot H_{m0}^3}} \cdot \sqrt{\frac{tan(\alpha)}{0.067 \cdot \xi_{m-1,0}}}\right) \tag{6.54}$$

$$HBN = d - h_{bottom} + R_C \tag{6.55}$$

Upfront, it must be noted that the computed crest heights of figure 6.14 are not an accurate representation of reality since optimization of the dike profile can lead to a substantially different outcome. However, the observed standard dike profile does indicate the influence of statistical uncertainty.

The results of the computation are very much alike the run-up computation. The influence of the additional statistical uncertainty is more apparent for the larger fetch lengths.

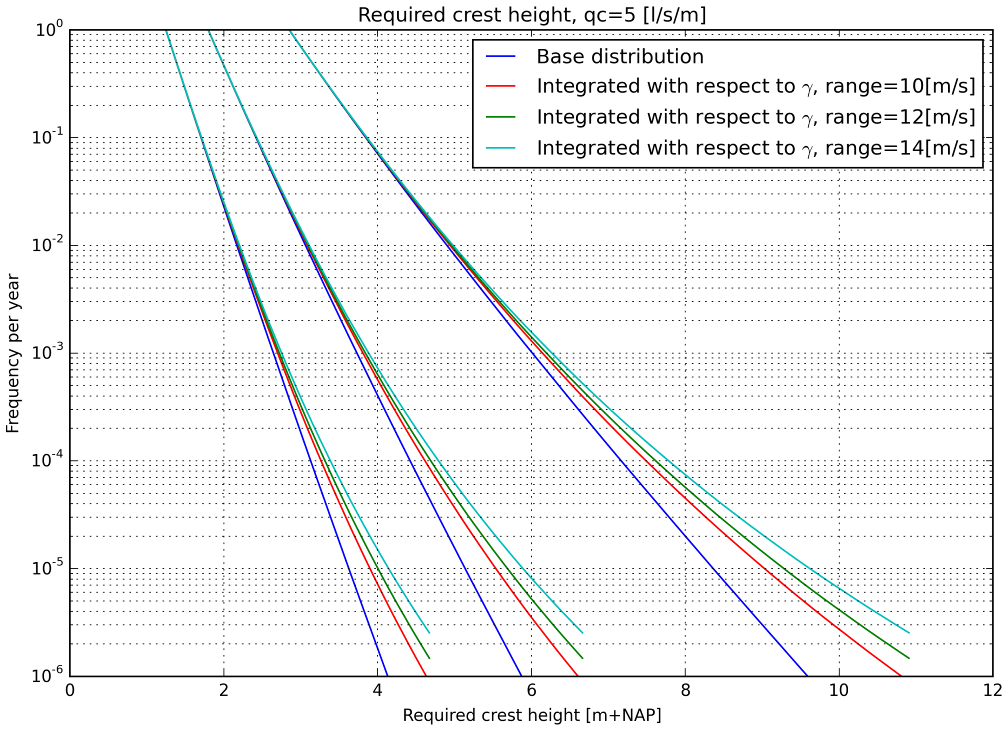

Figure 6.14: Influence of uncertainty on the required crest height due to overtopping for three different fetch lengths

Table 6.7: Percentage change of required crest height due to uncertainty at a frequency of 0.00001 per year.

Fetch length	95% CI: 10 [m/s]	95% CI: 12 [m/s]	95% CI: 14 [m/s]
5 km	+5%	+8%	+14%
10 km	+8%	+12%	+14%
25 km	+10%	+13%	+16%

6.3.4. EVALUATION OF MODEL

The aim of this analysis is to provide a transparent and sensible implementation method of statistical wind uncertainty into a model. To make this task tangible, the effects are presented in terms of increased wind speed, wind set up, wave height, wave period and run-up level exceedance probability. The boundary conditions have been chosen so that the results strife to be realistic. Although several simplifications are taken in this elaboration, the overall trend of enhanced conditions is assumed to be relevant for more complex instances as well. This is because statistical uncertainty would also show its effect in more refinement physical models.

The proposed model displays a method wherein the significance of uncertainty becomes larger for less frequent cases. Whereas the previously elaborated wind uncertainty methods, which is used for the robustness surcharges of Rijkswaterstaat (2014a), assumes a constant uncertainty significance for all frequencies. The implication of the method of this analysis is an overall increase of loading conditions. This is most apparent for very infrequent cases.

An observation in the elaboration of how to include uncertainty into a model is that the integration order can be exchanged. Changing the order of integration for double integrals is a common mathematical technique, and in this analysis, it is proven numerically for a practical example. The benefit of this is that during modelling a singular adjusted expression for wind speed data adjusted for uncertainty can be established prior to the elaboration of other loading conditions. This simplifies the process and reduces computation time.

The inclusion of uncertainty imposes enhanced loading conditions for the considered examples. The results will be presented in percentage increases. The effect on wind set-up is 10-20%. For significant wave height and wave period this is 5-10% and 3-5% respectively. The run-up height for a standard 1-to-3 profile is expected to increase with 5-10%.

In the elaboration an interpolation is performed for the $u - u_{10}$ conversion. Since the interpolation is based on the table provided Hydra-Zoet 1.6.3, the range is limited to u = 50 [m/s]. It is for this reason that many graphs as visualised in the section cannot extend any further. This issue might be solved by an alternative expression or a curve fitting. For the sake of this analysis, the interpolation suffices since the frequencies of interest are within the limits.

The outcome of this analysis is exploratory and should be considered with care. Many subjective choices play an essential role in the elaboration. This can be the choice of designation of uncertainty, curve fitting or the used formulas for that matter. Several will be highlited in the next section.

6.3.5. DISCUSSION

Many choices in this analysis are subjective of nature. Below, several considerations will be discussed. First, the topics associated with the analysis in itself will be discussed. Second, the implications for further development.

MODELLING

CURVE FITTING EQUATION FOR BASE DISTRIBUTION

A first point of attention is the subjective choice of the used curve fitting equation. This analysis assumes an exponential fit for the base distribution. Other fits could be considered as well. In figure 6.15 on the next page two other equations are fitted as well to the statical wind data. Time limitation have forced the project to not elaborate these other fittings any further. For further development it is strongly advised to considered different fitting equations nonetheless.

UNCERTAINTY DESIGNATION

The designation of the 95% confidence interval of uncertainty is crucial for any of the results in this chapter. This analysis has highlighted three different designations, namely $4 \cdot \sigma = 10$, 12 and 14 [m/s]. In all of the examples the impact of a different designation of uncertainty is apparent.

The uncertainty should comprehend deviations due to measurement errors and modelling errors of the RW-model. Quantification of these errors is near to impossible. However, using engineering judgement, it can be

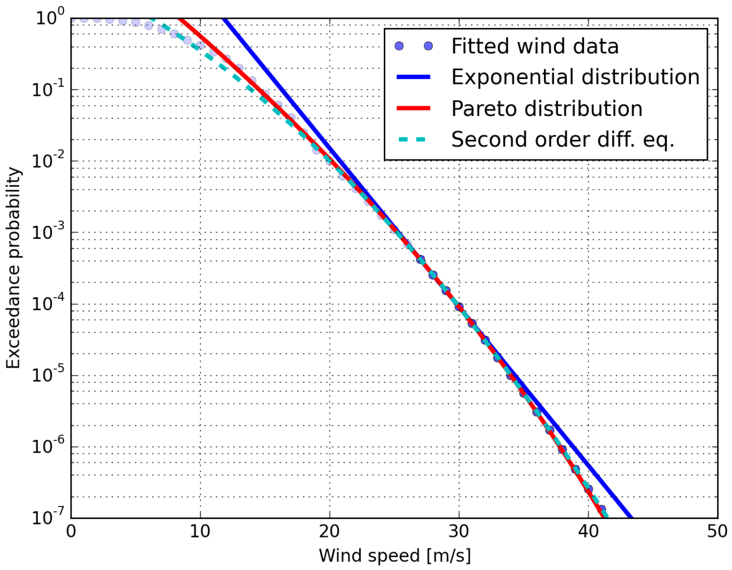

Figure 6.15: Three different curve fittings.

argued that when extreme wind conditions are considered which never have been encountered before, the error might as well be of the same order of magnitude. Also, relating to reverse psychology, it is as naive to say that the only real value is that of the retained statistical value.

If it is desired to retain an as good as possible subjective judgement, then a panel of experts might be a option to consider. If the pool is large enough, then the bias can be reduced.

SHORTCOMINGS OF USED FORMULAS

An attempt is done to reduce the error in wind set-up by using a more fundamentally wind set-up equation than the commonly used wind set-up formula. Hence, the force induced by wind stresses and hydrostatic pressure are:

$$F_{wind} = c\rho_a u_{10}^2 F \tag{6.56}$$

$$F_{water} = \frac{1}{2}\rho_w g(d+\Delta d)^2 - \frac{1}{2}\rho_w g d^2 \tag{6.57}$$

Equilibrium between the two forces results into:

$$F_{wind} = F_{water} \tag{6.58}$$

$$c\rho_a u_{10}^2 F = \frac{1}{2}\rho_w g(d+\Delta d)^2 - \frac{1}{2}\rho_w g d^2 \tag{6.59}$$

$$= \frac{1}{2}\rho_w g(2d\Delta d + \Delta d^2) \tag{6.60}$$

Rearrangement leads to the "abc-formula":

$$\Delta d^2 + 2d\Delta d - \frac{2c\rho_a u_{10}^2 F}{\rho_w g} = 0 \tag{6.61}$$

$$C = \frac{c\rho_a}{\rho_w g} = 0.35\cdot 10^{-6} \tag{6.62}$$

$$\Delta d = \sqrt{2CFu_{10}^2 + d_0^2} - d_0 \quad \text{for} \quad d/F < 0.001 \tag{6.63}$$

Using formula equation (6.63), one assumes that influence of Δd cannot be neglected. This has a limiting effect on the elaborate wind set-up. Although, a more elaborate equation is used, simplifications still remain. Effects of bathemetry and a changing water roughness coefficient c are not addressed for instance.

Early wave growth models like the Bretschneider are parametrised by empirical research. Subsequently, an error is introduced in this parametrisation. Unfortunately, the error of these formulas is unknown, and probably remains unknown since data of old studies like this one are often hard to come by. However, a study by Bart (2013) has taken the opportunity to analyse the error in the wave growth formulas of Young & Verhagen (1996). Bart (2013) observes a standard deviation of $s_\epsilon = 3.7507 \cdot 10^{-5}$. The formulas shown below are retained from and modified by Holthuijsen (2007).

$$\tilde{H} = \tilde{H}_\infty \left[tanh(k_3 \tilde{d}^{m_3}) tanh\left(\frac{k_1 \tilde{F}^{m_1}}{tanh(k_3 \tilde{d}_3^{m})} \right) \right]^p \tag{6.64}$$

$$\tilde{T} = \tilde{T}_\infty \left[tanh(k_4 \tilde{d}^{m_4}) tanh\left(\frac{k_2 \tilde{F}^{m_2}}{tanh(k_4 \tilde{d}_4^{m})} \right) \right]^q \tag{6.65}$$

With the dimensionless parameters:

$$\tilde{H} = \frac{gH}{u_{10}^2} \tag{6.66}$$

$$\tilde{T} = \frac{gT}{u_{10}} \tag{6.67}$$

$$\tilde{d} = \frac{gd}{u_{10}} \tag{6.68}$$

Wherein:

$H_\infty = 0.24$
$k_1 = 4.14 \cdot 10^{-4}$
$m_1 = 0.79$
$p = 0.572$
$k_3 = 0.343$
$m_3 = 1.14$
$T_\infty = 7.69$
$k_2 = 2.77 \cdot 10^{-7}$
$m_2 = 1.45$
$q = 0.187$
$k_4 = 0.10$
$m_4 = 2.01$

The tests of the Young's & Verhagen's research are performed at Lake George in Australia. Bart (2013) states that the lake is approximately 20x10 km and 2 metres deep. Aside from that this is slightly smaller and shallower then the IJssel Lake, the formula still seems suitable since the tests were performed in a lake area.

The Young and Verhagen model is compared to the Bretschneider model in figure 6.16 and figure 6.17. The aim of this analysis is to put emphasize on the way one should interpret a result of a model. Both models are broadly used, however, they display notable different results.

Still an error is to be expected with either the Bretschneider or Young & Verhagen formulas since the formulas do not fathom 2-d flows or the real bathymetrie for that matter. These first generation wave growth models might be improved by a third generation wave growth model like SWAN (categorization by Stijnen et al. (2008)).

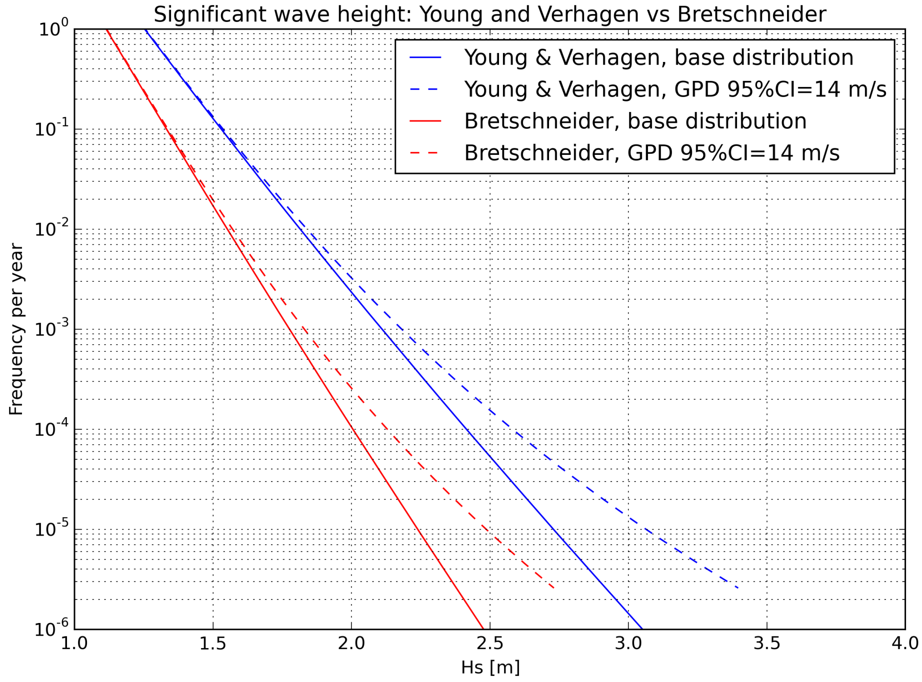

Figure 6.16: Significant wave height. Young & Verhagen vs Bretschneider. Fetch = 25 kilometer.

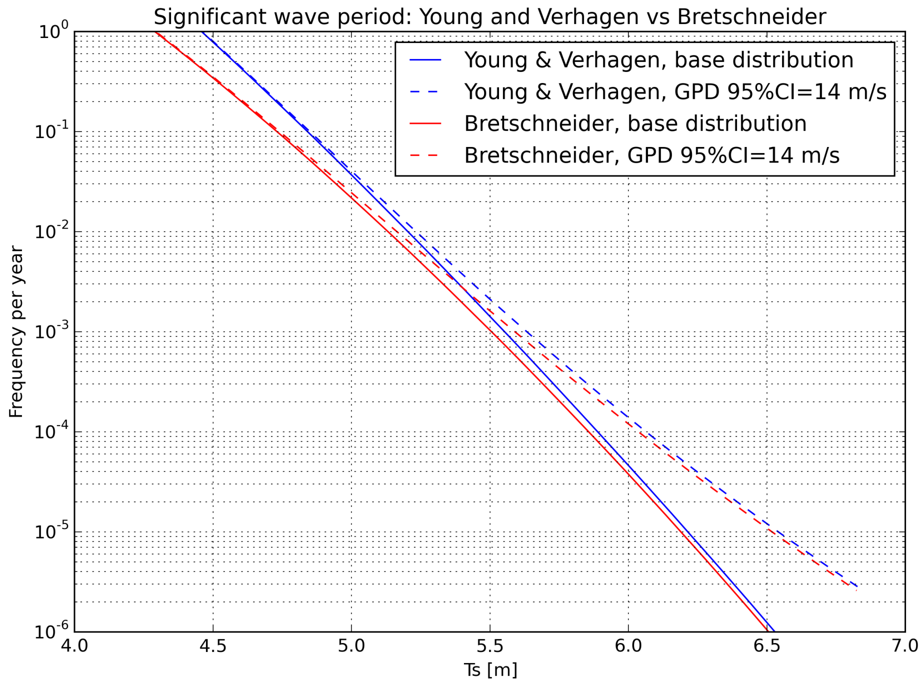

Figure 6.17: Significant wave period. Young & Verhagen vs Bretschneider. Fetch = 25 kilometer.

Normative wind direction versus omnidirectional

As explained in the analysis, only one normative wind direction is elaborated. In this way, an attempt is made to cover the most probable extreme wind scenario. In reality, the wind is not constrained by predefined wind directions. It can be investigated whether it is useful to elaborate the overall influence of all possible wind directions.

Practical considerations

The considerations and assumptions lead to an increase in loading conditions. One the one hand, the effect is considerably, therefore it should be addressed in future dike designs and assessments. On the other hand, a well-founded prove of the quantification of the uncertainty's significance is lacking.

Current developments show that there is a desire to include uncertainties in the dike design. Model uncertainty was first mentioned in the TAW (2008) and was also introduced in the TAW (2009). The influence of statistical uncertainty was first addressed by Nicolai et al. (2010) and is supported by Chbab (2013) before it was taken up by the Rijkswaterstaat (2014c). Although, some of this studies are exploratory of nature, these authors do take a predominantly stance towards the reality of uncertainty. This notion becomes easier to justify with the new trajectory approach in which a risk based design is especially put forward (Rijkswaterstaat, 2013). In this sense, a platform is created in which a seemingly extraordinary investment can be validated by considering the damage it prevents in the future.

A currently in development model which continues on this framework of uncertainty inclusion is the Hydra-Ring model, a probabilistic toolbox for the WTI2017. The inclusion of statistical wind uncertainty can be processed in three different ways:

1. Adjusted statistical local wind data. In conjunction with local data and the integrated version $P_{io}(U > u)$, predefined tables of $P_{io}(U > u|F, r)$ can be established. This is somewhat in the same fashion as the HR2006. The advantage of this is that no extra computations have to be done by the end-user. A prerequisite for this is that the uncertainty is hard coded.

2. A two step process, wherein a singular integrated version of $P_{io}(U > u)$ can be computed before the elaboration of loading conditions. This is similar as what is done in section 6.3.3.

3. Single step, wherein uncertainty is treated as a random variable for each elaboration for a loading condition. This method is highlighted in option 1 section 6.3.2. This option can be seen as the most transparent method.

The three options can be considered to be arranged from least to most transparent. This also coincides with an increasing computation time. If it can be assumed that the Hydra-Ring model should strive to be comprehensive in its core and transparent to its end-users. A trade off has to be made, since none of the options comply with both requirements.

The inclusion of predefined tables does not seem preferable because the underlying assumptions of such a modified table are easily taken for granted. This 'black-box' can not be considered an upgrade compared to existing models. Options 2 and 3 can contribute to a transparent process. The latter option might be easiest to fathom for the end-user, however the computational benefits of option 2 easily outweigh the computational intense method of option 3.

6.4. IMPLEMENTATION OF P_{io} INTO A CURRENT MODEL

In this chapter an attempt is made to implement the uncertainty definition derived from the former the chapter. In the practical consideration on 110, a recommendation is made to further investigate a probabilistic approach in future modelling, option 2 and 3. The two models readily in hand are the PC-Ring model and Hydra-Zoet model. Unfortunately, neither of these models are suitable for simple implementation of these options. Therefore, inclusion as suggested by this these options should be considered during the development phase of future models to suffice in flexibility. However, option 1 wherein prior alternative statistical wind data is defined and 'hard-coded' in a model is easier to implement in the current models. Hydra-Zoet in particularly can be easily modified with this purpose. Since, the considered statistical wind uncertainty moreover has its influence on the loading conditions of a dike design, the Hydra-Zoet is considered suitable for this exercise. An additional benefit of the Hydra-Zoet model compared to the PC-Ring model is that the capping of a river discharge can be included.

Again, the two test cases will be elaborated in this exercise. First, the coast of Flevoland where the IJsselmeerdijk is situated. Second, the dike along the river Waal near the city of Tiel. For both cases, which differ in water system, enhanced wind speed are expected to have an influence on the required dike height (HBN). Lake cases have been broadly highlighted throughout this report. In general, because this system features relatively large fetches, the wind conditions have an important role in the elaboration of loading conditions. For upper-river systems the considerations are different. In contrast, the fetches are limited by the dimensions of the river. Furthermore, the capping of the river discharge introduces an extra aspect to be reckoned with. When a dike design calculation is performed in which an extreme return period is determinative. An maximum local water depth is introduced by the maximum capped river discharge. For this reason one might think that an extreme conditions are governed by extreme wind conditions. This line of reasoning will be evaluated in this exercise.

The proposed robustness surcharges from the OI2014 (Rijkswaterstaat, 2014c) stem from research by Nicolai et al. (2010). From that research it becomes apparent that for lake systems model uncertainty and statistical uncertainty are deemed significant, whereas for upper-river systems only model uncertainty is. The later notion in the previous paragraph about statistical uncertainty for river systems is not tested in the research by Nicolai et al. (2010) since capping of the river discharge is not possible in the PC-Ring model. The forthcoming exercise shows if this reasoning holds when capping is introduced.

6.4.1. ALTERNATIVE TABLES OF STATISTICAL WIND DATA

In the Hydra-Zoet model, statistical wind data is implemented in the form of a table. This table can be found in table D.2 of appendix D. It consists out of 17 columns. The first row denotes the wind speed $U > u$. The remaining 16 columns represent the exceedance probability of each consecutive wind direction in a 12-hours interval.

Using the routine of the previous chapter, this table can be generated for the wind speed exceedance probability including uncertainty with a 95%CI of choice. The aim of the routine in chapter 6 is to give a good estimation for extreme return periods. The GPD relation does not model well the behaviour of more regular return periods, the upper left part of the graph. For this reason it is chosen to use a combination of the RW-model and the proposed model to establish a more 'realistic' representation of the wind speed exceedance probability for the whole range of return periods.

A slicing and smoothing technique is applied to establish a visually good relation for $P(U > u)$. It is chosen that the threshold value u_{thres} of accounted wind speed data for the base distribution services as the boundary between the RW-model and the established GPD-model. For example, in figure 6.3 is shown that wind speed data of $u >= 27[m/s]$ is used for a fit for the base distribution. Then the RW-model data is used for $u < 27[m/s]$, and the GPD-model for $u >= 27[m/s]$. For some wind directions (as is the case in this example for West wind), the established GPD relations show a slightly higher exceedance probability when compared to the RW-model. To prevent a 'kink' or 'step' in the relation, an interpolation is applied between the u_{thres} and $u_{thres} - 1$, for instance $P(u = 26[m/s])$ and $P(u = 27[m/s])$. To add resolution to the table of statistical wind data, a roster of $0.1[m/s]$ is chosen instead of $1.0 m/s$. The available data is also extended to $60[m/s]$ instead of $42[m/s]$. Wind speed direction is not adjusted since this has not been researched. The combination variant of West wind is shown in figure 6.18. The blue line represents the relation which is used for the generation of the statistical wind data table for West wind.

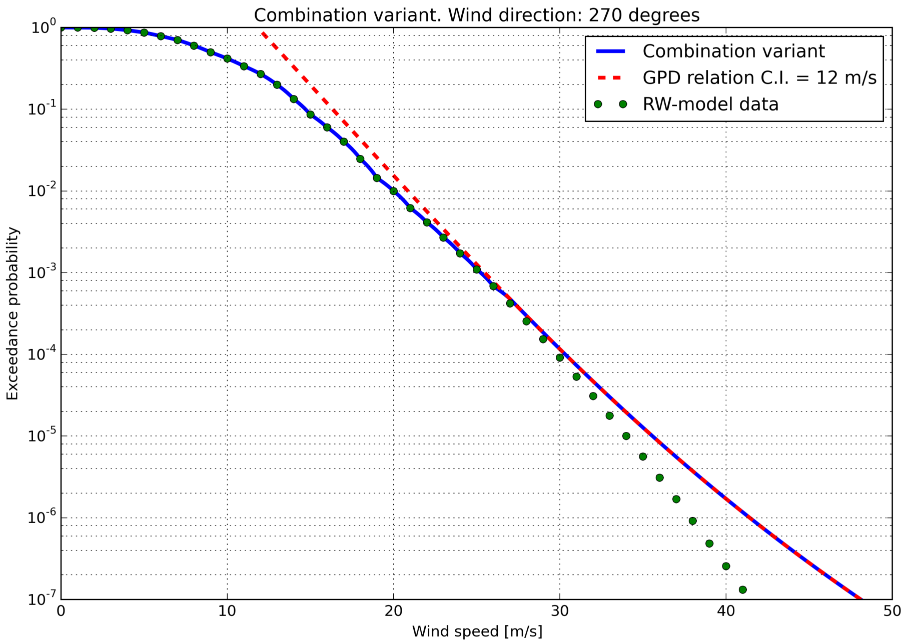

Figure 6.18: Combination variant of the RW-model and the GPD-model for West wind.

The mean μ_w and standard deviation σ_w for the shape parameter γ are chosen to be equal for all wind directions. This causes the uncertainty significance to decrease for wind directions where extreme wind speeds are less probable compared to the West wind case to which the μ_w and σ_w are tailored. Tailored μ_w and σ_w for each particular wind direction are not investigated further. However, this assumption seems plausible, because a constant 95%CI for each wind direction certainly is not.

Table 6.8: 95% confidence intervals for each wind direction. First row [degrees], second to fourth [m/s].

22.5	45	67.5	90	112.5	135	157.5	180	202.5	225	247.5	**270**	292.5	315	337.5	360
6	5	5	5	5	5	6	6	7	9	9	**10**	10	10	8	6
7	6	6	6	6	6	7	7	9	11	11	**12**	12	12	9	8
8	7	7	7	6	7	8	8	11	13	13	**14**	14	14	11	9

6.4.2. HYDRA-ZOET RESULTS FOR THE IJSSELMEERDIJK AND TIEL

In this section the results will be shown of a Hydra-Zoet computation for the IJsselmeerdijk and the dike near the city of Tiel. A varying boundary condition is the use of alternative statistical wind data. In-line with the previous chapter, three uncertainty inclusion instances will be elaborated. Namely, the 95% CI of $10[m/s]$, second $12[m/s]$ and third $14[m/s]$. Other boundary conditions are a normative overtopping discharge of $5[l/m/s]$, (effective) fetch lengths and water depths as defined by the HR2006 or retained from Hydra-Zoet 1.6.3 database, a dike profile of VNK (2012) & VNK (2014).

IJSSELMEERDIJK F260

Dike location F260 is elaborated. Its location can be found on page 43. As stated before, the location is characterised by a relative long fetch length since it faces the IJssel Lake directly to the North-West. The normative mean high water is denoted as the MHW-level. The needed crest height of the dike is denoted as the HBN-level. The results are visually summarized in figure 6.19.

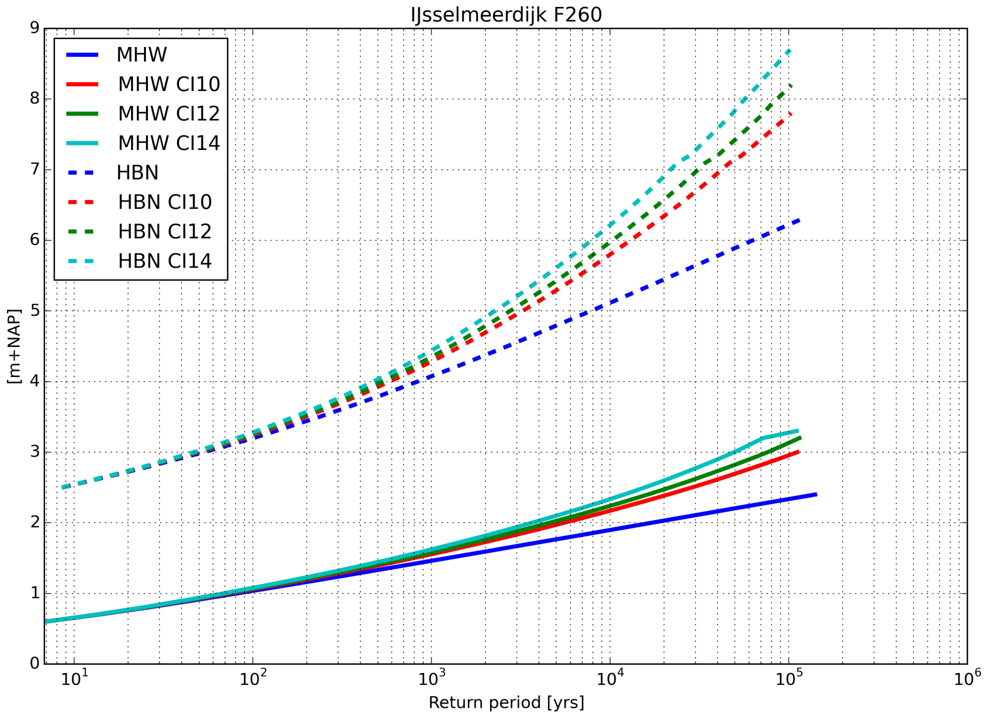

Figure 6.19: The impact of alternative wind statistical data for HBN and MHW computation for F260.

Figure 6.19 displays an enhanced bend when uncertainty is introduced. This might have to do with the properties of the GPD relation, since this has the same feature. Physical limits to the MHW and HBN are not very apparent. To put this in perspective, higher wind speeds cause a higher wind set up, which in turn allow for the development of higher wave loading on the dike. Enhanced conditions due to uncertainty show a significant impact even at more conventional return periods. The MHW CI14 relation looks unstable from $T > 70^4$ years. In any case, the attribution of the uncertainties significance seems most important.

Two return periods are specifically highlighted. The return period associated with the current safety standard and that of the design standard according to the OI2014. The MHW-level differences are presented in centimetre height according to equation (6.69), whereas the the wave conditions are presented in relative differences as equation (6.70) and equation (6.71).

$$\Delta MHW_X = MHW_X - MHW_{CI0} \tag{6.69}$$

$$R.H_{s.X} = \frac{H_{s.X}}{H_{s.CI0}} \tag{6.70}$$

$$R.T_{m-1.0.X} = \frac{T_{m-1.0.X}}{T_{m-1.0.CI0}} \tag{6.71}$$

Table 6.9: Difference of MHW-level for alternative wind uncertainty statistics at the IJsselmeerdijk.

Return period $[years]$	ΔMHW_{CI10} $[cm]$	ΔMHW_{CI12} $[cm]$	ΔMHW_{CI14} $[cm]$
4,000	+19	+24	+31
187,500	+96	+120	+117

Table 6.9 shows a moderate increase of water level at a return period of 4,000 years. The OI2014 robustness surcharges is defined as $+40cm$ for statistical and model uncertainty. The results of this computation can be assumed consistent with this. This is not the case for extreme return periods in this computation. At $T = 187,500$ years the influence is about 1 metre.

Table 6.10: Relative difference of H_s for alternative wind uncertainty statistics at the IJsselmeerdijk.

Return period $[years]$	$R.H_{s.CI10}$ $[-]$	$R.H_{s.CI12}$ $[-]$	$R.H_{s.CI14}$ $[-]$
4,000	1.05	1.06	1.08
187,500	1.14	1.16	1.19

Table 6.11: Relative difference of $T_{m-1.0}$ for alternative wind uncertainty statistics at the IJsselmeerdijk.

Return period $[years]$	$R.T_{m-1.0.CI10}$ $[-]$	$R.T_{m-1.0.CI12}$ $[-]$	$R.T_{m-1.0.CI14}$ $[-]$
4,000	1.02	1.02	1.03
187,500	1.08	1.10	1.11

Wave conditions to be found in table 6.10 and table 6.11 show an overall increase. The relative difference show that the significance is greater for wave height then for wave period.

The results for extreme return periods differ significantly from the PC-Ring analysis from section 5.2.3. In this analysis for the MHW-level, the influence was about 40 centimetres for the accounted variables at the same extreme return period of 187,500 years. Proper wave conditions could not have been retained due to computational problems. Thought, the comparison between analysis is hard to justify since boundary conditions are different, foremost the designation of uncertainties. However, the comparison does give insight about the sensitivity of the attribution of uncertainty. From this, two observations can be made. First, the attribution is most influential for the end result. Second, both analysis imply an undeniable increase in loading conditions.

TIEL DIKE 43.TG001.TG003

After seeing the effects of enhanced wind conditions for a lake system, next, the effects are investigated for an upper-river system. In this section the location of interest is the dike near the city of Tiel along the Waal, 43.tg001.tg003.

For this analysis, a capping of the river discharge is assumed of $16,500 m^3/s$ at Lobith. Furthermore, a discharge of $16,000 m^3/s$ for a return period of 1,250 years is reckoned. This complies with the KNMI'06 for 2015W+ scenario. In so, the analysis gives answer to two questions. What is the effect of a physical maximum discharge for the MHW- and HBN-level? As stated before, the Hydra-Zoet model is used for this computation. This inherently means that the river discharge and wind speed are treated as independent parameters for an upper-river system. The results are depicted in figure 6.20.

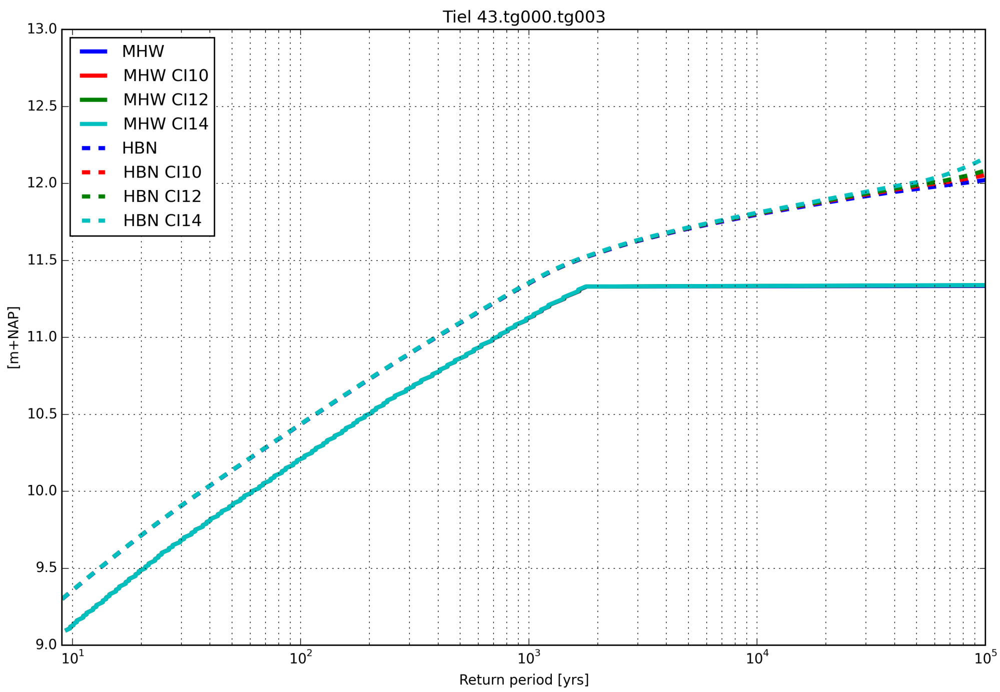

Figure 6.20: The impact of alternative wind statistical data for HBN and MHW computation for 43.tg000.tg003.

The first notable observation from figure 6.20 is the consequence of capping the river discharge. The MHW relations reach a maximum of $11.33[m+NAP]$. When this level is reached, more severe (less frequent) instances are established by an increase in wave loading and set up.

Whereas differences imposed by statistical wind uncertainty are of significant order for the former IJssel Lake analysis, differences are kept to a minimum in this exercise. At the current scale of figure 6.20, it is hard to distinguish any difference for the HBN-level and no difference is to be seen for the MHW-level.

Looking closer at the 'illustration points' of the computation, it is observed that wind speeds at high return periods, like $T = 62,500$ years, do not exceed the $14.0[m/s]$ for normative wind directions. The influence of statistical wind speed uncertainty only becomes significant from $u > 20[m/s]$ as is visualised in figure 6.8 on 95. The normative wind direction is East-North-East for all instances, except for the 100,000 years instance where the direction is East. Wind speeds and water conditions combination of other wind directions do give rise for more severe wind conditions, but this combination has a very low probability of occurrence.

As for the IJsselmeerdijk, comparable tables of results are displayed below. Again return periods of the current safety standard and that of the design standard as described by the OI2014 have been highlighted. From the tables can quickly be concluded that the influence of statistical wind speed uncertainty is close to nothing.

Table 6.12: Difference of MHW-level for alternative wind uncertainty statistics at 43.tg001.tg003.

Return period $[years]$	ΔMHW_{CI10} $[cm]$	ΔMHW_{CI12} $[cm]$	ΔMHW_{CI14} $[cm]$
1,250	0	0	0
62,500	1	1	1

Table 6.13: Relative difference of H_s for alternative wind uncertainty statistics at 43.tg001.tg003.

Return period $[years]$	$R.H_{s.CI10}$ $[-]$	$R.H_{s.CI12}$ $[-]$	$R.H_{s.CI14}$ $[-]$
1,250	1.00	1.00	1.00
62,500	1.00	1.05	1.02

The decrease in relative H_s from CI12 to CI14 can be explained since the design wind direction changes from East-North-East to East.

Table 6.14: Relative difference of $T_{m-1.0}$ for alternative wind uncertainty statistics at 43.tg001.tg003.

Return period $[years]$	$R.T_{m-1.0.CI10}$ $[-]$	$R.T_{m-1.0.CI12}$ $[-]$	$R.T_{m-1.0.CI14}$ $[-]$
1,250	1.00	1.00	1.00
62,500	1.00	1.00	1.00

In the current assumption that river discharge and wind conditions are uncorrelated and therefore independent, only one scenario can be thought of wherein statistical wind uncertainty can become of significance. This is a scenario where 'super' extreme return periods are considered, extreme periods which far exceed our human scope.

MIRRORED TIEL DIKE 43.TG001.TG003

The results from the analysis on the previous page indicate that design loading conditions concern a storm situation with eastern wind. However, from wind statistics it is known that high wind speeds are most probable from a western wind direction. These usually occur during winter months. In the following example a fictional dike location is analysed. It is situated across the previous dike location. In so, a design condition with western wind can be evaluated.

A dike location is chosen across the city of Tiel. Its location can be seen in figure 6.21. Local specifications have been estimated to be the same as in the previous analysis. The fetch length orientation and dike normal are mirrored. A radar plot of the fetch lengths can be seen in figure 6.22. The geometric dike profile is retained as well, the slope is roughly 1-to-3.

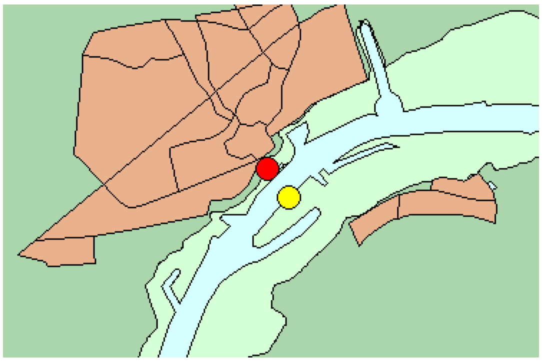

Figure 6.21: Locatie van 43.tg000.tg003

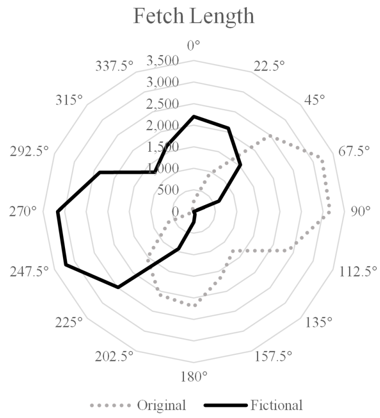

Figure 6.22: Fetch lengths of original and fictional dike location.

The results of the computation are presented in figure 6.23. Again, return periods 1,250 years and 62,500 years are highlighted in table 6.15, table 6.16 and table 6.17 for MHW, H_s and T_{m1-0} respectively. Although from the graph can be observed that the influence of statistical uncertainty is higher then in the previous example, still the influence for the considered return periods is small. An interesting observation is that wind speeds in for the extreme return period of 62,500 are in the order of 19-20 $[m/s]$, whereas in the previous example the wind speed did not exceed 14 $[m/s]$.

Table 6.15: Difference of MHW-level for alternative wind uncertainty statistics at fictional location.

Return period $[years]$	ΔMHW_{CI10} $[cm]$	ΔMHW_{CI12} $[cm]$	ΔMHW_{CI14} $[cm]$
1,250	0	0	0
62,500	0	1	1

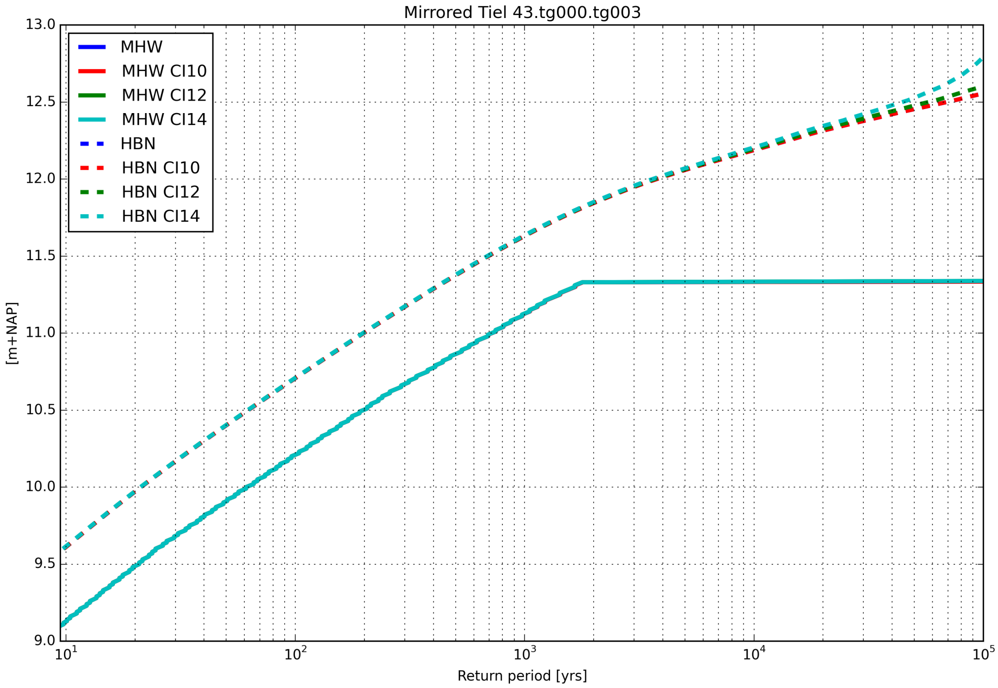

Figure 6.23: The impact of alternative wind statistical data for HBN and MHW computation for fictional location.

Table 6.16: Relative difference of H_s for alternative wind uncertainty statistics at fictional location.

Return period $[years]$	$R.H_{s.CI10}$ $[-]$	$R.H_{s.CI12}$ $[-]$	$R.H_{s.CI14}$ $[-]$
1,250	1.00	1.00	1.00
62,500	1.00	1.01	1.04

Table 6.17: Relative difference of $T_{m-1.0}$ for alternative wind uncertainty statistics at fictional location.

Return period $[years]$	$R.T_{m-1.0.CI10}$ $[-]$	$R.T_{m-1.0.CI12}$ $[-]$	$R.T_{m-1.0.CI14}$ $[-]$
1,250	1.00	1.00	1.00
62,500	1.00	1.00	1.00

6.4.3. Evaluation of implementation of P_{io} in Hydra-Zoet

In the exercises of this chapter, modified statistical wind data is used to evaluate two practical cases using the Hydra-Zoet model. The wind data has been modified to the extent that the influence of statistical wind speed uncertainty can be incorporated in the elaboration. The 95% confidence intervals of 10, 12 and 14 $[m/s]$for a return period of $T = 100,000$ years are investigated. Using the method explained in chapter 6, modified statistical wind speed tables can be generated, henceforth they can be implemented in the Hydra-Zoet model.

The use of the Hydra-Zoet model gives several benefits. One, the model approaches the task in a probabilistic manner. Hence, 'all' combinations of wind directions, wind speeds and water loading conditions are evaluated. Two, the more commonly used overtopping formula can easily be used. Three, the current VNK dike profile is implemented. Four, river discharge capping is investigated for upper-river systems.

As mentioned, two dike locations are highlighted in this chapter. Namely, the IJsselmeerdijk and the dike near the city of Tiel. The impact of statistical wind uncertainty is clear and significant for the first case. As the location is categorized as being located in a lake system, the enhanced wind speed conditions have a great effect on the subsequent wind set up and wave loading.

For the second case, the impact of uncertainty is almost none existent. The notion that wind conditions have an important role when capping of the river discharge is assumed, is true. However, governing wind speeds have not reached to the extent that uncertainty plays a role. Which is when $u_{wind} > 20[m/s]$ according to figure 6.8. The computations show that design wind speeds do not exceed the $14[m/s]$ for the dike location of Tiel, for the mirrored fictional dike location the design wind speeds are higher but do not exceed $20[m/s]$.

Again, it must be noted, that the attribution of the uncertainty significance is the primary factor in the established of the displayed enhanced loading conditions. As such, the boundary conditions of this analysis impose a vast increase in loading conditions. For the IJssel Lake system, the MHW-levels are comparable to the computed levels from section 5.2.3. The significant wave height shows an increase of +14%, whereas the significant wave period shows an increase of approximately +9% (compared to the instance without uncertainty influence). This is more then what is observed in the analysis of section 5.2.3. For upper-river systems, no apparent increase can be determined.

Observations of this analysis about the inclusion of statistical wind speed uncertainty are largely in line with results of Nicolai et al. (2010). Both analyses display a vast increase of loading conditions when statistical wind speed uncertainty is added for lake systems. However, for upper-river systems, the influence does not have to be addressed since the effects are almost non existent.

7

CONCLUSIONS AND RECOMMENDATIONS

Using the Ontwerpinstrumentarium 2014 (OI2014) as the starting point, this master thesis investigates the underpinnings of various elements within this document. The emphasize is put on uncertainty handling in dike designs. This thesis highlights three main topics. Which are: the exploration and identification of the challenges the OI2014 bring, quantification and evaluation of uncertainties in the robustness surcharge in future designs and an extended view on the implementation of statical wind uncertainty.

The assessment conditions the OI2014 imposes on dike section level are of an unprecedented order for the considered cases of Flevoland and the river Waal. However, by evaluating the method on trajectory level, a computation leads to the set safety standard in both cases. However, it is of importance to realize that the method by the OI2014 implies a national average dike design. By looking into an alternative contribution factor distribution based on most relevant failure mechanisms, a more optimized dike design can be realised. In the case of the coast of Flevoland, a required increase of crest height can be reduced from 2.0 to 1.3 metres. Likewise, along the Waal a required increase can be reduced from 1.0 to 0.7 metres.

Robustness surcharges presented in the OI2014 have been evaluated and extended to a framework of extreme return periods in this thesis. Retaining the same assumptions from Nicolai et al. (2010), the evaluation at extreme return periods shows that the enhanced loading conditions are just within the boundaries of the OI2014's robustness surcharge. In this context, the robustness surcharge remains applicable for extreme return periods, since the surcharge parameters are not seriously exceeded. A refinement to the wave period multiplication factor is advisable, since this parameter has been overestimated. Furthermore, the sensitivity analysis on additional uncertainties showed an exceptionally prominent influence of statistical wind uncertainty for enhanced hydraulic loading conditions.

An alternative implementation of statistical wind uncertainty is proposed in this thesis, this is explained in chapter 6. The model is based on the premise that uncertainty increases when higher return periods are considered, whereas previous models assume a constant deviation for all return periods. In the end, alternative statistical wind data are established wherein the influence of uncertainty is incorporated. The low return period data are identical to the Rijkoort-Weibull model, and the high return period data are modelled using the new model based on the Generalized Pareto Distribution. For lake systems this results in an overall increase for all conditions when compared to the case of no additional uncertainty. This indicates that the current robustness surcharge is insufficient. The normative water level is subjected to a 1 meter increase, whereas wave conditions increase by 16% and 10% for wave height and wave period respectively. For upper-river systems the effects are negligible. Furthermore, the analysis shows that the prior integration of statistical wind uncertainty is possible for water systems where the system is dominated by one variable. This method of modelling greatly relieves the required computing, and is therefore highly advised to be included in future models.

The results of this thesis are based on several assumptions. First, the implications of uncertainty in hydraulic loading have not been evaluated for other failure mechanisms than 'overtopping and overflow'. Especially for river systems, like the Waal, other failure mechanisms like piping need to be taken into account. Second, the limited extendibility of the Hydra-Zoet model and PC-Ring model limit the possibilities of extensive uncertainty evaluation. This problem seems only solvable if extendibility is addressed during the development of future models. Third, the prior integration of uncertainty is only valid under the assumption that correlations of the considered random variable can be disregarded due to a water system which is either strongly dominated by the considered random variable or others.

Overall can be concluded that uncertainties pose a more prominent role with the change to a new safety standard. Studies like the one before you contribute to a greater understanding of the faced uncertainties. Refinement can not be confused with lessening of loading conditions. In several special cases, refinement of the contribution factor can lead to a decrease of required crest height. However, as the wind uncertainty analysis shows, loading conditions are also expected to increase when compared to Nicolai et al. (2010). In so, the subjective decision about the allocation of uncertainty inextricably leads to a change in robustness surcharges.

In follow up of this study, future research is advisable on the following topics:

1. Optimization study towards an adjusted contribution factor and length effect factor for envisioned dike designs.
2. Investigate implementation of prior integration of uncertainty in future modelling. This method might be applicable to river discharge uncertainty for upper river systems.
3. In context of uncertainty allocation refinement, two new studies about model uncertainty can contribute to a more accurate handling of model uncertainty. Bart has contributed interesting information about standard deviations about the Young and Verhagen wave growth formula carried out by Bart (2013). Also, the water roughness coefficient for wind set-up are investigated at the Technical University of Delft. Of interest will be what the implications of these elements are in a in a probabilistic model.
4. The implications of a physical maximum river discharge for the river Waal on such be further. The possible physical maximum river discharge for the river Waal such be further elaborated, this in turn has its effect on the development of loading conditions and its inseparable uncertainties.
5. Improved estimation techniques since the FORM computation is highly unstable at extreme return periods for lake systems.

A

SENSITIVTY ANALYSIS IN MODEL UNCERTAINTY OF OVERTOPPING

The failure mechanism referred to as overtopping and overflow failure of the dike occurs when a critical amount of water is discharged over the dike. This may lead to erosion of the inner slope. Eventually, this can result in a breach in the dike system. Inundation of the hinterland is shortly followed.

OVERTOPPING

The limit state function for the critical overtopping discharge can be formulated as:

$$Z = m_{qc} q_c - m_{qo} q_o / P_t \tag{A.1}$$

In this formula two factors are appointed to take uncertainties into account. Model uncertainty of the resistance m_{qc} and the load m_{qo}. The resistance parameter for critical overtopping q_c can be addressed in two ways. As a deterministic critical overtopping discharge following from literature can be used, or as a variable following from the CIRIA model which assumes the critical overtopping discharge to be dependable on the quality of the grass cover. When the factor m_{qc} is chosen to be a stochastic, then this will actually lead to stochastic behavior of q_c.

FACTOR q_c

Selecting a critical overtopping discharge is not a crystal clear procedure. Until now there has been no golden rule on what the critical overtopping discharge should be. Some ranges are defined by van der Meer (2012) and The EurOtop Team (2007). Their classification and can be seen in table A.1 and table A.2 respectively.

Table A.1: Design rules of Rijkswaterstaat (2014a) as proposed by van der Meer (2012)

Requirement	q_c [l/s/m]
practical no overtopping	0.1
permeable inner slide	5
reinforced inner slope, or thick clay layer	10

Table A.2: Design rules of The EurOtop Team (2007)

hazard type and reason	q_c [l/s/m]
no damage to crest and rear face of embankment if not protected	0.1
no damage to crest and rear face of grass covered embankment of clay	1-10
no damage if crest and rear slope are well protected	50-200

These tables are generally used for the design and assessment of a dike. They show several similarities in the lower regions of the critical overtopping discharge, however the biggest difference is that EurOtop allows for far greater overtopping discharges in its upper class. This has to do with the definition of classes in table of The EurOtop Team (2007), which extends to other flood defenses then dikes alone. It can be concluded that these studies show consensus about the maximum allowable overtopping discharge.

FACTOR m_{qc}

The resistance in the limit state function of equation (A.1) is denoted as the critical overtopping discharge and an additional factor. This factor should comprehend all uncertainties of the assumptions and approaches used to define such a requirement. As it is implemented in PC-Ring, the critical discharge is given a stochastic value which has a normal distribution, mean and standard deviation of $[\mu;\sigma] = [1.0;0.5]$. This values seem rather arbitrary chosen values. In this paragraph, an attempt is done to justify these values.

For the factor m_{qc} two options remain. Option one would be $m_{qc}= N[1.0;0.0]$, in this way q_c would be seen as solely a deterministic value. No uncertainties in the allocation of q_c are considered. This is understandable, since the allocation of q_c has been done by the user's preference. From the viewpoint that the user defines what is acceptable (and what not) this seems like a sound reasoning. The biggest task lies in the proper designation of q_c.

Another sensible thought would be to consider $\sigma_{m_{qc}} \nleq 0$, and by doing so introducing a factor of uncertainty. Although many research has been done towards critical overtopping discharges in order to comprehend this parameter, hardly any has given a sufficiently accurate explanation to why this parameter should be seen as a deterministic value. Considering this value to be a random variable means that the erosion that should occur given the predefined critical overtopping discharge might differ from what is expected. This can be due to a variation in the expected strength of the grass cover or a mis-appointment in the quantification of the critical overtopping discharge. The resistance of a grass cover is dependable on many factors and is far from constant in time and space. Following these considerations it can be justified to assign some other value then zero for $\sigma_{m_{qc}}$. However, no documentation exists on what this value should be. A standard value of $\sigma_{m_{qc}} = 0.5$ can be found in PC-Ring. This means that when a critical overtopping discharge of $5l/s/m$ is chosen, the probability that the real value of q_c will lie between 4.5 and 5.5 is approximately 68%. For this research, we assume the possibility of parameter $\sigma_{m_{qc}}$ to be 1.0 to see the effects of an increase of uncertainty.

ALLOWING FOR A HIGHER q_c

Many dikes and other flood defenses have been designed using the 2% rune-up requirement. van der Meer (2012) tries to put this into perspective. This requirement leads to 2 out of 100 waves that will reach the top and will cause overtopping. Therefore, if the average wave period is 6 seconds, only 12 waves will reach the crest during one hour. A rough estimation would be that this corresponds with 0.1 l/s/m overtopping discharge. For sea- and lake dikes the waves are expected to be larger, corresponding with an 1.0 l/s/m overtopping discharges. Still these figures do not give a real tangible representation of the expected erosion of the dike, let alone failure of the dike. For this, experiments have been done with a wave simulator which shows that overtopping discharges of 0.1 and 1.0 l/s/m hardly cause any erosion to the inner slope. When the overtopping discharge is increased to 10 l/s/m, first signs of erosion do occur on a high quality grass cover. Various (more severe) erosion patterns are to be expected if the slope is not of good quality or has an inhomogeneous vegetation.

According to Rijkswaterstaat (2014a), the presented values for overtopping discharge in table A.1 are maybe too conservative. Conservative in this context means that at this value, failure of the revetment or grass cover is unlikely and failure of the dike is practically impossible. This can be visually explained using figure A.1. The document notes that for design purposes different values should be considered. A qualitative research has shown that for design purposes far greater critical overtopping discharges can be used. The research points out that failure of a dike is reached when a deterministic overtopping discharge between 10 and 50 l/s/m would occur. Another specification would be an overtopping discharge with a log-normal distribution with $\mu = 40 l/s$ and $\sigma = 50 l/s$. In the end it is concluded that these values are not recommended at this time, since they are not sufficiently substantiated and tested.

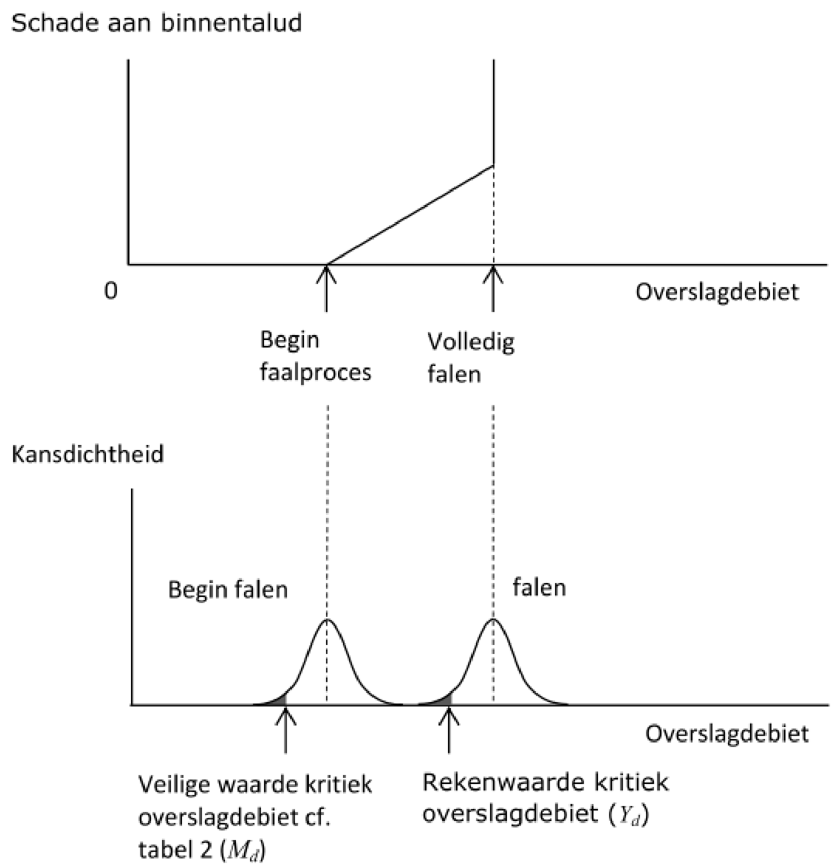

Figure A.1: Inner slope concept of failure (Rijkswaterstaat, 2014a).

Though the idea of conservative values of table A.1 is also backed in van der Meer (2012). In the report they derive a method to quantify the load on the grass cover as the cumulative overtopping discharge. The top layer is expected to be most vulnerable to failure when the maximum flow velocity occurs during an overtopping wave. Subsequently van der Meer (2012) states that the total time of the flow velocity which exceeds the critical flow velocity is important for erosion of the top layer. This leads to the following equation (A.2). Using an overtopping discharge simulator, the volume of each wave is known, and subsequently the maximum velocity can be derived. Tests result in the following damage definition as proposed in table A.3.

$$\text{cumulative overtopping discharge} = \sum \left(u^2 - u_c^2\right) \tag{A.2}$$

Table A.3: Damage definition, source: (van der Meer, 2012)

Damage definition	$\sum \left(u^2 - u_c^2\right)$
Start of erosion of top layer	500 $[m^2/s^2]$
Several bare spots in top layer	1,000 $[m^2/s^2]$
Failure of top layer	3,500 $[m^2/s^2]$

The table A.4 shows results of a test for a dike with a overtopping discharge of $q_c = 5l/s/m$ on a sand dike during a 6 hours simulation. In the right column the cumulative overtopping load is presented. Comparing these test result with the damage definitions from table A.3, and when wave heights of 3 meters are considered to be comparable to a normative storm, it follows that only minor damage is expected to a top layer. From this test can be concluded that $q_c = 5l/s/m$ is a conservative value. The cumulative overtopping load at $q_c = 10l/s/m$ seems to be in an acceptable range. In the end it will come down to the duration of the storm and the wave height. The overtopping load would not be a constant parameter during the whole storm. As the storm increases in strength, surge will increase, and wave height will do as well. The total time of $H_s = 2-3m$ is maybe only a matter of a couple of hours. A thorough simulation might give better insight about this phenomena. For this research, the impact of a $q_c = 10l/s/m$ will be investigated. An argument to research this value is that this will lead to a more tailored design.

Table A.4: Sand dike, $u_c = 4m/s$, duration is 6 hours, source: (van der Meer, 2012)

Significant wave height [m]	$\sum \left(u^2 - u_c^2\right)$	$[m^2/s^2]$	
	$q_c = 5$ l/s/m	$q_c = 10$ l/s/m	$q_c = 30$ l/s/m
$H_s = 1m$	24	132	1,548
$H_s = 2m$	348	894	4,110
$H_s = 3m$	714	1,578	5,652

CASE STUDY: IJSSELMEERDIJK

The considerations of the previous section will be further examined. They will be applied in the case study of the Flevokust at trajectory 8_3. The PC-Ring model will be used to evaluate the different options. In all, five possible options will be considered. First the standard settings of the PC-Ring model are used to compute a failure probability of the system. Second, a deterministic approach is looked at following the users choice of a normative q_c. Third, the possibility of an increase of uncertainty is introduced. Fourth, an increase in allowable q_c is considered following research done by van der Meer (2012) and Rijkswaterstaat (2014a). And last, the use of the CIRIA model which approaches q_c as a value which follows from the strength parameters of the inner slope. The options are summarized in table A.5. Due to the limitations of PC-Ring, other distributions such as the log-normal distribution as proposed by Rijkswaterstaat (2014a) can not be evaluated further.

The IJsselmeerdijk has a grass cover on its inner slope. This indicates that the top layer is fairly permeable. Also, considering that the embankment is made out of sand, it can be concluded that this is hardly an impervious soil body. If we assume that the expected normative significant wave height during storm conditions is larger then 2 meters, then from table A.1 it follows that a critical overtopping discharge of 5 l/s/m is a 'safe' figure to reckon with. Therefore, this value for q_c will be used as the standard critical overtopping discharge in the following computations.

Table A.5: Considered options for evaluation for variations in q_c and m_{qc}.

Option	m_{qc}	q_c
1	N[1;0,5]	5
2	N[1;0]	5
3	N[1;1]	5
4	N[1;0,5]	10
5	N[1;0,5]	CIRIA

RESULTS

In the following paragraph results will be shown in which a greater insight is gained about the impact of certain design choices regarding overtopping. PC-Ring is used to compute these results. in this model, the dike system is approached as series-system composed out of elements/sections. The model takes account for correlation between parameters of different dike sections.

In figure A.2 the expected return period of failure of the dike system is presented. What immediately shows is that the CIRIA-model results in a lower probability of failure of the system. Furthermore, variation in failure probability are as expected. Limiting the standard deviation or allowing for a higher overtopping discharges result in a lower probability of failure. When q_c is seen as a purely deterministic value, the expected return period shows an increase of 47% compared to the standard settings. Allowing for more overtopping discharge gives an increase of 150%. Increasing the standard deviation, as has been done in option 3 with N[1;1], shows a higher probability of failure. An decrease in return period by 48%. The use of the CIRIA model shows an increase of 692%.

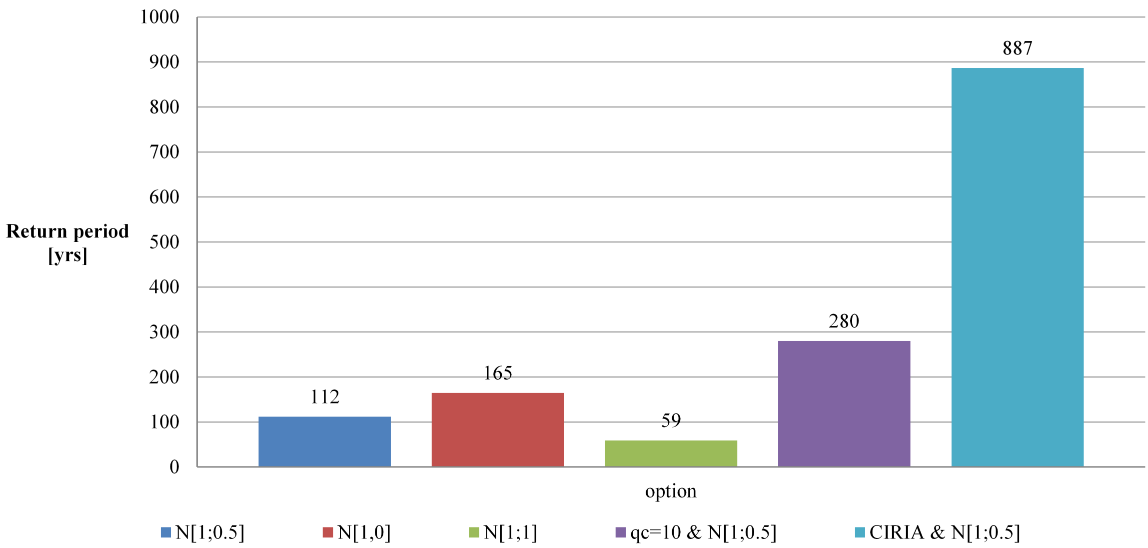

Figure A.2: Failure of dike trajectory 8_3 according to PC-Ring, several options for m_{qc} are considered.

Figure A.3 shows results of the reliability indexes per dike sections. The failure probability of the system as seen in figure A.2 is decomposed, or rather made up, out of the dike sections which belong to the system. Several observations can be made. Dike section 43 shows great dependency on which model is used to compute its failure probability. The CIRIA model gives the impression of a far safer dike then the other options observe. This may indicate that the CIRIA model does allow for greater overtopping discharges then $q_c = 10$ l/s/m for this specific dike section. The high failure probability as computed by options 1 to 4 for dike section 43 will be the main cause of the high failure probability of the system. When comparing the CIRIA model results at other dike sections to the other options, the results are comparable to that of the $q_c = 10$ l/s/m. Furthermore, the failure probability is relatively higher at dike sections 52 and 53 then other dike sections of the trajectory. This can be explained by the location of the dike sections. Their specific location allows for more severe loading conditions due to a relatively large fetch length which extends all across the IJssel lake and is almost perpendicular to the normal of the dike. This effect can be justified when looking at dike section 54 where the orientation changes and the failure probability reduces.

Overall, the observation holds that an increase of the standard deviation of m_{qc} leads to a higher computed failure probability, and the allowance of more overtopping discharges leads to a lower computed failure probability. The difficult task of choosing an appropriate critical overtopping discharge can be remedied by using a model such as the CIRIA model, however one should be cautious with its results. In some cases this may lead opportunistic values. And since a series system is "as strong as its weakest link", misinterpretation can have devastating consequences. Further research is needed in these specific cases, therefore applicability of the CIRIA model becomes highly questionable.

For dike design purposes, the allowance of a higher overtopping discharge should be carefully thought out. Other failure mechanism might come in to play when the overall stability of the dike is undermined. Also, the dike will be harder to inspect during extreme overtopping scenarios. In this sense allowing for a higher overtopping discharge can be theoretical sound, but in reality unpractical.

As shown in this section, uncertainties in overtopping can have a significant influence on the elaborated failure probability of a system. This is less of a concern when a flood defense has a sufficiently robust design, so that variations do not cause alarming conditions. However, in cases where the margin of robustness is small, uncertainties can have a crucial impact. This can also be of importance during a dike design when it is preferred to optimize dimensions, such as the dike height or the materials used. There emerges a trade-off between allowable uncertainty and the benchmark which is considered. The biggest challenge in this would be the comprehensiveness in the quantification of uncertainty.

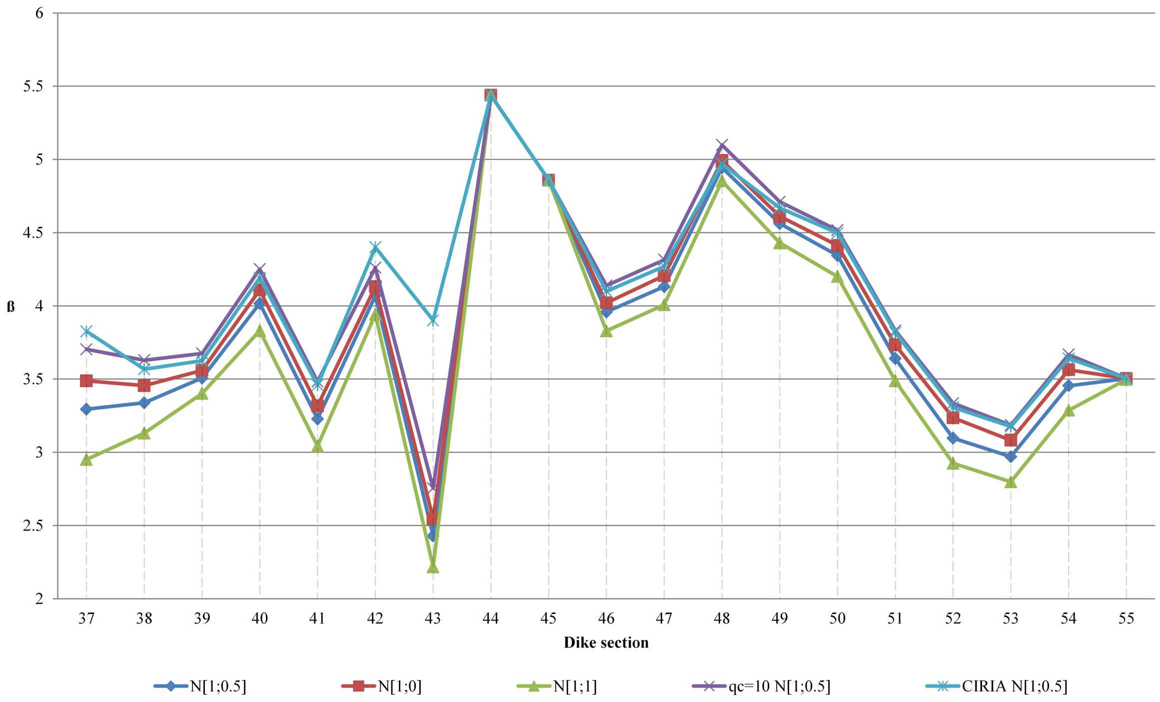

Figure A.3: Failure of dike sections of trajectory 8_3 according to PC-Ring, several options for m_{qc} are considered.

Due to the Flevodijk its design, other failure mechanisms are less of a concern. For instance, the sand dike makes it nearly impossible for piping to occur. Assuming that the design rules of piping are accurate enough to come to the conclusion that this phenomenon indeed does not play a role in the establishment of the system's failure probability, it can also be concluded that its uncertainties do not impose variation in the failure probability. Following this reasoning, it should be considered to come up with smart designs instead of trying to balance uncertainties in multiple elements. Feasibility can be assessed using a cost-benefit analysis.

B

Robustness calculations for three locations at the Flevoland coastline

Table B.1: Post- versus pre-processing of robustness surcharge in Hydra-Zoet.

Parameter	Unit	Hydra-Zoet	post-Surcharge	Hydra-Zoet	post-Surcharge
F260 IJsselmeerdijk		1/4,000	1/4,000	1/187,500	1/187,500
Lake level	m+NAP	-0.16	-0.16	-0.16	-0.16
U	m/s	31.7	31.7	37.6	37.6
htoe	m+NAP	1.71	2.11	2.46	2.86
Hs	m	2.27	2.497	2.7	2.97
Tm-1.0	s	5.9	6.49	6.3	6.93
Wave direction	degrees	323	323	323	323
Wind direction		NW	NW	NW	NW
Exceedance frequincy	%	70.5		74.8	
Required crest	m	4.69	5.91	6.539	7.94
F300 IJsselmeerdijk					
Lake level	m+NAP	-0.19	-0.19	-0.19	-0.19
U	m/s	32.3	32.3	39.6	39.6
htoe	m+NAP	1.71	2.11	2.39	2.79
Hs	m	1.91	2.101	2.33	2.563
Tm-1.0	s	5.5	6.05	6	6.6
Wave direction	degrees	323	323	323	323
Wind direction		NW	NW	NW	NW
Exceedance frequincy	%	54.4		71.7	
Required crest	m	3.814	4.85	5.454	6.74
hm3.4 Oostvaardersdijk					
Lake level	m+NAP	-0.32	-0.32	-0.26	-0.26
U	m/s	35.3	35.3	40.9	40.9
hteen	m+NAP	1.18	1.58	1.77	2.17
Hs	m	2.11	2.321	2.42	2.662
Tm-1.0	s	5.6	6.16	6	6.6
Wave direction	degrees	273	273	273	273
Wind direction		W	W	W	W
Exceedance frequincy	%	54.8		64.4	
Required crest	m	4.732	5.95	6.14	7.59

Unit	Hydra-Zoet	post-Surcharge	Hydra-Zoet	post-Surcharge	pre-processing	pre-processing
years	1/4,000	1/4,000	1/187,500	1/187,500	1/4,000	1/187,500
m+NAP	-0.16	-0.16	-0.16	-0.16	-0.16	-0.16
m/s	31.7	31.7	37.6	37.6	31.7	37.6
m+NAP	1.71	2.11	2.46	2.86	2.11	2.86
m	2.27	2.497	2.7	2.97	2.49	2.97
s	5.9	6.49	6.3	6.93	6.5	7
degrees	323	323	323	323	323	323
	NW	NW	NW	NW	NW	NW
%	70.5		74.8		70.5	74.7
m	4.69	5.91	6.539	7.94	5.893	7.995
m+NAP	-0.19	-0.19	-0.19	-0.19	-0.19	-0.17
m/s	32.3	32.3	39.6	39.6	32.3	37.7
m+NAP	1.71	2.11	2.39	2.79	2.11	2.8
m	1.91	2.101	2.33	2.563	2.1	2.58
s	5.5	6.05	6	6.6	6	6.6
degrees	323	323	323	323	323	323
	NW	NW	NW	NW	NW	NW
%	54.4		71.7		54.2	69.3
m	3.814	4.85	5.454	6.74	4.84	6.759
m+NAP	-0.32	-0.32	-0.26	-0.26	-0.31	-0.26
m/s	35.3	35.3	40.9	40.9	35.3	40.9
m+NAP	1.18	1.58	1.77	2.17	1.59	2.18
m	2.11	2.321	2.42	2.662	2.33	2.67
s	5.6	6.16	6	6.6	6.2	6.6
degrees	273	273	273	273	273	273
	W	W	W	W	w	W
%	54.8		64.4		52.3	61
m	4.732	5.95	6.14	7.59	6.234	7.843

C

ALTERNATIVE APPROACH: THE ROBUSTNESS FACTOR PROCESSED IN THE HYDRA-ZOET DATABASE

A question arises when the robustness factors from table 5.1 are used for the design of a dike. If it is acknowledged that the lack of understanding about the physical process of water hydraulics and the inapplicability of statistical analysis might lead to more extreme conditions, then a discussion about the proper inclusion of a robustness surcharge can be held. Considering the current Hydra-Zoet model, in which step of the process should this robustness surcharge be applied to come to a most satisfying result?

In the current approach by den Bieman and Smale (2014), only one (dominant) wind direction is elaborated for the determination of the robust design values. Therefore, the method implies that in this enhanced situation the overall loading distribution stays the same. However, it is conceivable that enhanced loading conditions might change the marginal contribution of each wind direction. In turn, the dominant wind direction can be different then the one observed in the former method.

To address this issue, a modification will be made in the 'Hydra physics database'. In figure C.1 the structure of the Hydra-Zoet model is visualized. The 'Hydra physics database' can be found as the third step. Because the WAQUA and SWAN models take significant computation time, databases of expected water level and wave conditions are pre-generated (in Dutch these are called "productieberekeningen"). Thereafter, the database is used in the semi-probabilistic Hydra-Zoet model in which statistical data and dike characteristics are also incorporated.

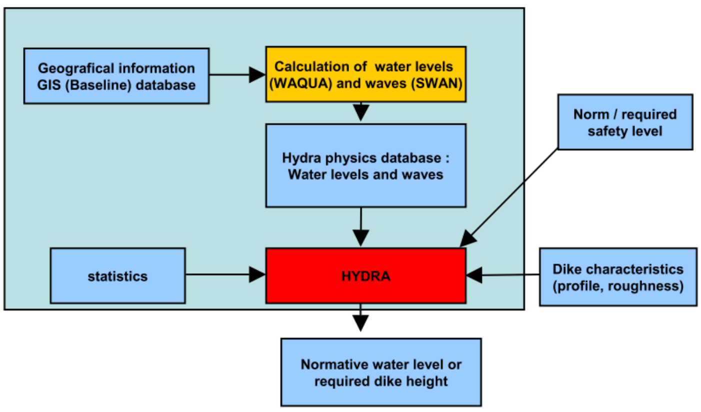

Figure C.1: Structure of the Hydra-Zoet model (Geerse et al., 2011)

By applying the robustness factors of table 5.1 in the 'Hydra physics database', the benefit is gained that the enhanced conditions are used in the probabilistic determination of the hydraulic load. The same locations which were considered in the previous section will be analysed using this modified database.

The table C.1 shows the differences between the parameters retained from the conventional robustness method (as carried out in the previous section) and the alternative method in which a modification is made in the 'Hydra physics database'. The results show that the differences between the two methods is not very apparent looking at the dike locations of Flevoland. In all cases the lake level, MHW level, and wave conditions are roughly the same. The exceedance frequency of the dominant wind directions does change in several cases, especially for the more extreme cases of 1/187,500 conditions. For the Marker Lake dike "Oosvaardersdijk" this results in a notable change in required crest height, for the IJssel Lake cases hardly any disparities show.

Table C.1: Differences between post- and pre-processing of the robustness factor.

Parameter	**Unit**	**F260 IJsselmeerdijk**		**F300 IJsselmeerdijk**		**hm3.4 Oostvaardersdijk**	
Return period	years	1/4,000	1/187,500	1/4,000	1/187,500	1/4,000	1/187,500
Lake level	m+NAP	0.00	0.00	0.00	-0.02	0.01	0.00
U	m/s	0.00	0.00	0.00	1.90	0.00	0.00
htoe	m+NAP	0.00	0.00	0.00	-0.01	0.01	-0.01
Hs	m	-0.01	0.00	0.00	-0.02	0.01	-0.01
Tm-1.0	s	0.01	-0.07	-0.05	0.00	0.04	0.00
Wave direction	degrees	0	0	0	0	0	0
Wind direction		same	same	same	same	same	same
Exceedance frequency	%	0	-0.1	0.2	2.4	2.5	3.4
Required crest	m	-0.02	-0.05	-0.01	-0.02	0.28	0.25

The results suggest that no significant disparities occur when this alternative approach for the robustness surcharge is used. This might be due to the fact that a distinct dominant wind direction is the primary contributor to the hydraulic load for the locations considered. Since some minor disparities do show from this analysis, the significance of this alternative approach might become larger for other locations where the dominant wind direction is not so obvious and is therefore recommended to look into.

Also, the cases in which a more extreme conditions are incorporated do show a higher sensitivity to the alternative approach. This means that the impact the robustness surcharge (and its uncertainties) becomes larger when one looks at loading conditions affiliated to these extreme return periods.

Still, analysis like these show the importance to carefully think threw the implications such a robustness surcharge has. Hydraulic loading conditions are derived using methods which strive to give an as accurate result as possible. A robustness surcharge puts these results into perspective again, since the ones accurate answer is deemed not accurate any more. Following this observation, a designer has to question himself to whether it is wise to further improve physical models and statistical analysis, or by properly quantifying the impact of its the uncertainties. As shown in this chapter, the latter can have a significant influence and will therefore be further analysed in the next chapter.

D

STATISTICAL WIND DATA

Table D.1: Wind direction probability, taken from Hydra-Zoet 1.6.3

Direction	**Probability**
22.5	0.045241
45.0	0.055728
67.5	0.064416
90.0	0.057525
112.5	0.041446
135.0	0.044442
157.5	0.058224
180.0	0.074503
202.5	0.090682
225.0	0.095975
247.5	0.090882
270.0	0.075901
292.5	0.057525
315.0	0.050834
337.5	0.049536
360.0	0.047139

Table D.2: Wind speed exceedance probability for 12-hrs interval. Taken from Hydra-Zoet 1.6.3.

u	NNO	NO	ONO	O	OZO	ZO	ZZO	Z
0	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00
1	9.96E-01	9.97E-01	1.00E+00	1.00E+00	9.99E-01	9.99E-01	9.99E-01	1.00E+00
2	9.79E-01	9.85E-01	9.95E-01	9.87E-01	9.84E-01	9.85E-01	9.89E-01	9.87E-01
3	9.16E-01	9.49E-01	9.65E-01	9.30E-01	9.04E-01	9.34E-01	9.44E-01	9.47E-01
4	7.78E-01	8.71E-01	8.88E-01	8.17E-01	7.53E-01	7.79E-01	8.35E-01	8.58E-01
5	6.33E-01	7.58E-01	7.68E-01	6.60E-01	5.56E-01	5.91E-01	6.65E-01	7.17E-01
6	4.67E-01	6.20E-01	6.41E-01	5.14E-01	3.82E-01	4.15E-01	5.14E-01	5.79E-01
7	3.37E-01	4.78E-01	5.12E-01	3.60E-01	2.29E-01	2.69E-01	3.64E-01	4.20E-01
8	2.17E-01	3.45E-01	3.81E-01	2.25E-01	1.15E-01	1.50E-01	2.40E-01	2.91E-01
9	1.50E-01	2.35E-01	2.72E-01	1.29E-01	5.35E-02	8.07E-02	1.43E-01	2.11E-01
10	1.04E-01	1.52E-01	1.79E-01	7.71E-02	1.96E-02	3.89E-02	7.65E-02	1.38E-01
11	5.95E-02	8.06E-02	9.85E-02	4.72E-02	1.10E-02	1.65E-02	3.91E-02	9.12E-02
12	3.17E-02	4.47E-02	4.81E-02	2.52E-02	8.70E-03	1.04E-02	1.86E-02	5.81E-02
13	1.69E-02	1.80E-02	1.35E-02	1.09E-02	6.00E-03	6.92E-03	1.30E-02	3.28E-02
14	7.42E-03	7.44E-03	7.59E-03	5.80E-03	4.50E-03	5.40E-03	6.50E-03	1.99E-02
15	3.71E-03	3.77E-03	3.91E-03	2.84E-03	2.20E-03	2.45E-03	4.39E-03	1.22E-02
16	1.74E-03	1.77E-03	1.86E-03	1.28E-03	8.98E-04	9.97E-04	2.24E-03	6.96E-03
17	7.77E-04	7.71E-04	8.18E-04	5.26E-04	3.32E-04	3.69E-04	1.09E-03	3.71E-03
18	3.30E-04	3.11E-04	3.32E-04	1.98E-04	1.12E-04	1.26E-04	5.03E-04	1.86E-03
19	1.35E-04	1.17E-04	1.24E-04	6.81E-05	3.45E-05	3.95E-05	2.23E-04	8.78E-04
20	5.36E-05	4.08E-05	4.32E-05	2.15E-05	9.69E-06	1.13E-05	9.43E-05	3.92E-04
21	2.06E-05	1.33E-05	1.39E-05	6.20E-06	2.50E-06	2.99E-06	3.80E-05	1.64E-04
22	7.73E-06	4.03E-06	4.16E-06	1.64E-06	5.95E-07	7.23E-07	1.46E-05	6.48E-05
23	2.83E-06	1.13E-06	1.15E-06	4.01E-07	1.31E-07	1.61E-07	5.32E-06	2.41E-05
24	1.01E-06	2.97E-07	2.96E-07	9.03E-08	2.70E-08	3.31E-08	1.83E-06	8.41E-06
25	3.54E-07	7.25E-08	7.05E-08	1.88E-08	5.21E-09	6.24E-09	5.96E-07	2.76E-06
26	1.21E-07	1.63E-08	1.55E-08	3.62E-09	9.28E-10	1.07E-09	1.82E-07	8.49E-07
27	3.99E-08	3.43E-09	3.19E-09	6.52E-10	1.60E-10	1.65E-10	5.22E-08	2.44E-07
28	1.28E-08	6.71E-10	6.13E-10	1.22E-10	2.83E-11	2.43E-11	1.40E-08	6.55E-08
29	3.97E-09	1.20E-10	1.04E-10	0.00E+00	0.00E+00	0.00E+00	3.52E-09	1.65E-08
30	1.19E-09	1.05E-11	3.84E-12	0.00E+00	0.00E+00	0.00E+00	8.26E-10	3.87E-09
31	3.37E-10	7.82E-13	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.80E-10	8.46E-10
32	9.56E-11	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	3.55E-11	1.66E-10
33	1.17E-11	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	4.41E-12	2.07E-11
34	1.47E-13	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
35	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
36	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
37	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
38	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
39	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
40	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
41	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
42	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00

u	ZZW	ZW	WZW	W	WNW	NW	NNW	N
0	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00	1.00E+00
1	1.00E+00	1.00E+00	9.98E-01	9.98E-01	1.00E+00	9.99E-01	9.97E-01	9.66E-01
2	9.94E-01	9.95E-01	9.90E-01	9.93E-01	9.92E-01	9.88E-01	9.81E-01	9.49E-01
3	9.71E-01	9.73E-01	9.66E-01	9.71E-01	9.56E-01	9.53E-01	9.31E-01	8.56E-01
4	9.24E-01	9.22E-01	9.17E-01	9.26E-01	9.06E-01	8.80E-01	8.27E-01	6.94E-01
5	8.21E-01	8.42E-01	8.64E-01	8.69E-01	8.28E-01	7.86E-01	7.03E-01	5.54E-01
6	7.16E-01	7.59E-01	7.92E-01	7.85E-01	7.55E-01	6.98E-01	5.99E-01	4.21E-01
7	6.02E-01	6.63E-01	7.02E-01	7.04E-01	6.67E-01	5.79E-01	4.78E-01	3.13E-01
8	4.82E-01	5.79E-01	6.21E-01	6.01E-01	5.60E-01	4.70E-01	3.59E-01	2.15E-01
9	3.73E-01	4.87E-01	5.27E-01	4.98E-01	4.62E-01	3.76E-01	2.36E-01	1.42E-01
10	2.87E-01	4.10E-01	4.50E-01	4.16E-01	3.64E-01	2.90E-01	1.71E-01	1.01E-01
11	1.96E-01	3.28E-01	3.55E-01	3.37E-01	2.87E-01	2.17E-01	1.08E-01	7.49E-02
12	1.33E-01	2.61E-01	2.86E-01	2.70E-01	2.17E-01	1.52E-01	7.41E-02	5.12E-02
13	9.49E-02	1.84E-01	2.12E-01	1.99E-01	1.43E-01	1.04E-01	4.57E-02	2.88E-02
14	5.91E-02	1.36E-01	1.47E-01	1.33E-01	9.35E-02	5.99E-02	3.41E-02	9.58E-03
15	3.96E-02	1.02E-01	9.93E-02	8.59E-02	6.19E-02	4.61E-02	2.23E-02	5.29E-03
16	2.37E-02	6.51E-02	6.73E-02	6.01E-02	4.33E-02	2.86E-02	1.26E-02	2.80E-03
17	1.43E-02	4.40E-02	4.74E-02	4.03E-02	3.41E-02	1.72E-02	7.32E-03	1.43E-03
18	8.18E-03	2.80E-02	2.83E-02	2.47E-02	2.05E-02	9.20E-03	4.47E-03	7.03E-04
19	5.18E-03	1.80E-02	1.71E-02	1.44E-02	1.19E-02	6.17E-03	2.62E-03	3.35E-04
20	3.16E-03	1.00E-02	1.00E-02	1.00E-02	6.74E-03	4.07E-03	1.50E-03	1.54E-04
21	1.86E-03	7.20E-03	4.57E-03	6.18E-03	4.56E-03	2.64E-03	8.28E-04	6.82E-05
22	1.06E-03	3.82E-03	2.95E-03	4.13E-03	3.02E-03	1.69E-03	4.50E-04	2.90E-05
23	5.84E-04	2.74E-03	1.85E-03	2.70E-03	1.97E-03	1.07E-03	2.38E-04	1.18E-05
24	3.13E-04	6.89E-04	1.14E-03	1.73E-03	1.26E-03	6.68E-04	1.24E-04	4.63E-06
25	1.62E-04	3.98E-04	6.93E-04	1.10E-03	7.96E-04	4.13E-04	6.34E-05	1.74E-06
26	8.18E-05	2.26E-04	4.12E-04	6.82E-04	4.94E-04	2.52E-04	3.16E-05	6.25E-07
27	4.00E-05	1.27E-04	2.41E-04	4.20E-04	3.01E-04	1.51E-04	1.55E-05	2.15E-07
28	1.91E-05	6.95E-05	1.39E-04	2.55E-04	1.81E-04	8.93E-05	7.39E-06	7.08E-08
29	8.81E-06	3.76E-05	7.88E-05	1.53E-04	1.07E-04	5.21E-05	3.45E-06	2.23E-08
30	3.96E-06	2.01E-05	4.41E-05	9.11E-05	6.27E-05	2.99E-05	1.57E-06	6.73E-09
31	1.72E-06	1.05E-05	2.42E-05	5.32E-05	3.60E-05	1.69E-05	6.97E-07	1.93E-09
32	7.27E-07	5.41E-06	1.31E-05	3.10E-05	2.03E-05	9.36E-06	3.02E-07	5.51E-10
33	2.99E-07	2.76E-06	7.02E-06	1.77E-05	1.13E-05	5.12E-06	1.27E-07	6.73E-11
34	1.19E-07	1.38E-06	3.70E-06	1.00E-05	6.21E-06	2.75E-06	5.23E-08	8.44E-13
35	4.58E-08	6.78E-07	1.93E-06	5.62E-06	3.36E-06	1.46E-06	2.10E-08	0.00E+00
36	1.71E-08	3.29E-07	9.92E-07	3.09E-06	1.79E-06	7.57E-07	8.18E-09	0.00E+00
37	6.19E-09	1.56E-07	5.04E-07	1.70E-06	9.40E-07	3.87E-07	3.11E-09	0.00E+00
38	2.18E-09	7.32E-08	2.54E-07	9.15E-07	4.87E-07	1.95E-07	1.13E-09	0.00E+00
39	7.31E-10	3.36E-08	1.26E-07	4.86E-07	2.49E-07	9.67E-08	4.09E-10	0.00E+00
40	2.37E-10	1.52E-08	6.20E-08	2.55E-07	1.25E-07	4.70E-08	0.00E+00	0.00E+00
41	7.93E-11	6.71E-09	3.01E-08	1.32E-07	6.22E-08	2.25E-08	0.00E+00	0.00E+00
42	4.05E-11	2.92E-09	1.45E-08	6.73E-08	3.05E-08	1.07E-08	0.00E+00	0.00E+00

Table D.3: U_{pot} to U_{10} conversion. Retained from Hydra-Zoet 1.6.3.

Up (m/s)	**U10** (m/s)
0	0
1	1.12
2	2.25
3	3.37
4	4.49
5	5.61
6	6.74
7	7.86
8	8.97
9	10.06
10	11.14
11	12.21
12	13.28
13	14.34
14	15.39
15	16.44
16	17.49
17	18.53
18	19.56
19	20.59
20	21.62
21	22.64
22	23.66
23	24.68
24	25.69
25	26.69
26	27.69
27	28.69
28	29.69
29	30.68
30	31.67
31	32.65
32	33.64
33	34.62
34	35.59
35	36.56
36	37.53
37	38.5
38	39.47
39	40.43
40	41.39
41	42.34
42	43.3
43	44.25
44	45.2
45	46.14
46	47.08
47	48.03
48	48.96
49	49.9
50	50.83

E

INFORMATIVE MAPS OF THE NETHERLANDS

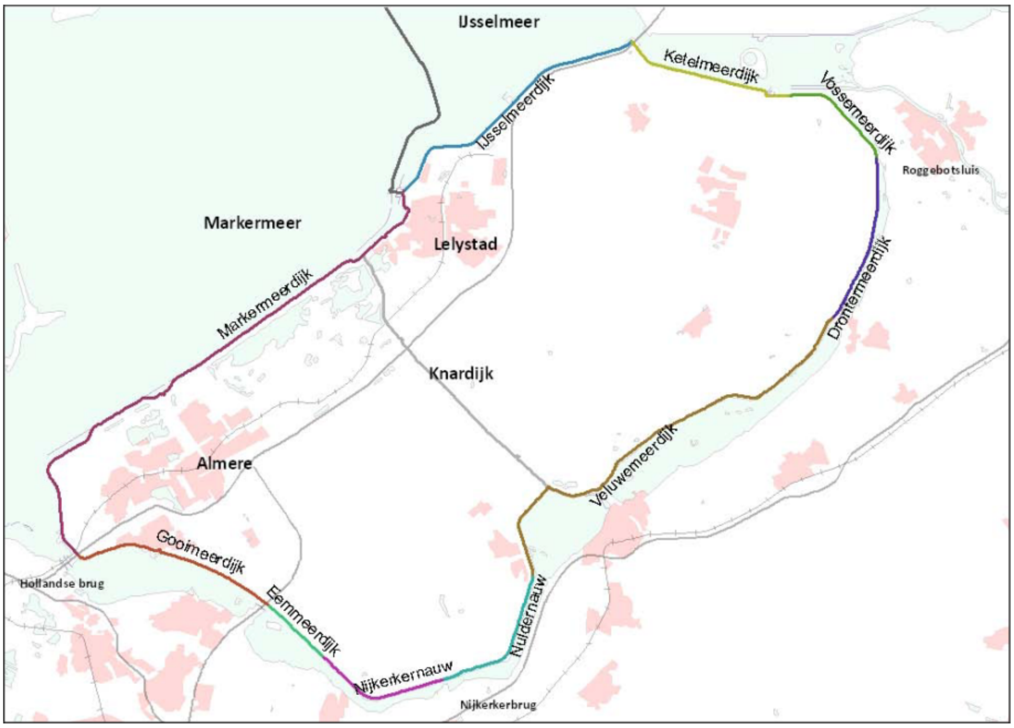

Figure E.1: Dike ring 8 (VNK, 2012)

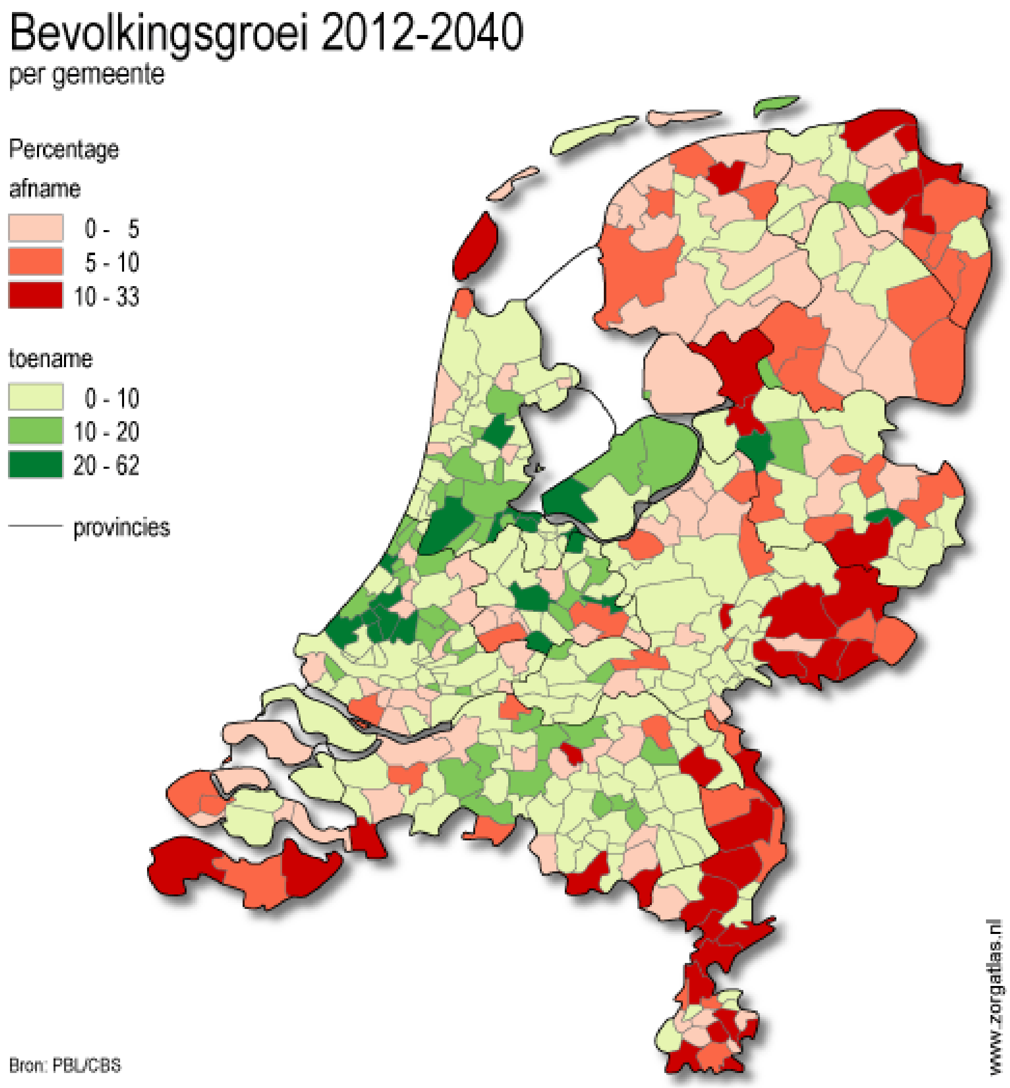

Figure E.2: Population growth (VNK, 2012)

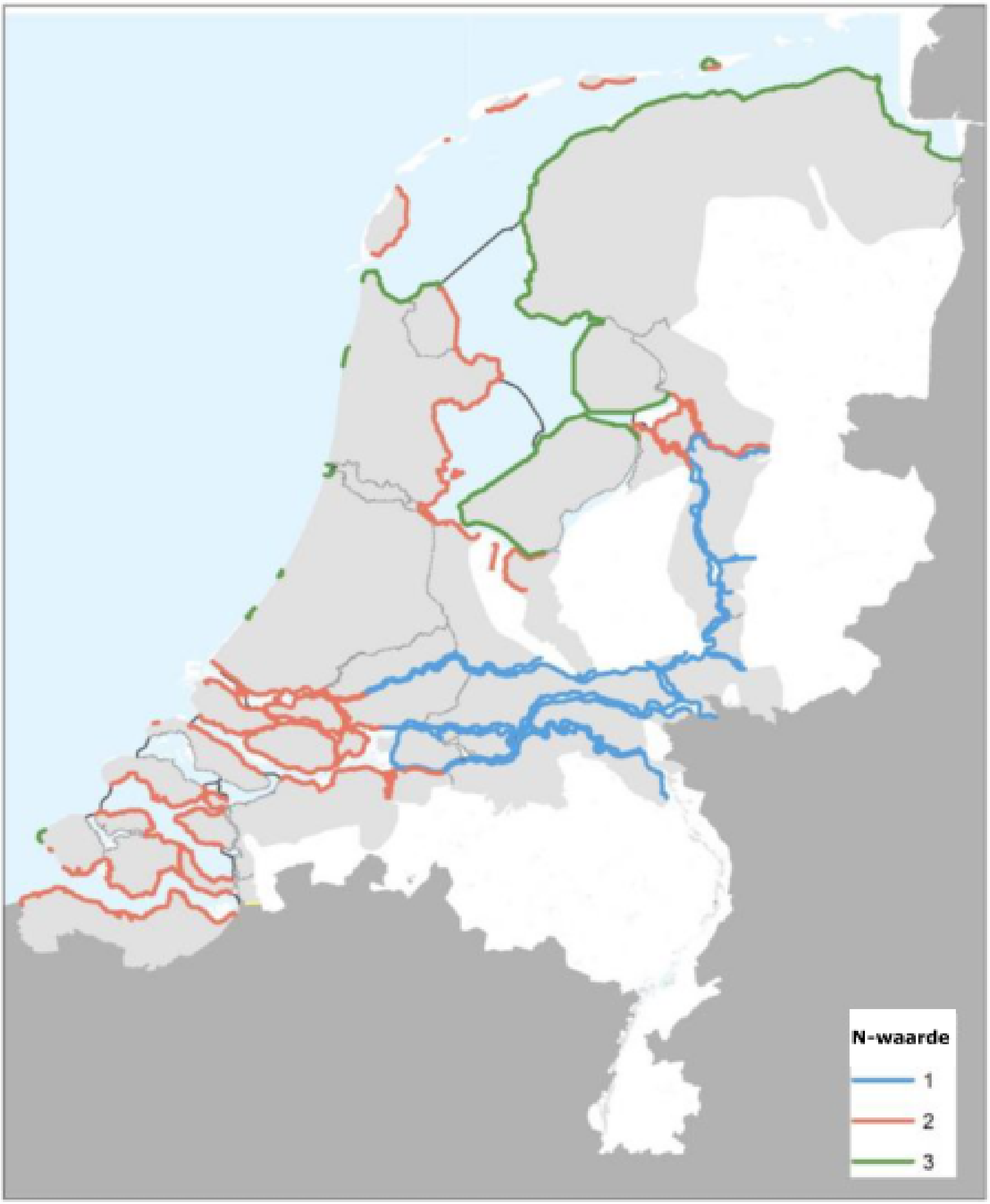

Figure E.3: Length effect values [N] Rijkswaterstaat (2014c).

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