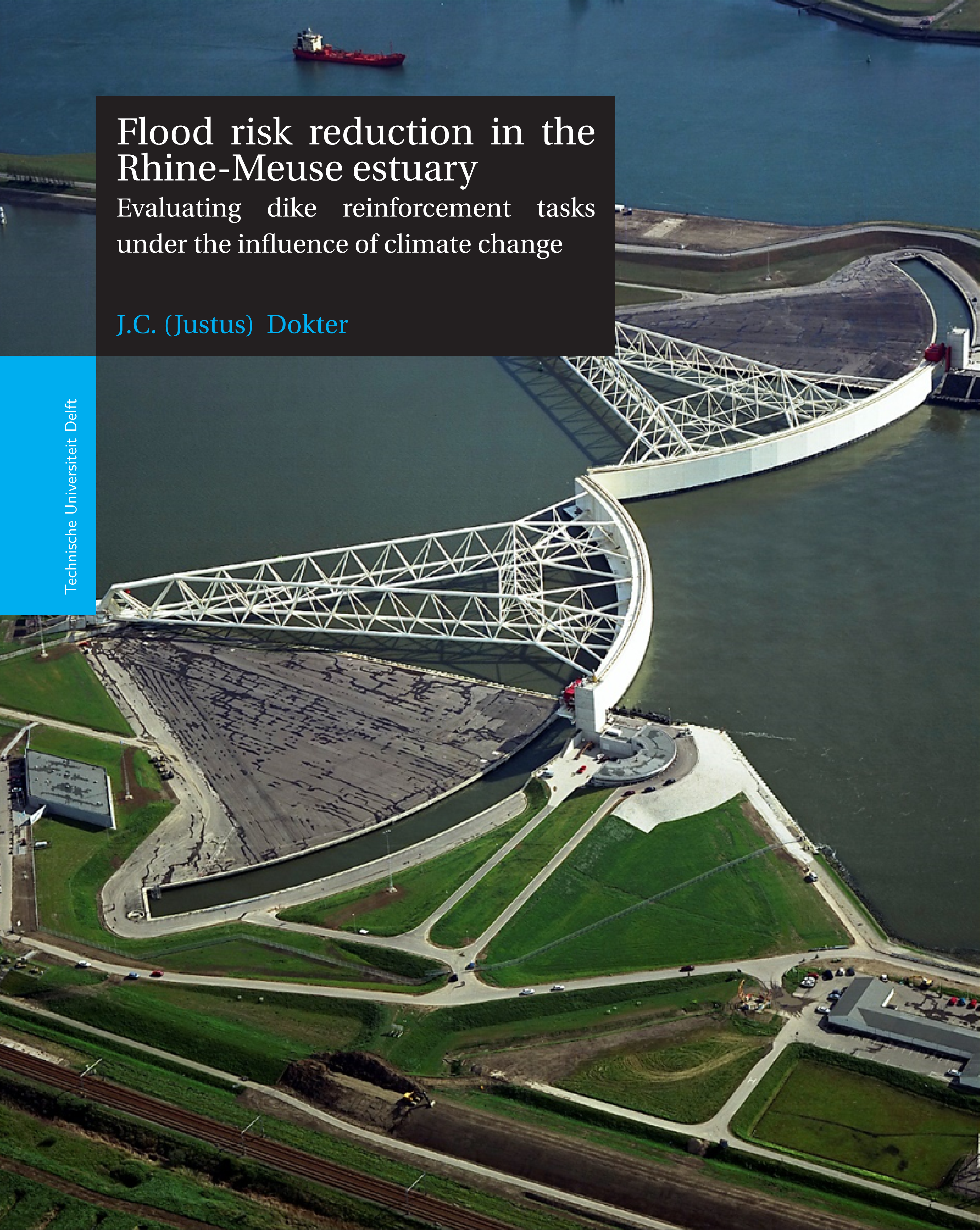
Flood risk reduction in the Rhine-Meuse estuary

Evaluating dike reinforcement tasks under the influence of climate change

J.C. (Justus) Dokter



Technische Universiteit Delft

Delft University of Technology

Challenge the future

Flood risk reduction in the Rhine-Meuse estuary

Evaluating dike reinforcement tasks under the influence of climate change

Master Thesis

by

J.C. (Justus) Dokter

in partial fulfillment of the requirements for the degree of

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This thesis is confidential and cannot be made public until December 1, 2015.

Cover photo: Areal photo of the Maeslant barrier in closed situation
Retrieved from: `https://beeldbank.rws.nl` *Rijkswaterstaat*

PREFACE

This report contains my thesis about flood risk reduction in the Rhine-Meuse estuary, where an alternative strategy to provide the required level of flood safety for the coming decades is compared to the strategy proposed by the Delta Programme. With this thesis, I finish my Master programme Hydraulic Engineering at the Delft University of Technology. The research has taken place in association with $\text{HKV}_{Lijn\ in\ Water}$ and Rijkswaterstaat.

During my research I found out that many aspects play a role in outweighing the alternatives, which go way further than solely balancing expected costs for dike reinforcements with gained flood risk reduction. Aspects related to ecology, politics, economics and climate changes are only a few amongst others that are just as important in making a final decision, while none of these aspects is taken for granted. This is exactly what makes the field of flood risk interesting to me, because without getting bogged down in details, it is possible to deal with uncertainties and variables in a nuanced, analytic and structured manner. Also the social relevance of flood risk contributes to my enthusiasm for this topic.

I would like to thank my graduation committee for their support and feedback during this research. On the first place I would like to thank Matthijs Kok for chairing my committee. Thanks to Matthijs I found this very interesting research topic and his feedback has challenged me truly. Furthermore I would like to thank Jules Verlaan and Ad van der Toorn for their constructive criticism during several meetings, which improved the quality of this report. I would like to express my gratitude to my supervisor Ton Botterhuis for all his dedicated support and feedback. Thanks to Ton I was able to structure the problem and work my way through this research. Lastl, I want to thank Harold van Waveren of Rijkswaterstaat for giving me the ability to join several meetings in which relevant topics within strategy 'Sluices' were discussed on high level between Rijkswaterstaat, HKV and the group of engineers led by Frank Spaargaren. Not many students get an opportunity to be involved in such inspiring and challenging discussions with such experienced engineers. I really appreciate that I got this opportunity.

I have had a great time while conducting my research at HKV Lijn in Water. I really enjoyed doing my thesis research at HKV and learned a lot of the professionals (Bart, Bastiaan, Bob, Fred, Gerbert, Jan, Joost, Karolina and Wouter) who were always willing to help me.

Furthermore, special thanks go out to Vivian and my family in their full support on all possible levels. Finally, completing a study at the Delft University of Technology is not just about the study, which I know for sure thanks to my friends of Laga and Sioux, who have helped me becoming a broad engineer.

J.C. (Justus) Dokter
Delft, November 2015

Summary

Large parts of the Netherlands are exposed to the threats of flooding due to the influence of high water coming from the North Sea region, due to high discharge of rivers like the Meuse and Rhine or due to a combination of North Sea water levels and river discharges. In order to cope these threats, strategies are developed in the Delta Program in which measures with respect to flood risk reduction are elaborated. This study focusses on measures and strategies for the Rhine-Meuse estuary, which consists of the downstream branches of the rivers Rhine and Meuse and is characterized by influences of river discharges and the water level of the North Sea, which is under influence of tide and storm surges.

The current preferred strategy by the Dutch government, further referred to as strategy 'DP2015', accounts for flood safety of the Rhine-Meuse estuary by means of functioning of the Europoort barrier (both the Maeslant barrier and Hartel barrier) in combination with an extensive dike reinforcement program and room for the river measures. In 2017 new safety standards for flood risk become effective and it is expected that many dike trajectories within the Rhine-Meuse estuary will be rejected according these standards and need to be reinforced. Dikes will be assessed according the standards, by determining the actual failure probabilities P_f [-/year] compared to the maximum allowed failure probability.

Opposed to strategy 'DP2015', an alternative strategy in order to meet the required level of flood safety in the coming decades is developed by a group of six engineers under guidance of ir. Frank Spaargaren, a retired engineer who has been in charge of the construction works of the Eastern Scheldt barrier. The engineers state that the current strategy 'DP2015' is too expensive and that a safer and less expensive solution is obtained when the hydraulic loads are reduced significantly. A alternative strategy, further referred to strategy 'Sluices', is developed by the engineers in which the Maeslant barrier is replaced by a closed dam with navigation locks, sluices and pumping stations. These new complexes will be located at the Nieuwe Maas nearby the Benelux-tunnel and Oude Maas nearby 'Het Scheur'.

Both strategies are very different from each other in their main principles. The 'DP2015' strategy aims to maintain and improve the current manner of resisting high waters, whereas 'Sluices' wants to adapt the functioning of the Rhine-Meuse estuary on system scale by the permanent closure of the rivers due to the dams. In outweighing the alternatives, the strategy that gives the most optimal design for safety - where required standards are met with respect to the life cycle costs of the comprehensive strategy - is the most beneficial one. A demarcation is made for this study as it is too comprehensive to study all the matters within the strategies. It is chosen to limit the scope of this research to the expected dike reinforcements tasks up to 2100. Reinforcement tasks are expressed in both meters (Δh for dike heightening and ΔL for dike widening) and in costs (M€) and will be necessary when the occuring failure probability for a trajectory exceeds the allowed failure probability according the safety standards.

Instead of analysing the Rhine-Meuse estuary as a whole, it is investigated which dike trajectories are most distinctive with respect to investments in dike reinforcements in the near future. It was found that these trajectories are 16-1, 16-2, 16-3 and 16-4. The trajectories make part of dike ring 16 'Alblasserwaard & Vijfheerenlanden' and are located in the so-called 'transition zone' where hydraulic loads acting on the dikes are determined by both influence of high water levels from the North Sea (being tide and storm surge amongst others) as determined by discharge of the rivers Rhine and Meuse. The failure probabilities for these trajectories are largely determined by failure due to piping and failure due to overtopping/overflow. The failure probabilities for these mechanisms are calculated for the situation in 2015 and in 2100 where influence of climate change is taken into account.

The largest risk reduction of strategy 'Sluices' compared to strategy 'DP2015' is found in trajectory 16-2, where the failure probabilities due to overtopping/overflow and piping were reduced significantly. Based on the results of risk reduction, the shortage on dike height and width is determined. For trajectory 16-1 and 16-4

the shortage on dike height and berm width is more or less equal for both strategies, as a reduction in hydraulic loads due to pumping stations in combination with sluices and retention measures is effective up to Schoonhoven (on the Lek) and Gorinchem (on the Waal), when 'Sluices' is compared to strategy 'DP2015'. For trajectory 16-3 a risk reduction is found up to about half way of the trajectory.

With the calculated shortage on dike height and berm width, a cost calculation is made in nominal and net present terms for the trajectories. A nominal saving of €158 million is made for dike ring 16 in strategy 'Sluices' (€1.561 billion) compared to 'DP2015' (€1.719 billion). Costs drivers for dike reinforcements are mainly determined by initial costs which cannot be avoided in case a reinforcement has to take place. However, when a reinforcement for a dike trajectory may be postponed to a later moment, a cost reduction is realized in terms of net present values (NPV). Costs for both strategies are calculated in terms of NPV's and it was found that a reduction of €446 million (37%) is realized within strategy 'Sluices' with respect to 'DP2015', in case the principles are followed as stated in subsection 6.3.4. Strategy 'DP2015' costs €1,211 million, whereas for 'Sluices' net present costs of €764 million are calculated. The difference in net present costs is clarified by a combination of a reduction in nominal costs (and thus net present costs) and the finding that reinforcements of trajectory 16-2 can be postponed by 37 years for both dike widening and dike heightening. It is noted that the cost calculation in terms of present values is very sensitive to applied principles and boundary conditions.

Several recommendations are made with respect to further research and used methodology. It is recommended to incorporate the duration of high water events in the applicable models, as the factor time is an important parameter in the occurrence of failure due to piping. Currently, this is not taken into account in calculations for failure due to piping. Furthermore it is advised to make use of survived loads on piping, as this can lead to a reduction or increase of the calculated failure probabilities by a factor 2-20. Third it is recommended to adapt further research on the effect of strategy 'Sluices' for other trajectories and other failure mechanisms than the ones investigated in this report. Last it is recommended to verify the configuration of operation mode of the pumping stations. In the current proposed configurations, pumping stations will be in operation for almost each high tide event, which requires a high amount of energy consumption.

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Glossary

Aquifer	A permeable sub-layer which conducts groundwater well
Aquitard	A low permeable blanket layer laying on the aquifer with a low conductivity
DP2015	The strategy for flood protection according to the Delta Program of 2015
Failure probability	Probability on loss of water retaining capacity for a dike stretch causing the hydraulic load on a backlaying dike stretch to increase substantially
Flood probability	Probability on loss of water retaining capacity for a dike stretch causing the protected area behind the dike stretch to flood in such a way that it results in fatalities or substantial economic damage
Hinterland	The protected area at the inner side of a dike system (also the term polder is used)
Levee	Dike
Outer dike area	higher lying area that is not protected by a dike system
Sluices	The strategy for flood protection proposed by *ir.* Spaargaren and 5 other engineers

Acronyms

CBA	Cost benefit analysis
DPR	Delta Programma Rivieren
DPRD	Delta Programma Rijnmond Drechtsteden
EPK	Europoortkering (Europoort barrier)
H2	Strategy ‘Sluices’
HBN	Hydraulisch belastingiveau (Hydraulic Load Level)
LIR	Lokaal individueel risico
LSF	Limit state function
MHW	Maatgevend hoogwater (Normative High Water)
MK	Maeslantkering (Maeslant barrier)
NAP	Normaal Amsterdams Peil (Amsterdam ordnance datum)
NPC	Net present costs
NPV	Net present value
OI2014	Ontwerp Instrumentarium 2014
Ref	Strategy ‘DP2015’
SLR	Sea level rise
VNK	Veiligheid Nederland in Kaart (Flood Risk in the Netherlands)
WTI	Wettelijk Toetsinstrumentarium

List of Symbols

a	[-]	Part of length of trajectory that is sensitive to the respective failure mechanism
b	$[m]$	Length of independent, equivalent sections for the respective failure mechanism
D	$[m]$	Thickness aquifer
d_{70}	$[m]$	70% Quantile of the grain size distribution
d_{70m}	$[m]$	Mean d_{70} in small laboratory scale tests
Δh	[m]	Additional dike height
ΔL	[m]	Additional berm widht
η	[-]	Coefficient of White or drag factor
ω	[-]	Failure mechanism contribution factor
$F_{geometry}$	[-]	Geometry parameter
$F_{resistance}$	[-]	Resistance parameter
F_{scale}	[-]	Scale parameter
g	$[m/s^2]$	Gravitational constant
γ_s	$[kN/m^3]$	Dry volumetric weight of sand grains
γ_w	$[kN/m^3]$	Wet volumetric weight of sand grains
H_c	$[m]$	Critical difference in water level
k	$[m/s]$	Specific conductivity of aquifer
L	$[m]$	Length
N	[-]	Length effect factor
μ	[-]	Mean
P_f	$[1/year]$	Probability of failure
Q	$[m^3/s]$	River discharge
r	[-]	Discount rate
RP	[year]	Return period
RS	[m]	Robustness surcharche
σ	[-]	Standard deviation
θ	[°]	Internal friction angle sand grains
v	$[m^2/s]$	Viscosity of water

1
Introduction

1.1. General

Large parts of the Netherlands are exposed to the threats of flooding due to the influence of high water coming from the North Sea region, due to high discharge of rivers like the Meuse and Rhine or due to a combination of North Sea water levels and river discharges. In order to cope these threats, the Netherlands have been protecting themselves against floods for centuries. However, in 1953 a large flood occurred and more than 1,800 people were killed. As a reaction to this flood disaster, the Delta Works were constructed in the past decades. The main philosophy of the Delta Works was to shorten the coast line by building large dams and barriers along the south west delta. In 1997 the Delta Works were finished by the completion of the Maeslantbarrier, but this did not stop the Dutch government from keeping improving the flood safety in the Netherlands. Continuous research is ongoing and new insights are obtained about strategies to keep the Netherlands safe.

1.1.1. Delta policies since 2008

In 2008 the Dutch government appointed a new Delta committee (Deltacommissie, 2008, p. 5) which had the task to construct a long term vision for the flood safety of the Dutch hinterland. The developments on climate change played a large role in the resulting report, but also the combination of safety with other aspects like life and work, agriculture and energy formed the background of the report. The Delta committee had several recommendations which were implemented by the Cabinet. One of the recommendations consisted of an annual program where strategies for the flood safety of the Dutch delta and river areas are proposed. This resulted in the Delta Program which comes out annually since 2010.

The annual Delta Programs develop the main strategies that will be undertaken the coming years in order to control the flood threats in an efficient manner. The current strategy works with a set of different scenarios, which are based on future developments in terms of climate change and socio-economic progress. In total, a set of four different scenarios is taken into account, ranging from low socio-economic developments combined with a mild change in climate to high socio-economic developments with a severe changing climate up to 2050 and 2100 (Bruggemann et al., 2013).

The expected hydraulic loads occurring at a flood defence system are based on both statistical and physical analysis, in which historic data is taken into account. In combination with mentioned climate scenarios, design water levels - or 'ontwerpwaterstanden' in Dutch - are calculated (see also subsection 2.3.3). The socio-economic developments predict future value and demographic characteristics of the hinterland, providing information about the consequences of a flood. The required safety of the hinterland, given a design water level is determined according to several safety standards, as will be discussed in section 2.3.

The Delta Program is divided in several sub-programs based on important decisions that are made with respect to safety, fresh water supply and spatial adaptation. Furthermore strategies are defined based on hydraulic characteristics, resulting in strategies for different areas of the Netherlands that are related with the water systems dominating the area (Rivers, IJssellake, Rhine-Meuse estuary, Coast, Waddensea, South-West Delta). The scope of this report is limited to the matters that are dealing with water safety and the scenarios that will be used in this study assume a severe changing climate (Warm/Steam scenario).

CONSEQUENCES DUE TO NEW SAFETY STANDARDS

In 2017, new safety standards will become effective (Infrastructuur en Milieu, 2015c). It is expected that many dike trajectories will be rejected according these new standards and need to be reinforced. Dikes will be assessed according the standards, by determining the actual failure probabilities P_f [-/year] compared to the allowed failure probability. There are four major failure mechanisms for dikes that contribute to the overall failure probability, of which two will be assessed in this thesis, being 'overtopping/overflow' and 'piping' (see section 2.5 for a description of the mechanisms). Once the failure probabilities of dike trajectories exceed the allowed failure probabilities, reinforcements should take place. With respect to the mechanism of 'overtopping/overflow', this means that a dike needs to be heightened ($+\Delta h$). With respect to 'piping', the leakage length under the dike should be increased ($+\Delta L$). The cheapest solution to do so, is by an increase of the berm width of dikes and alternatively a seepage screen could be placed which is more expensive, see figure 1.1 for an indication of the reinforcement measures. Once the reinforcement tasks are known in terms of dike heightening and berm widening, it is possible to calculate costs for these reinforcements.

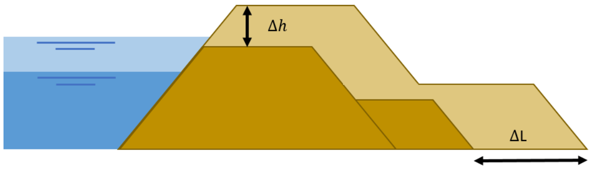


Figure 1.1: Reinforcement measures for 'overtopping/overflow' and 'piping'

1.1.2. THE RHINE-MEUSE ESTUARY

The area of interest is described by the Rhine-Meuse Estuary and consists of the downstream branches of the rivers Rhine and Meuse and is characterized by influences of river discharges and the water level of the North Sea, which is under influence of tide and storm surges. To protect the hinterland from the high waters of the Rhine-Meuse estuary, areas are protected by dike rings and trajectories. The dike ring areas (partly) located in the Rhine-Meuse estuary are: 14 t/m 25, 34, 34a and 35 (Chbab, 2012, p. 5). In Dutch, the Rhine-Meuse estuary is called the 'Rijn-Maas monding' and in literature also the synonym 'lower river area' or 'beneden rivierengebied' is found. The estuary can be divided into three parts, characterized by the dominance of either the water levels from the North Sea or the water levels due the discharge of the both rivers. In between, one can speak of a transitional area, where both influences of river discharges as sea water level are noticeable. There is no such thing as a hard line between the different areas, but it is more an indication about which mode is dominant. In figure 1.2 an overview is given of the Rhine-Meuse estuary, connected with the North Sea at the west side and upper river area at the east side:

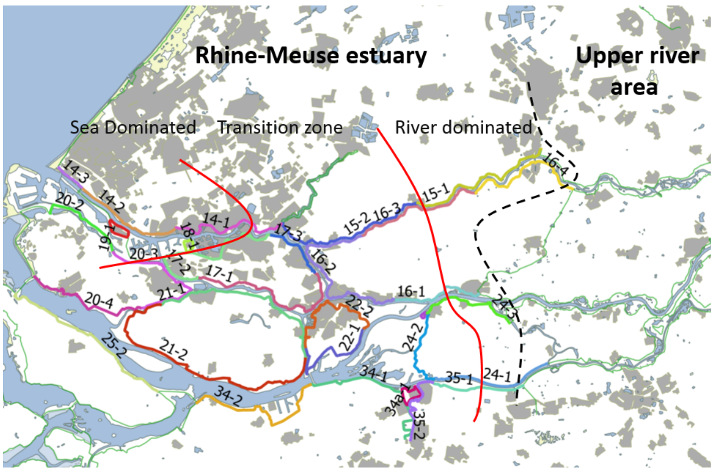


Figure 1.2: Overview of the Rhine-Meuse estuary, with sea dominance, transition zone and river dominance (indicative). Dike ring stretches are given as well.

1.1.3. STRATEGY: 'DP2015'

The preferred strategy by the Dutch government, further referred to as strategy 'DP2015', accounts for flood safety of the Rhine-Meuse estuary by means of the functioning of the Europoort barrier (both the Maeslant barrier and Hartel barrier, see fig 1.3) in combination with an extensive dike reinforcement program and room for the river measures. With respect to the functioning of the Europoort barrier, an additional research will be undertaken in order to improve its failure probability from 1/100 per closure to 1/200 per closure, taking partial functioning into account. On the long term, the Maeslant barrier will be replaced at its technical and economical life time after 2070. It is assumed that the new structure will have a failure probability of at least 1/1,000 per closure (Vos, 2014, p. 9). More about the system functioning in found in section 2.1. The new standard specifications that have to be met for allowable flood probability of dike trajectories are stated in figure 1.4.

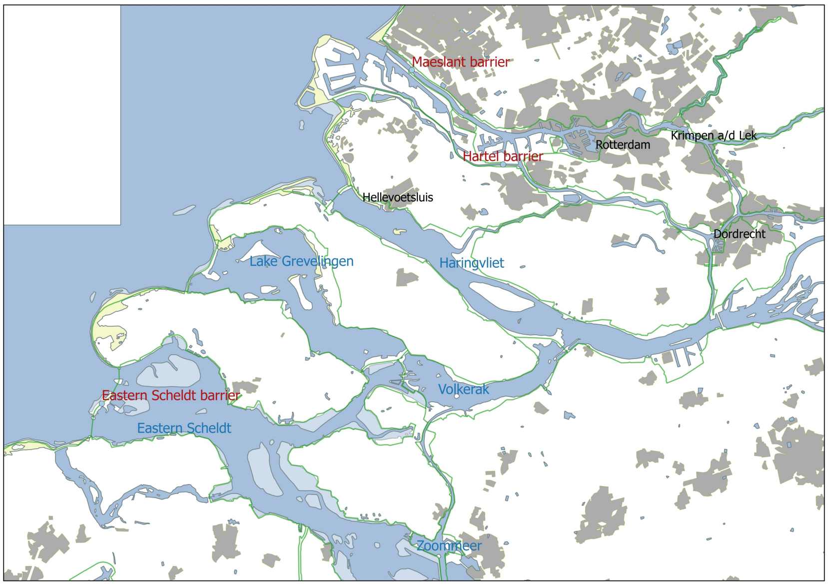


Figure 1.3: Overview important storm surge barriers in brown within the strategy 'DP2015'

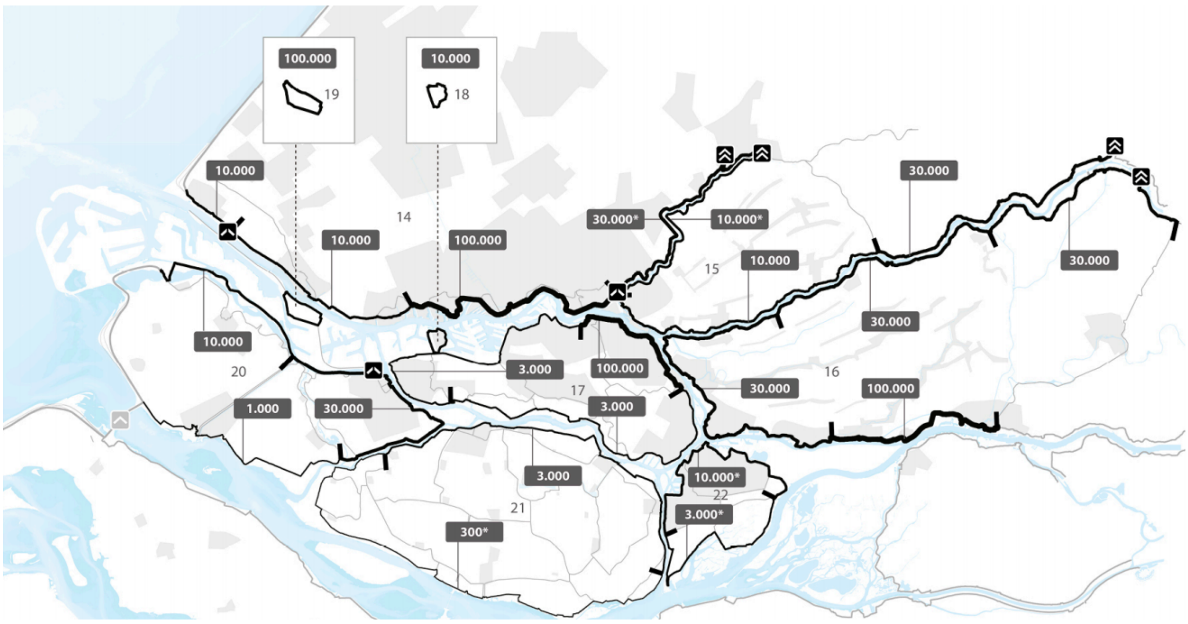


Figure 1.4: New safety standards for flood defences in the Rhine-Meuse estuary (Deltaprogramma, 2014a, p. 5)

1.1.4. Alternative strategy: 'Sluices'

In contrast with mentioned strategy 'DP2015', engineer Frank Spaargaren proposed, together with five other involved engineers, a different strategy to provide the required level of flood safety for the coming decades (further mentioned as 'alternative strategy' or alternative 'Sluices'). Spaargaren is a retired engineer who has been in charge of the construction works of the Eastern Scheldt barrier and still is concerned with the Delta strategies. Spaargaren states that the current preferred strategy is too expensive and that a safer solution could be obtained when the hydraulic loads are reduced significantly. This is done by replacing the Maeslant barrier by a closed dam with navigation locks, sluices and pumping stations. One of the main concerns that the group of engineers led by *ir.* Spaargaren has in the current preferred strategy, is that there is a large uncertainty in expected costs of dike reinforcements for the coming decades combined with rather large uncertainties in the reliability of levees (dikes). A resolution from parliamentarian Geurts is adopted, stating that this variant needs further investigation (Geurts, 2014). In figure 1.5, a schematization is given of the proposed alternative by Spaargaren, where the Europoort barrier is removed and replaced by sluices nearby the Beneluxtunnel and 'Het Scheur'. Furthermore, strategy 'Sluices' makes use of retention in the Eastern Scheldt in order to reduce high water levels. This is done by widening the flow profile of the Volkerakdam and Philipsdam in order to reduce the resistance and let river discharge easily access the Eastern Scheldt. It is known that the Eastern Scheldt barrier has a large leakage area, letting high waters from the North sea still enter the basin. Spaargaren wants to reduce this leakage area from $1,250 m^2$ to $350 m^2$ in order to decrease the flow from the North sea and increase the retention capacity in the Eastern scheldt. The same standards as in figure 1.4 hold, but the design to meet these standards change:

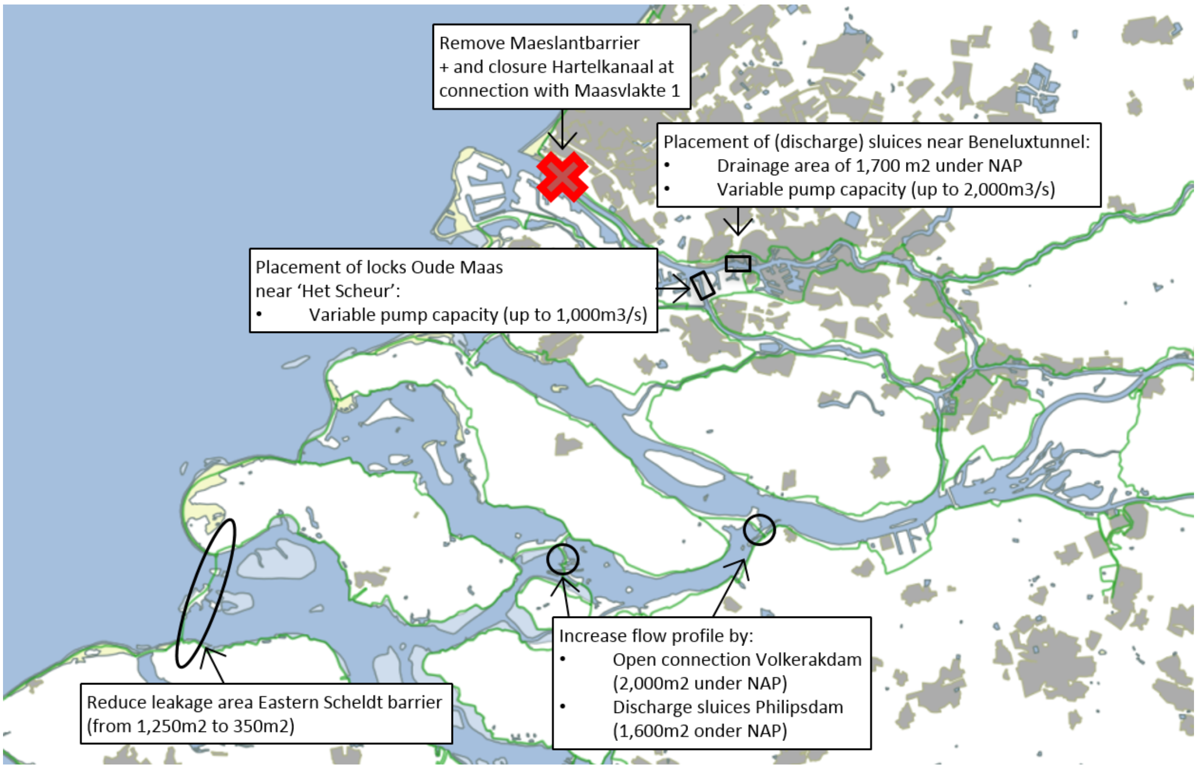


Figure 1.5: Overview alternative proposed by *ir.* Spaargaren on January 16, 2015

Denote alternative strategy with 'Sluices'

In literature, the alternative strategy is denoted with 'Plan Sluizen' in Dutch, referring to the strategy as proposed by *ir.* Spaargaren. In this report, it is chosen to denote this strategy with 'Sluices', which is not a fully bounding translation of 'Plan Sluizen' as sluizen refer to both sluices and navigation locks. In case alternative strategy 'Sluices' is mentioned, all the measures as seen in figure 1.5 are referred to.

1.2. PROBLEM ANALYSIS

This section provides more insight in the problem analysis. First it is stated how the alternative strategies will be outweighed in the evaluation of motion Geurts (Geurts, 2014) and next the most important differences between the strategies are elaborated. This is done in order to get a clear view on the knowledge gaps and to point out the parts that are relevant for this thesis, which is done in subsection 1.2.3.

1.2.1. BACKGROUND

In 2017, new safety standards become effective for the flood safety of the Netherlands (Infrastructuur en Milieu, 2015c). It is expected that in order to meet the new specifications combined with expected climate change, large parts of the dike rings within the Rhine-Meuse estuary need to be reinforced.

In order to meet these requirements, the two strategies 'DP2015' and alternative 'Sluices' are developed. Both strategies are very different from each other in their main principles. The 'DP2015' strategy aims to maintain and improve the current manner of resisting high waters, whereas 'Sluices' wants to adapt the functioning of the Rhine-Meuse estuary on system scale by the permanent closure of the rivers due to dams. According the motion of Geurts (Geurts, 2014), the alternative strategy 'Sluices' needs to be further investigated in a consistent manner with respect to earlier studies for the 'DP2015' strategy. In outweighing the alternatives, the strategy that gives the most optimal design for safety - where the required standards are met with respect to the life cycle costs of the comprehensive strategy - is the most beneficial one.

To determine which strategy is most cost effective, first the main differences between the strategies will be elaborated and it is shown how the costs differ globally from each other. Second, a demarcation is made for this study as it is too comprehensive to study all the matters within the strategies. Therefore it is chosen to limit the scope on the expected dike reinforcements as will be explained later.

1.2.2. DIFFERENCES BETWEEN STRATEGIES 'SLUICES' AND 'DP2015'

There are several beneficial aspects in the alternative strategy 'Sluices' which should be balanced with the disadvantages that arise by following this strategy. The aspects involved in the alternative strategy 'Sluices' that differ with respect to the preferred strategy 'DP2015' are discussed below, based on a cost benefit analysis (CBA) that is undertaken for the evaluation of the strategies (Stone et al., 2014). The mentioned cost benefit analysis of Stone et al. was nevertheless insufficient to make a definitive decision, as principles described in this CBA have been changed and new insights are obtained. A more extensive cost benefit analysis is now under development in which the new insights and changes in principles will be included to obey the motion Geurts. In November 2015, the renewed CBA will be presented to The House of Representatives. Extra aspects that will be treated in the renewed CBA and are not included in the previous CBA, are also stated below, based on the plan of approach (Labrujere & Van Waveren, 2015):

DIFFERENCES IN HYDRAULIC LOADS

The most essential benefit in the alternative 'Sluices', contains the reduction in design water level at the upstream side of the complexes. When a permanent closure is realized with dams, locks, sluices and pumping stations, high waters from the North Sea will not any longer be able to penetrate into the system, as the gates will be closed when the water level at the sea side exceeds the water level at the river side. It is expected that the failure probability of the gates in the locks and sluices will be significantly smaller than the current failure rate of the Maeslantbarrier which is 1/100 per closure (or 1/1,000 in 2070 (Kallen, Botterhuis, & Kok, 2012, p. ii)), as navigation locks will be always closed at minimal one of the gate sides and the discharge gates are only opened in a situation where there is a positive gradient between the head at the riverside and seaside. In figures 1.6 the differences in normative high waters in 2100 are given for both strategies with respect to 2015 (in section 2.3 the definition for normative high waters is explained, where the difference is made between normative high water, or 'maatgevend hoog water, MHW' in Dutch and hydraulic load level or 'hydraulisch belastingniveau, HBN'). Figures 1.6 and 1.7 show clearly the development in water levels in 2100 with respect to the situation in 2015 for both the strategies. In the extension of the reduced water levels by the application of strategy 'Sluices', it is expected that the dike reinforcement tasks for the coming decades will be reduced as a result of this water level reduction.

Figure 1.6: Differences in normative water level (MHW) in 2100 compared to 2015 for 'DP2015' (Botterhuis & Stijnen, 2015b)

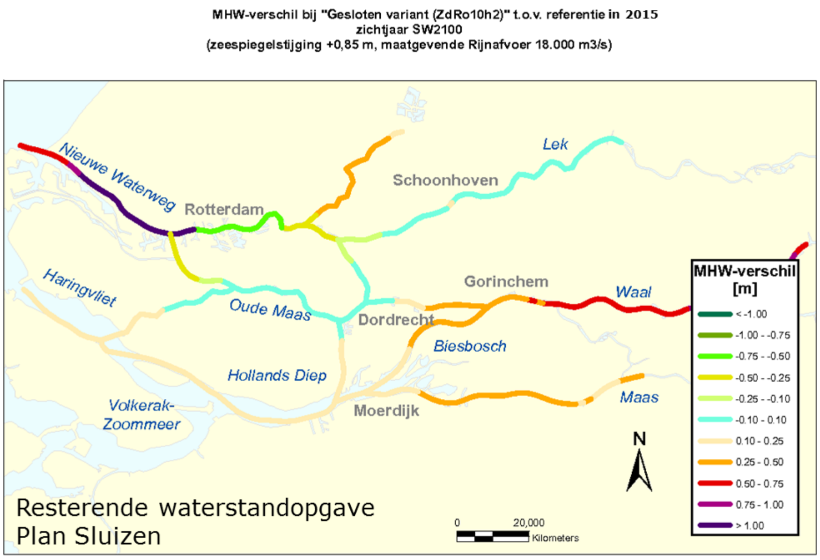


Figure 1.7: Differences in normative water level (MHW) in 2100 compared to 2015 for 'Sluices' (Botterhuis & Stijnen, 2015b)

DIFFERENCES WITH RESPECT TO NAVIGATIONAL ASPECTS

Due to the permanent closure of the Oude and Nieuwe Maas in the alternative 'Sluices', the average transportation time of inland vessels over the rivers and the seegoing vessels to harbour will increase. This extra expected transportation time can be expressed in costs, which should be taken into account in the CBA. In Stone et al. (2014), costs for the extra transportation time are calculated and also initial investment costs for the navigational locks are expressed. Cost drivers for navigational aspects are amongst others: The amount of locks necessary, the dimensions of locks and the expected waiting costs for ships passing. The waiting costs are dependent on the the expected amount of ships passing the locks and the type of ships (barges or seagoing vessels). While the new CBA is under development, also new key figures for expected growth in shipping became available and will be implemented.

Navigational costs will not necessarily be larger in the alternative 'Sluices' than when the 'DP2015' strategy is followed. In case the Maeslantbarrier keeps maintained in the 'DP2015' strategy, financial loss due to waiting should also be incorporated in case the barrier is closed. At that moment, ships are not able to pass the Europoort anymore and need to circumnavigate or wait until the barrier opens again. With expected sea level rise the frequency of the closure will grow as well and the reliability of accessibility of the port of Rotterdam decreases.

Cost calculations with respect to navigational aspects are calculated by Ecorys, a policy research and consultancy company. The described findings will also be submitted in the report to The House of Parliaments in November 2015.

DIFFERENCES IN INVESTMENTS

Both strategies require different investments.
For the 'DP2015' strategy, investments costs are mainly driven by the following aspects:

- Replacement of the Maeslantbarrier (in 2070)
- Dike reinforcements to meet the required norms (up to 2100)

In alternative 'Sluices', one could think of the following investments:

- Hydraulic structures:
  - Navigation locks
  - Pumping stations
  - Discharge sluices
  - Removal of Maeslantbarrier
  - Widening of flow profile of Philipsdam and Volkerakdam (see figure 1.5)
  - Reduction of flow profile in the Eastern Scheldt barrier
- Dike reinforcements to meet the required norms (up to 2100)
- Bottom protection nearby sluices

For above investments, the costs of hydraulic structures will be larger than in the 'DP2015' strategy. However, regarding dikes it is expected that a reduction in necessary investments can be made with respect to the preferred strategy. If the water levels are reduced far enough by implementation of the alternative strategy 'Sluices', reinforcements might be postponed or even cancelled. It is mentioned that not only investment costs should be included, but also operational costs, think of energy consumption and regular maintenance at the structures. Last, at the seaside of the complex the dikes and outer dike areas will have to deal with a increased design water level due the absence of the Maeslantbarrier in the alternative strategy.

ECOLOGICAL DIFFERENCES

Implementation of alternative 'Sluices' requires interventions on system scale. It should be investigated what the influence of these changes is for ecological aspects.

FRESHWATER SUPPLY

The Rhine-Meuse estuary is also facing salination which threatens the supply of freshwater. In dry periods, the discharge of the rivers Rhine and Meuse is too low to maintain a stable equilibrium in the separation between salt and fresh water. As a consequence of this, salt water from the North sea penetrates deep in the estuary and freshwater inlets for drinking water and irrigation are threatened (Arnold, Bos, Doef, Kielen, & Van Luijn, 2011). In case the alternative strategy is applied, this problem might be solved as the North Sea is not able to reach high in the system any longer.

DEVELOPMENTS AND PREVENTED DAMAGE OUTER DIKE AREA

The last part that shall be investigated in the new CBA holds with developments and prevented damage of the outer dike area. Outer dike area is land that lies outside the primary flood defenses, for which no norms apply with respect to flood safety, but is often laying high above sea level. In the Port of Rotterdam many economic and chemical activities take place in the outer dike area. The amount of risk that this outer dike area is imposed to should be investigated for the 'DP2015' strategy and outweighed with the 'Sluices' strategy.

Next to the chemical activities that take place in outer dike areas, also many people live in outer dike areas (approximately 100,000 according to Labrujere and Van Waveren (2015)). In combination with urban developments the alternative strategy could be an opportunity for the value of this area.

1.2.3. KNOWLEDGE GAPS AND FOCUS OF THE RESEARCH

The evaluation of the alternative strategy 'Sluices' compared with 'DP2015' requires a broad research which is too extensive to cover as a whole. The total costs of the execution of one of the two strategies is in the order of billions of euros and requires specific knowledge on each of the mentioned topics.

STUDY FOCUSSED ON DIKE REINFORCEMENTS

From a civil engineering point of view, it is interesting to conduct more research on necessary dike reinforcements. Further research on this topic for both the strategies corresponds not only with personal study background, but the costs for dike reinforcements are also expected to be one of the main drivers within the strategies. In case less dike reinforcements are necessary or reinforcements can be postponed within the alternative strategy, a reduction in costs will be realized.

Besides the fact that the costs for dike reinforcements are a large driver for the strategies, it is also interesting to conduct research in these fields from a scientific point of view. New safety standards are developed and dikes will be assessed for these norms starting in 2017 (Infrastructuur en Milieu, 2015a). On many points it is still unclear how these new standards will have their effect on the dike reinforcements for both the strategies.

Last, in line with the new standards, new insights have led to a better understanding of failure mechanisms of dikes (Vergouwe, 2014). With respect to the failure probabilities of dikes for the different strategies also more knowledge should be obtained, in order to assess the influence of measures on the several failure mechanisms of dikes. It is for instance unknown to what extend a reduction in hydraulic load leads to a reduction in failure probability due to piping, as this failure mechanism also could occur under moderate circumstances. For the failure mechanism of overtopping/overflow this is for instance more clear, as the height of a dike included with its slope is directly related to its ability to withstand a certain hydraulic load.

APPLY RESEARCH ON DISTINCTIVE DIKE RINGS

Initially, research will be applied on distinctive dike rings. The length of the dikes within the Rhine-Meuse estuary is larger than 500km and according to earlier studies like Vergouwe (2014), there is a large difference in flood risk between several dike rings and trajectories. In order to make a distinction in this and to investigate the most relevant parts within the Rhine-Meuse estuary, it is chosen to perform a study on the dike rings where a reduction in reinforcements would lead to the largest benefits with respect to expected dike investment costs.

1.3. PROBLEM STATEMENT

Following from the problem analysis, the following problem statement is derived:

In evaluating and comparing the alternative strategy 'Sluices' with the preferred strategy 'DP2015', the main problem is that it is unknown *how necessary reinforcements are determined and influenced by a reduction in hydraulic loads.*

Three sequential aspects are determined within the problem statement, based on previous problem analysis:

1. It is unknown what the failure probabilities of dike trajectories within the Rhine-Meuse estuary will be for both the strategies (in 2015 and 2100) and whether they will or will not meet the required safety standards.
2. In case the safety standards are not met for certain dike trajectories, it is unknown to which extend dikes should be reinforced in terms of dike heightening ($+\Delta h$) and dike widening ($+\Delta L$).
3. It is unknown how costs (M€) are calculated as a function of necessary dike reinforcements (with respect to Δh and ΔL)

1.4. RESEARCH QUESTION

1.4.1. MAIN RESEARCH QUESTION

Following from the problem statement, the following research question is formulated:

How do reinforcement tasks according to strategy 'Sluices', where a reduction in hydraulic loads is realised, relate to dike reinforcements of the Deltaprogram 'DP2015' for the Rhine-Meuse estuary until 2100?

Reinforcement tasks are expressed in both meters (in Δh and ΔL) and costs (in M€) and will be executed when the occurring failure probability of a trajectory exceeds the allowed failure probability according the safety standards as stated in Infrastructuur en Milieu (2015b). In this research, the design horizon is set on 2100. In other words: When a dike is reinforced, it should be designed to 'just' meet the safety standards in 2100.

1.4.2. SUB QUESTIONS

The following sub-questions are defined for this research, in order to find a clear answer of the main research question:

1. How is the actual and allowable flood risk for dike rings currently calculated?
   (a) How is the safety standard for dike rings determined and built up?
   (b) What are the contributing failure mechanisms for dikes and how is their failure probability calculated?
   (c) What are the main underlying principles for dike reinforcements?
2. Which dike trajectories are most distinctive in the Rhine-Meuse Estuary in terms of costs for reinforcements?
3. How do failure probabilities of dike rings change for both strategies and the most important failure mechanisms, taking into account climate scenarios?
   (a) For the failure mechanisms 'overtopping/overflow'
   (b) For the failure mechanism 'piping'
4. What is the difference in costs for dike reinforcements between the two strategies?

1.5. READING GUIDE

In chapter 2, the literature study is elaborated. The system under discussion is described and it is stated how flood risk is calculated and which safety standards are in effect. The system analysis forms the basis on which calculations are made. In chapter 3, a description of the used methodology will be given. In chapter 4 the results on the influence on the necessary dike height are discussed whereas in chapter 5 results are elaborated for the failure mechanism piping. Chapter 6 states a discussion on the applied method and obtained results. Costs for dike reinforcements in dike ring 16 are calculated and an interpretation of results is given in the translation to other trajectories than the ones assessed in this study. In chapter 7 conclusions and recommendations are given. In figure 1.8 a schematization is given of the sequence of chapters and elaborated sub questions per chapter.

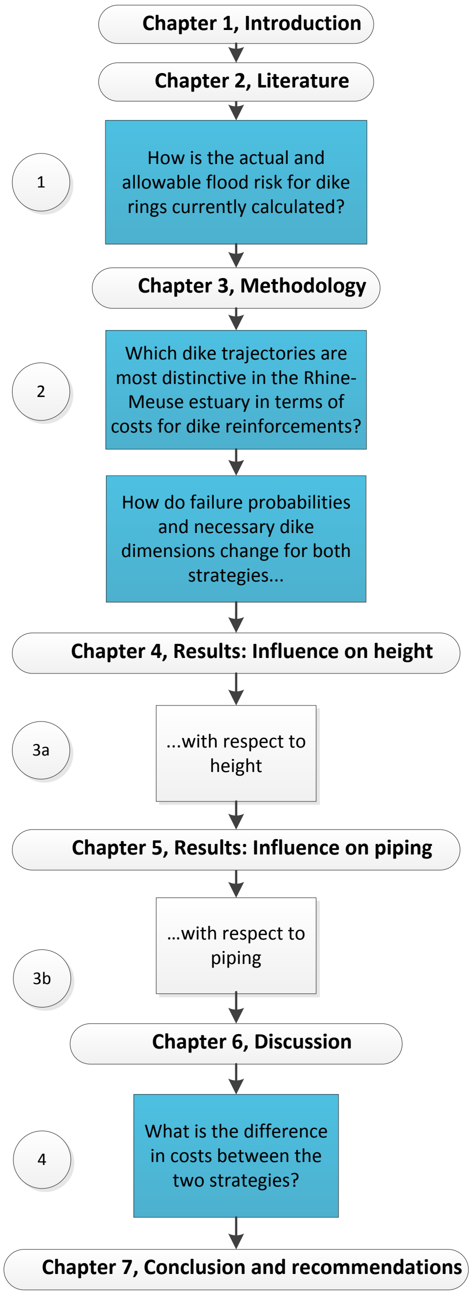


Figure 1.8: Overview of chapters and the concerning research questions

2

LITERATURE STUDY

In this chapter, relevant information known from literature is elaborated. First, the water system under discussion in both the strategies is analysed in section 2.1. It is shown which area is influenced in either one of the strategies and then the scope is refined to the relevant dike stretches. In the second section, section 2.2, a description is given about the estimated costs for dike reinforcements according to current state of view. In order to understand up to what extend dike reinforcements are necessary, a section is devoted to current Dutch flood safety philosophy in section 2.3. The new standard specifications according to this philosophy will be nation wide implemented from 2017 and are therefore described in 2.4. The mathematical calculation of these norms for the failure mechanism of interest in this study, piping and overflow/overtopping, are finally described in 2.5.

2.1. GEOGRAPHICAL AREA

The geographical area that is of relevance for the system under consideration consists of a large part of the south-west of the Netherlands. In the reference strategy of the Delta programme 'DP2015', the area of consideration is more or less equal to the Rhine-Meuse estuary, however in the alternative strategy 'Sluices' a large part of the river discharges will be deflected southwards where it temporary debouches in the Eastern Scheldt. Hence, also the Southwest Delta makes part of the system under consideration. The subdivision of the areas in two parts (Rhine-Meuse estuary and Southwest Delta) is also done by policy makers; The Delta programme is for instance divided in several sub-programmes such as 'Sub-programme Rivers (DPR),' 'Sub-programme Rijnmond-Drechtsteden (DPRD)' and 'Sub-programme South-west Delta (DPZWD)'. Between the different sub-programmes often different boundary conditions and assumptions are applied which makes it relevant to follow this distinction. In figure 2.1 an overview of the system is stated.

2.1.1. SOUTHWEST DELTA

In subsection 1.1.2 it is already shown which area is described by the Rhine-Meuse Estuary, so only the Southwest Delta is described here. The Southwest Delta consists of the Provence Zeeland, the southern part of Zuid-Holland and the western part of Noord-Brabant (Deltaprogramma, 2013, p.5). After the flood disaster of 1953, the system is changed radically by the construction of the Delta Works. Before the construction of these works, the estuary had an entire open connection with the sea, resulting in influence of tide and salt water estuaries. In the current situation, only the Western Scheldt has still an entire open connection with the North Sea, whereas the Eastern Scheldt and Haringvliet are now closed with storm surge barriers and lake Grevelingen is entirely closed from a connection with the North Sea due to the construction of the closed 'Brouwersdam'. Due to the recent developments in the safety philosophy against floods, new safety standards have been proposed. Deltaprogramma (2014c, p. 6) proposes these specifications and describes furthermore which dike reinforcement tasks are planned in the coming decades according to the preferred strategy. In table 2.2 the proposed level of safety is stated, which will also be elaborated more deeply in section 2.4.

PROPOSED RETENTION MEASURES IN THE SOUTHWEST DELTA

One of the decisions coming from the Delta programme is to make use of the 'Volkerak-Zoommeer' as a retention basin. In case of high river discharges in combination with rather high surge levels at the north sea, the discharge of the Rhine and Meuse will be partly deflected to the Volkerak-Zoommeer. More investigation at the frequency of this measure will be adapted (Deltaprogramma, 2014c, p.8), but it is clear that reinforcement measures are necessary for the dike stretches along this lake.

In the alternative strategy 'Sluices' not the Volkerak-Zoommeer will be used as a retention basin, but the Eastern Scheldt. To make this measure more effective, also the functioning of the Eastern Scheldt storm surge barrier will be improved. This will be realized by changing the closure regime of the barriers and decreasing the leakage area from $1,250m^2$ to $350m^2$. In case the Eastern Scheldt barrier needs to close as a consequence of an expected extreme storm surge at the North Sea, the barrier closes during low tide, leaving a high capacity for retention. When the leakage area is still $1,250m^2$ the basin will however still fill after some time. By applying these measures the effectiveness of this measure is significantly increased (Botterhuis & Stijnen, 2015b). In figure 2.1 an overview is given of the entire system:

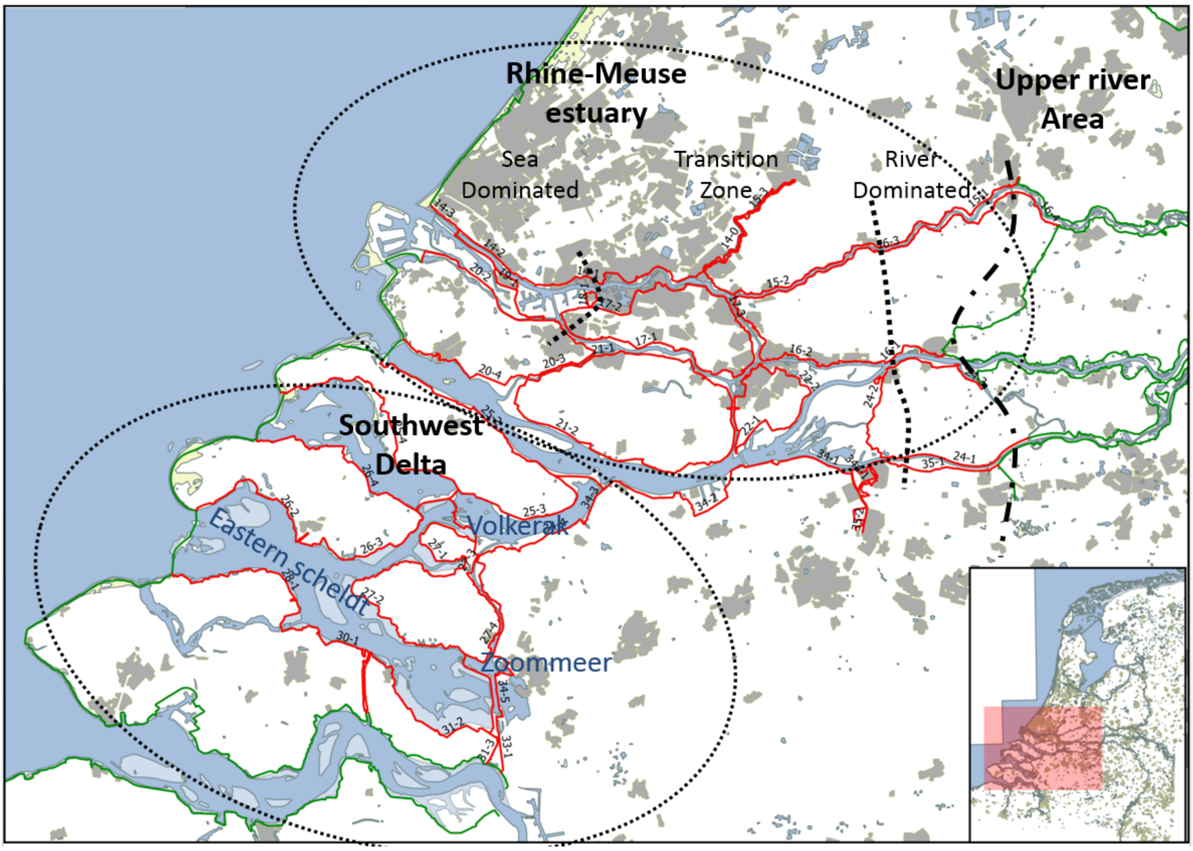


Figure 2.1: Overview of the considered system consisting of the Rhine-Meuse estuary and South-west Delta. The Rhine-Meuse estuary is described by three different areas, with sea dominance, transition zone and river dominance in the current situation (indicative). Dike-ring stretches are given as well

2.2. ESTIMATED COSTS IN REFERENCE STUDIES

During the realization of the Delta Programme (Deltaprogramma, 2014b), several studies have been executed to predict the costs of necessary measures. De Delta Programme has been divided in sub-programmes as earlier discussed in 2.4.2. In these sub-programmes, preliminary studies have been executed to estimate the costs of dike reinforcements for the coming decades for each separate programme. In this case the sub-programmes referred are 'Deltaprogramma Rivieren' and 'Deltaprogramma Rijnmond-Drechtsteden'. An analysis of the expected costs is undertaken in (Asselman & van der Zwan, 2014) and are briefly stated in 2.2.1.

Besides the studies that have been executed to estimate the dike reinforcement costs in the preferred strategy, also studies have been done at the costs for the alternative strategy 'Sluices'. In 2014 Deltares has executed a Societal Cost Benefits Analysis in which the estimated cost until 2100 were estimated for several configurations of this alternative strategy. In this study all the societal costs have been accounted for, including the estimated costs for dike reinforcements. In 2.2.2 more detail will be provided on the several cost studies and they will be compared with previous executed reinforcement projects to check whether this is a realistic first order estimate of expected costs.

2.2.1. COMPARISON OF SUB-PROGRAMMES

Asselman and van der Zwan (2014) have evaluated the used cost methodology for the sub-programme 'Rivers' in 2014. In this report, it is elaborated how the costs were calculated and some comparisons for dike stretches in this programme have been made with the interfering sub-programme Rijnmond-Drechtsteden for the dike stretches 15-1, 15-2, 16-1, 16-2, 16-3 & 16-4. In this evaluation it became clear that there could be a difference of up to 40% (Asselman & van der Zwan, 2014, p.22) for the same stretches between the two sub-programmes. The differences in the result can be explained by their different assumptions, principles and design horizon, but it clearly shows that there is a large sensitivity in outcome for these assumptions and principles. It is tried to find a reasoning behind this and some of the differences were found as below:

- The sub-programme DPRD ('Deltaprogramma Rijnmond-Drechtsteden') calculates with a design horizon of 50 years after reinforcement measures have been executed, while DPR ('Deltaprogramma Rivieren') designed their reinforcements at a design horizon of 2050 - saying that the executed reinforcement measures just would have been strong enough in 2050
- Robustness surcharge: This is a additional surcharge of 0.3m in the design water level which is taken account for in DPRD, while it is excluded in DPR
- DPR assumes that all the dikes along the rivers do not provide enough safety against the failure mechanism of piping, while this might be in fact the case as DPRD took into account

From above arguments it becomes clear that there is not a clear statement for which cost calculation is more correct, for now it only can be stated that 1) not the latest insights in actual failure probability as found in VNK have been taken into account and 2) that there are many uncertainties influencing the total costs. In above report costs were estimated at €1.4 billion up to €2.2 billion for dike rings 15 and 16, making it worthwhile to investigate the costs at a more consistent and transparent manner.

2.2.2. RESULTS FROM PREVIOUS COST BENEFIT ANALYSIS

BACKGROUND OF THE ANALYSIS

The cost benefit analysis (Stone et al., 2014) is performed by Deltares on behalf of Rijkswaterstaat in July 2014 after a statement of *ir.* Spaargaren et al. that the alternative strategy was not elaborated well enough in the Delta programme. The analysis is performed in an earlier stage where the current configuration in the alternative strategy as proposed by *ir.* Spaargaren (see figure 1.5) was not developed yet, but the analysis is performed on a preliminary design. In the analysis, 5 different configurations were evaluated, including the alternative strategy 'Sluices' in an earlier stage and the preferred strategy 'DP2015'. The largest difference between the earlier configuration in the alternative strategy compared with the latest design (dd January 16, 2015), is found in the use of pumping stations; in the earlier design these were absent, while they are currently proposed to be implemented with a capacity of $3,000 m^3/s$ resulting in significant lower design water levels. As the analysis tried to take as much societal costs and benefits into account, the following aspects were discussed in the report:

- Investment costs hydraulic structures (without pumps)
- Costs of dike reinforcements
- Costs of erosion prevention measures
- Economic damage for navigation
- Economic damage of outer dike regions due floods
- Economic damage of the hinterland due floods
- Freshwater supply
- Ecological costs/benefits

USED METHODOLOGY

The dike reinforcement costs for the Delta programme 'Rijnmond-Drechtsteden' are calculated via a software program called 'Blokkendoos'. In this module, it is possible to determine the total investment costs in nominal and net present. The time horizon is set on 2100 to compare the different variants. The assumptions and principles for this calculation can be found in Appendix D of Stone et al. (2014). The used calculation method is called ‘onder/overhoogte (under-/overheight)’, referring to the previous method of assessing the strength of dikes in which one calculated the strength according to exceedance probability instead of flood probability.

RESULTS

From the societal cost benefits analysis, it is found that the total costs of dike reinforcements for the preferred strategy vary between 5.118 billion and 5.575 billion euros depending on the speed of climate change, while the total costs for the alternative strategy ‘Sluices’ are only lower in the Rest-climate scenario (4.718 billion euros) and are found to be even higher (5.712 billion euros) than the preferred strategy in case the Steam-climate scenario becomes truth. *Ir.* Spaargaren assumed that his proposed alternative would significantly cost less regarding dike reinforcements, which is contradicting to the results found, see table 2.1.

Table 2.1: Expected investment and maintenance costs for dike reinforcements in nominal value (Stone et al., 2014, p.27)

	Preferred strategy		Alternative strategy (2032)	
Climate scenario:	Rest	Steam	Rest	Steam
Costs [M €]	5,118	5,575	4,718	5,712

On a first evaluation of the results, the fact that the costs of the alternative strategy regarding dike reinforcements are higher than the preferred strategy, seems not logical. The following remarks were therefore made with respect to these results:

- The Alternative strategy can be implemented at its earliest by 2032 because of the realization time of the dams, locks and sluices. About 15 years will be necessary between the decision moment for the alternative strategy and the actual realization of the sluices, given that the decision for the alternative strategy is made in 2017; until then, it was assumed that the preferred strategy is followed. In reality this will not be the most cost effective strategy.
- The configuration of the sluices that was calculated in the alternative strategy, did not contain pumping stations. The river discharge could therefore only be debouched in the North-sea by meanings of discharge sluices (‘spuisluizen’) resulting in locally even higher design water levels in the Steam climate scenario for dike-ring 16.
- Due to the removal of the Maeslant storm surge barrier, the design water level between the original location of this barrier up to the location of the sluices will increase, resulting in higher dike reinforcement investments.

Concluding above results, it becomes clear that a sub-optimal configuration was analysed in the report of Stone et al. (2014) in which small differences in assumptions and principles leads to large differences in the results. Besides, an outdated methodology was used to assess the standardization of dike-rings.

2.2.3. COMPARISON WITH CURRENTLY APPLIED REINFORCEMENT PROJECTS

The results of Stone et al. (2014) give investment costs in dike reinforcements in the order of €8 to 10 million per kilometer for the various strategies (total costs divided by the length of 576 km dike stretch). This number seems to be rather high; A rule of thumb for dike reinforcements shows that dike reinforcements generally cost € 2.5 to € 5.0 million per km (Maaskant, 2015), in case a dike section has to be reinforced. The results of Stone et al. (2014) would then imply that each dike section in each dike strection within the area of interest should be reinforced twice until 2100.

2.2.4. COSTS PER TRAJECTORY

A more recent study by Botterhuis (2015) calculated the expected costs for per trajectory for the ‘DP2015’ scenario for the Rhine-Meuse estuary. In figure 2.2 these costs are stated per trajectory and it is clearly shown that the costs for dike reinforcements are high for trajectories 14-1, 15-1, 15-2, 16-1, 16-2, 16-3 and 22-2. Information of the study of Botterhuis (2015) will be used to determine the trajectories that will be calculated. This will be done in section 3.2.

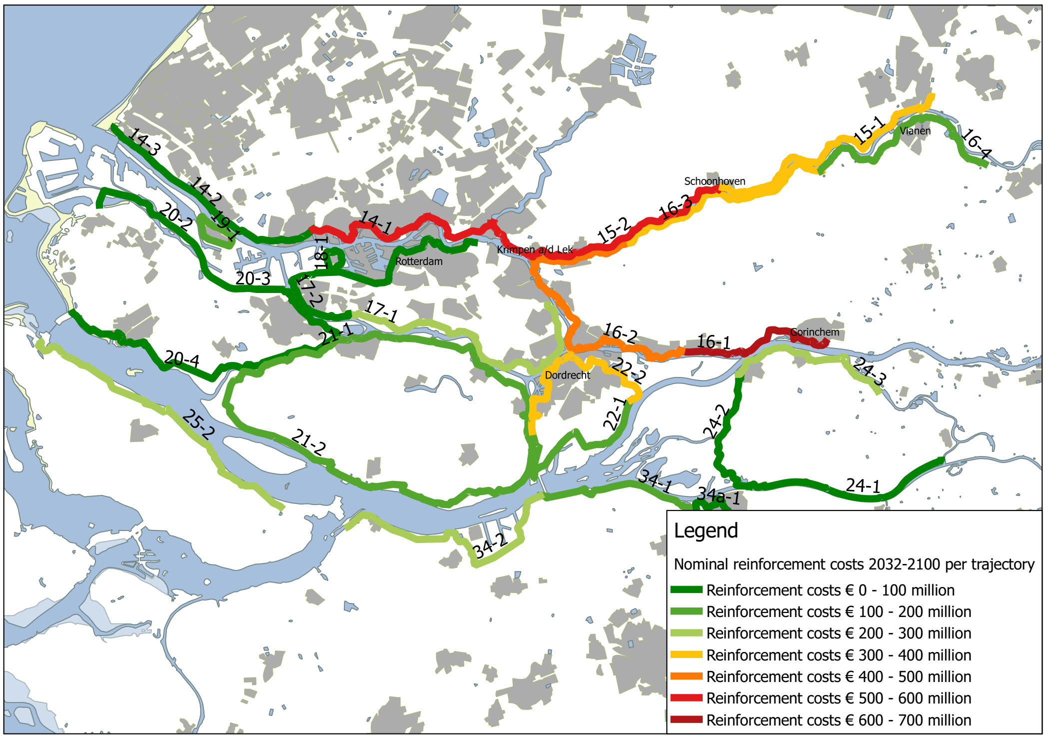


Figure 2.2: Expected costs 'DP2015' strategy according to Botterhuis (2015).

2.3. FLOOD SAFETY PHILOSOPHY

Last years, new research in the flood risk of the Netherlands has improved the safety standards in the Netherlands providing a more equal distribution of risk and better strategies on where to adapt reinforcement measures. The Dutch government is the main responsible in defending the Netherlands against threats from high water levels. This part briefly discusses several developments in the flood safety philosophy in the Netherlands and sketches an overview of the main risk principles.

2.3.1. THE CONCEPT OF FLOOD RISK

As it is impossible to guaranty a 100% safe delta region, there is always some risk of flooding in the Netherlands. The maximum allowable risk is determined by the government and is a boundary condition for the further standardization of flood defences in the Netherlands. Risk can be described as a function of the probability of an event and the consequences of this event (CUR190, 1997, p. 3-2). In a basic formula form it follows:

$$Risk = probability \cdot consequence \tag{2.1}$$

In above formula the probability states the probability of a flood and is expressed in frequency per year $\left[T^{-1}\right]$. The consequences of flood risk are often expressed in economic losses [€] or in number of fatalities [−]. By applying these units it is possible to make a calculation about the expected annual damage, divided in several types of risk. There are three types of risk that are related with flood safety in the Netherlands, namely individual risk, economical risk and societal or group risk (Vergouwe, 2014). For each risk type a maximum allowable fatalities per year is determined or an economic optimum is found between reinforcements of flood defences (investments in prevention measures) and expected annual damage. Below a short description is given for the three risk types.

Individual risk

The local individual risk in the Netherlands is set on 10^{-5} per year from 2050. This is the maximum probability per year that an average unprotected person, permanently present at any location in the Netherlands is killed due a flood. In new standards, the Local Individual Risk is the basic standard in the entire Netherlands, additional safety may be added in case of economic and societal risk.

Economic risk
Economic risk expresses the annual expected losses due to a flood. Once the failure probabilities are known and an estimation of the losses given a flood can be made, a cost benefit analysis (CBA) can be realized to calculate the investments for protection measures. A risk optimum is found at the point where the sum of expected annual losses and expected investment costs to reduce the risks are at a minimum. According to Deltaprogramma (2014b, p. 16) extra safety factors are set for regions with high economic value or with vital and vulnerable infrastructure.

Societal risk
The third type of risk contains societal risk and is also known as group risk. This is associated with the risk that occurs to large groups and is also based on the perspective of people on risks; an extreme event with many fatalities has a higher impact than many smaller events with few fatalities (Vergouwe, 2014, p. 26). In regions where societal risk would have a large impact, the Dutch government intends to set a higher safety standard. The concept related to the avoidance of large societal risks is called risk aversion.

2.3.2. CALCULATING RISK

From equation (2.1) it can be seen that risk is calculated by the product of failure probability for a flood defence system times the consequences given a flood. This study focusses on the change in failure probabilities P_f and the difference in consequences is neglected in this study or already taken implicit into account in the standard specifications as will be discussed in section 2.4.

FAILURE PROBABILITY

The failure state often described by a the z-function or limit state function (LSF), stating (CUR190, 1997, P. 5-2):

$$Z = R - S \tag{2.2}$$

It is said that a flood defence system (partly) fails when the load or solicitation (S) is larger than the resistance of the system (R), failure occurs if $S > R$. In above equation Z is not a fixed value, but a function that depends on the characteristics of the load on and resistance of the considered structure. Loads occurring at a dike are for instance permanent loads and hydraulic loads. The permanent loads are determined by self-weight of the dike and phreatic pressure in the dike. The hydraulic load is influenced among others by wind speed, wind direction, swell, set up, tide, river discharge and the duration and development of the loads acting on the dike. Furthermore one can think of loads due to drifting ice, ship collision, floating debris, traffic, vandalism et cetera. The resistance is amongst others characterized by the dike dimensions, geotechnical parameters and conditions, presence of fore land, the presence of a berm and conditions of the top-layer (Kremer, Van der Meer, Niemeijer, Koehorst, & Calle, 2001).

Within most of the loads and strength parameters, there are (large) variations inherent to these parameters, think of a variable wind speed or variation in sea water level. Next to inherent uncertainties there are modeling uncertainties, that may follow from simplifications in a model or because used formulas are based on empirical relations (Kanning, 2012). Both these uncertainties can be taken into account in one calculation. If both the load as the resistance are described with normal distributions, failure occurs for the events where the load exceeds the resistance and $z < 0$. The failure probability (P_f) is given by:

$$P_f = P(Z < 0) = P(S \geq R) \tag{2.3}$$

Both the load as the resistance are often computed with a normal distribution, each has a mean (μ) and a standard deviation (σ). The normal distribution is described by (CUR190, 1997, p. B-7):

$$f(x,\mu,\sigma) = \frac{1}{\sigma\sqrt{2\pi}} \exp^{-\frac{(x-\mu)^2}{2\sigma^2}} \tag{2.4}$$

For an arbitrary example, the load and resistance distributions could be represented as in figure 2.3:

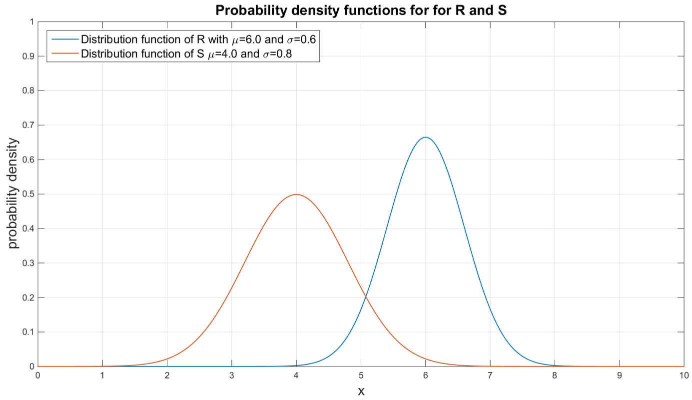


Figure 2.3: Probability density function for a load S and a resistance R

To determine the failure probability of the combination between the load and strength, a joint probability density function is set up, where contours of equal probability density are sketched. In below figure 2.4, the area (or in fact volume as it is a contour plot) that lies under the orange line determines the failure probability where $z < 0$. In case the load and strength functions are determined by a set of variables with each a mean μ and a standard deviation σ it becomes very difficult to solve the expression for $P_f(Z < 0)$ analytically and one uses other methods to solve the equation. Examples of this are the First Order Reliability Method (FORM) or Monte Carlo (MC) simulations. The FORM analysis makes use of linearisation around the limit state function at the design point; the point at which failure is most likely to occur and lies on the line $z = 0$ at the point where the joint probability density is the largest (Vrouwenvelder & Steenbergen, 2003). In a Monte Carlo simulation a large number of samples is drawn according the statistics of the variables. The failure probability is then equal to the number of outcomes for where $z < 0$ divided by the number of possible outcomes (CUR190, 1997).

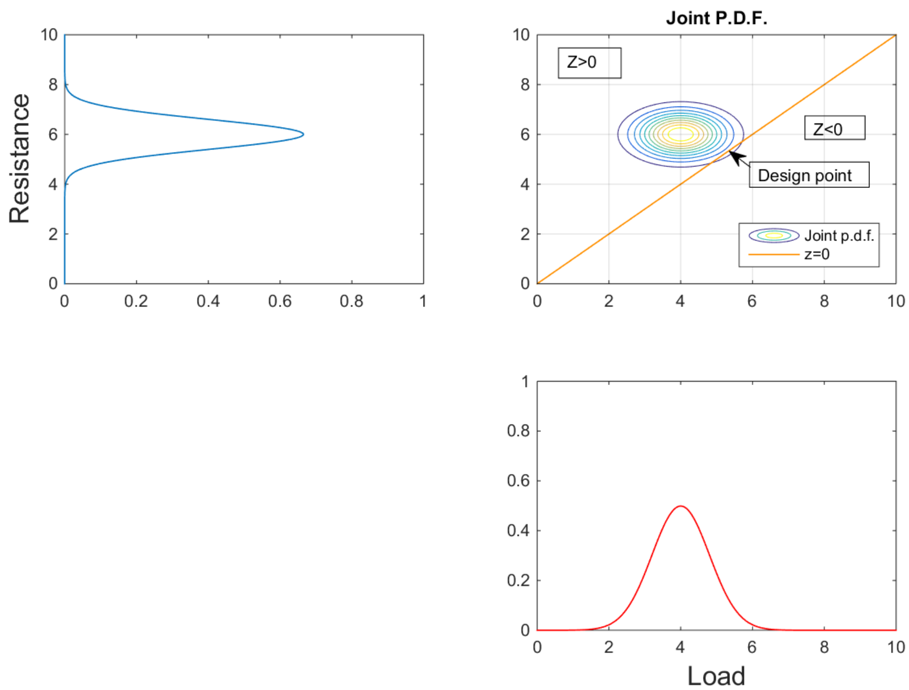


Figure 2.4: Joint p.d.f. with contours of equal probability density and probability of failure for the area under the line $z = 0$

From above figure it becomes clear, that only in the cases where the solicitation is larger than the resistance, the construction will fail. This principle could be applied for all different mechanisms, however the distribution type may differ. In each case, there are uncertainties or variations which influence the mean and standard deviation of the loads and resistance. By looking at above figure, it becomes clear that there are several possibilities to reduce the failure probability. One can reduce the loads on a dike by reducing the water level or increase the strength of the dike by raising the dike. Furthermore, much benefit to reduce the failure probability can be obtained when the variations or uncertainties that are described by the standard deviation can be reduced, see also subsection 2.6.3.

2.3.3. HYDRAULIC LOADS

Design water level

The main loads occurring at a dike are permanent and hydraulic loads. The permanent loads are determined by self-weight of the dike and phreatic pressure inside the dike. Hydraulic loads for the Dutch delta are among others determined by upstream boundary conditions from river discharges of the Meuse and Rhine and lower boundary conditions from sea-water level, wind speed, wind direction and closure situation of several storm surge barriers (Van Velzen, Beyer, Berger, Geerse, & Schelfhout, 2007, p. 110). The total load is then determined by a combination of several factors, and a design water level follows from a particular set of combinations. The design water level determines the locally required height and strength of the flood defence and results from the normative high water level or hydraulic load level. Both water levels are elaborated below:

Normative high water level or 'Maatgevend hoogwater' (MHW)

The normative high water level is the water level that is measured at the middle of a waterway and is expected on average once in a number of years with a certain return period, regarding the standard that is set by the government (the return period is often in the range of 250 to 10,000 years). The MHW follows from either a high water level of the North Sea, high water levels due discharge of the Rhine and Meuse or by a combination of sea and river water levels, regarding the geographical location and boundary conditions (see figure 2.1). The MHW determines the design water level, in case lasting high water levels from sea or river discharge are leading with respect to the hydraulic load level. The MHW is normative for the failure mechanisms piping and macro stability, because these failure mechanisms require a certain development time before the deterioration process starts.

Hydraulic load level or 'hydraulisch belastingniveau' (HBN)

The hydraulic load level is defined as the water level that determines the design level for a certain location, in case local conditions (wind and wave set up) at the toe of a dike are leading with respect to the normative high water level. Local conditions like the lay-out and positioning of a dike may lead to very different HBNs compared with the MHW for a certain location. A strong easterly wind for instance will hardly influence the MHW for a sea dominated region, as there is no set up from the North Sea at the Dutch coast (that is facing western directions). However, this easterly wind might lead to a large HBN as it causes local wind set-up and wave run-up at the toe of a dike. Local wave conditions may therefore induce wave overtopping, even in absence of the normative high water level. For the failure mechanisms overtopping and overflow, the HBN is normative as these failure mechanisms are strongly influenced by the local conditions of the dike. See also figure 2.12 later on in this chapter, where the HBN is dominant with respect to the MHW due to local conditions.

2.3.4. BUDGETTING FORMULA FOR FAILURE MECHANISMS

The strength of a dike was in previous regulations almost entirely determined by its height. The main assessments that were undertaken to test whether a dike provided enough safety was by checking its actual height and comparing this with the normative high water level. When a dike was larger than this MHW and some margin, it was said that a dike had 'over height'. One computed the safety of a flood defense system by calculating the exceedance probability of a polder. However, ongoing research showed that a dike can fail far before a dike is over-topped by high water levels. Other failure mechanisms like piping play a significant role in the safety of a flood defense system and more care is taken for this and other failure mechanisms in the new standards, one speaks now of the flood probability by assessing flood defense systems rather than exceedance probability (Vergouwe, 2014). As a result of these new insights, budgetting for reinforcement tasks is also changed. By budgetting one gives certain failure mechanisms an allowance for their contribution to the total failure probability. As example: If a dike had a safety standard of maximum failure probability of $P_f = 10^{-4}$ in the past of failure probability, 90% of this failure probability was apointed to 'overtopping/overflow' and 10% to other failure mechanisms like piping and macrostability (see section 2.5 for a description of the several failure mechanisms). Hence, the maximum allowable failure probability due to height (both 'overtoppping and overflow') was $9 \cdot 10^{-5}$ and for the other failure mechanisms $1 \cdot 10^{-5}$. In new proposed norms, this budgetting is spread over more failure mechanisms, more evenly spread and depending on the location of the dike.

2.4. CURRENT AND FUTURE STANDARD SPECIFICATIONS

Distinctions in dikes and flood defences are applicable according to their retaining function, dimensions and safety standard. In this section the several distinctions are described starting with the classification according to retaining function and dimensions in 2.4.1. In 2.4.2 the distinction according to safety standards will be discussed.

2.4.1. CLASSIFICATION OF FLOOD DEFENCES

The Netherlands counts dozens of dike rings. According to (Verkeer en Waterstaat, 2008), a dike ring is defined as 'an area enclosed by a system of flood protections or high ground to protect the area from flooding'. Dike rings make part of the primary flood defence, which is defined as 'Flood defence around dike ring areas that border water on the other side' (sea, river, large lakes). The primary flood defence system can be categorized in four different categories (Vergouwe, 2014), see table 2.2 where it is shown how the dike stretches in the discussed area are categorized:

Table 2.2: Classification of flood defences

Category	Definition
A	A flood defence which retains open water directly
B	A flood defence which is connecting two other defence systems, often there is water laying at both sides of the defence (open water and inland water)
C	A flood defence that is retaining open water indirectly
D	A flood defence protecting Dutch areas but is situated abroad

In below table, the dimensions of the dike stretches with their categories that fall within the scope of the study:

Table 2.3: Division of flood defences according to their category within the scope of the study

Considered area	Length [km]	# Trajectories	Remarks
Category A	576.5	31	
Category B	20.5	3	These are the trajectories between the Nieuwe Waterweg and Hartelkanaal
Category C	39.7	2	These are the trajectories along the Hollandse IJssel
Total	636.7	36	

Next to the categorization of flood defences by their retaining function, flood defences are also specified by their dimensions, see table 2.4 where a distinction is made according to dimensions of a (part of a) dike.

Table 2.4: Subdivision of dike rings in trajectories and sections

Name	Definition
Dike ring	An area enclosed by a system of flood protections or high ground to protect the area from flooding
Dike trajectory	Subdivision of a primary water defence which has an equal standard specification along its water line
Dike section	Subdivision of a dike trajectory, considered to have the same strength and load characteristics along the section. The maintenance of a dike section is taken care of by one and the same operator

2.4.2. SAFETY STANDARDS

In previous flood safety policy, all dike sections of an entire dike ring had to meet the same safety standard according to exceedance probability. However, new insights led to a different flood safety philosophy and divided each of the dike rings in one or more dike stretches. According to Deltaprogramma (2014b) each dike stretch will now get a different standard specification as also is shown in table 2.6. Instead of a retaining height according to an *exceedance* probability of a design water level, each dike stretch will have to meet requirements according to *food* probability, where besides load characteristics also strength characteristics are taken into account on an extensive manner.

CALCULATION OF FLOOD PROBABILITY

In 'Veiligheid Nederland in Kaart' (VNK), the failure probability of dike sections in the current situation is calculated (Vergouwe, 2014), here the failure probability is defined as the probability that due to an undesired event a flood of the hinterland occurs. One of the assumptions made in these calculations is that river widening measures from 'Room for the River' have been executed (Maaskant, 2015). From the failure probabilities of dike sections it is possible to calculate the failure probability of an entire dike trajectory. This is of relevance since the dike trajectories as a whole must conform the standards. However, one must take the several dependencies of failure mechanisms into account by calculating the total failure probability of a dike trajectory; one cannot just add the several failure probabilities from the dike sections and failure mechanisms, this would lead to a over conservative calculation. In below table an indication is given in the approach as elaborated by VNK: For each dike section the four most dominant failure mechanisms are assessed, namely overflow and overtopping, macrostability at the inner slope, uplift and piping and erosion of the outer slope. For the last three failure mechanisms, it is assumed that their failure behaviour is independent between other dike sections. The variety in subsoil in the Netherlands is rather large, requiring the sections to assess independently as a conservative choice. The height of dike sections and the height of design water levels is calculated dependent; the water level occurring at a dike section at the beginning of a trajectory will be roughly the same as the level occurring at a dike section at the end of the same trajectory. To compute the overall failure probability due to the failure mechanism overflow and overtopping, the maximum failure probability that is found of all dike sections is normative. For the other failure mechanisms one has to add up all single contributions to the overall failure probability. The total failure probability of a dike trajectory is finally equal to the sum of the contribution of the four failure mechanisms, see also table 2.5.

Table 2.5: Schematization of failure probability for a dike stretch by assessing all individual dike sections within the stretch

Dike section (#)	Overflow & overtopping	Macrostability inner slope	Uplift & piping	Erosion outer slope	Combined
1	P_{11}	P_{12}	P_{13}	P_{14}	$\sum_{j=1}^{4} P_{1j}$
2	P_{21}	P_{22}	P_{23}	P_{24}	$\sum_{j=1}^{4} P_{2j}$
$\vdots$	$\vdots$	$\vdots$	$\vdots$	$\vdots$	$\vdots$
n	P_{n1}	P_{n2}	P_{n3}	P_{n4}	$\sum_{j=1}^{4} P_{nj}$
$P_{mechanism}$	$\max\{P_{i1}\}$	$\sum_{i=1}^{n} P_{i2}$	$\sum_{i=1}^{n} P_{i3}$	$\sum_{i=1}^{n} P_{i4}$	
$P_{trajectory}$:					$\max\{P_{i1}\} + \sum_{i=1}^{n} (P_{i2} + P_{i3} + P_{i4})$

NEW SAFETY STANDARDS

The insights from VNK did not only lead into better insight in the current state of dikes and their related failure probabilities, it also led to the development of new proposed standard specifications of maximum allowable flood probability. From the determined local individual risk of 10^{-5} per year as a probability that a single person dies as the consequence of a flood disaster at any location in the Netherlands (with a possibility to flee) in combination with extra societal risk or economic risk, new standard specifications for flood defences are deducted. In a schematic way the formula to determine the safety standard for a dike stretch is given as follow:

$$Pf_{norm} = \frac{LIR}{P_{mortality} * (1 - P_{evacuation})} \tag{2.5}$$

where:

Pf_{norm}	= Allowed failure probability of a dike stretch according to new insights which will be rounded to the closest discretized standard specification
LIR	= Local Individual Risk, set on 10^{-5} by the Dutch government as maximum allowable risk due to flood in 2050
$P_{mortality}$	= Probability that an individual person (averaged for all ages and its background) who is in the flood zone area dies when a flood happens
$1 - P_{evacuation}$	= The fraction of people who could not be evacuated or did not flee in time in case of a flood

Following on above equation a more in depth analysis is applied with also taken the societal and economical risks into account. This led to table 2.6 in which the new standard specifications according to Deltaprogramma (2014b, pp 155-163) and Infrastructuur en Milieu (2015c) are stated. For now, only the safety standards for the 'A-category' is stated as these are already proposed in the new law. The last two columns in table 2.6 both show standard specifications. The last column provides insight in the lower limit or boundary value; the actual flood probability of the dike stretch may never exceed this value. However, because of periodic assessments of dikes and the fact that it takes time to design, finance and construct a dike reinforcement a signal value is also accounted for which is shown in the second last table, this is determined as 'standard specification' (trajectnorm), 'signal value' (signaalwaarde) or 'middle probability' (middenkans), see also figure 2.5:

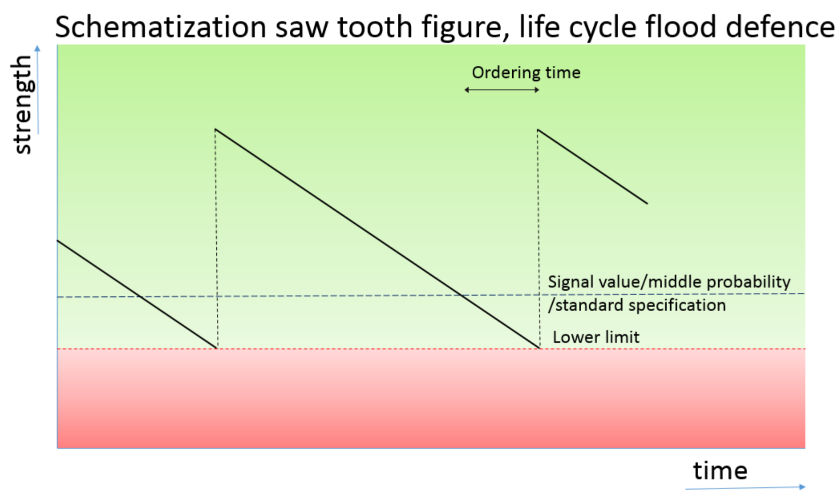


Figure 2.5: Difference between middle probability and lower limit due to deterioration and construction time of reinforcements

Next to the standard specifications, the dike-ring name is given and the sub-programme to which a certain dike stretch belongs to. The latter is based on policy considerations, but it gives a clear view that there is a certain overlap for instance for the dike stretch 16 where both sub-programme Rivers and sub-programme Rhine Estuary Drechtsteden are accounted for. Besides, each sub-programme has made its own cost calculation in the reference situation with their own assumptions and principles, this will be discussed in next chapter.

Table 2.6: Dike trajectories that are within the scope of the study with their corresponding sub-programme and standard specifications, DPR = Sub-programme Rivers, DPRD = Sub-programme Rijnmond-Drechtsteden and DPZWD = Sub-programme South-west Delta

Dike-ring name	Stretch	Length [km]	Subprogramme	Signal value (1/...)	Lower limit (1/...)
Zuid-Holland	14-1	20.3	DPRD	100,000	30,000
	14-2	16.5	DPRD	10,000	3,000
	14-3	4.4	DPRD	10,000	3,000
Lopiker- en Kripmenerwaard	15-1	23.1	DPRD	30,000	10000
	15-2	24.4	DPRD	10,000	3,000
Alblasserwaard en Vijfheerenlanden	16-1	15.1	DPR & DPRD	100,000	30,000
	16-2	31	DPR & DPRD	30,000	10,000
	16-3	19.9	DPR & DPRD	30,000	10,000
	16-4	19.6	DPR & DPRD	30,000	10,000
Ijsselmonde	17-1	26.9	DPRD	3,000	1,000
	17-2	26.6	DPRD	3,000	1,000
	17-3	9.4	DPRD	100,000	30,000
Pernis	18-1	5.2	DPRD	10,000	3,000
Rozenburg	19-1	8.1	DPRD	100,000	30,000
Voorne-Putten	20-2	13	DPRD	10,000	3,000
	20-3	21.9	DPRD	30,000	10,000
	20-4	19.8	DPRD	1,000	300
Hoeksche Waard	21-1	30.4	DPRD	3,000	1,000
	21-2	40.3	DPRD	300	100
Eiland van Dordrecht	22-1	17.5	DPRD	3,000	1,000
	22-2	21.5	DPRD	10,000	3,000
Biesbosch	23-1	2.6	DPR	3,000	1,000
Land van Altena	24-1	18	DPR	10,000	3,000
	24-2	13	DPR	1,000	300
	24-3	15.3	DPR	10,000	3,000
Goeree Overflakkee	25-2	26.9	DPZWD	1,000	300
West-Brabant	34-1	24.4	DPZWD	1,000	300
	34a-1	23	DPZWD	3,000	1,000
	34-2	9.9	DPZWD	1,000	300
Donge	35-1	13.8	DPR	10,000	3,000
	35-2	14.7	DPR	3,000	1,000
Total	31	576.5	-	-	-

2.5. FAILURE MECHANISMS

Failure of a retaining structure may occur as a consequence of various causes. This section will describe the most important failure mechanisms for (river) levees. Failure mechanisms for other types of retaining structures, like dunes and hydraulic structures will not be discussed as this study focusses on investments in dike reinforcements especially. This section starts with a definition of failure in subsection 2.5.1, followed by an overview of possible failure mechanisms for dikes in 2.5.2. Next, the tow failure mechanisms, overtopping/overflow and piping, which are the most dominant failure mechanisms for dike ring 16 according to the study of VNK (Vergouwe, 2014) are elaborated more in detail. See section 3.2 for the clarification to assess dike ring 16.

2.5.1. DEFINITION OF FAILURE

In CIRIA (2013, p. 156) failure is defined as: 'The inability to achieve a defined performance threshold (response to a given loading) or performance indicator, for a given function. Failure is a state.'

For a levee system, consisting of various retaining structures and levee segments, the state of failure is defined by the 'unintentional inundation of the levee area' (CIRIA, 2013). This inundation can be either the consequence of a structural failure of (parts of) the flood defence or the consequence of a hydraulic failure, in which the structure itself remains intact, but where a critical amount of water is flowing over the dike (for the failure mechanisms of overtopping and overflow). Both failure mechanisms can induce the other failure mechanism to occur.

2.5.2. OVERVIEW FAILURE MECHANISMS

In figure 2.10, an overview of failure mechanisms for dikes is sketched:

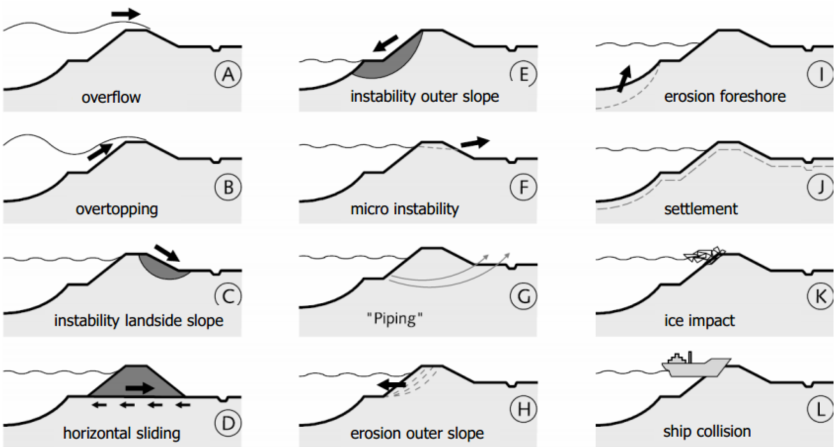


Figure 2.6: Failure mechanisms for dikes (Schiereck, 1998, p. 79)

In table 2.7, it is explained how these failure mechanisms are induced. One should remark that the occurrence of a failure mechanism, will not lead automatically to failure of the water retaining function. The described failure mechanisms are so-called 'initial failure mechanisms'. For instance for the failure of a dike to due piping, resulting in a flood, three separate events have to occur which are heave, uplift and piping (Schweckendiek & Calle, 2013). Furthermore instability of outer slope may not lead immediately to flooding in case the water level is still lower than the crest level.

Table 2.7: Inducing forces on failure mechanisms

Failure Mechanism	**Induced by**
A) Overflow	Water level exceeds crest level of dike; water starts to flow over the dike
B) Overtopping	The crest level of waves exceeds crest level of dike. Water (partly) flows over the dike and erosion of inner slope may occur and water infiltrates the dike
C) Instability landside slope	The driving moment of the(wet) earth body exceeds the resisting moment that is driven by shear stress. This failure mechanism is amongst others described by the Bishop method
D) Horizontal sliding	Horizontal forces due to hydrostatic pressure exceed the horizontal shear capacity of the dike
E) Instability outer slope	The excess pore pressure in the dike is increased due to infiltration during high water. When the water level suddenly drops the driving moment becomes larger than the resisting moment.
F) Micro instability	When there is high water for a longer duration, the phreatic level in the dike develops to a maximum. At the toe of the inner slope, grains start to wash out when the phreatic level exceeds a critical value (Bałachowski, 2014)
G) Piping	Excessive pressure in the aquifier causes the aquitard to lift up (Uplift), next groundwater flows towards the leak (Seepage) and the flow starts to erode granular material (Heave). A pipe starts to develop in upstream direction until a continuous pipe is formed and erosion accelerates, resulting finally in a structural collapse by undermining (Schweckendiek & Calle, 2013).
H) Erosion outer slope	Erosion of outer slope is mainly induced by wave forces; especially for revetments
I) Erosion foreshore	A foreshore reduces wave load and gives stability to dike. Bow thrusts of ships or dredging activities may induce erosion of foreshore.
J) Settlement	Consolidation, creep, land subsidence and extraction may cause a levee system to reduce its retaining height (permitting other failure mechanisms like overflow and overtopping to occur in an earlier stage)
K) Ice	Drifting ice gives an extra horizontal load on the dikes. Ice may shear over the dike, inducing erosion.
L) Ship collision	Ship that has lost rudder might hit the dike

According to (Vergouwe & Van den Berg, 2013, p. 46), the failure mechanisms Overtopping and Overflow in combination with Uplift and Piping are the most dominant failure mechanisms for dike ring 16, the dike ring that will be assessed in this study (again, see section 3.2 for the clarification to assess this dike ring). Table 2.8 shows the calculated return periods for the entire dike ring in VNK. Macrostability has also a large failure probability and contribution to the overall failure probability as also can be seen in the diagram 2.7, but this is largely determined by the heigh failure probability of a single dike section (Vergouwe & Van den Berg, 2013, p. 155), leaving piping and overtopping/overflow as the normative failure mechanisms.

Table 2.8: Dominant failure mechanisms dike ring 16 (Vergouwe & Van den Berg, 2013, p. 46)

Failure mechanism	$P_f[year^{-1}]$
Overtopping and overflow	$1/1,460$
Macrostability innerslope	$1/360$
Uplift and Piping	$>1/100$
Erosion outer slope	$1/80,000$

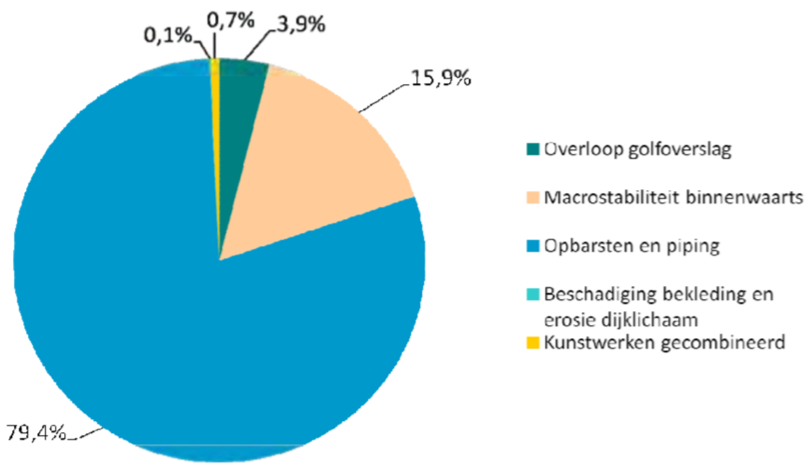


Figure 2.7: Relative contribution of failure mechanisms to failure probability of dike ring 16 (Vergouwe & Van den Berg, 2013)

2.5.3. OVERFLOW & OVERTOPPING

OVERFLOW

As described in table 2.7, overflow occurs in case the water level exceeds the crest level of a dike. Water then starts to flow over the dike. Overflow is related with the still water level, hence flow over a dike due to waves is excluded in this mechanism. Overflow does not directly lead to failure, as inundation of the lower lying area does not directly takes place. Often, first a breach is formed and flow increases. Due to the breach, structural failure of the levee occurs, leading to inundation of the area. The limit state function (LSF), with $P_f(Z<0)$ for this mechanism reads (Steenbergen, Vrouwenvelder, & Koster, 2008):

$$Z = h_d - h_{water} \tag{2.6}$$

For this study, it is assumed that the duration of a overflow situation does not influence the failure of a levee, so when $h_{water} > h_d$ failure occurs. This is a conservative assumption as in reality a certain time is needed either to inundate an area entirely or to develop a breach in a dike. Furthermore one could think of a critical flow that is necessary before erosion starts, in this study this critical flow is also neglected as conservative assumption.

OVERTOPPING

In case of the overtopping mechanism, local waves due to wind and seiches are included in the failure mechanism. When there is a critical flow over the levee, the revetment of the levee at the inner side may fail, inducing a breach. Another possible fail scenario is that a critical flow infiltrates the levee at the inner slope up to a moment where the granular material in the dike starts to flow out with a breach as a consequence. In the new design instrumentation (Infrastructuur en Milieu, 2015a, p. 18), maximum allowable critical flows are determined. For this study a maximum allowable critical flow of $5l/m/s$ will be maintained, which is a conservative value according to the instrumentation. The LSF, with $P_f(Z<0)$ reads:

$$Z = m_{qc}q_c - m_{q0}q_0 \tag{2.7}$$

where:

m_{qc} = Model uncertainty factor for the critical overtopping q_c
q_c = The critical average flow at which a dike collapses, set at $5l/m/s$
m_{q0} = Model uncertainty factor for the appearing overtopping q_0
q_0 = Occuring overtopping flow, which can be determined in various ways

The allowed failure probability P_f depends on the standard specifications given for a dike trajectory or dike cross-section. This is discussed in 2.4 and shall be explained in more detail for this mechanism in section 3.5.

The occuring overtopping q_0 can be modelled in various ways. The calculation takes place within the Software program Hydra-B, via the equations of Van der Meer (Geerse, 2003, p 59). More information about overtopping flow is found in the overtopping manual, which provides formulae for different types of breaking- and non-breaking waves (Pullen et al., 2007).

2.5.4. PIPING

Failure of a levee due to piping occurs in case the three related mechanisms to piping occur. These mechanisms are 'uplift', 'heave' and 'piping'. Schweckendiek and Calle (2013) describe these as: 'Excessive pressure in the aquifier causes the aquitard to lift up (Uplift), next groundwater flows towards the leak (Seepage) and the flow starts to erode granular material (Heave). A pipe starts to develop in upstream direction until a continuous pipe is formed and erosion accelerates, resulting in a structural collapse by undermining'. In below figure 2.9 the development of the failure mechanisms piping is given. As stated, failure due to piping occurs when the Limit State Function is reached for all three mechanisms (Uplift, Heave and Piping). These mechanisms will be discussed next, whereupon the actual computation of this failure mechanism is described.

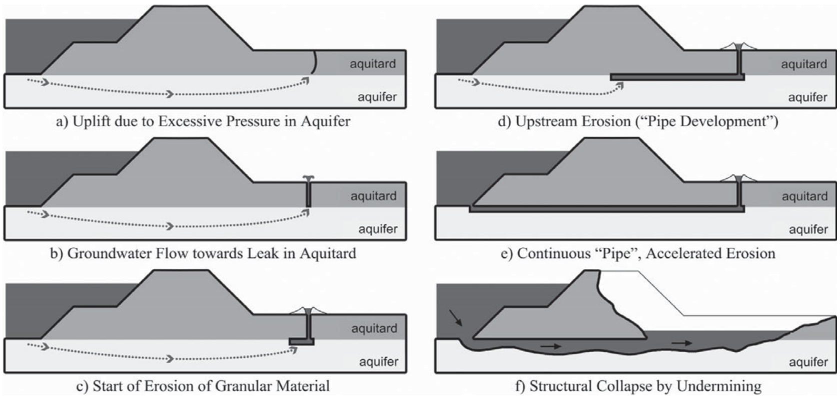


Figure 2.8: Failure of a levee due the development of the piping mechanism (Schweckendiek & Calle, 2013)

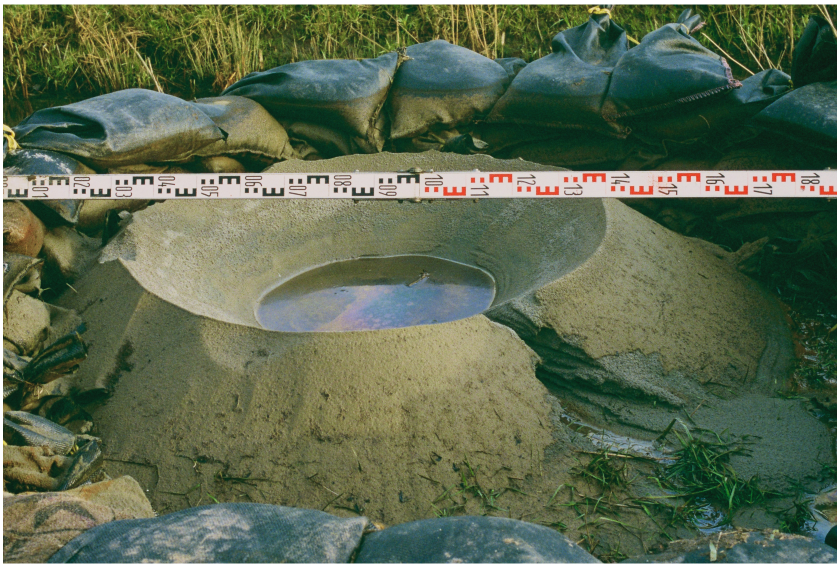


Figure 2.9: Exit point of a pipe (P. Cappenwijk, via beeldbank.rws.nl)

UPLIFT

Uplift is the mechanism in which the aquitard is lifted up by the excessive pressure that is developed in the aquifer. The LSF for uplift is given in the following equation (Schweckendiek & Calle, 2013), failure occurs for the situations where $Z_u < 0$:

$$Z_u = m_u \Delta\phi_c - m_\phi \Delta\phi \tag{2.8}$$

where:

m_u = Model factor that states the uncertainty in the model that determines the critical water level [m]
$\Delta\phi_c$ = Critical potential difference, given in equation (2.9) [-]
m_ϕ = Model factor which defines damping [-]
$\Delta\phi$ = Estimated potential difference [m]

$$\Delta\phi_c = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} d \tag{2.9}$$

where:

γ_{sat} = Saturated volumic weight of the blanket layer [kN/m^3]
γ_w = Volumetric weight water [kN/m^3]
d = Blanket layer thickness [m]

LSF FOR HEAVE:

The mechanism of heave occurs when granular material starts to erode at the place where the hinterland is lifted up. Its LSF is given by (Schweckendiek & Calle, 2013):

$$Z_h = i_c - (m_\phi \phi) - h_b / d \tag{2.10}$$

where:

i_c = The critical exit gradient as determined by (Terzaghi, n.d.) see 2.11
h_b = Surface level at exit point [m]
h = Local occuring water level [m]

$$i_c = \frac{\gamma_{sub}}{\gamma_w} \approx \frac{G_s - 1}{1 + e} \tag{2.11}$$

where:

G_s = Specific gravity of sand grains (= 2.65)
e = Void ratio

LSF FOR PIPING:

Recently the formula for piping has been revised. The formula for piping is now described by the (revised) rule of Sellmeijer:

$$Z_p = m_p H_c - (h - h_b - 0.3d) \tag{2.12}$$

where:

m_p = Model factor that states the uncertainty in the model that determines the critical water level
H_c = Critical difference in water level, given in equation 2.13 [m]
h = Local (occuring) water level [m]
h_b = Water level at exit point [m]
d = Blanket layer thickness [m]

$$H_c = F_{resistance} \cdot F_{scale} \cdot F_{geometry} \cdot L \tag{2.13}$$

$$F_{resistance} = \eta(\frac{\gamma_s}{\gamma_w} - 1) tan(\theta) \tag{2.14}$$

$$F_{scale} = \frac{d_{70m}}{\sqrt[3]{\frac{\upsilon k L}{g}}} \left(\frac{d_{70}}{d_{70m}}\right)^{0.4} \tag{2.15}$$

$$F_{geometry} = 0.91(D/L)^{\frac{0.28}{(D/L)^{2.8}-1}+0.04} \tag{2.16}$$

where in equations 2.13 to 2.16:

L = Leakage length [m]
η = Coefficient of White or drag factor [-]
γ_s = Dry volumetric weight of sand grains $[kN/m^3]$
γ_w = Volumetric weight water $[kN/m^3]$
θ = Internal friction angle sand grains [°]
v = Viscosity of water $[m^2/s]$
k = Specific conductivity of aquifer $[m/s]$
g = gravitational constant $[m/s^2]$
d_{70} = 70% quantile of the grain size distribution [m]
d_{70m} = Mean d_{70} in small scale laboratory tests[m]
D = Thickness aquifer [m]

The parameters regarding to the geometry of a levee are shown in figure 2.10.

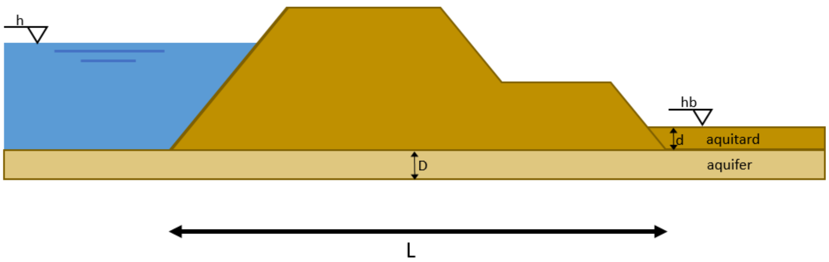


Figure 2.10: Parameters regarding to the geometry of a levee for piping

COMPARISON REVISED SELLMEIJER WITH OLD FORMULA

Recently the formula of Sellmeijer has been revised. However, in former VNK projects often the old formula is used (Steenbergen et al., 2008). The old Sellmeijer is used in case the aquifer consists of one layer. An other calculation method of VNK was to make use of and extern software programme named 'MPiping' in which critical heads were calculated when the aquifer consisted out of two layers.
The LSF of the old Sellmeijer reads the same as the revised sellmeijer and its formula is given by:

$$Z = m_p H_c - (h - 0.3d - h_b) \tag{2.17}$$

The former formula for H_c as used by VNK reads:

$$H_{c,old} = \alpha c L(\frac{\gamma_s}{\gamma_w} - 1)(0.68 - 0.1\ln(c))\tan(\theta) \tag{2.18}$$

with α a factor that reflects the effect of a finite thickness of the aquifer. The formula for α reads:

$$\alpha = \left(\frac{D_1}{L}\right)^{\frac{0.28}{\left(\left(\frac{D_1}{L}\right)^{2.8} - 1\right)}} \tag{2.19}$$

Coefficient c is determined by sand properties in the aquifer:

$$c = \eta d_{70}\left(\frac{1}{\kappa L}\right)^{\frac{1}{3}} \tag{2.20}$$

where κ is given by:

$$\kappa = \frac{v}{g}k \tag{2.21}$$

Evaluating both formulas, the revised and old Sellmeijer formula, shows that the revised formula leads to a much smaller resistance. In table 2.9 some chosen values are given in order to determine the critical heads for both formulas. Filling in the parameters of this table in equations 2.13 to 2.16 and 2.18 to 2.21 gives the following results for the calculated critical head as resisting force:

$$H_{c,revised} = 6.67m \tag{2.22}$$

$$H_{c,old} = 7.74m \tag{2.23}$$

The large difference of 1.15m in critical head between the two formulas can be explained as follows: It has not only to do with the revision of the Sellmeijer formula, but more important is a change in principle in the use of the Sellmeijer formula. In the former application of the Sellmeijer formula for sections with one aquifer layer, the so called ‘4-forces’ model is applied. This is the case when the grain is embedded between other grains. In following years the model is improved to a ‘2-forces’ model where the grain is no longer embedded, but lying at the surface. It is no longer supported by any other grains. In the computations of VNK this renewed insight is only taken into account partly: When the bottom section is dimensioned with only 1 aquifer layer, the ‘4-forces’ model is used, whereas when the bottom has 2 separate aquifer layers, the revised ’2-forces’ model is used within MSeep (Steenbergen et al., 2008, p. 24) and (Knoeff, 2009, p. 3).

Table 2.9: Chosen parameters for an arbitrary example to determine the critical head H_c or H_p with the new or old Sellmeijer formula

Parameter	Choosen value	Units
η	0.25	$[-]$
γ_s	27	$[kN/m^3]$
γ_w	10	$[kN/m^3]$
θ	37	[deg]
d_{70m}	2.08E-04	$[m]$
d_{70}	2.00E-04	$[m]$
g	9.81	$[m/s^2]$
v	1.33E-06	$[m^2/s]$
k	1.00E-04	$[m/s]$
D_1	3.00E+00	$[m]$
L	5.00E+01	$[m]$

2.6. OTHER ASPECTS RELATED WITH FAILURE

This section describes three aspects that are related to the failure probability of a dike ring or trajectory. First the failure of the Maeslant barrier is discussed and the contribution of failure of the Maeslant barrier to the overall failure probability on dike sections is addressed as this mechanism is an important load driver in the evaluation of the reference strategy ‘DP2015’. Next, the principle of the length effect is considered, which is not a failure mechanism itself but is a phenomena which influences the strength of a retaining structure or system. The section ends with aspects regarding proven strength of a levee.

2.6.1. FAILURE OF MAESLANT BARRIER

In section 1.1.3, the reference strategy ‘DP2015’ according to the Delta program is elaborated. One of the key drivers of this strategy consists of the functioning of the Maeslant barrier. As the Maeslant barrier is a retaining structure in the category B (see 3.2), its function is to reduce water levels behind the barrier instead of directly protecting the hinterland. While the functioning of this barrier does not directly influence the strength characteristics of the earlier mentioned failure mechanisms, it does influence the load occurring at the levees behind the barrier. Currently the failure probability of the Maeslant barrier is calculated at 1/109 per closure (Kallen et al., 2012, p. ii). The Maeslant barrier is open in most of the situations and only has to close in case the predicted water levels in Rotterdam and Dordrecht are respectively 3.0m and 2.9m (the so called closure criterion) and the water levels at sea are exceeding the water levels in the estuary due to for instance a storm surge. Hence the reduction in water level due to functioning of the water levels occurs only in rare events when high water levels are expected.

Last, the influence of functioning of the Maeslant barrier is at its largest just behind the barrier while this influence will decay more upstream as the influence of river discharge on water levels is increasing in upstream direction (see also figure 2.1). In figure 2.12 the normative water levels are shown for dike section 14003012 (Located just behind the barrier nearby Rotterdam, see also figure 2.11) and for dike section 22001013 (nearby Dordrecht) in which the influence of the failure probability of the Maeslant barrier can be derived. Figure 2.12 holds for the reference strategy 'DP2015' in 2015 with a failure probability of 1/100 per closure and in 2100 with a failure probability of 1/1,000 per closure. The influence of climate change with sea level rise and an extreme river discharge of $18,000 m^3/s$ with a frequency of 1/1,250 per year instead of $16,000 m^3/s$ is shown as the lines in 2100 lie above the lines of 2015, while also the influence of the failure probability of the Maeslant barrier on the water levels can be derived: For section 14003012 the normative water levels are significantly lower up to a return period of approximately 2,000 years, while water levels with a return period above 2,000 years show a steeper line, an indication for the failure of the Maeslant barrier in these events. One might expect that due the increased functioning of the Maeslant barrier in 2100 from 1/100 to 1/1,000 the bend in the line would shift to the right; however due to sea level rise the frequency of closure will also increase thus limiting the effect of an improved barrier.

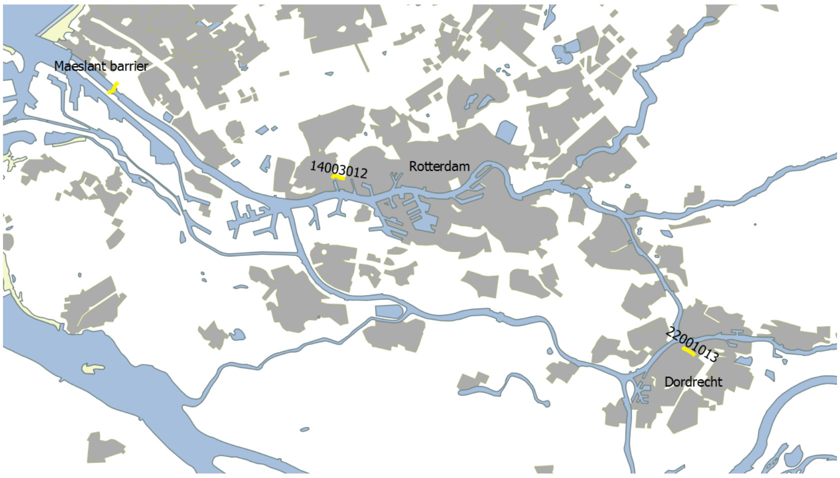


Figure 2.11: Locations of dike sections 14003012 and 22001013

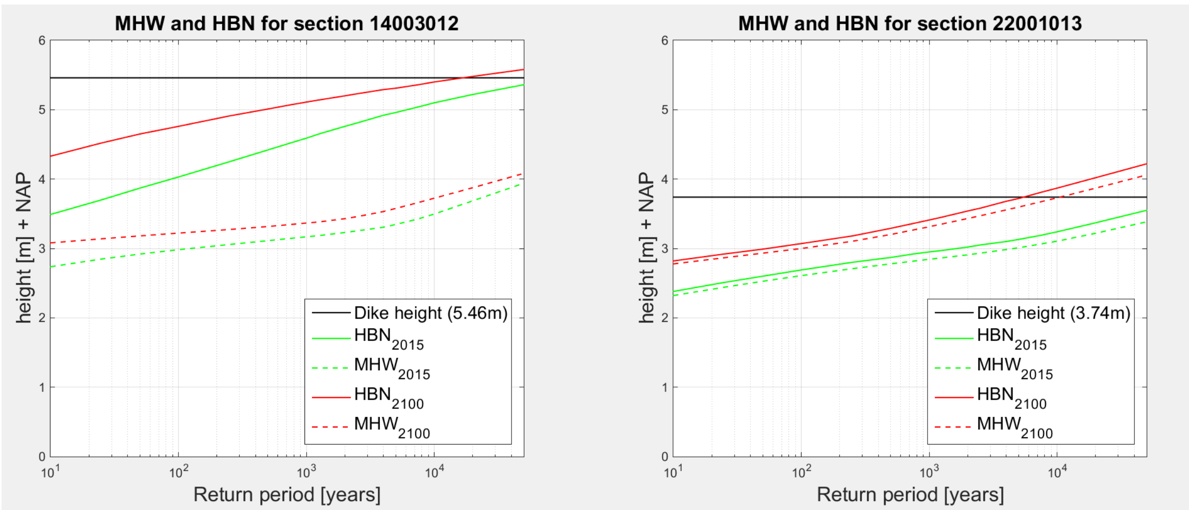


Figure 2.12: MHW and HBN at toe of dike dike sections 14003012 and 22001013 for strategy 'DP2015' in 2015 and 2100

2.6.2. LENGTH-EFFECT

According to Kanning (2012, p. iii) the length-effect causes the probability of failure to increase with the length of the structure, as a weak link might be encountered due to rapid fluctuation of soil uncertainties in space. The length effect can best be compared by a chain; if there is a critical load acting on the chain, the chain will always break at the weakest link. The longer the chain, the higher the probability of a weak link.

To coop with the length effect in the assessment of failure probabilities of dikes, a formula is developed to include the length effect. For the piping mechanism, it is given by (Infrastructuur en Milieu, 2015a, p. 11):

$$N_{piping} = 1 + \frac{a \cdot L_{trajectory}}{b} \tag{2.24}$$

where in equations 2.13 to 2.16:

a	= Part of length of trajectory that is sensitive to the respective failure mechanism (0.4 for trajectories 16-1, 16-2, 16-3 and 16-4) [-]
b	= Length of independent, equivalent sections for the respective failure mechanism [m]
$L_{trajectory}$	= Length of the dike trajectory for which the norm is valid [m]

The formula for length effect shall be used to assess the calculated failure probabilities of dike section with respect to their proposed standard specifications.

2.6.3. SURVIVED LOAD

Survived load or in Dutch 'Bewezen sterkte' can provide valuable information in reducing failure probabilities. If an extreme water level has occurred in the past and the dike has proven itself by withstanding this load, the probability of failure given this extreme water level will reduce (Schweckendiek, 2010). In the most extreme scenario, assuming no deterioration of the levee over time and that strength characteristics are not affected due to the high water level, the probability density function for failure can be translated to a p.d.f. with higher strength characteristics as shown in next figure (compare to figure 2.3):

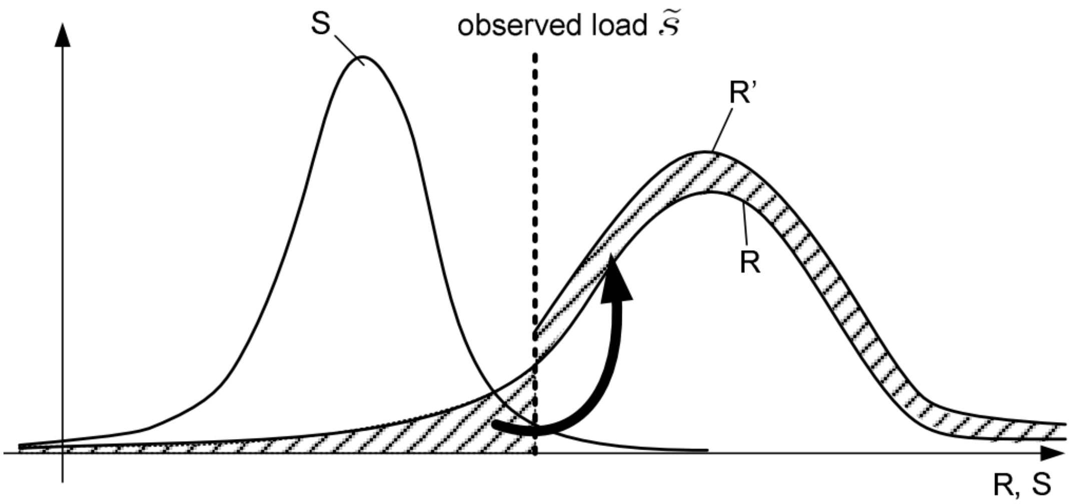


Figure 2.13: Reassessing reliability after survived load at point $\tilde{s}$ (Schweckendiek, 2010)

The reduction in failure probability by assessing survived loads is expected to be in the order of 2 to 20 for the Rhine-Meuse estuary according to Schweckendiek (2010, p. 13). However piping is also a time-dependent process, saying that the failure probability increases as the duration of a load also increases as the formation of pipes and development of the pipes takes time. For the assessment of survived loads, a pipe could have formed but not have progressed enough to cause structural failure. In this case, the reliability is falsely improved as the next time a high water occurs, the previously formed pipe could start to grow again. These insights are also endorsed by Förster, Van den Ham, Calle, and Kruse (2012, p.18), where a pipe formed under a first high water could lead to failure by a second high water event under a less critical head difference than was found in the first high water. On the other hand, it is not known for sure whether pipes formed during the first high water event, will be compressed again and more research is needed.

In assessing dikes, one have to keep in mind above principles. Schweckendiek and Calle (2013, p. 367) states therefore also that the reliability is only improved when no signs of seepage, heave or piping are observed, survival load really contributes. In other cases the failure probability even increases.

3
Methodology

In this chapter the research methodology is elaborated. From the literature study it became clear that recent insights with respect to flood probabilities and length-effects are not fully accounted for in cost calculations of dike reinforcements. Also a renewed configuration of strategy 'Sluices' is available, which was not accounted yet in the study of (Stone et al., 2014). It is chosen to perform a new cost analysis on the system for both strategies, based on new standard specifications and insights with respect to length-effects and failure mechanisms. The work approach for this analysis with its boundary conditions, principles and assumptions is found in 3.1. Following this approach, most distinctive dike trajectories are determined in section 3.2 with respect to expected costs for dike reinforcements. For these trajectories it is found that the failure mechanisms of overtopping/overflow and piping lead to the largest contribution in failure probability. For both mechanisms it is elaborated how these will be assessed, which is done in sections 3.3 and 3.4. In the last section of this chapter, it is expressed how the calculated failure probabilities are compared to the norm. The cost calculation with its underlying principles are entirely described in another chapter, chapter 6.

3.1. Approach

The methodology in this report aims to be consistent between the different alternatives and takes recent insights in flood probability from VNK into account. In figure 3.1 it is shown which steps are undertaken to evaluate the expected dike reinforcements in both the reference strategy 'DP2015' as the alternative 'Sluices'. These steps are elaborated next.

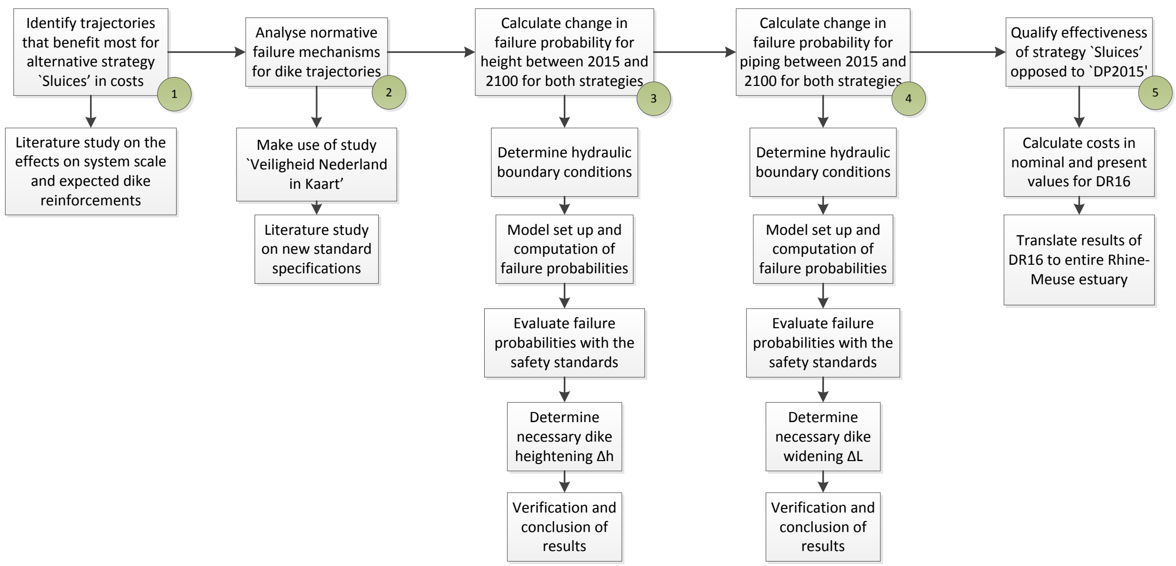


Figure 3.1: Overview of methodology for this thesis

From practical point of view, the main driver is to understand which dike sections will be rejected for both strategies and, once a section is rejected, to what extend it should be reinforced. According to new policies, each dike trajectory will be assessed every 12 years and the flood risk may never exceed the safety standards as set in subsection 2.4.2 (accounting for the lower limits). In this study, it is analysed on beforehand which dike sections within the trajectories will be rejected for the years 2015 and 2100 for both the strategies. Safety standards for dike sections are derived from the prevailing standards for dike trajectories in section 3.5. Once a dike section is rejected, it will be reinforced with a design horizon up to 2100. In other words: it should 'just' meet the safety standards from subsection 2.4.2 in 2100. In next sections and subsections, the steps in the scheme will be elaborated.

3.1.1. ELABORATE STEPS IN VARIOUS CHAPTERS

STEP 1: IDENTIFY MOST CRITICAL DIKE SECTIONS

In the first step (step 1 of figure 3.1) the dike trajectories are analysed. This will be done in section 3.2. Here it is elaborated that for dike ring 16, with the trajectories 16-1, 16-2, 16-3 and 16-4 a large reinforcement program is expected in the 'DP2015' strategy when new safety standards become effective.

STEP 2: ANALYSE NORMATIVE FAILURE MECHANISMS FOR DIKE TRAJECTORIES

In the second step the normative failure mechanisms are determined and elaborated in more detail. In subsection 2.5.2 it was elaborated that piping and overtopping/overflow are the largest contributors to the overall failure probability of dike ring 16. For these mechanisms the limit state functions are determined and models will be made in order to compute failure probabilities. The computed failure probabilities are compared to the maximum allowed failure probabilities and once a dike is rejected because of a shortage in height or leakage length, it is determined what increase in dike height (Δh) or berm width (ΔL) is necessary.

STEP 3: CALCULATE CHANGE IN FAILURE PROBABILITY FOR OVERTOPPING/OVERFLOW

The changes in failure probability due to a shortage in dike height, related to the mechanisms overtopping/overflow, will be assessed in chapter 4. The methodology for this step is described in more detail in section 3.3.

STEP 4: CALCULATE CHANGE IN FAILURE PROBABILITY FOR PIPING

The changes in failure probability in piping will be assessed in chapter 5. The methodology for this step is described in more detail in section 3.4.

STEP 5: QUALIFY EFFECTIVENESS OF STRATEGY 'SLUICES'

The effectiveness of strategy 'Sluices' will be discussed in chapter 6. A discussion is set forth about the applied assumptions and costs are calculated for dike ring 16. Furthermore a qualitative reflection will be given on the reduction in costs in the alternative strategy on the entire Rhine-Meuse estuary compared to the 'DP2015' strategy, by translating the effects (in both cost reduction and change in hydraulic loads) on dike ring 16 to other trajectories. In the last chapter, conclusions are made and recommendations are elaborated.

3.2. IDENTIFY DISTINCTIVE DIKE TRAJECTORIES

3.2.1. ASSESS ONLY TRAJECTORIES WITHIN THE RHINE-MEUSE ESTUARY

The total length of the flood defences marked in red in figure 2.1 is approximately 989.9km containing category-A, -B and -C defenses within both the Rhine-Meuse estuary and South-West Delta (see the definition in subsection 2.4.1). Only the category-A defenses within the Rhine-Meuse estuary will be analysed. The following assumptions and principles are taken into account by the demarcation to these sections:

- At this moment, only new standard specifications for the category-A defences are proposed. It makes no sense yet therefore to analyse the other types of defences (which are a minority) as it is not yet decided which safety standard they have to fulfil.
- In strategy 'DP2015', the Volkerak-Zoommeer will be used as a retention basin in the near future and requires therefore quite extensive dike reinforcements. However, in the alternative strategy, the Volkerak-Zoommeer will also be part of the retention system as the discharges of the Rhine and Meuse will be deflected to the Eastern Scheldt via the Volkerak-Zoommeer. It is assumed that the dike reinforcements necessary for this lake in both strategies will be of same order magnitude.

- In strategy ‘Sluices’, the Eastern Scheldt will be used as a retention basin, whereas this is not the case in strategy ‘DP2015’. However, because of the improvements in closure regime and leakage area of the Eastern Scheldt barrier, the design water levels and occurrence of water levels according to several return periods are expected to be of the same order magnitude in both situations as analysis by HKV have shown (Botterhuis & Stijnen, 2015a). It is therefore expected that there will only be slight differences in the dike reinforcement tasks between both strategies, which makes it less relevant to assess for the overall comparison.
- Expensive dike reinforcements take mostly place in urbanized areas where the space for reinforcements is limited, requiring more sophisticated solutions. The dike trajectories located in the South-West Delta are barely located in urbanized areas, whereas the dikes in the Rhine-Meuse estuary are more frequently located in urbanized areas.

With above assumptions, one has to bear in mind that the costs of the decreased leakage area in the Eastern Scheldt are only present in the alternative strategy ‘Sluices’ and not in ‘DP2015’. However, the costs for this reduction is estimated in the order of millions (Botterhuis & Stijnen, 2015a). In figure 3.2 the remaining area under consideration in this study is shown:

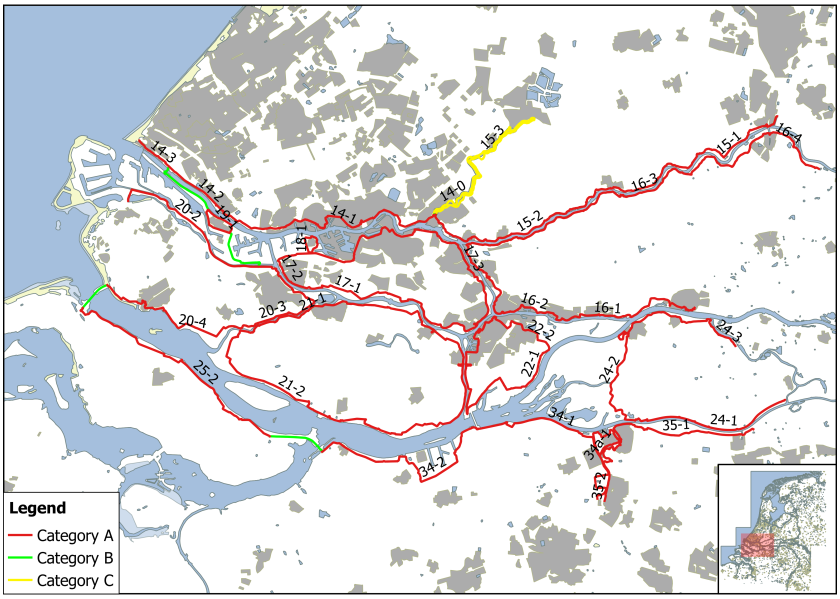


Figure 3.2: Overview of dike stretches that are within the scope of this research. The dike stretches in red are A-category flood defences, green marks the B-category flood defences and, the yellow dike stretches mark the extra C-category sections in case the Hollandse IJssel is taken into account

In in table 3.1, the total length of the dike stretches in the discussed area is given.

Table 3.1: Length of dike stretches within the discussed area

Area	Length dike stretches [km]
Total length of dike stretches in figure 2.1	989.9
Total length of dike stretches within scope in figure 3.2	576.5

3.2.2. ASSESS TRAJECTORIES 16-1, 16-2, 16-3 & 16-4

The Rhine-Meuse estuary consists of 576.5km dike stretches of category-A, which is still too much to assess in this study. It is chosen to limit the scope of this study to the most distinctive dike trajectories with respect to expected costs according current literature studies like Botterhuis (2015) and Stone et al. (2014). Relative cost reductions for these trajectories, will lead to largest savings in absolute terms. In figure 2.2 it is shown that the most expensive dike trajectories with respect to reinforcement tasks for the ‘DP2015’ were 14-1, 15-1, 15-2, 16-1, 16-2, 16-3 and 22-2. All these trajectories are (largely) located in the so called transition zone, meaning that the local water levels are under influence of both tide and storm surges from sea as under influence of high river discharges (see figure 1.2). In the study by Botterhuis (2015) nominal costs for dike reinforcements per trajectory are estimated and in Van Waveren (2015) the year of reinforcement is stated. From this it followed that the trajectories within dike ring 16 'Alblasserwaarden Vijfheerenlanden' are expensive trajectories to reinforce and will on short notice be reinforced between 2030 and 2040. The main contributor to the overall failure probability of the trajectories within dike ring 16 is the piping mechanism, followed by macrostability and overtopping/overflow. However, as earlier stated in subsection 2.5.2, the contribution of 16% due to macrostability is almost entirely determined by a single dike section. Assuming that this section will be reinforced on very short notice, the two contributing failure mechanisms within dike ring 16 are piping and overtopping/overflow (Vergouwe & Van den Berg, 2013). In table 3.2 the expected costs per trajectory are stated, in combination with year of reinforcement and contributing failure mechanisms. Among other mechanisms, the failure mechanisms of dunes and failure of hydraulic structures belong. Note that these costs are expected according to the assumptions as made within Botterhuis (2015) and are calculated for the 'DP2015' strategy.

Table 3.2: Normative trajectories with respect to expected reinforment costs

Traj.	Expected costs (Botterhuis, 2015)	Year of reinforcement (Van Waveren, 2015)	Influence height (Vergouwe, 2014)	Influence piping	Influence macro-stability	Influence erosion	Influence other
14-1	€528,000,000,-	2040-2050	41%	15%	0%	9%	35%
15-1	€337,000,000,-	2024-2030	4%	17%	72%	6%	1%
15-2	€531,000,000,-	2030-2040					
16-1	€652,000,000,-	2030-2040	4%	80%	16%	0%	0%
16-2	€482,000,000,-	2030-2040					
16-3	€352,000,000,-	2030-2040					
16-4	€191,000,000,-	2030-2040					
22-2	€391,000,000,-	2050-2100	2%	65%	15%	11%	7%

The expected nominal costs for dike ring 16 are total estimated at €1,677,000,000,-. This amount makes it worthwhile to conduct especially research at this dike ring. On the other hand, costs for this dike ring will already be made in the period 2030-2040. Even if the alternative strategy 'Sluices' is feasible, it is unlikely that they will be constructed in time to avoid these costs entirely. To avoid this, a revision of prioritization should be applied.

3.2.3. EVALUATED SECTIONS

The same sections within the trajectories of dike ring 16 are evaluated as the sections that are evaluated for piping in the VNK study (Vergouwe, 2014), the concerning sections are marked in figure 3.3. There are 57 dike sections within trajectories 16-1 to 16-4, of which 23 sections are assessed. The reason to evaluate only these specified sections, follows from the mentioned VNK study in which only the sections are evaluated that are believed to be significantly influenced by the piping mechanism based on a former study (Vergouwe & Van den Berg, 2013, p 5.). As for piping the mentioned sections are most relevant to investigate, it is chosen in line with this to also investigate these sections on overtopping/overflow. This implicates that for the failure mechanism of overtopping/overflow, dominant dike sections may be excluded in this study while these would be relevant. It is assumed that for the overall results still a good view will be extracted for the influence of height on system scale, even if some sections are excluded from the analysis. Dike ring 16 consists of 4 trajectories with trajectory safety standards of 30,000 [1/year] for trajectory 16-1 and and 10,000 [1/year] for the other trajectories (see also table 2.6).

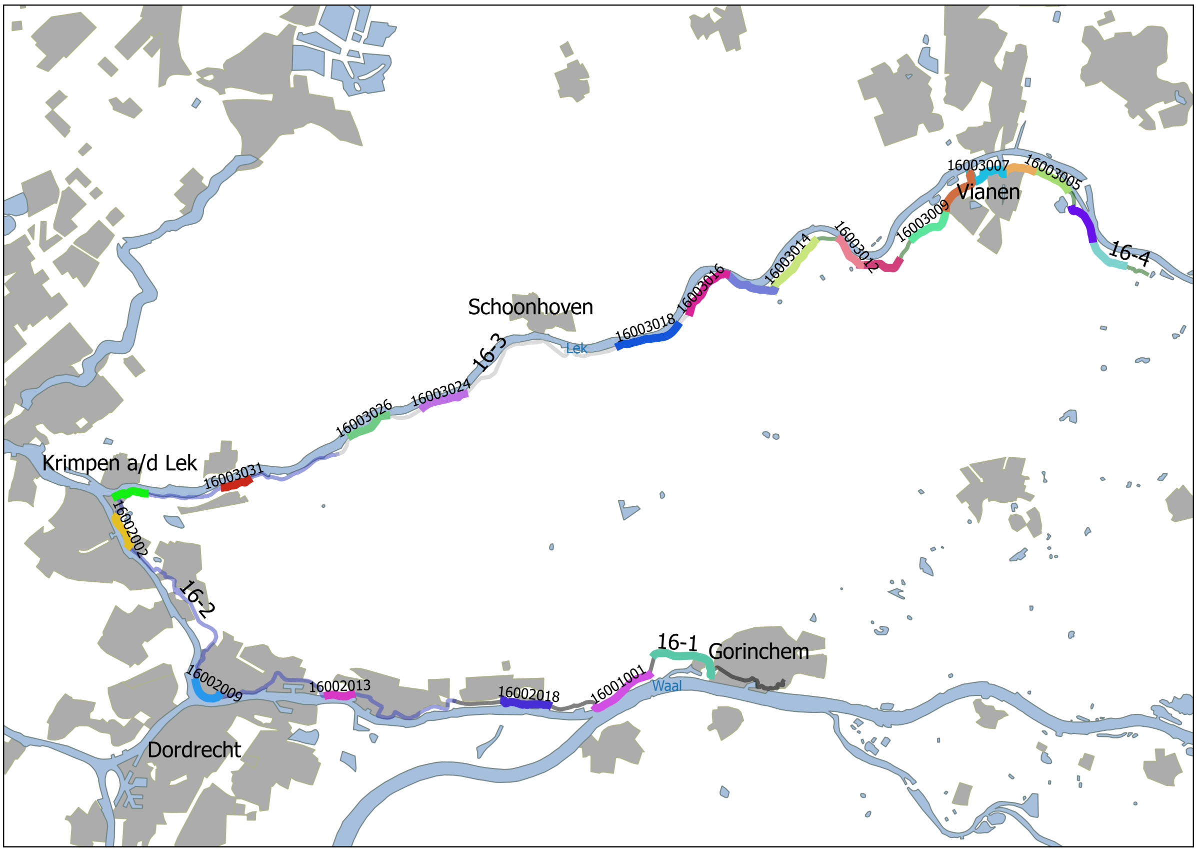


Figure 3.3: Dike sections evaluated for both height and piping within dike ring 16

3.3. CALCULATION OF FAILURE PROBABILITY FOR OVERTOPPING/OVERFLOW

The first mechanism that will be analysed, overtopping/overflow, is related with the height, slope and orientation of flood defences . The section will start with the hydraulic boundary conditions for the two strategies 'DP2015' and 'Sluices' in 2015 and 2100. Next, the schematization of the model is given in 3.3.2.

3.3.1. HYDRAULIC BOUNDARY CONDITIONS

HYDRAULIC BOUNDARY CONDITIONS

For both the strategies, the hydraulic boundary conditions are equal when it comes to expected sea level rise and extreme discharges of the rivers Rhine and Meuse. In table 3.3 the expected extreme river discharges and sea level rise are shown for the Warm/Steam scenario, the scenario that is followed in this study. The extreme river discharges of the Rhine and Meuse are expected to increase in 2050 and 2100 with respect to 2015. The water levels of the rivers in the Rhine-Meuse estuary, are Rhine dominant, saying that the water levels are mainly determined by river discharges of the Rhine. Therefore the extreme river discharges for the Rhine are adjusted in the 2050 and 2100 scenarios. In figure 3.4 it is shown how the work lines for the 2050 and 2100 scenarios are adjusted with respect to the current situation in 2015. The work line gives the extreme discharge on the y-axis that is exceeded only once in a certain return period (as given on the x-axis). The adjustments are made for river discharges with a return period of 25 years or more.

Table 3.3: Extreme river discharges for the Warm/Steam Deltascenarios (Kroekenstoel, 2014)

Year	1/1,250 year Rhine [m^3/s]	1/1,250 year Meuse [m^3/s]	Sea level rise [m]
2015 (S0)	16,000	3,800	0.08
2050 (S1)	17,000	4,200	0.35
2100 (S2)	18,000	4,600	0.85

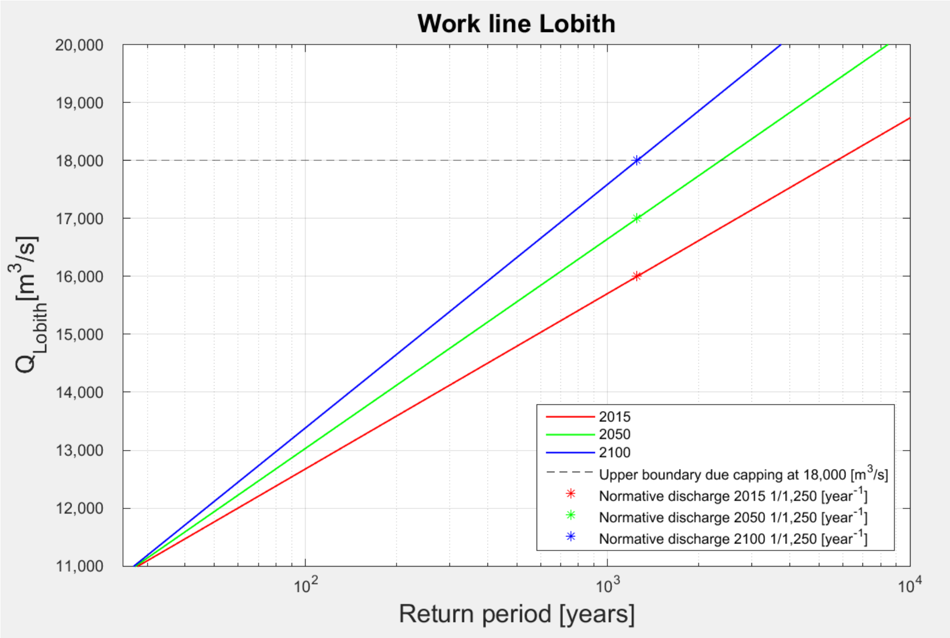


Figure 3.4: Work line for the Rhine at the location Lobith for return periods between 25 and 10000 years in 2015, 2050 and 2100

CHOSEN DESIGN WATER LEVEL: HYDRAULIC LOAD LEVEL, HBN

In the computation of failure probabilities and necessary dike heightening, the hydraulic load level is used. For an explanation of this design water level, see subsection 2.3.3.

3.3.2. MODEL SCHEMATIZATION

As input files for the calculation of failure probabilities, hydraulic databases for two different strategies in two different years (2015 and 2100) will be used. In table 3.4 the most important parameters of the different databases HKV (2015) are shown. In the databases both HBN's and MHW's are computed. Next the principles in computation of the water levels are discussed and differences between databases are explained.

Table 3.4: Generated databases with hydraulic information to compute failure probabilities, in this report the 'DP2015' strategy (ref) and alternative strategy 'Sluices' (H2) are assessed for 2015 and 2100

Filename	Strategy	Reference year	P_f MK	Deltamodel calculations
U06Ref_S0_3636lg_01	DP2015 (ref)	2015 (S0)	0.01	DM02 and DM03
U06Ref_S1_3636lg_01	DP2015	2050 (S1)	0.005	DM02 and DM03
U06Ref_S2_3636lg_01	DP2015	2100 (S2)	0.001	DM02 and DM03
U07ZdRo10h2_S0_3636lg_01	Sluices (H2)	2015	n.a.	DM02
U07ZdRo10h2_S1_3636lg_01	Sluices	2050	n.a.	DM02
U07ZdRo10h2_S2_3636lg_01	Sluices	2100	n.a.	DM02

COMPUTATION OF WATER LEVELS

The water levels are computed with a software program called Hydra, the hydraulic boundary conditions as described in 3.3.1 are used. For the configuration of the alternative strategy 'Sluices', a discharge capacity for the pumping stations of $3,000m^3/s$ is added and the free discharge sluices are opened any time that there is a positive head difference between the east- and west side of the sluices. Hence the sluices are opened for any time that $h_{sluices,riverside} > h_{sluices,seaside}$. The pumping stations are in operation for any time $h_{sluices,riverside} < h_{sluices,seaside}$ and $h_{sluices,seaside} > 1.0m + NAP$[i], which is a rather overdimensioned assumption. In reality, the pumping stations might only work in case an extreme river discharge is expected.

[i]NAP is defined as the Amsterdam ordnance datum in 1990

VNK DIKE PROFILE

In the computation of hydraulic load levels, the levels are computed by an interaction between load parameters - like wind velocity, wind direction fetch and river discharge - and strength parameters - like angle of orientation of a dike section, geometry of a levee and presence of a foreshore. Concerning the load characteristics, the same dike profiles will be used as used in the study of Veiligheid Nederland In Kaart (Vergouwe, 2014). As this was a study where in depth actual failure probabilities of dike sections are evaluated, it is expected that these profiles will lead to failure probabilities which are closer to reality than for instance dike profiles that are used in other studies like water safety (Dutch: water veiligheid) in the 21st century for which 'WV21' dike profiles are used (Kind, 2011). This will be addressed in chapter 4.

DELTA MODEL VERSION 02 VERSUS DELTA MODEL VERSION 03

Water levels are in VNK calculated according to principles from the Deltamodel version 02. Main principles according to calculations with this model are (Botterhuis, 2013):

- Discharge on the river Lek follows from a ratio that is not set on a maximum limit in 2015 and that is set on a maximum limit of $3,380m^3/s$ (ten Brinke, 2013) in the calculations for 2050 and 2100. The maximum limit is realized by increasing the discharge on other branches (de Waal and IJssel).
- Extreme discharge waves are not capped in 2015 and are capped on $18,000m^3/s$ in 2050 and 2100. It is expected that for river discharges of the Rhine above $18,000m^3/s$, dikes will break upstream the Dutch border (for instance in Germany), therefore it is physically not possible that a discharge above this level could occur in the Netherlands.

For the Deltamodel version 03, the principles differ with above principles, which is more correct for this study:

- Discharge on the river Lek is set on a maximum limit of $3,380m^3/s$ in *all* the calculations (2015, 2050 and 2100).
- Extreme discharge waves are capped in 2015 at $16,500m^3/s$, in 2050 at $17,000m^3/s$ and in 2100 at $18,000m^3/s$. It is expected that higher discharges cannot be reached as dikes upstream will break by discharges above these limits, leading to a positive effect of retention for the lower areas.

In first instance, calculations are made according the DM02 version, however, this might lead to situations where failure probabilities in 2100 are lower than in 2015 even if no extra measures are undertaken, due the artificially choices for capping in later reference years with respect to 2015.

Calculations in this study will be made with the DM03 version. In the 'DP2015' strategy, water levels are available computed for both DM02 and DM03. In order to get also hydraulic load levels for DM03 for the 'Sluices' strategy, the following translation will be made:

$$RP_{H2S0,DM03} = RP_{RefS0,DM03} + (RP_{H2S0,DM02} - RP_{RefS0,DM02}) \tag{3.1}$$

$$RP_{H2S2,DM03} = RP_{RefS2,DM03} + (RP_{H2S2,DM02} - RP_{RefS2,DM02}) \tag{3.2}$$

Where:

$RP_{H2S0,DM03}$ = Return period given h in 2015 in strategy 'Sluices' with deltamodel version 3 [m]
$RP_{RefS0,DM03}$ = Return period given h in 2015 in strategy 'DP2015' with deltamodel version 3 [m]
$RP_{H2S0,DM02}$ = Return period given h in 2015 in strategy 'Sluices' with deltamodel version 2 [m]
$RP_{RefS0,DM02}$ = Return period given h in 2015 in strategy 'Sluices' with deltamodel version 2 [m]
$RP_{H2S2,DM03}$ = Return period given h in 2100 in strategy 'Sluices' with deltamodel version 3 [m]
$RP_{RefS2,DM03}$ = Return period given h in 2100 in strategy 'DP2015' with deltamodel version 3 [m]
$RP_{H2S2,DM02}$ = Return period given h in 2100 in strategy 'Sluices' with deltamodel version 2 [m]
$RP_{RefS2,DM02}$ = Return period given h in 2100 in strategy 'Sluices' with deltamodel version 2 [m]

In above equations, the return periods will first be translated to return periods on logarithmic scale and afterwards calculated back to return periods in years again.

3.3.3. REQUIRED OUTPUT

As output, hydraulic load levels as a function of the return periods will be compared to the dike heights of the concerning sections. The hydraulic load level occurring given its safety standard (which will be described in section 3.5) is compared to the dike height in 2015 and 2100 for both strategies. In first instance, bottom subsidence is excluded out of these results. An example for the hydraulic load levels of dike section 16002009 are given in figure 3.5. It is shown that in the 'DP2015' strategy (Ref), in 2100 a shortage of 0.44m in dike height is present in case no dike heightening has taken place in between, while the calculated failure probability P_f is in the order of $0.9 \cdot 10^{-4}$ (1/Return period). This is an unacceptable situation and reinforcements should have taken place before. The scheduling of these reinforcements are elaborated in subsection 6.3.2, where also bottom subsidence will be included. In chapter 4 the results will be presented on a more visual manner, by representing results according the geographical location of the sections.

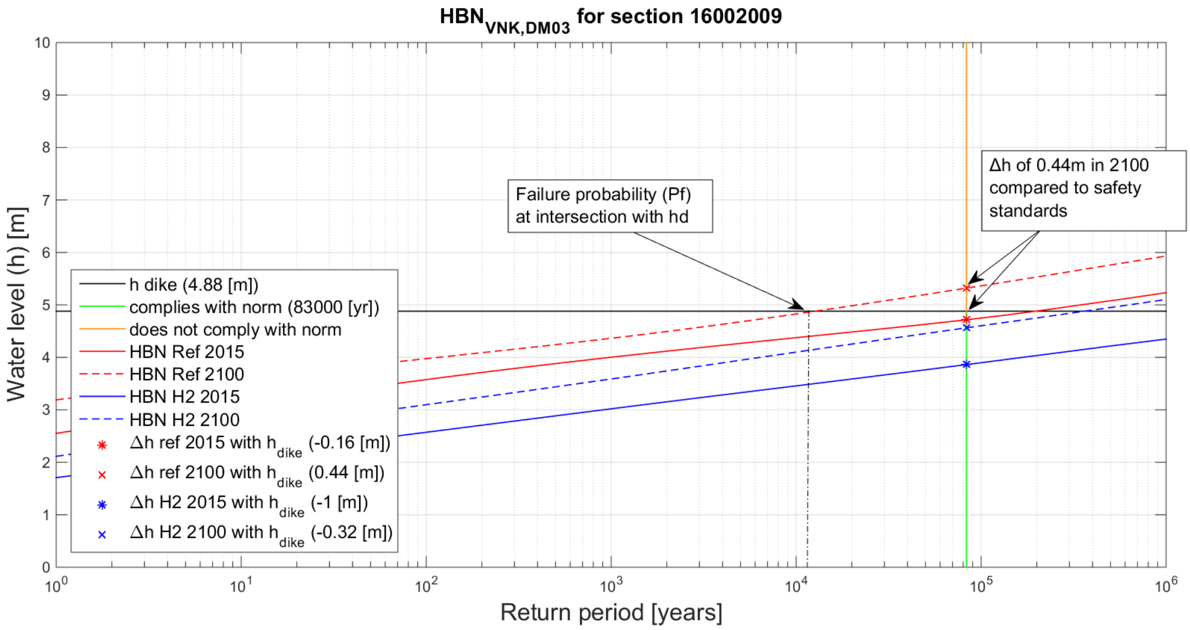


Figure 3.5: Computed HBN lines for dike section 16002009 for the 2 strategies in 2015 and 2100

3.4. CALCULATION OF FAILURE PROBABILITY DUE TO PIPING

In the following section, the approach for the calculation of failure probability due to piping is described. This section starts with a statement on the evaluated sections, applied strategies and used designs water levels. Then it is showed how the limit state function for piping is simplified, by assessing only the actual development of a pipe. Piping is a failure mechanism where there is a large interaction between load and strength characteristics. Therefore, in subsection 3.4.3 the strength characteristics are described for piping, followed by the interpretation of the acting load in 3.4.4. Finally the method to calculate the failure probability is given in 3.4.5.

3.4.1. EVALUATED SECTIONS, STRATEGIES AND DESIGN WATER LEVELS

The same dike sections for the failure mechanism piping are evaluated as done for height. The concerning sections are already discussed in subsection 3.2.3.

With respect to the evaluated strategies, also the same strategies are applied, namely the reference strategy according to the Delta Program in 2015 and 2100 denoted as 'DP2015' 2015 and 'DP2015' 2100. In figures also the term 'Ref' is used. The reference strategy 'DP2015' is compared with the alternative strategy 'Sluices' in 2015 and 2100. The alternative strategy 'Sluices' is also denoted with 'H2' in figures.

The used design water levels differ from the applied water levels in the previous chapter. For the mechanism of piping, influence of locally generated waves is minimal as the duration of the high water levels due to the waves is too small to develop the critical gradient in the subsoil. Furthermore the development of a pipe that is necessary within the piping mechanism to induce failure, takes much longer time than the short time of the waves related with HBN's. The normative water levels for this mechanism are therefore the MHW's instead of the HBN's. See also section 2.3 where the differences in water levels are explained.

3.4.2. SIMPLIFICATION OF THE MECHANISM

In section 2.5.4 it is stated that for the failure of a dike due to piping, three separate mechanisms have to occur at the same time; Uplift, Heave and Piping. As a conservative assumption, in line with the proposed design rules for levees (Infrastructuur en Milieu, 2013, p. 42), the state of failure of a dike section due to piping will be reached when only the piping mechanism occurs, hence the failure mechanisms of heave and uplift are excluded in further analysis (see figure 3.6). This assumption holds for no or very thin blanket layers (Schweckendiek, 2010). As shown in 2.5, the failure probability of a section given water level h, is determined by the following limit state function:

$$Z_p = m_p \cdot H_c - (h - h_b - 0.3d) \tag{3.3}$$

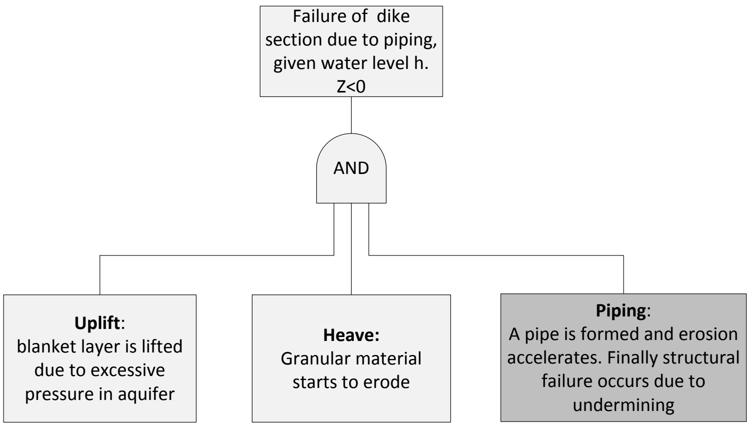


Figure 3.6: Fault tree for piping, with a conservative choice to assess only 'Piping'

3.4.3. STRENGTH CHARACTERISTICS

Via the described formulas of section 2.5, one can now compute the failure probabilities for a dike section for given water levels. The figure that describes these failure probabilities is called a fragility curve in which the failure probability Pf increases from 0 to 1 for rising water levels. Figure 3.7 gives the computed fragility curve for dike section 16001001, a dike section located along the river Waal nearby Gorinchem. The failure probabilities in the fragility curve for each water level are computed with a Monte Carlo simulation, in which variables are modelled stochastically with a mean μ and a standard deviation σ. Hence in each situation at which a water level is standing at a dike, one speaks of a failure probability. In appendix B.2 it is described how the fragility curves for the dike sections are constructed. The used parameters and structure of the model are also described in more detail in the appendix.

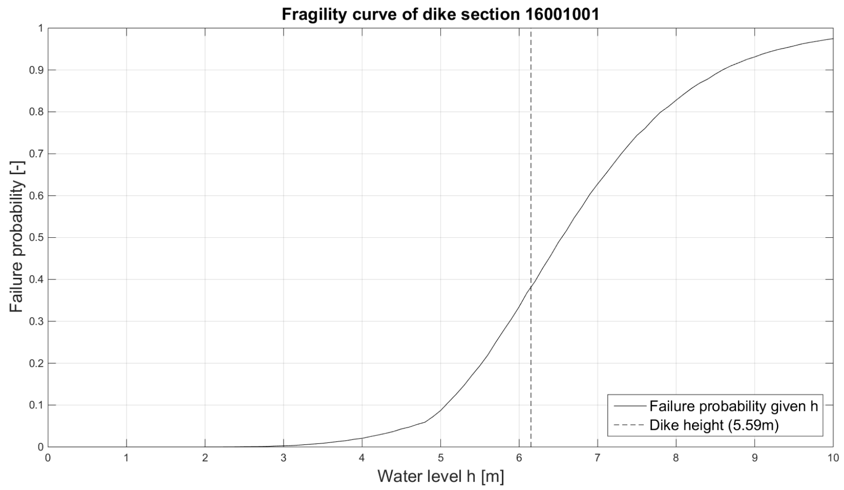


Figure 3.7: Fragility curve for section 16001001

3.4.4. LOAD CHARACTERISTICS

With the failure probabilities given a certain water level h described by means of fragility curves, the next step is to compute a probability density function of h. The probability density function describes the probability density of certain water levels to occur over an interval of time [-/year]. The integral of the probability density function over a certain interval of h, states the probability [-/year] that a water level between this interval is reached. Finally, the sum of both the probability density function of water level h with the fragility curve for a dike section, leads to the total annual failure probability due to piping.

(NON-)EXCEEDANCE PROBABILITY OF WATER LEVEL h

In the first place exceedance probabilities for water level h need to be determined, this is done via the formulations described in appendix B.3. Following on the exceedance probabilities, the non-exceedance probabilities are determined by:

$$P_{non-exceedance} = 1 - P_{exceedance} \tag{3.4}$$

Figures 3.8 and 3.9 show the exceedance and non-exceedance probabilities for normative high water levels at dike section 16001001 for the two strategies in 2015 and 2100.

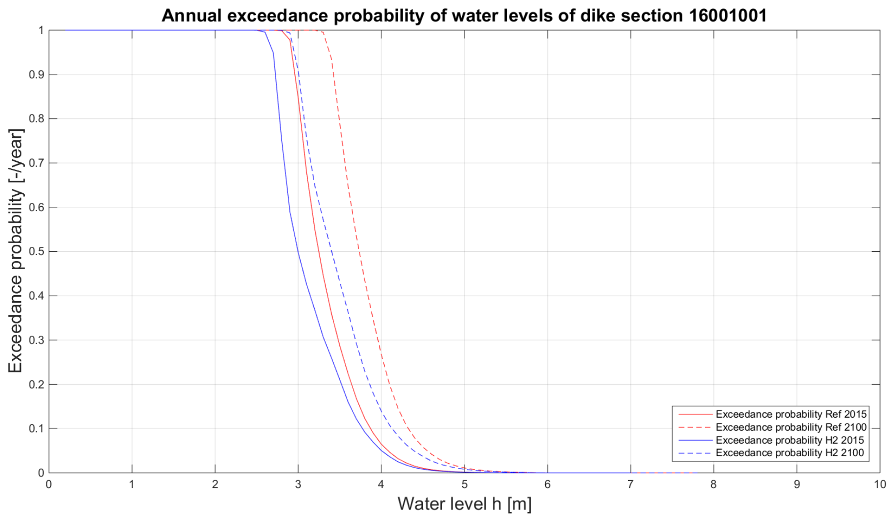


Figure 3.8: Exceedance probabilities for section 16001001, for the two strategies in 2015 and 2100

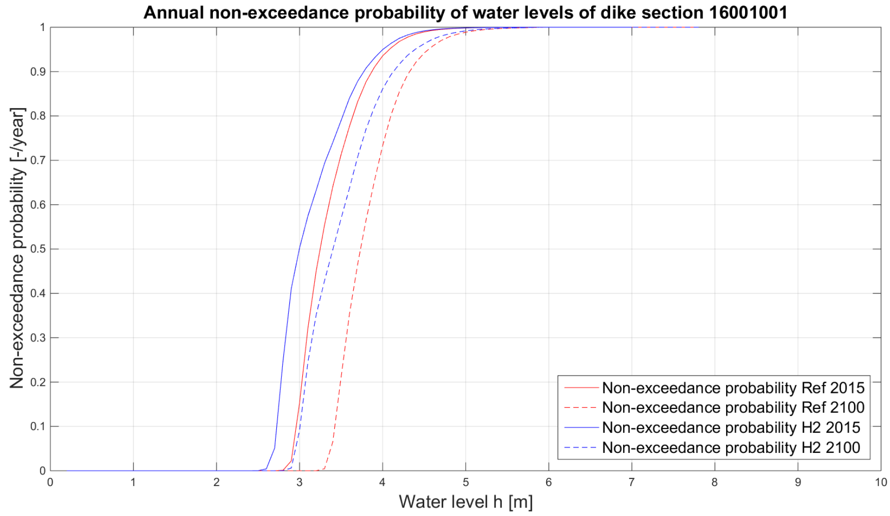


Figure 3.9: Non-Exceedance probabilities for section 16001001, for the two strategies in 2015 and 2100

PROBABILITY DENSITY FUNCTIONS

The figure that describes the probability of non-exceedance is also called the probability distribution function. The derivative of the probability distribution function gives the probability density function which is required to calculate failure probabilities in combination with fragility curves. The probability density function for dike section 16001001 is given in figure 3.10. The derivation of the probability density function from the probability distribution function for non-exceedance probabilities is given in equations B.3 and B.4 of the appendix.

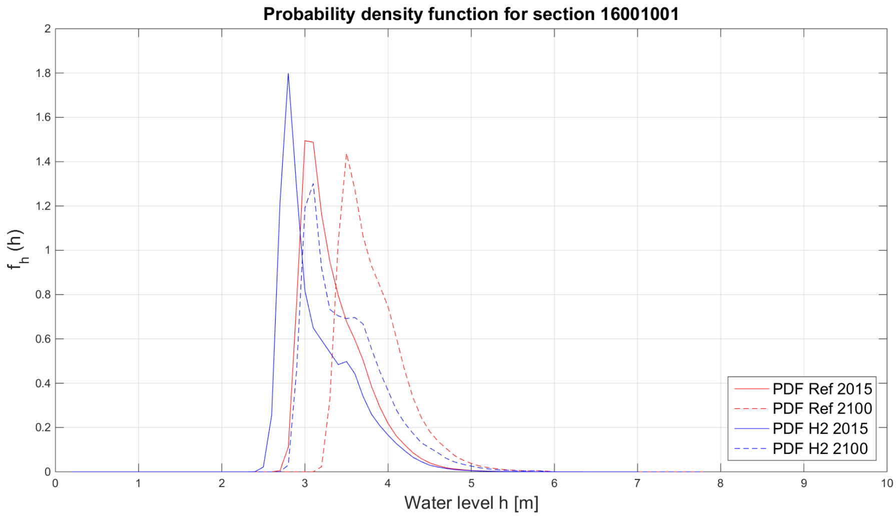


Figure 3.10: Probability distribution function for section 16001001, for the two strategies in 2015 and 2100

3.4.5. CALCULATION OF FAILURE PROBABILITY FOR A DIKE SECTION

The integration of the combined figure in which the probability density function is multiplied by the fragility curve (or failure probability given a certain water level) finally leads to the overall failure probability of a dike section (figures 3.11 and 3.12). The failure probability is given by:

$$P_f(z<0) = \int_{-\infty}^{\infty} \underbrace{P_f(Z<0|h)}_{\text{fragility curve}} \cdot \underbrace{f(h)}_{\text{pdf}} \cdot \mathrm{d}h \tag{3.5}$$

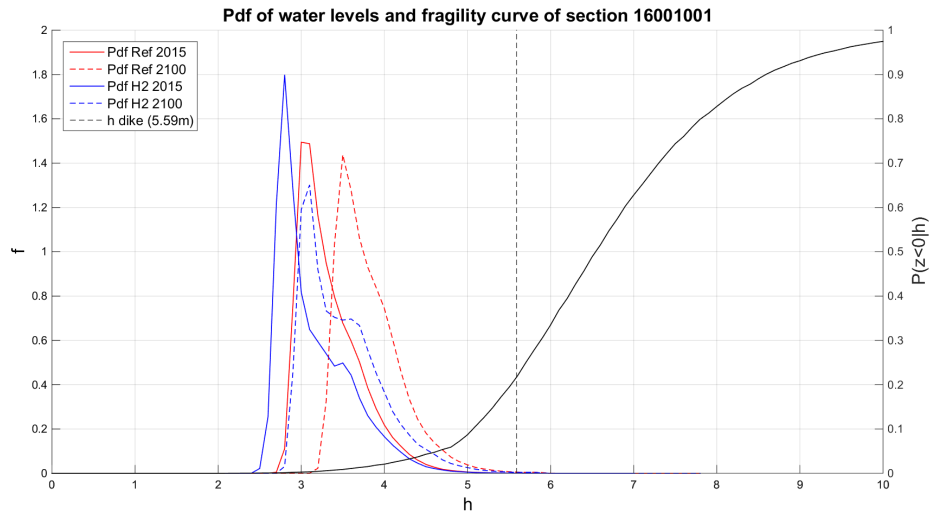


Figure 3.11: Both pdf and fragility curve for section 16001001, for the two strategies in 2015 and 2100

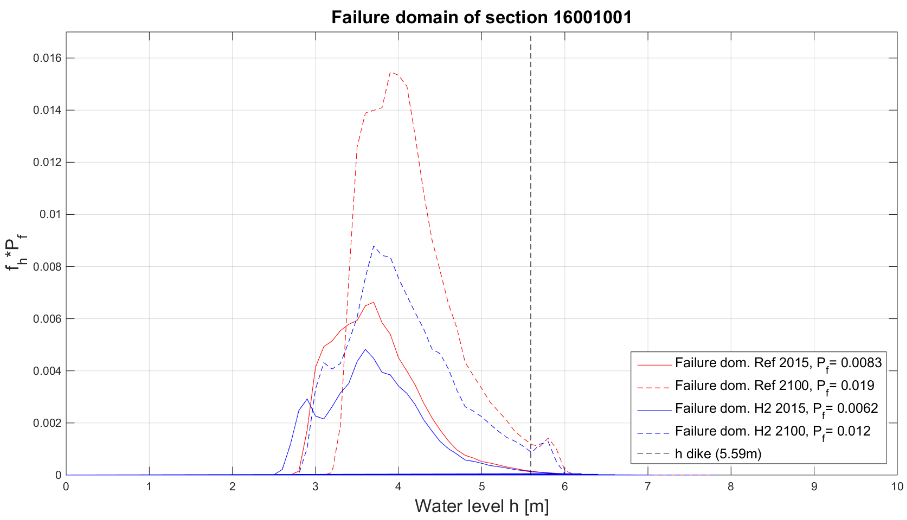


Figure 3.12: Failure domain for section 16001001, for the two strategies in 2015 and 2100. The area under the graph is equal to the failure probability

3.4.6. REQUIRED OUTPUT

The previous subsections showed that quite some steps are necessary in order to compute failure probabilities due to piping. In the first place, the calculated failure probabilities will suffice in order to get an insight in the flood risk. Once a section or trajectory is rejected according its standards, it needs to be reinforced. In general, reinforcements to cope the piping problem take place by increasing the leakage length by means of a piping berm. One could also place a seepage screen, but generally this is a more expensive solution. Once its found that dike sections are rejected, calculations are made to compute the shortage in berm width. Both aspects (failure probabilities and shortage in berm width), are presented in chapter 5.

3.5. ANALYSIS OF SECTIONS WITH SAFETY STANDARDS

The calculated failure probabilities will be compared to new safety standards which have to be met from 2050 for the entire Netherlands. Starting in 2017, dike sections will however already be assessed to these new specifications (Infrastructuur en Milieu, 2015a). An important aspect that should kept in mind, is that even when the hydraulic load reduction within strategy ‘Sluices’ leads to a reduction in flooding probability, high costs for dike reinforcements are necessary in case the standard specifications are not met. The requirements for height due to overtopping and overflow are explained in 3.5.1 and for piping in 3.5.2.

3.5.1. STANDARD SPECIFICATIONS FOR HEIGHT

In section 2.4, insight is provided in the new standard specifications as proposed by (Infrastructuur en Milieu, 2015a). In this section, standard specifications for an entire trajectory were given, and it is partly discussed how separate failure mechanisms contribute to the overall failure probability of a dike section. To determine the maximum contribution of each failure mechanism, a 'budgeting' formula is developed. For the failure mechanism of overtopping/overflow, a separate formula is proposed to provide a first insight in the reinforcement task that has to be undertaken for this mechanism. The formula for the overflow/overtopping mechanism reads (Infrastructuur en Milieu, 2015a, p. 18):

$$P_{norm,crosssection,height} = \frac{P_{max} \cdot \omega_{heigth}}{N_{height}} \tag{3.6}$$

Where:

$P_{norm,crossection,height}$	= The norm (maximum allowable failure probability) for a dike cross section for the failure mechanism of height [-/year]
P_{max}	= The max allowable failure probability for a trajectory [-/year]
N_{height}	= The factor for length effect, for trajectories 16-1, 16-3, 16-4 equal to 1 and for trajectory 16-2 equal to 2
ω	= The partial factor that is allowed for this failure mechanism (0.24 in case of height (overtopping/overflow)) [-]

Above formula states that the norm for a trajectory can be translated to a norm for an arbitrary cross section of the dike within the trajectory. The N-factor is a measure for the length effect of the specified failure mechanism within the trajectory. As the length-effect is small for the failure mechanism overtopping/overflow within a trajectory, it is assumed that the norm for a cross section is equal to the norm for the dike section in which the cross section is taken. This is a safe choice in case $N << n_{dike\ sections \in trajectory}$, because the height characteristics for the load and strength parameters do then hardly change over the length of a dike section. Furthermore from section 2.4 it followed that $P_{mechanism\ height,trajectory} = max\{P_{i1}\}$. However, each dike section should at least meet the norm as stated in equation 3.6. The norms per cross section - and thus dike section - within each trajectory for height, are given in table 3.5.

Table 3.5: Safety standards for each cross section within a trajectory for the failure mechanism 'overtopping/overflow'

Trajectory	Lower limit trajectory [year]	N [-]	Norm cross-section [1/year]	Min. return period [year]
16-1	30,000	1	8.00E-06	125,000
16-2	10,000	2	1.20E-05	83,333.3
16-3	10,000	1	2.40E-05	41,666.7
16-4	10,000	1	2.40E-05	41,666.7

3.5.2. STANDARD SPECIFICATIONS FOR PIPING

In a similar way with the standard specification for height in 3.5.1, a new norm is in development for the piping mechanism. The formula for the norm of a cross section reads:

$$P_{norm,crosssection} = \frac{P_{max} \cdot \omega}{N_{piping}} \tag{3.7}$$

$P_{norm,crossection}$	= The norm (maximum allowable failure probability) for a dike section [-/year]
P_{max}	= The max allowable failure probability for a trajectory [-/year]
N_{piping}	= The factor for length effect for piping, as given in equation 2.24
ω	= The partial factor that is allowed for this failure mechanism (0.24 in case of piping in the Rhine-Meuse estuary) [-]

In contrast with overtopping and overflow, the translation of the norm from a cross section to a dike section cannot be made 1:1 as this would lead to an over-conservative norm because of the high values for N. High values for N give an indication that there is a high variety in the strength and load characteristics over the length of a trajectory and influence the norm for a *cross* section. In order to translate this norm to a *dike* section again, the following step is made:

$$P_{norm,dikesection} = P_{norm,crosssection} \cdot L_{section} \cdot \frac{b}{a} \tag{3.8}$$

where (in line with equation 2.24):

a = Part of length of trajectory that is sensitive to the respective failure mechanism (0.4 for trajectories 16-1, 16-2, 16-3 and 16-4) [-]
b = Length of independent, equivalent sections for the respective failure mechanism (300 [m])
$L_{section}$ = Length of the dike section for which the norm is valid [m]

Table 3.6 shows the proposed norms per dike section that have to be met in 2050. In chapter 5, the comparison of the results per dike section with the norm will be given.

Table 3.6: Piping safety standards for dike sections

Section	Trajectory	Length tr. [km]	Lower limit [yr]	Norm cross sect. [1/yr]	Length section [m]	Norm dike sect. [1/yr]
16001001	16-1	15.1	30,000	7.84E-06	2,330	2.44E-05
16001003	16-1	15.1	30,000	7.84E-06	2,577	2.69E-05
16002002	16-2	31	10,000	2.30E-05	1,200	3.69E-05
16002009	16-2	31	10,000	2.30E-05	1,203	3.70E-05
16002013	16-2	31	10,000	2.30E-05	874	2.69E-05
16002018	16-1	15.1	30,000	7.84E-06	1,732	1.81E-05
16003002	16-4	19.6	10,000	2.34E-05	1,623	5.06E-05
16003003	16-4	19.6	10,000	2.34E-05	1,429	4.46E-05
16003005	16-4	19.6	10,000	2.34E-05	1,481	4.62E-05
16003006	16-4	19.6	10,000	2.34E-05	1,273	3.97E-05
16003007	16-4	19.6	10,000	2.34E-05	1,786	5.57E-05
16003008	16-4	19.6	10,000	2.34E-05	2,042	6.37E-05
16003009	16-4	19.6	10,000	2.34E-05	1,539	4.80E-05
16003011	16-4	19.6	10,000	2.34E-05	1,590	4.96E-05
16003012	16-4	19.6	10,000	2.34E-05	1,200	3.74E-05
16003014	16-4	19.6	10,000	2.34E-05	2,421	7.55E-05
16003015	16-3	19.9	10,000	2.34E-05	1,961	6.11E-05
16003016	16-3	19.9	10,000	2.34E-05	2,052	6.40E-05
16003018	16-3	19.9	10,000	2.34E-05	2,318	7.23E-05
16003024	16-3	19.9	10,000	2.34E-05	1,592	4.96E-05
16003026	16-3	19.9	10,000	2.34E-05	1,527	4.76E-05
16003031	16-2	31	10,000	2.30E-05	929	2.85E-05
16003034	16-2	31	10,000	2.30E-05	1,140	3.50E-05

4

Results: Influence on height

In this chapter the influence of the changes in hydraulic load levels on the necessary height on dikes will be described. It will become clear how the hydraulic load levels given the safety standards per dike section will develop and next which dike sections need to be heightened according the two strategies in 2015 and in 2100.

4.1. Results for overtopping

In this section, the results for overtopping are given. First, the change in hydraulic load level with respect to the *calculated* reference scenario 'DP2015' in 2015, are given in 4.1.1. This is done for the 'DP2015' scenario in 2100 and the alternative strategy 'Sluices' in 2015 and 2100. The figures will give a clear view in the effect of the measures and climate changes, but do not say anything about the fact whether they do or do not meet the required standards. Therefore in 4.1.2 the difference in hydraulic load level for the norms per dike section are compared with the actual height of the dike.

4.1.1. Change in hydraulic load levels compared to reference situation

The results are based on the computations made as described in section 3.3.2, where the used dike profiles are in agreement with the profiles used in VNK. For the 'DP2015' strategy the Deltamodel version 03 is used and for strategy 'Sluices' the Deltamodel version 02 is used and translated via equations 3.1 and 3.2. The height $[m]$ of the hydraulic load level is computed for each dike section at the prevailing norm following from table 3.5. This leads for each dike section to a computation of hydraulic load level for both the strategies 'DP2015' (ref) and 'Sluices'(H2).

A verification of the hydraulic load levels for the strategy 'Sluices' led to a slight underestimation of the failure probabilities for 7 dike sections. The HBN's for this strategy were determined via the translation made in equation 3.1. Adjustments for this underestimation have been made and is already accounted for in the overviews that are shown next.

In the next figures the change in hydraulic load level (HBN $[m]$ calculated at the required norm) is given per dike section. The influence on measures of the alternative strategy has the most effect at the downstream side of dike ring 16. As the effects of the alternative strategy 'Sluices' are the largest downstream nearby the sluices, one expects to recognize this pattern also for the failure probabilities of dike sections due height. Figures 4.1, 4.2 and 4.3 show the relative change in hydraulic load level for different scenarios and strategies with respect to the calculated HBN's for the reference strategy 'DP2015' in 2015.

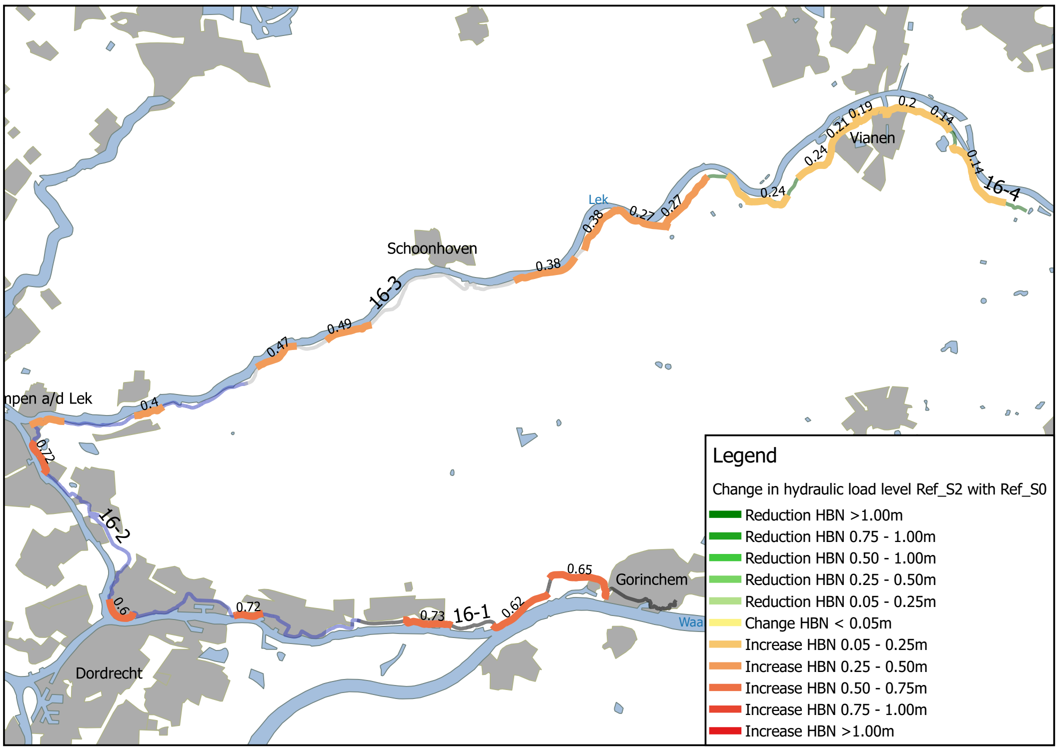


Figure 4.1: Change in HBN for dike sections within dike ring 16, ‘DP2015’ 2100 compared with ‘DP2015’ 2015

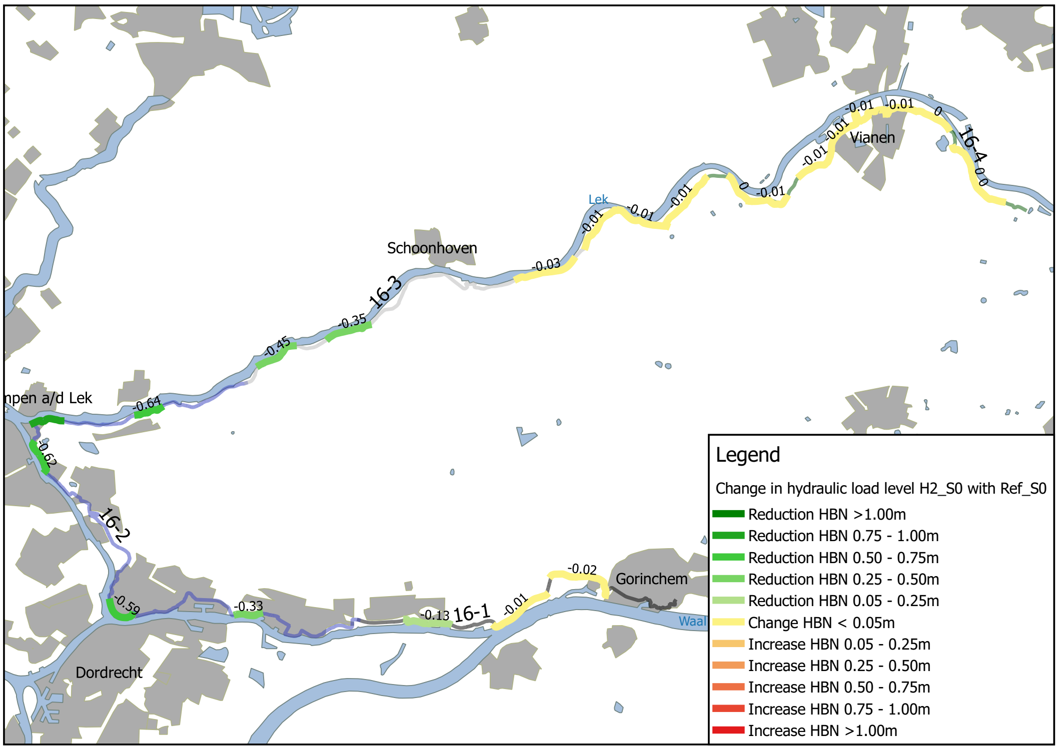


Figure 4.2: Change in HBN for dike sections within dike ring 16, ‘Sluices’ 2015 compared with ‘DP2015’ 2015

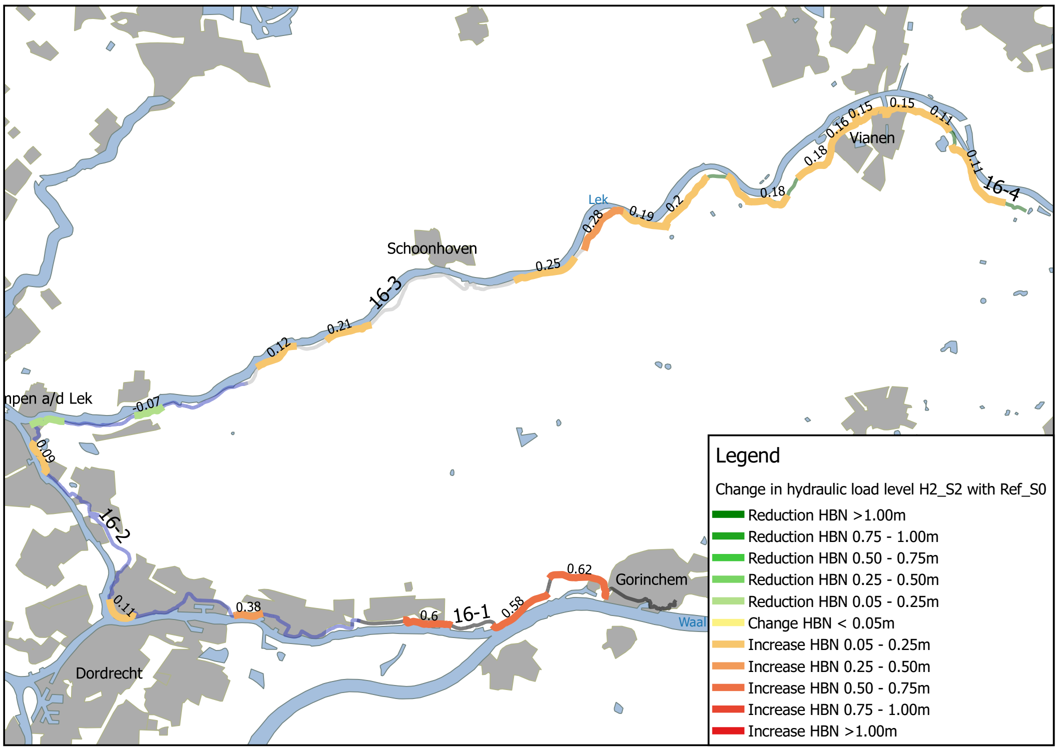


Figure 4.3: Change in HBN for dike sections within dike ring 16, 'Sluices' 2100 compared with 'DP2015' 2015

From the figures one clearly recognize the described pattern; nearby Dordrecht and Krimpen aan de Lek the reduction in hydraulic load levels for 'Sluices' 2015 are the largest, whereas in Gorinchem there is only a small benefit. In the 'Sluices' 2100 plot, this benefit nearby Gorinchem is changed in a loss and only positive effects nearby Krimpen aan de Lek are still noticeable compared with the current hydraulic load levels. The observation that the effects on the Lek (the stretch Krimpen - Schoonhoven - Vianen) turn out slightly more positive than at the Waal (Dordrecht -Gorinchem) is not only the result of the 'Sluices' strategy, but also follows from the policy decision to set the maximum discharge at the Lek at 3.380m^3/s (see part about 'The Lek ontzien' in subsection 3.3.2).

4.1.2. CALCULATED HYDRAULIC LOAD LEVELS COMPARED TO SAFETY STANDARDS

In the next figures, the results of hydraulic load levels (the occuring HBN [m] calculated given the return periods from the safety standards of table 3.5) are offset against the height of the dike (see also figure 3.5 where Δh - the 'remaining height' is clarified):

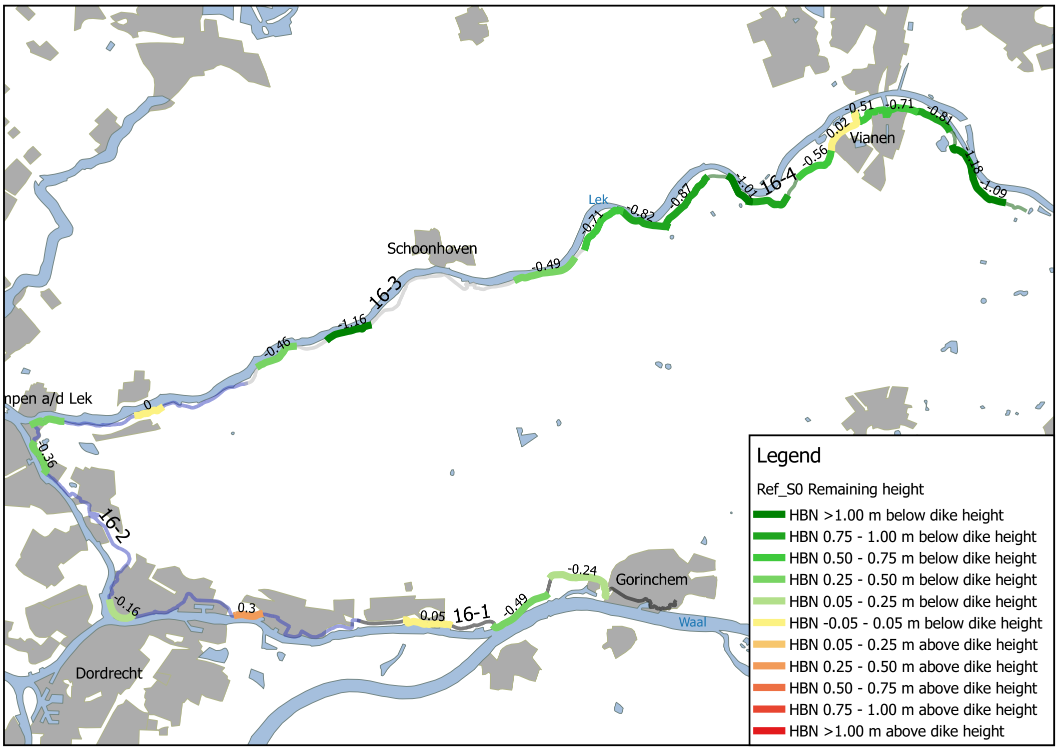


Figure 4.4: Difference between HBN for new norms and h_{dike} in DR16, 'DP2015' 2015

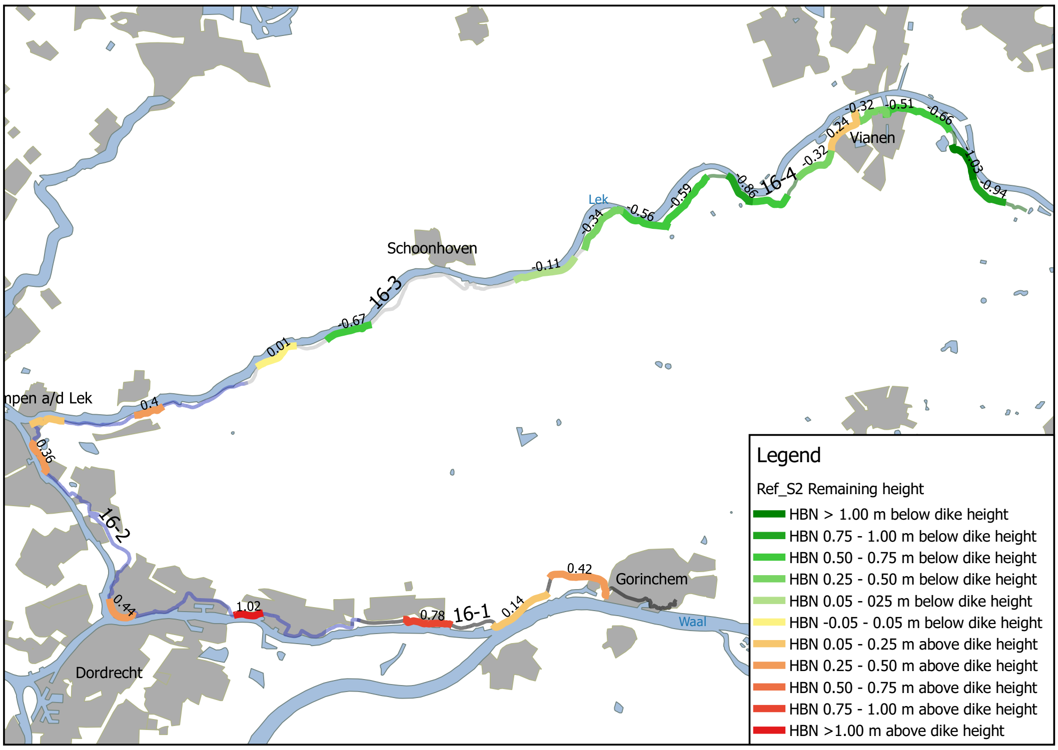


Figure 4.5: Difference between HBN for new norms and h_{dike} in DR16, 'DP2015' 2100

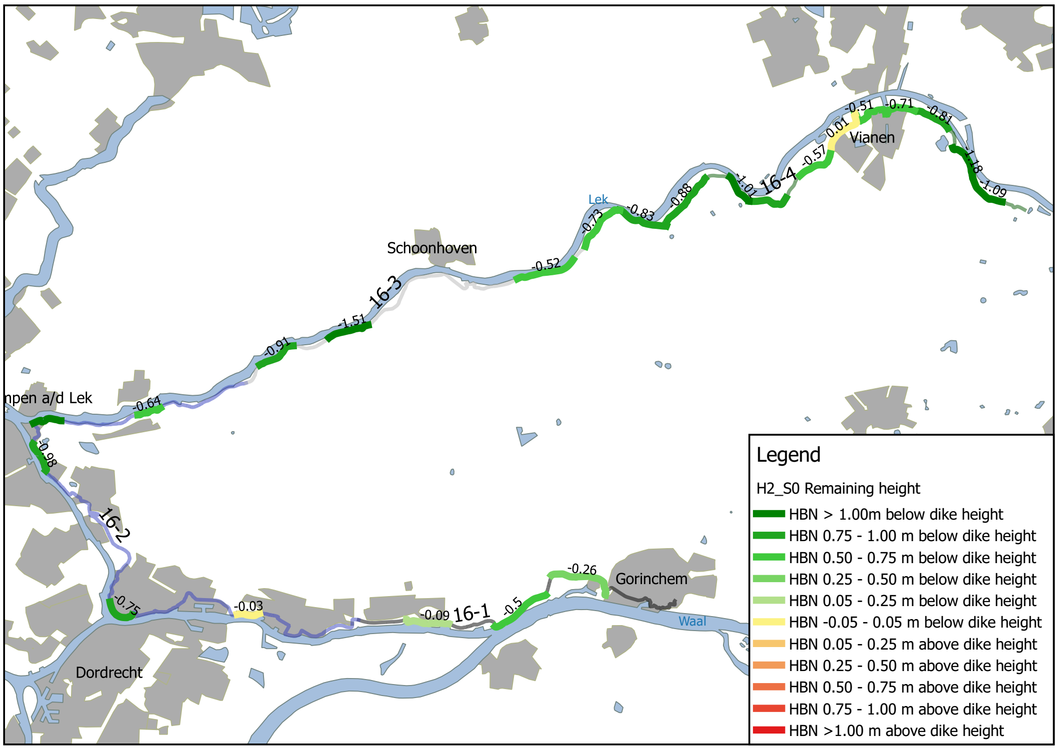


Figure 4.6: Difference between HBN for new norms and h_{dike} in DR16, 'Sluices' 2015

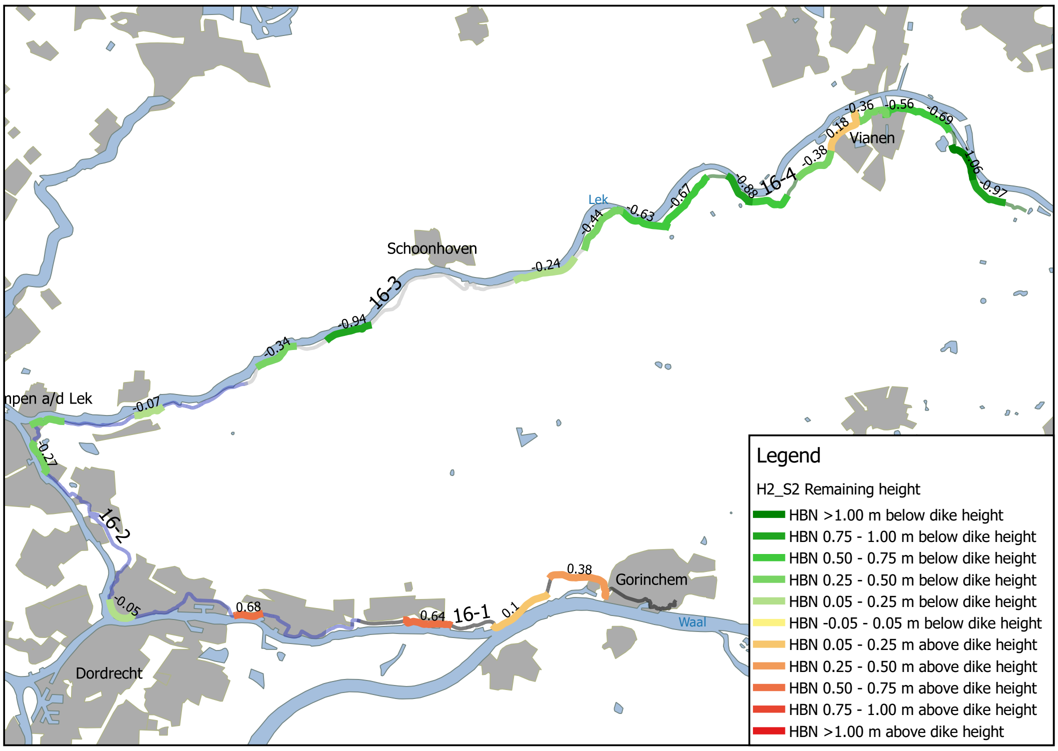


Figure 4.7: Difference between HBN for new norms and h_{dike} in DR16, 'Sluices' 2100

Analysis of figures

One could recognize the following patterns for the above figures:

- In figure 4.4, only one dike section does not meet the requirements for height. The dike height for this section is 0.30m too short. This is the situation according to the 'DP2015' strategy in reference year 2015.
- In figure 4.5, all dike sections from Gorinchem up to Krimpen aan de Lek and some dike sections at the Lek do no longer meet the safety standards for height in case no intermediate reinforcements are applied. Nearby Vianen, the dikes are still high enough, except for one dike section. Dike trajectories 16-1 and 16-2 need to be reinforced for height completely. This is the situation according to the 'DP2015' strategy in reference year 2100.
- In figure 4.6, all dike sections meet the required height. This is the situation according to the 'Sluices' strategy in reference year 2015, as if the sluices are already present.
- In figure 4.7, 4 dike sections need to be reinforced along the river Waal and one along the Lek nearby Vianen. This is the situation according to the 'Sluices' strategy in reference year 2100.Dike trajectory 16-1 need to be reinforced for height completely and 16-2 partly.

Based on above findings, one could argue that strategy sluices is effective in reduction the failure probability for the entire stretch along the Lek (in combination with the limitations of the maximum discharge at $3,380 m^3/s$) and along the Waal (or the river is called the 'Noord' here) up to Dordrecht. Note that only climate change and the effects of the measures is taken into account and that local subsidence is not discussed.

4.2. Verification: Comparison results with VNK outcomes

Figure 4.4 gives the indication that the dikes along dike ring 16 are generally high enough according to the new norms that will take effect in 2050. However, from VNK outcomes, higher failure probabilities are known for height. These differences between VNK results and results as shown above can largely be clarified due to the use of the Deltamodel version 02 within VNK and Deltamodel version 03 in this study. In below figures, the results between the VNK study and this study are compared with each other. The three compared results are:

1. Calculated return periods ($1/P_f$) of Veiligheid Nederland in Kaart (Vergouwe & Van den Berg, 2013) in PC-Ring.
2. Calculated return periods with hydraulic load levels from Deltamodel version 02 in Hydra-B and VNK-dike profiles from PC-Ring database.
3. Calculated return periods with hydraulic load levels from Deltamodel version 03 in Hydra-B and VNK-dike profiles from PC-Ring database.

Figure 4.8: Location of dike sections for the stretch Krimpen aan de Lek - Vianen

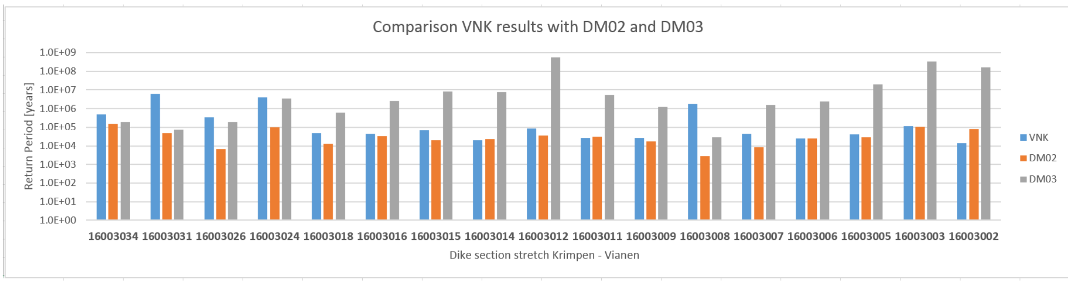


Figure 4.9: Differences in return periods ($1/P_f$) for results of VNK with results in this study, VNK compared with 'DP2015' in 2015 for DM02 and DM03

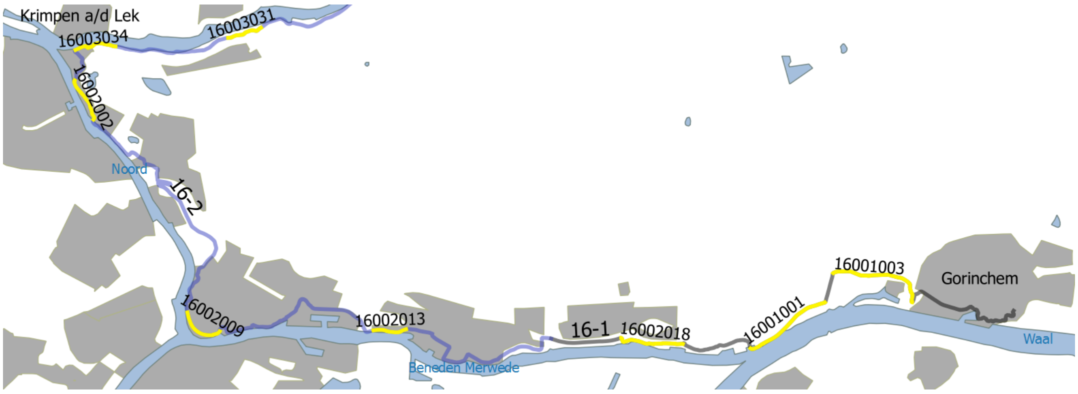


Figure 4.10: Location of dike sections for the stretch Krimpen aan de Lek - Gorinchem

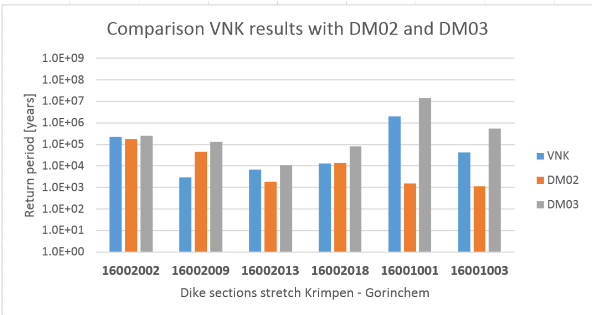


Figure 4.11: Differences in return periods ($1/P_f$) for results of VNK with results in this study, VNK compared with 'DP2015' in 2015 for DM02 and DM03

Above figures give reasonably well comparisons between the VNK results and DM02. Still there are some dike sections with large deviations like section 16001001 in figure 4.11 and section 16003008 in figure 4.9. These differences cannot be directly explained and could be a result for instance in the use of the software program that is used to calculate the results. As earlier mentioned, the results in VNK were computed within PC-Ring, while the results of this study are obtained with Hydra-B.

While the results of VNK are better comparable with the results of this study for the Deltamodel version 02, it is still chosen to compute results with version 03. The reason to do so is that within the assumptions of 02 lies that in future scenarios the maximum discharge at the Lek will be set on 3,380. In computing so, a safer solution will be found in 2100 for the Lek within the strategy of 'DP2015' with respect to 2015. This makes no sense in the evaluation of 'Sluices' with respect to 'DP2015' and would blur the results.

4.3. Conclusions & recommendations for height

4.3.1. Conclusions

Based on above findings, with respect for the failure mechanism overtopping which is related to the height of a dike, the following conclusions are stated:

Strategy 'DP2015', Ten dike sections need to be heightened before 2100

When the hydraulic loads of strategy strategy 'DP2015' in *2100* are compared to the safety standards, it can be seen that along the stretch Schoonhoven - Vianen, only one dike section needs to be heightened. Along the other stretches within dike ring 16, almost all other dike sections need to be heightened before 2100 in order to cope the climate changes. In total 10 of the 23 assessed dike sections need to be heightened in this situation up to 2100 (bottom subsidence is not taken into account).

Strategy 'Sluices': Five dike sections need to be heightened before 2100

The alternative strategy 'Sluices' is in *2015* effective in hydraulic load reduction for entire trajectory 16-2 and partly effective for trajectories 16-1 and 16-3. The effects of reduction in hydraulic load level reach for the Lek up to Schoonhoven and for the Waal (Beneden Merwede) up to the bifurcation nearby Gorinchem. More upstream of Schoonhoven (trajectory 16-4) and Gorinchem (trajectory 16-1), the alternative strategy does not have any effect in reducing the failure probability any longer for overtopping/overflow. These locations are too far away situated from the pumping stations to have any effect.

The alternative strategy 'Sluices' is in *2100* still effective in lowering hydraulic load levels with respect to the 'DP2015' 2015 situation for a small part at the Lek between Krimpen aan de Lek and Schoonhoven. The increase in Hydraulic load level is limited to an order of 0.10m for the entire stretch Krimpen aan de Lek - Schoonhoven and Krimpen aan de Lek - Dordrecht.

When the hydraulic loads of strategy strategy 'Sluices' in *2100* are compared to the safety standards, it can be seen that along the stretch Schoonhoven - Vianen only one dike section needs to be heightened. In the stretch Krimpen aan de Lek - Schoonhoven, no dikes need to be heightened, nor in the stretch Krimpen aan de Lek - Dordrecht. Dikes eastwards of Dordrecht need to be heightened before 2100. In total 5 of the 23 assessed dike sections need to be heightened in this situation before 2100 (bottom subsidence is not taken into account).

Effects due to 'De Lek ontzien'

The implicit taken assumption within both strategies 'De Lek ontzien' is effective in reducing the hydraulic load levels along the stretch Schoonhoven - Lek. In this measure, the maximum discharge over the Lek is limited to $3,380 m^3/s$ and a failure of levees upstream is assumed for extreme river discharges (with a probability of occurence less than 1/1,250 per year) as described in subsection 3.3.2.

4.3.2. Recommendations

In order to measure the effects of the alternative strategy compared to the 'DP2015' strategy, local bottom subsidence is excluded from the calculations so far. In case the velocity of bottom subsidence is high in this area, it should be taken into account in the calculations and it should be investigated whether strategy 'Sluices' is still effective. In the worst case, the sluices are just realized, but because of bottom subsidence the dikes still need to be reinforced leading to double costs. In chapter 6, a cost calculation is made where the year of reinforcement per trajectory is estimated, taking subsidence into account. The following aspects are recommended for further study and not threatened in this report:

- For the comparison of hydraulic load level (HBN) with the safety standards that count for the height of a dike, the lower limits from table 2.6 are taken. In reality it takes time between the moment that a dike is rejected for a certain failure mechanism and is reinforced as a consequence again. In further study this so-called ordering time or in Dutch 'besteltijd' should be taken into account, which is clarified in figure 2.5.
- Last, it is for a dike trajectory important that the safety standard of the entire trajectory is met. In equation 3.6 the partial formula was given for height within the dike trajectories 16-1 to 16-4. Authorities may deviate from this partial norm when they compensate it with stricter partial factors for other failure mechanisms. It is advised to evaluate the dike sections along the entire norm, which was also done in Vergouwe (2014).

5

Results: Influence on piping

In this chapter the influence of a reduction in normative water levels (MHW) in the different strategies on the failure probabilities due to the piping mechanism is described. In section 5.1 it is made clear how failure probabilities relatively change according the chosen year of reference and strategy. Besides, the calculated failure probabilities are compared to the new safety standards for piping (as stated in subsection 3.5.2). When the safety standards are not met, the shortage on berm width is calculated. In section 5.2 the results are verified with different comparisons. First the results are compared with the old application of the Sellmeijer formula and next they are compared to VNK outcomes. Furthermore the results are compared to the physical behaviour on piping and the effects for piping are offset against the effects on height. In 5.3, conclusions are drawn and recommendations are made.

5.1. Results for piping

Via the methodology as described in section 3.4 and appendix B an analysis is performed on the effects of the flood reduction measures by application of alternative 'Sluices' compared to strategy 'DP2015'. At first, relative changes in failure probability for dike ring 16 will be shown and analysed. Second, the results are compared to the safety standards and the shortage on berm width is derived.

5.1.1. Change in failure probability compared to calculated reference situation

In the following figures 5.1, 5.2, 5.3 the relative change in failure probability is shown for different scenarios and strategies with respect to the calculated failure probabilities for the reference strategy 'DP2015' in 2015. For factor > 1.0 there is an increase in failure probability, whereas a factor < 1.0 leads to a reduction:

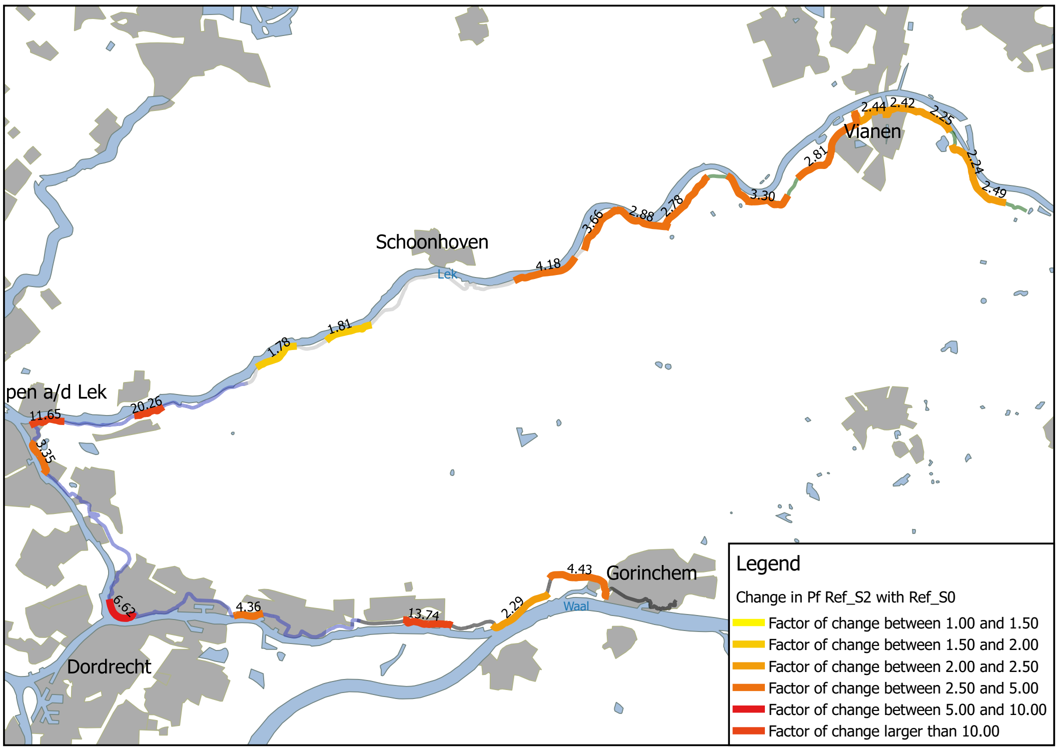


Figure 5.1: Relative change in failure probabilities of dike sections within dike ring 16, 'DP2015' 2100 compared with 'DP2015' 2015

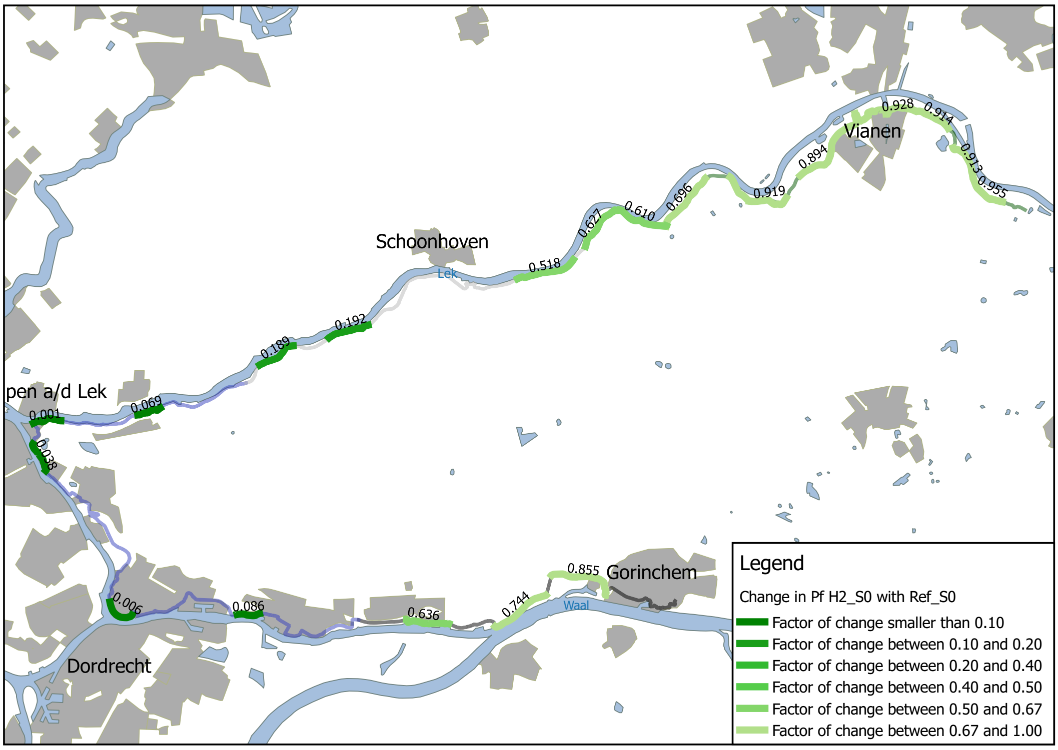


Figure 5.2: Relative change in failure probabilities of dike sections within dike ring 16, 'Sluices' 2015 compared with 'DP2015' 2015

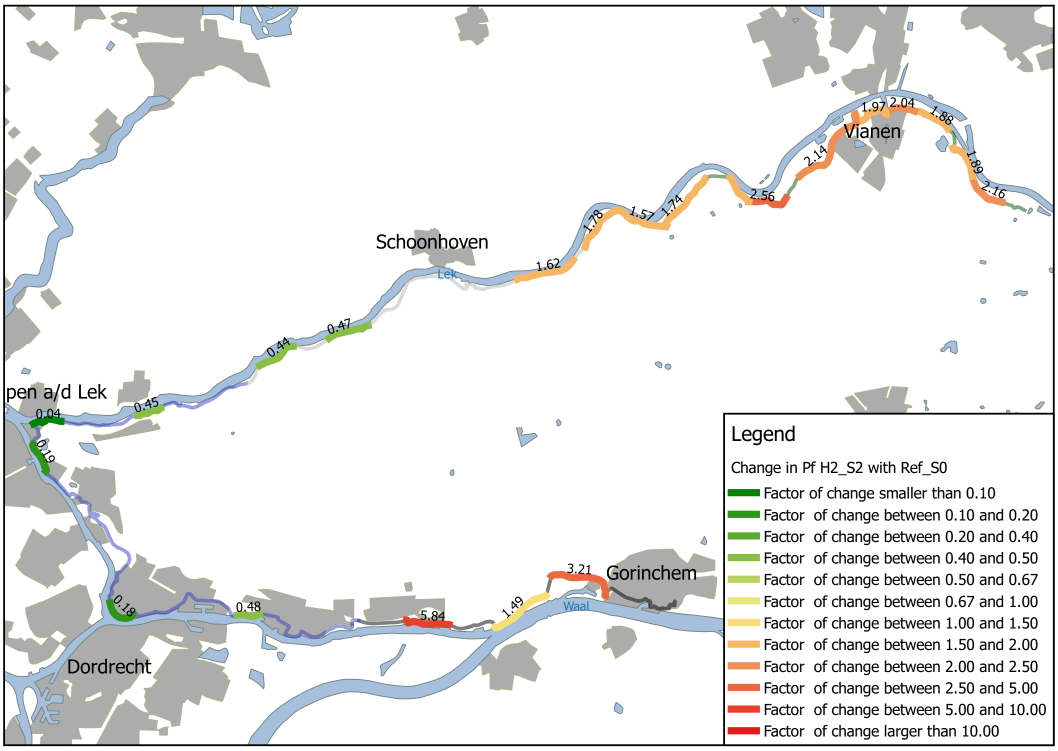


Figure 5.3: Relative change in failure probabilities of dike sections within dike ring 16, 'Sluices' 2100 compared with 'DP2015' 2015

ANALYSIS ON CHANGES IN FAILURE PROBABILITIES

The change in failure probabilities is the strongest at the downstream side of dike ring 16. This is in line with the pattern also recognized for height in chapter 4. The differences between 'Sluices' and 'DP2015' in 2100 are in the order of a factor 5 nearby Schoonhoven (an increase of factor 1.81 for 'DP2015' in 2100 nearby Schoonhoven in figure 5.1 divided by a reduction of factor 0.47 for 'Sluices' in 2100 in figure 5.3) up to 250 (the factor of 11.65 nearby Krimpen aan de Lek in figure 5.1 divided by 0.04 in 5.3). However, the factor changes in order magnitude of the failure probabilities are smaller than for height. This can be explained by the fact that the reduction in flood probability is not only a result of a reduction of extreme water levels, which is the case for height, but also of more moderate water levels.

The alternative strategy 'Sluices' leads to a significant reduction in failure probability in case this strategy is implemented in 2015. With expected sea level rise and higher extreme river discharges in 2100, the effect on the failure probabilities of dike sections is less clear; only seven sections have a lower failure probability within 'Sluices' 2100 compared to 'DP2015' in 2015. It is also noticeable that 'Sluices' has much more effect regarding change in failure probability on some sections than on others, the explanation for this can be found in the characteristics of the fragility curves combined with the effect of the probability density functions.

The five sections that would benefit most in terms of reduction in failure probability within 'Sluices' are sections 16002002, 16002009, 16002013, 16003031 and 16003034. Figures 5.4 and 5.5 show for section 16002009 that the reduction in failure probability is a combination of a shifted probability density function (to left) with a smaller probability of failure in this domain. The effect of more moderate water levels on the failure probability is also clearly shown (the highest values for probability density are found at water levels between 2 and 3.5m +NAP, more than 1m below the dike height):

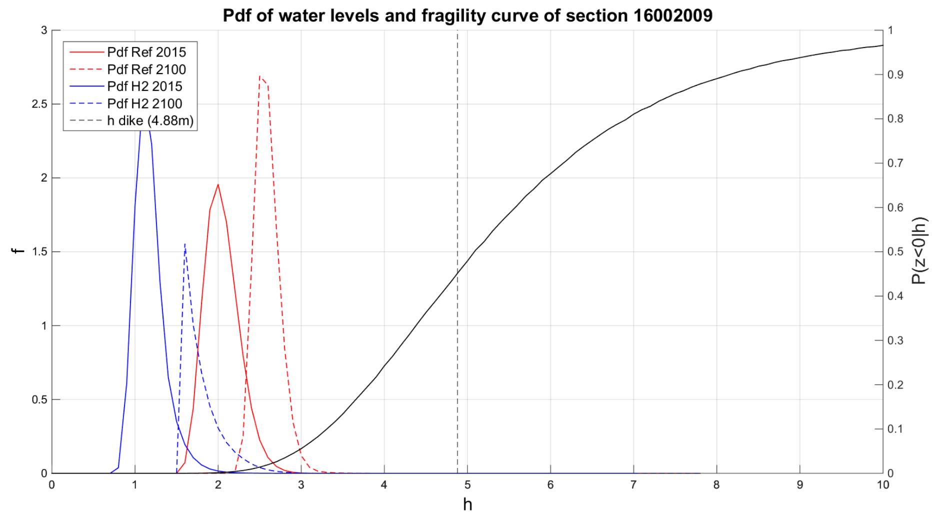


Figure 5.4: Both pdf and fragility curve for section 16002009, for the two strategies in 2015 and 2100

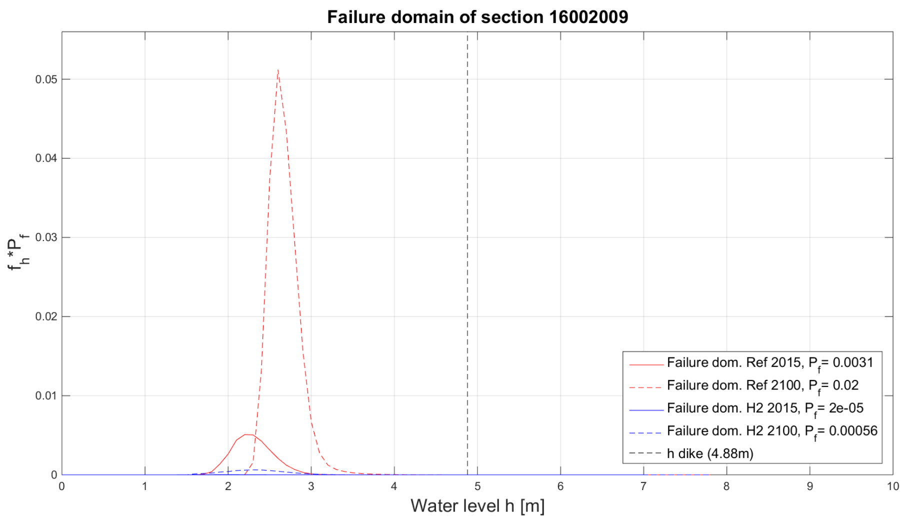


Figure 5.5: Both pdf and fragility curve for section 16002009, for the two strategies in 2015 and 2100

5.1.2. CALCULATED FAILURE PROBABILITIES COMPARED TO NEW STANDARD SPECIFICATIONS

In figure 5.6, the comparison of the results per dike section with the safety standards are given. It shows that for almost all dike sections the required standards as stated in section 3.5.2 are not met. This also holds for the reference situation of VNK and is in line with the conclusions from Vergouwe and Van den Berg (2013), where the total risk of failure due to piping exceeded >1/100 per year for dike ring 16 (see table 2.8). The sections that do meet the required standards, are largely located in trajectory 16-2.

The calculated failure probabilities for 'DP2015' in 2015 show large deviations for some dike sections with respect to the VNK outcomes. Most calculated sections differ with a factor of order 10 from the VNK reference situation, while some sections show exceptional high or low failure probabilities compared with the VNK results, see for instance dike section 16002018 and 16003024. In 5.2.2 a step by step refinery is made in order to compare the outcomes with the VNK results, but first the shortage on berm width for the two strategies is elaborated in subsection 5.1.3.

Section	Norm dike sect. [1/yr]	Pf VNK		Pf Ref 2015		Pf Ref 2100		Pf Sluices 2015		Pf Sluices 2100	
16001001	2.44E-05	3.15E-04	No	8.28E-03	No	1.90E-02	No	6.15E-03	No	1.23E-02	No
16001003	2.69E-05	2.10E-04	No	3.39E-05	No	1.51E-04	No	2.90E-05	No	1.09E-04	No
16002002	3.69E-05	5.30E-05	No	5.97E-05	No	2.00E-04	No	2.29E-06	Yes	1.14E-05	Yes
16002009	3.70E-05	1.15E-04	No	3.09E-03	No	2.04E-02	No	2.00E-05	Yes	5.64E-04	No
16002013	2.69E-05	2.68E-04	No	6.88E-04	No	3.00E-03	No	5.93E-05	No	3.33E-04	No
16002018	1.81E-05	1.52E-05	Yes	1.68E-08	Yes	2.30E-07	Yes	1.07E-08	Yes	9.78E-08	Yes
16003002	5.06E-05	1.15E-03	No	3.27E-05	Yes	8.13E-05	No	3.12E-05	Yes	7.07E-05	No
16003003	4.46E-05	6.45E-04	No	3.37E-04	No	7.55E-04	No	3.07E-04	No	6.36E-04	No
16003005	4.62E-05	2.61E-03	No	4.41E-03	No	9.91E-03	No	4.03E-03	No	8.28E-03	No
16003006	3.97E-05	2.05E-03	No	2.83E-03	No	6.85E-03	No	2.63E-03	No	5.79E-03	No
16003007	5.57E-05	1.57E-03	No	7.54E-03	No	1.84E-02	No	6.81E-03	No	1.49E-02	No
16003008	6.37E-05	1.69E-03	No	6.37E-04	No	1.70E-03	No	5.59E-04	No	1.29E-03	No
16003009	4.80E-05	3.26E-04	No	4.89E-04	No	1.37E-03	No	4.38E-04	No	1.05E-03	No
16003011	4.96E-05	1.81E-04	No	2.13E-03	No	7.03E-03	No	1.95E-03	No	5.44E-03	No
16003012	3.74E-05	1.60E-03	No	1.36E-02	No	3.44E-02	No	9.95E-03	No	2.18E-02	No
16003014	7.55E-05	3.56E-04	No	1.77E-03	No	4.90E-03	No	1.23E-03	No	3.08E-03	No
16003015	6.11E-05	2.97E-04	No	5.73E-03	No	1.65E-02	No	3.50E-03	No	8.99E-03	No
16003016	6.40E-05	5.56E-04	No	1.37E-03	No	5.02E-03	No	8.61E-04	No	2.44E-03	No
16003018	7.23E-05	5.57E-04	No	5.66E-04	No	2.36E-03	No	2.93E-04	No	9.15E-04	No
16003024	4.96E-05	5.61E-04	No	1.03E-01	No	1.87E-01	No	1.98E-02	No	4.82E-02	No
16003026	4.76E-05	3.14E-04	No	4.97E-02	No	8.87E-02	No	9.41E-03	No	2.17E-02	No
16003031	2.85E-05	6.76E-08	Yes	9.89E-06	Yes	2.00E-04	No	6.79E-07	Yes	4.48E-06	Yes
16003034	3.50E-05	5.89E-06	Yes	7.43E-06	Yes	8.65E-05	No	6.15E-09	Yes	2.95E-07	Yes

Figure 5.6: Comparison of calculated failure probabilities with safety standards

5.1.3. CALCULATED SHORTAGE OF BERM WIDTH

Based on the findings of figure 5.6 it is interesting to find out the shortage on berm width for the sections that do not comply to the standards. To do so failure probabilities are calculated, taking into account extra berm width (ΔL). Five extra calculations are made with steps of 25, 50, 75, 100 and 150m additional berm width. Next, via interpolation of the results the shortage on berm width is calculated. See figure 5.7 where the applied methodology is visualized for dike section 16002013, the shortage on berm width is shown for the two strategies in 2015 and 2100. This methodology is used for the 23 assessed sections of dike ring 16. The shortage in berm width is further visualized in figures 5.8, 5.9 5.10 and 5.11.

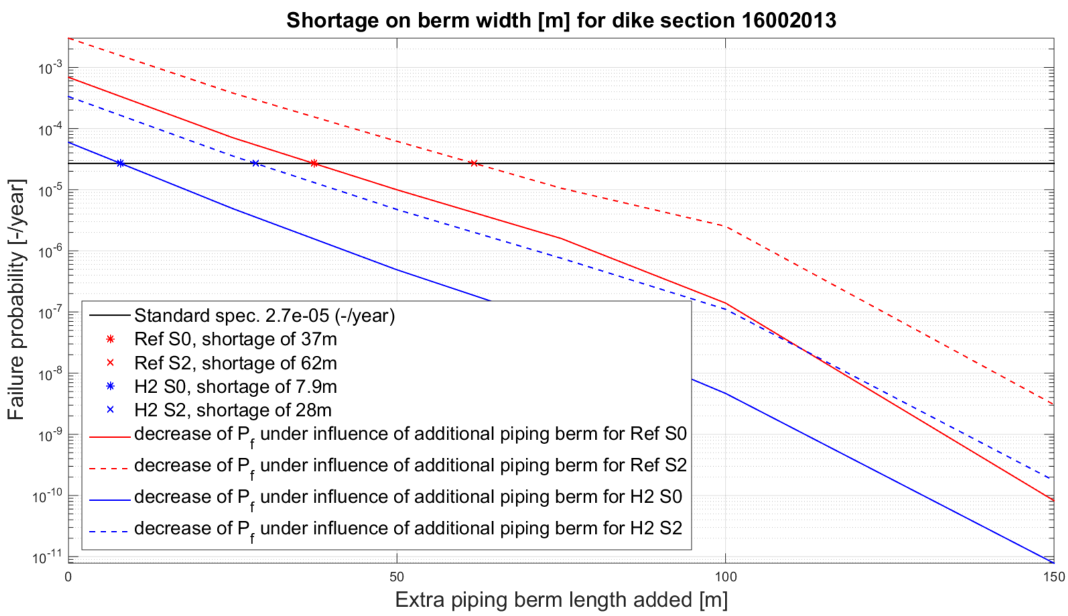


Figure 5.7: Computation of shortage on berm width via interpolation

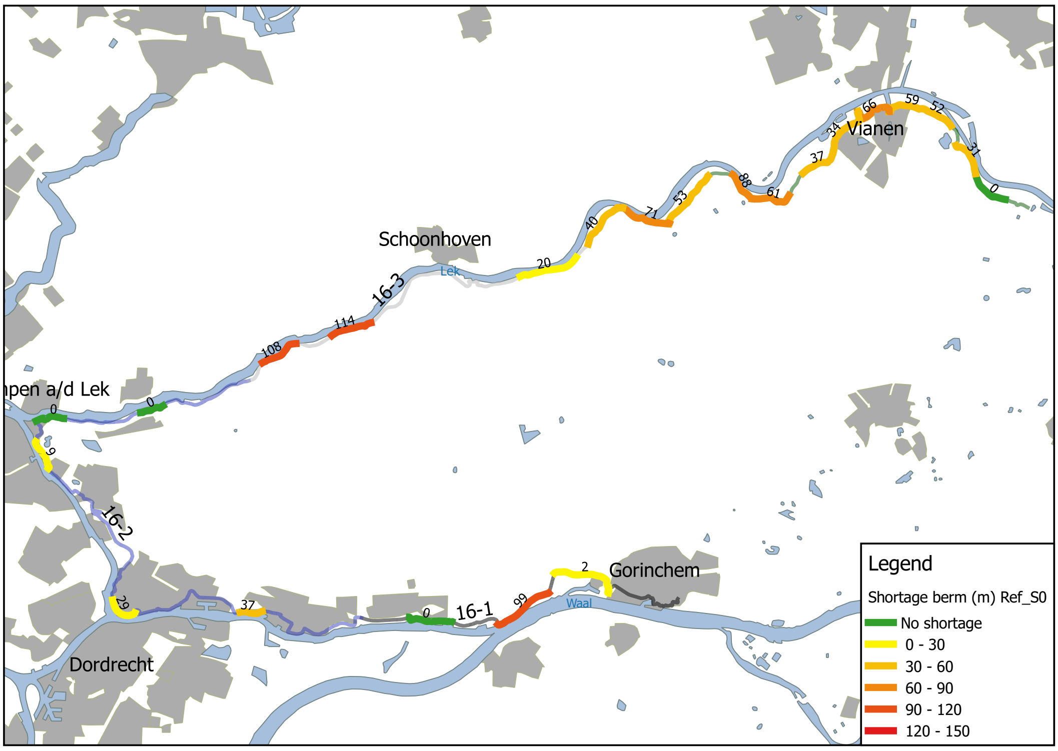


Figure 5.8: Shortage on piping berms for the assessed sections within dike ring 16, 'DP2015' 2015

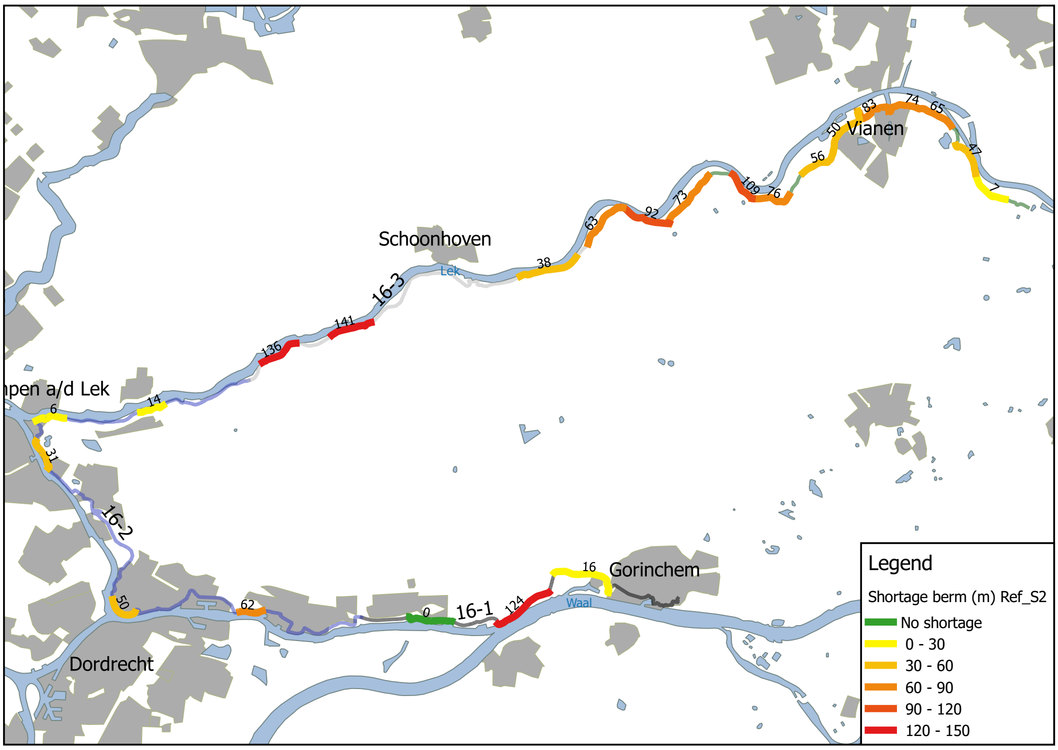


Figure 5.9: Shortage on piping berms for the assessed sections within dike ring 16, 'DP2015' 2100

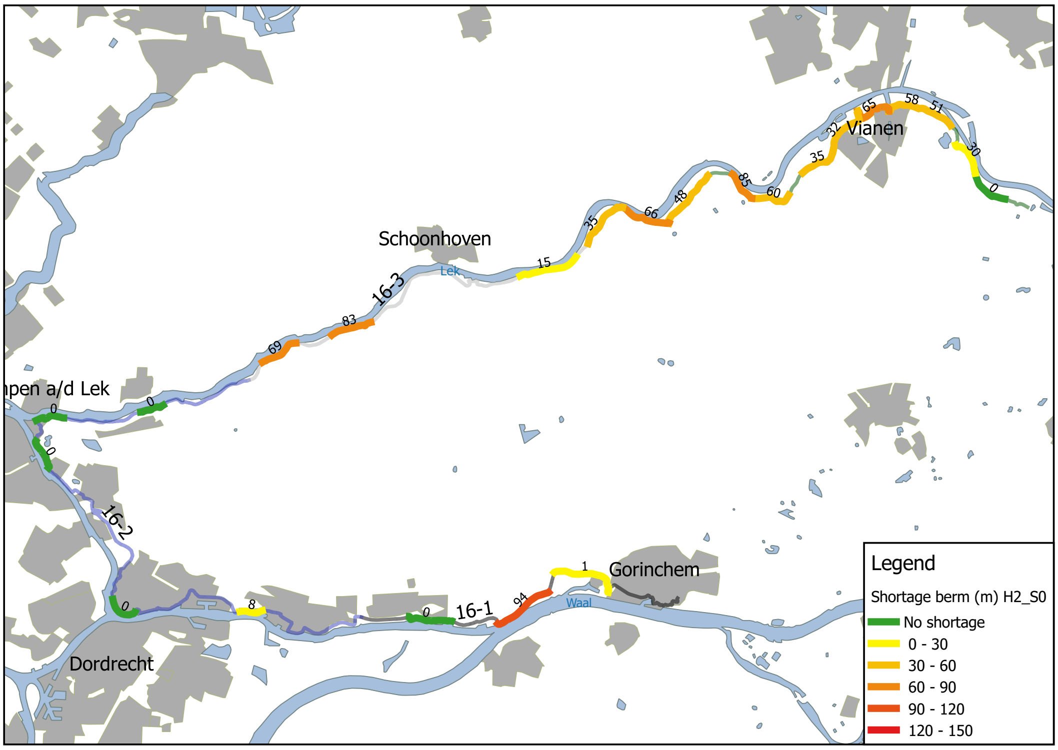


Figure 5.10: Shortage on piping berms for the assessed sections within dike ring 16, 'Sluices' 2015

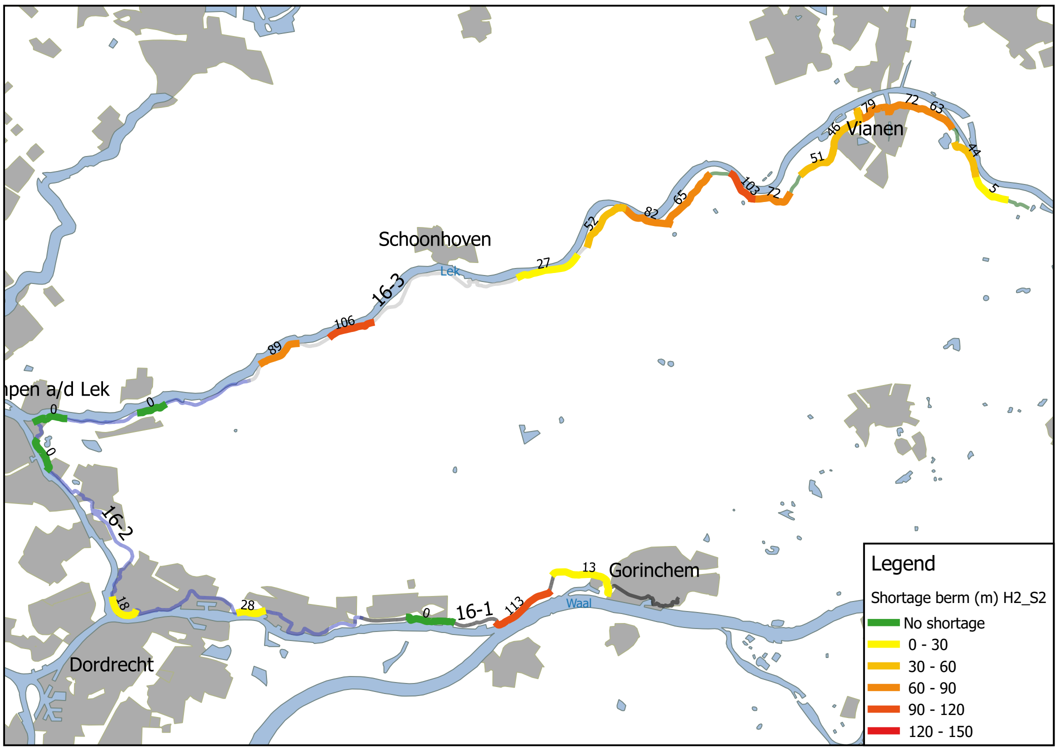


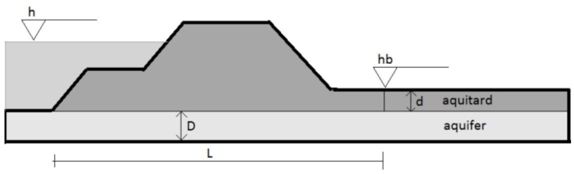
Figure 5.11: Shortage on piping berms for the assessed sections within dike ring 16, 'Sluices' 2100

5.2. VERIFICATION

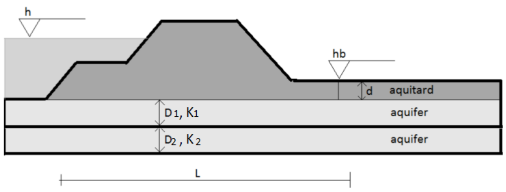
The obtained results for piping will be verified for three aspects in order to determine the validity of the outcomes. In the first place the outcomes will compared to a situation where the former Sellmeijer formula is applied, schematized with the '2-forces' model. Second, a comparison is made with VNK outcomes and differences are explained. Third, the results are compared with the physical behaviour of the piping mechanism and it is checked whether the model describes the mechanism of piping well enough for the calculation of failure mechanism in the Rhine-Meuse estuary.

5.2.1. COMPARISON RESULTS WITH FORMER SELLMEIJER FORMULA

In studies of VNK, the failure probability for piping is calculated by a combination of formulas. In case the bottom section is built up of only 1 aquifer, the former Sellmeijer formula is applied according to the '4-forces' model, on the assumption that the grain is embedded in the sand package, see also see section 2.5.4. When the bottom consists out more than one aquifer, the '2-forces' model is applied within M-Seep, on the assumption that the exit grain is lying at the surface of the sand package. See figure 5.12 for an indication of which model is used according to schematizations of the bottom sections. With the change in principles, one would expect larger failure probabilities for the '2-forces' model, which is indeed the case as seen in table 5.1, where 10 dike sections are analysed for both methods. The outcomes of this analysis will be used for the comparison with VNK results in 5.2.2. For dike section 16001001, the shape of both the fragility curves are plotted combined with the probability density functions of the water levels in figure 5.13.



(a) Calculation performed with former Sellmeijer, '4-forces' model



(b) Calculation performed with MSeep, '2-forces' model

Figure 5.12: Schematization of cross sections of a dike within PC-Ring and the applied model

Table 5.1: Differences in failure probabilities between '4-forces' model old Sellmeijer and revised Sellmeijer '2-forces' model

Dike section	P_f former Sellmeijer		Factor		P_f revised Sellmeijer
16001001	9.60E-05	·	86	=	8.28E-03
16001003	3.07E-07	·	110	=	3.39E-05
16002002	4.98E-12	·	1.20E07	=	5.97E-05
16002009	1.07E-05	·	288	=	3.09E-03
16002013	2.45E-06	·	281	=	6.88E-04
16002018	4.28E-11	·	392	=	1.68E-08
16003002	5.29E-08	·	618	=	3.27E-05
16003003	1.91E-06	·	176	=	3.37E-04
16003005	1.33E-04	·	33	=	4.41E-03
16003006	5.30E-05	·	53	=	2.83E-03
16003007	2.76E-04	·	27	=	7.54E-03
16003008	5.94E-06	·	107	=	6.37E-04

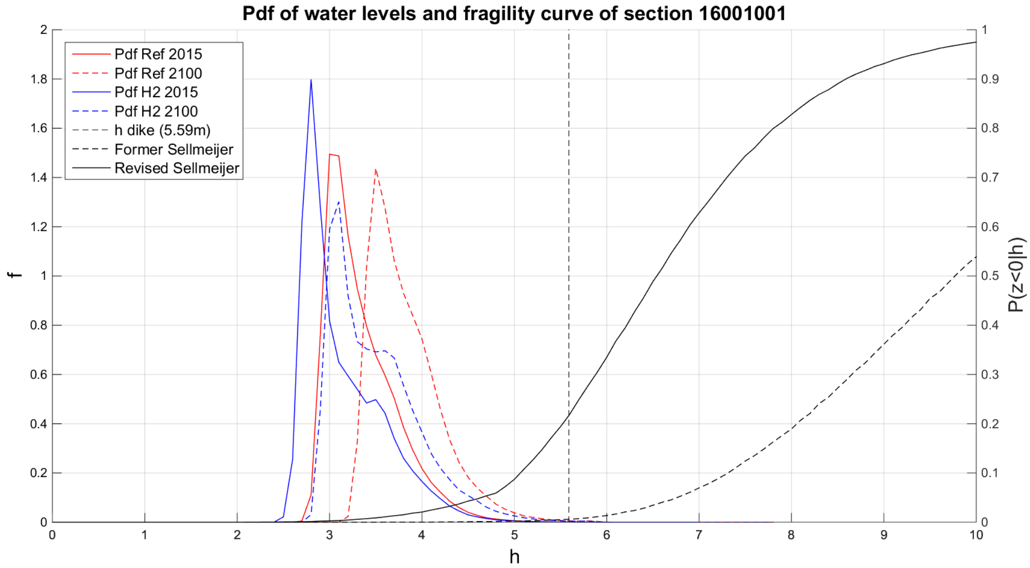


Figure 5.13: Comparison between fragility curves for the revised and former Sellmeijer formulae

5.2.2. COMPARISON RESULTS WITH VNK OUTCOMES

The failure probabilities P_f for VNK are calculated by a combination of the '4-forces' and '2-forces' model via the former Sellmeijer formula. It is therefore hard to make an accurate comparison between the results in this study and the results of VNK. Via the steps described in figure 5.14, the VNK computation is simplified to a model where it is schematized by only the '4-forces' model for the upper aquifer. The same is done for the computations in this study and instead of the revised Sellmeijer method with '2-forces', the former Sellmeijer formula is used (given in equation 2.18). By doing so, the results in table 5.2 are obtained. This verification is done for the 'DP2015' strategy in 2015, as the water levels are closest to the water levels used in VNK.

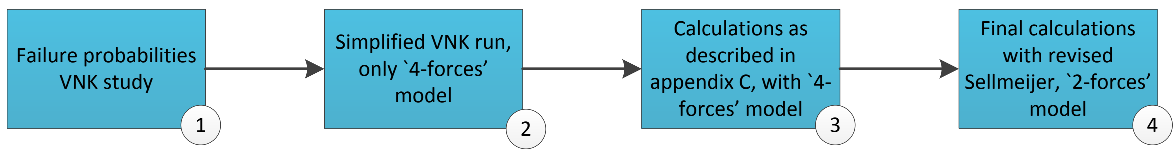


Figure 5.14: Computation steps for verification of VNK results with calculated failure probabilities

Table 5.2: Comparison of VNK results with computed results with two intermediate steps

Dike #	P_f VNK		Factor		P_f VNK former Sellm.		Factor		P_f former Sellm.		Factor		P_f revised Sellmeijer
16001001	3.15E-04	·	0.16	=	5.19E-05	·	1.85	=	9.60E-05	·	86	=	8.28E-03
16001003	2.10E-04	·	1.67E-03	=	3.51E-07	·	0.87	=	3.07E-07	·	110	=	3.39E-05
16002002	5.30E-05	·	4.90E-04	=	2.60E-08	·	1.92E-04	=	4.98E-12	·	1.20E07	=	5.97E-05
16002009	1.15E-04	·	1.48	=	1.71E-04	·	0.06	=	1.07E-05	·	288	=	3.09E-03
16002013	2.68E-04	·	0.03	=	8.94E-06	·	0.27	=	2.45E-06	·	281	=	6.88E-04
16002018	1.52E-05	·	5.08E-06	=	7.71E-11	·	0.56	=	4.28E-11	·	392	=	1.68E-08
16003002	1.15E-03	·	1.41E-03	=	1.63E-06	·	0.03	=	5.29E-08	·	618	=	3.27E-05
16003003	6.45E-04	·	0.03	=	1.84E-05	·	0.10	=	1.91E-06	·	176	=	3.37E-04
16003005	2.61E-03	·	0.82	=	2.14E-03	·	0.06	=	1.33E-04	·	33	=	4.41E-03
16003006	2.05E-03	·	0.09	=	1.91E-04	·	0.28	=	5.30E-05	·	53	=	2.83E-03
16003007	1.57E-03	·	0.38	=	5.88E-04	·	0.47	=	2.76E-04	·	27	=	7.54E-03
16003008	1.69E-03	·	0.04	=	6.67E-05	·	0.09	=	5.94E-06	·	107	=	6.37E-04

In table 5.2 some patterns are recognizable. In case the VNK computations are done only with the '4-forces' model for 1 aquifer, the failure probabilities are generally much smaller than the VNK results (differences between step 1 and 2), this is in line with expectations. An exception is formed for dike section 16002009 but the difference between the two outcomes are still small. Between step 2 and 3 there are larger differences noticeable. The differences between outcomes of step 2 and 3 are in the order of 10 for most of the outcomes, but for dike section 16002002 this does not hold, but can be explained by the fact that both failure probabilities are very small and that model uncertainties can already lead to large deviations on relative scale. The differences between step 3 and 4 are already discussed in subsection 5.2.1. The results of step 4 in table 5.2 show however to large differences with the first step to conclude that these outcomes are corresponding with the failure probabilities in reality, but the relative changes in failure probabilities as shown in figures 5.1 to 5.3 give a good insight in the effects of the strategies for the failure probabilities. Last it should be argued whether the most important parameters of the piping mechanism are correctly taken into account in the formulas, this will be discussed next.

5.2.3. COMPARISON OF OUTCOMES WITH PHYSICAL BEHAVIOUR OF PIPING

The calculated failure probabilities were shown in figure 5.6. For most dike sections, rather high failure probabilities were found for piping and that the norms would not be met, even if strategy 'Sluices' would been applied immediately. With respect to the calculated failure probabilities, the question arises whether these are not too conservative. This statement is explained by two aspects that are not yet considered in the calculations (of both VNK and this study) but could lead to lower failure probabilities.

THE USE OF SURVIVED LOAD

As described in 2.6.3, survived load could reduce failure probabilities for piping in the Rhine-Meuse estuary by a factor 2 to 20 according to Schweckendiek (2010). A discussion with an expert of HKV, Fred Havinga, made clear that survived load could be an important factor for increase of the calculated strength for dike ring 16. According to Havinga (2015), the observed sand boils in the area during high waters is been limited also during high waters.

DURATION OF HIGH WATERS

In the calculations of the failure probability for piping, the normative high water levels (MHW's) were modelled as if they stand against the dikes forever. In reality a high water level is not acting as a permanent load on the dike but will decrease after some time. Especially in the Rhine-Meuse estuary, the duration of a high water level is dependent on the duration of a storm. When the storm dies down or when high tide changes in low tide the water levels in the Rhine-Meuse estuary will also decrease. One could imagine that a certain time is necessary for the pore pressure to develop, but more important, that time is needed to develop the actual pipe from beginning until actual failure of a levee. Havinga (2015) endorses these findings and it is recommended to apply more research in the time dependency on the failure probability for piping.

5.2.4. COMPARISON EFFECTS FOR PIPING WITH RESPECT TO EFFECTS ON HEIGHT

The effects of the alternative strategy 'Sluices' for piping are not equal proportional to effects for height. Because also a reduction of more moderate water levels is realized, the effects on reduction of failure probabilities for piping are even more upstream noticeable than for overtopping. Compare for instance figure 5.3 with figure 4.3. Above Dordrecht an increase in HBN of 0.38 m is found for overtopping in 'Sluices' 2100 (dike section 16002013) with respect to 'DP2015' 2015, but still a reduction in failure probability for piping is found (factor of 0.48). This phenomena is explained in the figure below, where for more frequent events (return periods between 1 and 100 years) still an reduction in hydraulic load level is realized, but for extreme events (return period nearby the norm of 83,000 year) the hydraulic load level is larger. Note that for piping in reality the normative water level (MHW) is used and not the hydraulic load level (HBN):

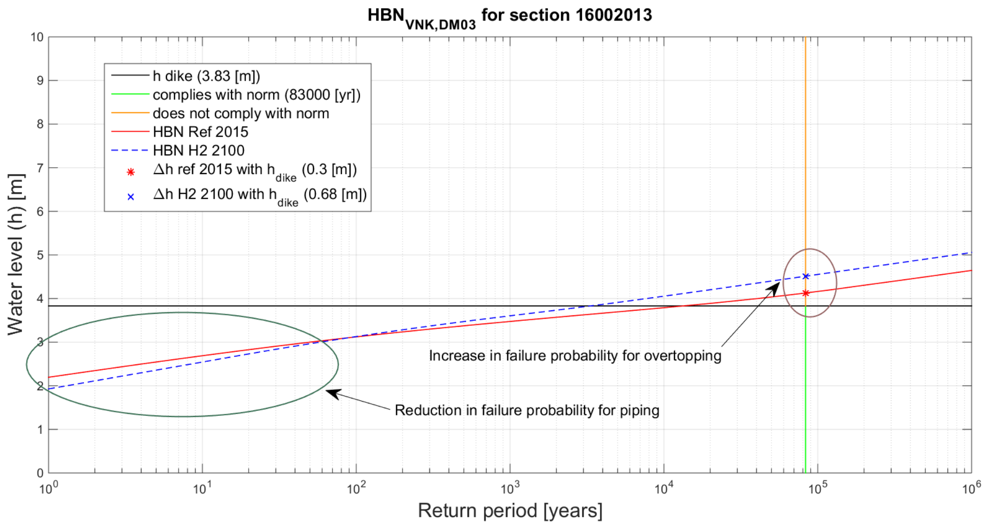


Figure 5.15: Explanation of reduction in P_f for piping in 2100 according to 'Sluices' and increase in P_f for overtopping at the same time with respect to 'DP2015' 2015

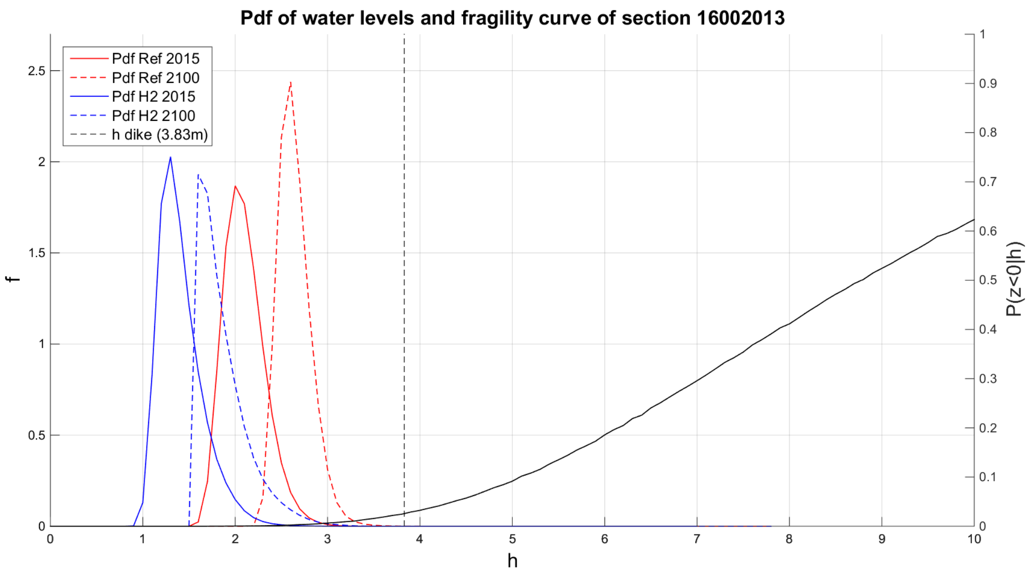


Figure 5.16: Corresponding fragility curve with probability density functions for dike section 16002013

5.3. CONCLUSIONS & RECOMMENDATIONS FOR PIPING

Above results show that there is an opportunity for the strategy 'Sluices' in significant reduction of flood risk in dike ring 16. However, as earlier stated, the reduction in flood risk will not be enough according to the results in absolute numbers. On the other hand, there is enough to argue about aspects that thus far have been excluded in calculations, for instance the absence of the time aspect in the Sellmeijer formula. In this section the main conclusions and recommendations will be stated with respect to piping.

5.3.1. CONCLUSIONS

Based on above findings, the following is concluded for the failure probabilities with respect to piping under the evaluation of both strategies.

- Strategy 'Sluices' 2015 leads to a reduction in failure probabilities in all sections for piping with respect to 'DP2015', in case this strategy is present in 2015. The reduction in failure probability ranges from a factor 5 (nearby Schoonhoven) to 1,000 (nearby Krimpen aan de Lek). In 2100, a reduction in failure probability with respect to 'DP2015' in 2015 is still found for trajectory 16-2 and 16-3, ranging from a factor 2 nearby (Schoonhoven and Dordrecht) to a factor 25 (nearby Krimpen aan de Lek). Upstream Schoonhoven, the failure probability increases between 50% and 100% in trajectory 16-3. For trajectories 16-1 and 16-4, again an increase in failure probability is found in 'Sluices' 2100 compared to 'DP2015' in 2015.

- In 2100, the calculated risk in strategy 'Sluices' is factor 5 to 250 lower than strategy 'DP2015' (in 2100) for trajectories 16-2 and 16-3 (partly).
- The positive effects from strategy 'Sluices' in reduction of failure probability in piping are noticeable more upstream than the positive effects in reduction of failure probability for height. This was explained in section 5.2.4.
- The positive effects for piping in the 'Sluices' strategy are in order magnitude smaller than the effects for height. This is a result of the fact that the reduction of more moderate water levels mainly lead to the reduction in failure probability for piping, whereas only the reduction in extreme high water levels leads to a reduction in failure probability for height.
- While strategy 'Sluices' leads to a significant reduction in risk, this reduction in failure probability is not enough to meet the standards of most dike sections. Therefore still a reinforcement task due to piping exists for dike ring 16 in case strategy 'Sluices' is adapted. There will still be a shortage on berm width for most dike sections, but this shortage is reduced. In chapter 6 the costs for piping berm will (partly) taken as a function of shortage of berm width.

5.3.2. RECOMMENDATIONS

With respect to the evaluation on piping sensitivity within dike-ring 16, the following is recommended:

- In the current models and formulas for the calculation of failure due to piping, there is no time aspect included, while in reality failure due to piping is also a function of the duration of a high water event. The longer a high water event lasts, the higher the failure probability. Storm surges from the North Sea typically have a smaller duration than for instance discharge waves from rivers. High water levels in the Rhine-Meuse estuary occur due to either one of those events or a combination of them, so it is useful to take duration of high water levels into account in the models and formulas.
- The effect of survived load on the failure probabilities for piping should be investigated, as this knowledge is currently hardly used but could lead to a significant reduction of failure probability.
- In the calculation of failure probabilities, a mix of the '4-forces' and '2-forces' model is applied in different situations. It is advised to only make use of the 2-forces model, as this model describes the actual performance of piping better (with a loose grain on the surface instead of embedded between other grains) and more consistency is acquired.

6

Discussion

This chapter discusses various aspects that came across during the execution of this research. In section 6.1 assumptions made with respect to the used water levels will be discussed. In section 6.2 a discussion follows on the assumptions with respect to evaluated dike sections and in section 6.3 the effects of risk reduction in strategy 'Sluices' compared to 'DP2015' in dike ring 16 is expressed in costs. Next, in subsection 6.4, there will be zoomed out from dike ring 16 to the Rhine-Meuse estuary, to interpret the results of this dike ring to the entire system. It is described which trajectories have an opportunity to benefit most of the measures within strategy 'Sluices' and which will be influenced negatively. Last, implications in the optimization are given that come along in case the sluices will be implemented earliest by 2030 in section 6.5

6.1. Assumptions in water levels

The used water levels (HKV, 2015) are calculated according the Deltamodel version 03. The results for height were verified by comparing the results of the Deltamodel version 02 with VNK results. The choice for Deltamodel version 03 was made in order to show as much as possible the effects of only the sluices and excluding other effects. In the Deltamodel version 02 for instance is capping of extreme river discharges for upstream dike sections taken into account in 2100, but not in 2015. For dike stretches along the Lek, this principle would lead to a reduction of design water levels in 2100 with respect to 2015, making it less clear which part of risk reduction esteemed from the alternative strategy and which part from this change in principle.

In the configuration of water levels for strategy 'Sluices', the pumping stations are in operation for any time $h_{sluices,riverside} < h_{sluices,seaside}$ and $h_{sluices,seaside} > 1.0m + NAP$. It can be argued whether this is a realistic configuration. In 2100, due to sea level rise of 0.85m, the pumping stations will almost be in operation for any time a high water occurs, which is roughly twice a day. The operation of the pumping stations will have a high energy consumption and during periods with lower amounts of river discharge it is not necessary to reduce the water levels in the Rhine-Meuse estuary as much as in the current configuration. High river discharges entering the Rhine-Meuse estuary can be predicted reasonably well a few days ahead and it can be argued that the pumping stations should only be in operation a few days in front of an expected extreme discharge wave. This revised configuration would reduce operation costs of the pumping stations, but would increase the failure probability of piping.

6.2. Assumptions with respect to evaluated dike sections

In table 2.5 it was found that the failure probability due to overflow and overtopping was determined by the dike section with maximum failure probability for these mechanisms. In this study, a limited amount of dike sections within the trajectories are evaluated, namely only the sections that were sensitive to piping. Trajectories 16-1 to 16-4 are divided by 57 dike sections in total, and only 23 sections are evaluated. It can be that the normative dike sections with respect to overtopping/overflow are excluded in this study. On the other hand, the reduction in hydraulic loads is largest nearby the sluices and decreasing as the location of the regarding dike section is situated more upstream. An evaluation of the other dike sections which are until now excluded can be realized by interpolating the reduction in hydraulic load levels.

The second aspect that is under discussion are the assumptions made within the calculation of flood probability due to piping. From chapter 5 it became clear that only relative changes in failure probability can be estimated, as the verification showed that the computed results compared to the outcomes of VNK differed significant:

- An explanation was found by the fact that a simplification of bottom schematization was applied and only 1 aquifer was adapted in the calculations, even for situations were the actual bottom was built up by more aquifers.
- Another explanation was found in the use of the Sellmeijer formula in subsection 5.2.1. The formula used in this study assumed only the '2-forces' model, where the grains under influence of piping were modelled to be located at the surface. In the studies of VNK, this '2-forces' model was only applied in case the bottom was schematized by 2 aquifers. When the bottom consisted of 1 aquifer, equation 2.18 was applied. This equation implicitly assumes that the grain is embedded in the sand layer and thus having a much higher resistance, leading to lower failure probabilities due to piping.

An aspect that is excluded in both the calculations made within this study and within VNK, is the time dependence on the failure probability of piping. This is something that should be incorporated in calculations for piping in the Rhine-Meuse estuary as the duration of high water events is often limited to a few days instead of weeks. In this case the duration of a high water event could be smaller than the time that is necessary for the piping mechanism to develop up to structural failure of a levee.

6.3. EXPRESS RISK REDUCTION WITHIN 'SLUICES' IN SAVED COSTS

So far, results have been presented either by changes in risk or by changes in necessary dike height or width. Regarding costs, the main principle was that in case a dike section was rejected compared to the new standard specifications, high costs would be inevitable. The expected reinforcement costs are then equal to the costs per trajectory of the study of Botterhuis (2015) (see also figure 2.2). The underlying assumption of this calculation, is that initial costs for dike reinforcements are high (equipment, replacement of top-layers, permits etc.) and that when a dike trajectory needs to be reinforced, an over-dimensioned reinforcement takes place in order to avoid reinforcements in the near future for the same section again (i.e. when a dike is 0.20m below the required height, one will not add only 0.20m but 1.20m to take further climate change and bottom subsidence into account).

With above assumptions in mind, it became clear that no significant cost reduction in dike reinforcements could be realized, while high investment costs for the sluices, locks, and pumping stations among others would be necessary in strategy 'Sluices', making it a non desirable strategy. This section provides more insight in expected cost reductions with respect to dike reinforcements and a calculation is made in order to get a first insight in the cost reduction for dike reinforcements within strategy 'Sluices' compared to 'DP2015' until 2100. First the approach of the cost calculation is explained.

6.3.1. APPROACH FOR COST CALCULATION

On a similar manner as done by Botterhuis (2015), costs for dike heightening are calculated in this study. The used data esteems from the 'Blokkendoos', which is a tool to budget costs for dike reinforcements according to different types of measures and strategies (Kind et al., 2015). The design horizon for dike reinforcements in the following calculation is set on 2100 for both the strategies: It is assumed that the dikes 'just' should meet the requirements in 2100. This means, that when a dike needs to be heightened or strengthened, one takes bottom subsidence, increase in hydraulic load level and a robustness surcharge into account up to 2100. In the first part, only costs for dike heightening are estimated, costs to reinforce dikes against the piping mechanism are discussed later in subsection 6.3.3. The formula for necessary increase in dike height is given by:

$$\Delta h = HBN(P)_{2100} - h_d + RS - Sub_{2100} \tag{6.1}$$

where:

Δh	= The increase in height at moment of execution [m]
$HBN(P)_{2100}$	= Hydraulic load level given its safety standard P per dike section in 2100 [m]
h_d	= Current dike height [m]
Sub_{2100}	= Expected bottom subsidence in 2100 with respect to 2015 [m]
RS	= Additional robustness surcharge [m]

Formula 6.1, for the determination of increase in dike height, is different to the formula used in the 'Blokkendoos'. In the 'Blokkendoos tool, one assumes a design horizon of 50 years after heightening is necessary. In the formula used for this calculation, the design horizon is fixed on 2100 and the moment of reinforcement is left out the equations as for now only a calculation will be made in terms of nominal costs. An extension to net present costs (NPC) will be made later on in a qualitative manner, as there are many aspects that influence the moment of reinforcement and thus the NPC.

The terms $HBN(P)_{2100} - h_d$ can easily be read from figures 4.5 and 4.7 where the difference in hydraulic load level in 2100 with respect to the dike height is stated. The expected bottom subsidence Sub_{2100} in 2100 differs per dike section. From the database of the 'Blokkendoos', information is extracted to determine the expected bottom subsidence (Deltares, 2015). The subdivision of dike sections within the 'Blokkendoos' differs to the used sub-division of dike sections in this study (in which VNK dike sections are used). Generally, the length of dike sections within the 'Blokkendoos' is longer. In table 6.1 information is found of the used dike sections in the 'Blokkendoos', with their corresponding trajectory and length.

Table 6.1: Sub division of Blokkendoos dike sections within dike trajectories

Tr.	Blokkendoos section	Length section [km]
16-1	16-1-4-B-1-Z[i]	-
	16-1-4-C-2-Z	6.6
	16-1-4-C-3-Z	5.8
	16-1-5-A-1-Z	4.8
	16-1-5-A-2-Z	4.6
16-2	16-1-2-A-1-Z	3.1
	16-1-2-B-1-Z	4.1
	16-1-2-B-2-Z	5.6
	16-1-3-A-1-Z	1.0
	16-1-3-A-2-Z	5.9
	16-1-3-A-3-Z	1.4
	16-1-3-B-3-Z	1.0
	16-1-3-B-4-Z	1.3
	16-1-4-A-1-Z	0.8
	16-1-4-B-1-Z	4.1
16-3	16-1-1-A-1-Z	3.7
	16-1-1-B-2-Z	3.6
	16-1-1-C-2-Z	0.7
	16-1-1-C-3-Z[ii]	5.1
	16-1-2-B-2-Z	-
	16-1-2-C-2-Z	1.5
	16-1-2-C-3-Z	2.3
16-4	16-1-1-C-3-Z	5.1
	16-1-1-D-4-Z	7.5
	16-1-1-D-5-Z	2.5
	16-1-1-E-5-Z	4.0

6.3.2. COSTS FOR DIKE HEIGHTENING

In the 'Blokkendoos', for each section reinforcement costs are determined per decimetre increase of a dike section (Deltares, 2015). The costs in the 'Blokkendoos' for increase are extracted from another software program called 'KOSWAT'. In this program, costs are estimated on a detailed level, taking into account several

[i]Dike sections 16-1-4-B-1-Z and 16-1-2-B-2-Z are attributed to trajectory 16-2. In the 'Blokkendoos' they made part of 2 trajectories, but to correspond with the actual trajectory length it is choosen to attribute the section only to one trajectory

[ii]The length of dike section 16-1-1-C-3-Z is divided by two because it made part trajectory 16-3 and 16-4. By doing so, the length of the trajectories within the 'Blokkendoos' corresponds more or less with the lengths of the trajectories in this study as given in table 2.6

aspects like dike dimensions, presence of houses and buildings near or at the dike and presence of infrastructure (Deltares, 2014). KOSWAT is programmed to find the cheapest solution for dike reinforcements, taking into account these aspects. When a dike needs to be heightened, the slope of the dikes will remain the same and the extension takes place at the inner slope (or sometimes at the outer side in case this is manually specified). See figure 6.1 for the simplification of the dike heightening.

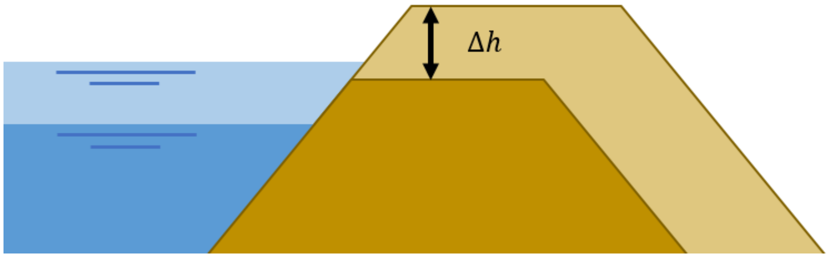


Figure 6.1: Principle of dike heightening within the 'Blokkendoos' based on input from KOSWAT

As the 'Blokkendoos' subdivision of dike sections is different to the subdivision of VNK dike sections, it is difficult to determine the bottom subsidence per VNK dike section and filling in the parameters of equation 6.1. Therefore, an average increase in HBN is estimated per trajectory per strategy in 2100. Based on earlier mentioned figures 4.5 and 4.7, the following difference between hydraulic load level and dike height per trajectory is expected. Note that bottom subsidence and a robustness surcharge are still excluded in this table:

Table 6.2: Average difference in HBN compared to required height according the safety standards in 2100 for 'DP2015' and 'Sluices'. Bottom subsidence and robustness surcharge is not yet included

	Av. diff. 'DP2015' [m]	Av. diff 'Sluices' [m]
16-1	0.5	0.4
16-2	0.6	0.1
16-3	-0.3	-0.5
16-4	-0.6	-0.6

The rate of subsidence differs per dike section, therefore it is excluded from table 6.2. Including the subsidence and robustness surcharge, finally the heightening task (Δh) per dike section can be determined for both strategies and costs can be determined. This is done for both strategies in table 6.3. In the table the Δh for both strategies is determined and costs are coupled with respect to this necessary elevation. Δh Is calculated by means of equation 6.1. In the calculation a robustness surcharge (RS) of 0.3m is used and for $HBN(P)_{2100} - h_d$ the values of table 6.2 are implemented. For sections where $\Delta h < 0.3m$, no costs are included as the difference in height is smaller or equal to the robustness surcharge. This surcharge is only necessary after a dike is rejected to the standard specifications (which is not the case for the sections with $\Delta h < 0.3m$).

Table 6.3: Nominal costs for dike reinforcements up to 2100 for 'DP2015' and 'Sluices', given Δh

Tr.	Dike section	Sub_{2100} [m]	Δh_{2100} 'DP2015' [m]	Δh_{2100} 'Sluices' [m]	Nom. costs 'DP2015' (M €)	Nom. costs 'Sluices' (M €)
16-1	16-1-4-C-2-Z	-0.50	1.3	1.2	183.6	181.6
	16-1-4-C-3-Z	-0.50	1.3	1.2	146.7	141.2
	16-1-5-A-1-Z	-0.50	1.3	1.2	121.6	120.8
	16-1-5-A-2-Z	-0.40	1.2	1.1	154.4	148.8
16-2	16-1-2-A-1-Z	-0.65	1.6	1.1	77.9	67.2
	16-1-2-B-1-Z	-0.65	1.6	1.1	98.8	89.2
	16-1-2-B-2-Z	-0.65	1.6	1.1	117.6	85.4
	16-1-3-A-1-Z	-0.60	1.5	1	9.0	8.5
	16-1-3-A-2-Z	-0.50	1.4	0.9	91.8	84.8
	16-1-3-A-3-Z	-0.50	1.4	0.9	14.5	10.3
	16-1-3-B-3-Z	-0.50	1.4	0.9	11.1	8.1
	16-1-3-B-4-Z	-0.50	1.4	0.9	11.5	8.1
	16-1-4-A-1-Z	-0.50	1.4	0.9	16.6	15.5
	16-1-4-B-1-Z	-0.50	1.4	0.9	96.6	86.2
16-3	16-1-1-A-1-Z	-0.65	0.7	0.5	63.2	58.9
	16-1-1-B-2-Z	-0.60	0.6	0.4	65.4	61.2
	16-1-1-C-2-Z	-0.60	0.6	0.4	5.5	4.6
	16-1-2-C-2-Z	-0.65	0.7	0.5	7.5	6.7
	16-1-2-C-3-Z	-0.65	0.7	0.5	21.6	20.9
	16-1-1-C-3-Z	-0.50	0.5	0.3	61.6	52.3
16-4	16-1-1-C-3-Z	-0.50	0.2	0.2	-	-
	16-1-1-D-4-Z	-0.35	0.1	0.1	-	-
	16-1-1-D-5-Z	-0.35	0.1	0.1	-	-
	16-1-1-E-5-Z	-0.35	0.1	0.1	-	-
Total:					1,376	1,260

From table 6.3 it becomes clear that the reduction in hydraulic loads within alternative 'Sluices' indeed only slightly reduces the costs. A nominal saving of **€116 million** (8%) is realized for dike heightening within dike ring 16. The percentage reduction differs over the trajectories, where trajectory 16-2 benefits the most of the alternative strategy, this is in line with the findings of chapter 4, where the reduction in hydraulic load level was the largest at trajectories 16-2 and 16-3. Table 6.4 shows the expected costs per trajectory, furthermore the results from the reference study of Botterhuis (2015) are stated in order to compare the results. It is shown that the costs in the reference study are slightly higher than in the calculation performed in this study. This can be explained by the following aspects:

- A difference in approach with respect to the chosen safety standard ('signal value' versus 'lower limit' of table 2.6)
- A slight difference in approach with respect to the used hydraulic loads
- A difference in design horizon (in the example the design horizon is set on 2100, whereas the design horizon in the reference study is set on 50 years after reinforcement)
- Costs for piping are left out of consideration so far in the example, while they are included in the reference study (costs for piping are discussed later)

Last, costs for trajectory 16-4 are avoided at all in the calculation, whereas the reference study calculates €191 million. This large difference is explained due to high initial costs, in case a slight heightening would be necessary in the calculated example, already initial costs of €107 million are made. These costs are now avoided because dike heightening is not necessary.

Table 6.4: Nominal reinforcement costs for dike heightening [M€]

Tr.	'DP2015'	'Sluices'	'Sluices'/'DP2015'	Botterhuis (2015) ('DP2015')
16-1	606	592	98%	652
16-2	545	463	85%	482
16-3	225	205	91%	352
16-4	-	-	-	191
Total:	1,376	1,260	92%	1,486

Finally, with respect to expected costs for dike heightening, a sensitivity analysis is performed to show which parts of equation 6.1 are the main cost drivers. For the four trajectories, the costs are split into four parts: initial costs (inevitable costs in case one has to start reinforcements), extra costs due to robustness surcharge of 0.3m, additional costs to cope bottom subsidence and last additional costs or cost savings either by the comparison of hydraulic load level in 2100 with the current dike height. In table 6.5 the subdivision in these four parts is given. It is clear that initial costs to start a dike heightening is by far the main driver in costs, for 70%-75% of the total budget. The initial costs are built up of various expenses, think of permits, design of the measures, equipment, demolition and replacement of infrastructure near or at the dikes, replacement of the top-layer of the dike, land purchase and demolition of buildings. As it is shown that it is impossible to cancel the costs dike reinforcements (soon or later the discussed trajectories need to be heightened), high costs are inevitable. However, postponement of investments also leads to cost reduction in terms of present values. This is discussed later, but first the costs for piping will be discussed.

Table 6.5: Subdivision of reinforcement costs into initial costs, costs for robustness surcharge, subsidence costs and costs due to change in hydraulic load level compared to the dike height

Tr.	Initial costs (M€)	RS (M€)	Sub_{2100}	Diff. with h_d 'DP2015' (M€)	Diff. with h_d 'Sluices' (M€)
16-1	421.3	42.6	69.8	72.6	58.7
16-2	345.8	33.3	69.8	96.4	14.4
16-3	164.6	26.7	91.1	-57.6	-77.5
16-4[iii]	107.4	34.7	55.6	-73.7	-73.7
Sum:	1,039.1	137.3	286.3	37.6	-78.1

[iii] Also costs for trajectory 16-4 are included in the table, but as earlier discussed dike heightening can be delayed up to 2100. This is not the case for the mechanism piping hence they are added here.

6.3.3. COSTS FOR PIPING

In the 'Blokkendoos' costs for measures to reduce the risk on piping, are implicitly included. According to Deltares (2014, p. 41) the first step in determining which sections need to be reinforced, is to check whether the flood risk due to the piping mechanism exceeds the proposed standard specifications. When this is the case, the regarding section is prioritized with high urgency to strengthen in the near future and simultaneously it is checked whether within the design horizon of the dike also a shortage for height is expected. In that scenario, the dike will both be strengthened as heightened. One realizes a reduction in failure probability due to piping by increasing the berm length at the inner slope. See figure 6.2 for the schematization of the regarding measure.

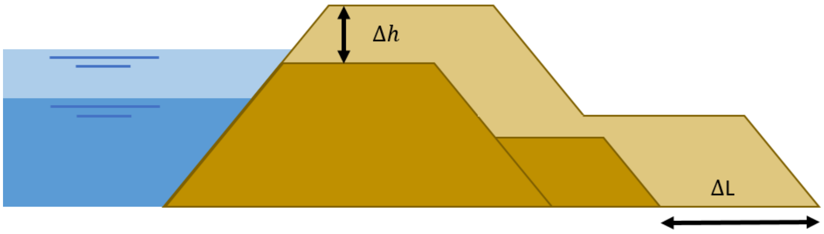


Figure 6.2: Principle of dike strenghtening and heightening within the 'Blokkendoos' based on input from KOSWAT

In the 'Blokkendoos' fixed costs are attributed to the dike sections where flooding due to piping plays a significant role. This means that regardless of climate change and measures to reduce hydraulic loads, the same initial costs are expected to solve the flooding problem due to piping. In case dikes also need to be heightened, 1/3rd of the initial costs of construction of a piping berm per meter dike heightening is budgeted. The total initial costs for strengthening of the dikes within dike ring 16 are estimated to be €73 million, extracted from the database of Deltares (2015), which is again a fraction of the costs compared to initial costs for dike heightening. It can be doubted whether these costs are realistic, because the wider a piping berm needs to be, the more change that other infrastructure/buildings get harmed by the measure (which would not be harmed when a dike is only heightened). Besides, the approach used in the 'Blokkendoos' leads no room for other measures in reduction of the piping problem. Therefore, an other approach in cost calculation for piping is proposed in the following paragraphs

In figures 5.9 and 5.11 the shortage in berm width was shown for the sections sensitive to piping in 2100 for both the strategies. In these figures it is clearly shown that the shortage in berm width is less in the alternative 'Sluices' than in the strategy 'DP2015'. From literature cost key figures are available for the construction of a piping berm. These costs range from €40,- to €60,- per meter additional berm width for the processing of earth or the construction of a seepage screen (in case there is no room for the placement of a berm) (Förster et al., 2012, p. 145). Furthermore, expenses have to be made partly as land purchase, which is often the most distinctive expense for piping berms (Ter Horst, 2015). In the subprogramme 'Rivers' of the Deltaprogramme (DPR), an expense of €176.62/m^2 is accounted for in case of urban area (Ouwerkerk, Wojciechowska, Barneveld, & Silva, 2014, p. 13). Assuming that half of the total piping berms in the trajectories needs to be placed in an urbanized area, the costs per meter berm width are roughly estimated at €100 per meter berm width. Per dike section, scenario and reference year, the costs for piping berms are found in table 6.6. The used dike sections esteem from the VNK studies (in contrast with the KOSWAT dike sections of table 6.1). In table 6.7 the costs are summarized per trajectory for the situation in 2100 and a relative comparison between the two strategies is made.

Table 6.6: Estimated costs for strengthening of dikes [M €] as a function of the shortage on berm width ΔL [m]

			'DP2015'				'Sluices'			
			2015		2100		2015		2100	
Trajectory	Dike #	Length [m]	ΔL	M €	ΔL	M €	ΔL	M €	ΔL	M €
16-1	16001001	2,330	99	23.1	124	28.8	94	21.9	113	26.4
	16001003	2,577	2	0.5	16	4.1	1	0.2	13	3.4
	16002018	1,732	0	0.0	0	0.0	0	0.0	0	0.0
16-2	16002002	1,200	9	1.0	31	3.7	0	0.0	0	0.0
	16002009	1,203	29	3.5	50	6.0	0	0.0	18	2.1
	16002013	874	37	3.3	62	5.4	8	0.7	28	2.5
	16003031	929	0	0.0	14	1.3	0	0.0	0	0.0
	16003034	1,140	0	0.0	6	0.7	0	0.0	0	0.0
16-3	16003015	1,961	71	13.9	92	18.1	66	12.9	82	16.1
	16003016	2,052	40	8.2	63	12.8	35	7.3	52	10.7
	16003018	2,318	20	4.7	38	8.8	15	3.4	27	6.2
	16003024	1,592	114	18.2	141	22.5	83	13.2	106	16.9
	16003026	1,527	108	16.4	136	20.8	69	10.5	89	13.7
16-4	16003002	1,623	0	0.0	7	1.1	0	0.0	5	0.8
	16003003	1,429	31	4.4	47	6.7	30	4.3	44	6.2
	16003005	1,481	52	7.7	65	9.6	51	7.6	63	9.3
	16003006	1,273	59	7.5	74	9.5	58	7.4	72	9.1
	16003007	1,786	66	11.8	83	14.8	65	11.6	79	14.2
	16003008	2,042	34	6.9	50	10.1	32	6.5	46	9.3
	16003009	1,539	37	5.7	56	8.6	35	5.5	51	7.9
	16003011	1,590	61	9.7	76	12.1	60	9.5	72	11.5
	16003012	1,200	88	10.5	109	13.1	85	10.1	103	12.4
	16003014	2,421	53	12.9	73	17.8	48	11.7	65	15.6

Table 6.7: Estimated nominal costs [M €] for piping in the two strategies up to 2100

Trajectory	'DP2015' in 2100	'Sluices' in 2100	'Sluices'/'DP2015'
16-1	32.9	29.7	90%
16-2	17.1	4.6	27%
16-3	83.0	63.7	77%
16-4	103.4	96.3	93%
Total:	236.4	194.3	82%

From table 6.7 it becomes clear that with respect to piping, nominal savings are made of **€42.1 million** (18%). The total nominal costs for strategy 'DP2015' become €1,719 million (€1,376m for heightening, + €237m for strengthening and another + €107m for initial costs in strengthening of trajectory 16-4). The total nominal costs for strategy 'Sluices' become €1,561 million (€1,260m for heightening, + €194m for strengthening and another + €107m for initial costs in strengthening). The total nominal savings in dike reinforcements for dike ring 16 within strategy 'Sluices' are then estimated at **€158 million** (9%) compared to strategy 'DP2015'.

6.3.4. VALUE OF POSTPONEMENT

This subsection provides insight in the relative change in costs in case these are calculated with the net present value formula. From previous subsections it became clear that strategy 'Sluices' saves €158 million in dike reinforcements for dike ring 16 compared to 'DP2015'. In relative terms a saving of 9% is realized and still dike reinforcements need to take place for approximately €1.55 billion within dike ring 16. However, when dike reinforcements may be postponed to later years, also savings are realized with respect to present values. Hence it should be known in which year reinforcements will need to be executed.

CALCULATE YEAR OF IMPLEMENTATION

In the 'Blokkendoos' it is in the first place checked whether the risk on failure due to piping meets the standard specifications and if not, the reinforcement is scheduled to take place in 2032 (the first moment that dikes will be assessed according to the new standard specifications + 15 years ordering time). Once the reinforcement to solve the piping issue takes place, immediately dike heightening takes place when standard specifications would not be met in the coming 50 years for this mechanism. According to Vergouwe and Van den Berg (2013), 23 of the 57 dike sections within dike ring 16 have a relative high risk on failure due to piping. These 23 sections were assessed in this study, and as became clear in chapter 5, the large majority of these sections will not meet the safety standards within both strategies. Hence for these dike sections, postponement of reinforcement measures is not an option. An exception is made for the 5 dike sections within trajectory 16-2. For three of the five sections there is no reinforcement task in 2100 for piping and for the other two the shortage in berm width is limited to 20-30m (see also figure 5.11).

Failure due to piping is for the other 34 dike sections not an issue, hence for these sections postponement could be an opportunity in terms of cost savings, as the largest contribution in failure is determined by overtopping/overflow. The expected year of dike heightening will however be calculated based on the 23 dike sections that have been assessed throughout the entire report, as it is assumed that these sections are representative for the reinforcement tasks with respect to height for the entire dike ring (see also section 6.2).

In this calculation, the hydraulic load levels in 2015 and 2100 are for both strategies plotted against the height of dikes, including bottom subsidence. The moment where the hydraulic load level exceeds the dike height, is taken as year in which dike heightening should take place. As expected bottom subsidence, the maximum subsidence rate per trajectory is chosen (0.5m for trajectories 16-1 and 16-4, 0.65m for trajectories 16-2 and 16-3, see table 6.3). It is noted that for both strategies, only the hydraulic load levels are plotted in 2015 and 2100. This suggests that in the 'DP2015' strategy a continuous improvement in function of improvement of the Maeslant barrier takes place (from 1/100 in 2015 to 1/1,000 in 2100) and that for the 'Sluices' strategy, the locks and sluices already are implemented in 2015. This is not the case, but it is assumed that for a first estimation the effects of this simplification is negligible. Adding extra points with data of the hydraulic load in different years would improve the results. In figure 6.3 it is shown how the alternative strategy leads to a postponement of 36 years for dike section 16002009 in case the alternative strategy is applied.

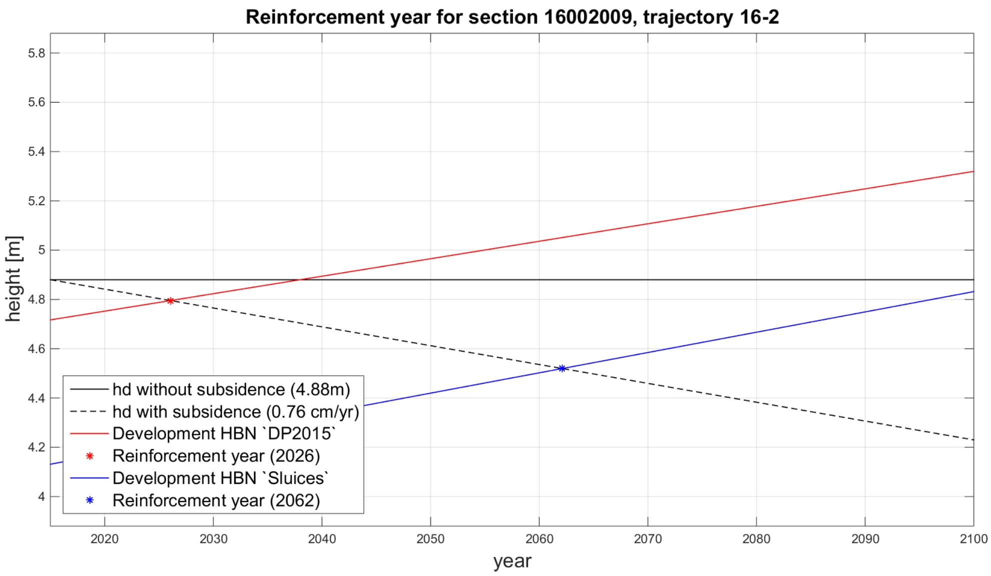


Figure 6.3: Necessary year of dike heightening in both the strategies

Similar to figure 6.3, for all other dike sections within dike ring 16 the year of implementation is calculated. Per trajectory, the necessary year of reinforcement is averaged for both strategies. The results are shown in table 6.8. In line with the trajectories where the largest reduction in hydraulic load is found for 'Sluices' compared to 'DP2015', the difference in reinforcement year is largest. In trajectory 16-2 a postponement of 37 years could be realized and in trajectory 16-3 the postponement is 14 years.

Table 6.8: Necessary year of dike heightening per trajectory and strategy

Trajectory	Year 'DP2015'	Year 'Sluices'	Difference [year]
16-1	2032	2036	4
16-2	2023	2060	37
16-3	2074	2088	14
16-4	2108	2114	6

EXPRESS COSTS IN TERMS OF NET PRESENT VALUES

Once the reinforcement years are known, it is possible to calculate the costs in terms of present values. The formula for present costs are given by equation 6.2:

$$NPC = \sum_{t=1}^{n} \frac{C_t}{(1+r)^t} \tag{6.2}$$

where:

NPC = the net present costs [M€]
n = final year in which costs are made [-]
t = year of expenditure [-]
r = discount rate [-]
C_t = expenditure in year t [M€]

The expenditures were found in table 6.4 and 6.6, with the price level set on 2009 (VAT included). As discount rate in this example, r is set on 5.5% equal to the discount rate that is used within governmental projects with 2.5% for real discount rate and an additional 3% risk premium (De Jager, 2011). Inflation is included in the real discount rate (Brealey, Myers, & Allen, 2014, p. 136).

Filling in the necessary reinforcement years, one can compute the percentage costs in terms of NPV compared to nominal costs. By doing so the following % are found in table 6.9. In this table it is clear that for trajectory 16-2 a significant profit can be made in postponing reinforcement measures. Also it is clear that expenditures need to be done in the far future, barely lead to net present costs due to the discount rate. The possibility to postpone reinforcements only counts for the dike sections for which the failure probability is dominated by overtopping/overflow and have a low probability of failure due to piping.

Table 6.9: Converted costs from nominal values to NPV, net present values are expressed as an percentage of the nominal costs

Trajectory	Year 'DP2015'	Year 'Sluices'	% NPV 'DP2015'	% NPV 'Sluices'
16-1	2032	2036	40%	32%
16-2	2023	2060	65%	9%
16-3	2074	2088	4%	2%
16-4	2108	2114	1%	0%

With the calculated nominal costs and net present values as a percentage of the nominal costs, the total net present costs can now be determined. In tables 6.10 and 6.11 the net present costs are calculated for both strategies for dike ring 16. The nominal costs are divided in three parts (columns 2-4):

1. Initial costs per trajectory in case a reinforcement task is started
2. Additional costs for dike heightening Δh as was found in table 6.5 (the additional costs Δh are the costs for robustness surcharge, bottom subsidence and the difference in HBN with dike height)
3. Additional costs for the length of a piping berm ΔL

In the calculation to the final net present costs, it is assumed that costs for piping berms are made in 2015, hence costs for piping berms in NPV are equal to the nominal costs. Only for dike trajectory 16-2 in strategy

'Sluices' it is assumed that the costs may be postponed to the same year in which the dike will be heightened. The calculation of for instance trajectory 16-1 in table 6.10 is made as follow:

In 2015, costs for piping berms will be made (€32.9 million). Three of the seven sections will be strengthened and thus 3/7th of the initial costs (being €180.6 million) is also made in 2015 and 3/7th of the costs for dike heightening as this is done simultaneously with the strengthening (begin €79.3 million). In total €292.7 million will be spend in 2015. The other four sections will be heightened in 2032 (see table 6.9), which is a remainder of €346.5 million in nominal terms and €138.6 in present terms. The total present costs will finally be €431.3 million for trajectory 16-1 in strategy 'DP2015'.

Table 6.10: Calculation of net present costs for 'DP2015' [M€]

Tr.	Init. costs	Costs Δh	Costs ΔL	Year reinf. piping	# Sections sensitive to piping	Nom. costs 2015	Nom. costs later	% NPV	NPC
16-1	421.3	185.0	32.9	2015	3/7	292.7	346.5	40%	431.3
16-2	345.8	199.5	17.1	2015	5/23	135.6	426.8	65%	411.6
16-3	164.6	60.2	83	2015	5/13	169.5	138.3	4%	175.0
16-4	107.4	16.6	103.4	2015	10/14	192.0	0.0	1%	192.0
Total:	1,039.1	461.3	236.4		23/57	789.8	911.6		**1,211.3**

Table 6.11: Calculation of net present costs for 'Sluices' [M€]

Tr.	Init. costs	Costs Δh	Costs ΔL	Year reinf. piping	# Sections sensitive to piping	Nom. costs 2015	Nom. costs later	% NPV	NPC
16-1	421.3	171.1	29.7	2015	3/7	283.6	338.5	32%	391.9
16-2	345.8	117.5	4.6	2060	5/23	0.0	467.9	9%	42.1
16-3	164.6	40.3	63.7	2015	5/13	142.5	126.1	2%	145.0
16-4	107.4	16.6	96.3	2015	10/14	184.9	0.0	0%	184.9
Total:	1,039.1	345.5	194.3		23/57	611.0	932.5		**763.9**

From tables 6.10 and 6.11 the net present costs are calculated at €1,211.3 million for strategy 'DP2015' and €763.9 million for strategy 'Sluices', a saving of **€447 million** is realized in present values, which is determined by a combination of postponement and lower nominal costs. It should be noted that this result is only a very first estimation in order to compare the strategies: In reality there are many uncertainties involved in the calculation of costs to present terms. The discount rate r is for instance an important parameter which is a rather uncertain value. Furthermore the moment of reinforcement determines largely the costs in NPV and can be influenced due to several reasons like the fact water authorities may postpone a reinforcement or expedite it. Next the used principles determine largely the outcome. In this case study it was for instance assumed that dike sections could be strengthened separately, while in fact trajectories will be reinforced as a whole or for a large part. Note also that for simplicity it was assumed in the cost calculation that costs are uniform divided amongst the trajectories.

6.4. INTERPRETATION OF RESULTS DIKE RING 16 FOR THE ENTIRE SYSTEM

From the findings for dike ring 16 an interpretation of the results is made for the entire system on a qualitative manner. Dike ring 16 is located in a river dominated area for the upstream trajectories (16-1 and 16-4) and in the transition zone for trajectories 16-2 and 16-3. It is found that the reduction in hydraulic loads in alternative strategy 'Sluices' is largest at the trajectories 16-2 and 16-3. The reduction in hydraulic load led to a reduction in flood risk for the failure mechanisms piping and overtopping/overtopping in 2100 with respect to 'DP2015' in 2015. It can be argued that at the downstream trajectories from trajectory 16-2 and 16-3, also these beneficial aspects will be even more noticeable with respect to risk reduction. As was shown in previous section, this reduction in risk will not necessarily lead to a reduction in costs. As from 2017 new safety standards become effective and even when the risk is reduced, it is questioned whether the standards are met (see for instance the results of chapter 4).

In figure 6.4 the overview of expected costs for the 'DP2015' strategy are stated again. It can be seen that trajectory 15-1 and 15-2 are expensive dikes for the reinforcement. However, in table 3.2 it was made clear that the main failure mechanism in this trajectory is macrostability which accounted for 72% of the total failure probability within these trajectories. The effect reduction in normative water levels and hydraulic load levels should still be investigated for this mechanism. The expected costs for dike ring 14 are mainly given by height (41%), piping (15%) and other mechanisms (35%), which is mainly failure of dunes at the westside of dike ring 14. As trajectory 14-1 does not consist out of dunes, it has high potential in reduction of reinforcement costs, because height and piping will be the main contributors. The year of reinforcement is expected between 2040 and 2050. Furthermore, trajectory 14-1 is located far closer to the sluice complex, where the reduction in normative water levels and hydraulic load levels are largest. Regarding trajectory 22-2, also possible benefit can be achieved, as the main contributor to the failure probability is piping by 65% and the expected year of reinforcement is between 2050 and 2100. However, from figure 5.3 it followed that the benefit nearby Dordrecht in 2100 is a factor 2 to 5 with respect to reduction in failure probability for piping. This reduction might still not be enough. It is recommended to investigate especially trajectories 14-1, 15-2 and 22-2 quantitatively, as the risk reduction in trajectory 16-2 has proven to be effective even up to 2100 in alternative 'Sluices' compared to 'DP2015' in 2015.

In the current configuration of strategy 'Sluices', the structures are located at the Nieuwe and Oude Maas (nearby the transition between trajectory 14-1 and 14-2 in figure 6.4). Hence, trajectories 14-2, 14-3 and 19-1 will become at the seaside of the structures. As the Maeslant barrier will be removed, trajectories 14-2 and 19-1 will be much more exposed to extreme water levels in strategy 'Sluices' than within the 'DP2015' strategy, leading to higher costs for dike reinforcements. It is recommended to apply further research on the costs for dike reinforcements for these trajectories, and outweigh them against the costs for locating the locks and dams more westwards than in the current configuration of 'Sluices'.

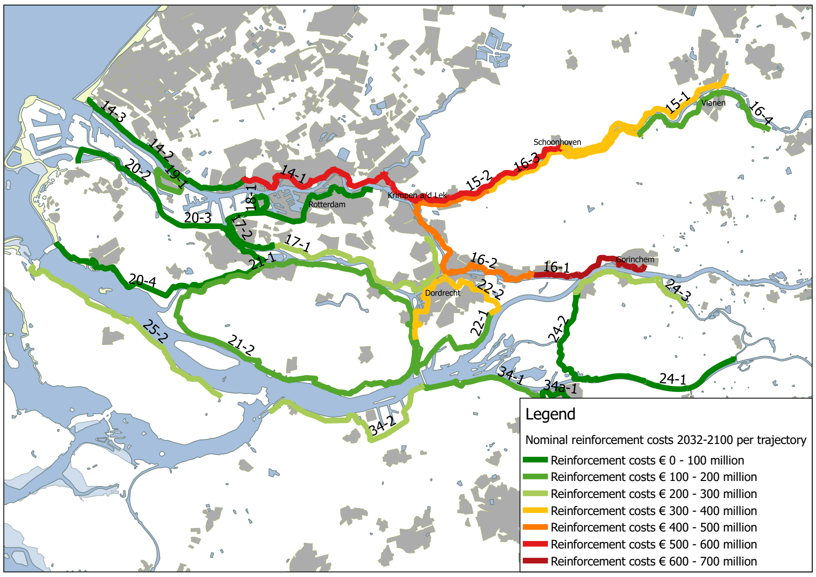


Figure 6.4: Expected costs 'DP2015' strategy according to Botterhuis (2015).

6.5. IMPLICATION DUE TO NECESSARY IMPLEMENTATION TIME 'SLUICES'

The last aspect that is discussed is the implication of implementation time for 'Sluices' in case this would be the favourable strategy. In case it is chosen that this strategy is beneficial by the end of 2015, it would still take at least 15 years before the sluices are finally realized. In this time, dike reinforcements for dike ring 16 should already have been executed as the new standards will be effective from 2017 and the prioritization states that these trajectories should be reinforced between 2024 and 2030. A revision in the prioritization could of course been made, but it is more realistic to assume that if the sluices will be implemented, this will not directly be done, but only when the sea level has risen significantly and more certainty is obtained in extreme river discharges. Further study is required to investigate the optimal moment of implementation of 'Sluices' related to the reinforcement programs that are undertaken for dikes.

7

CONCLUSIONS AND RECOMMENDATIONS

In this study the effects of reduction of hydraulic loads on the failure probability of dike trajectories within the Rhine-Meuse estuary in the strategy 'Sluices' are compared to the strategy 'DP2015', in order to find an answer on the research question:

How do reinforcement tasks according to strategy 'Sluices', where a reduction in hydraulic loads is realised, relate to dike reinforcements of the Deltaprogram 'DP2015' for the Rhine-Meuse estuary until 2100?

Reinforcement tasks are expressed in both meters (in Δh and ΔL) and costs (in M€). Dike reinforcements are necessary when the occurring failure probability of a trajectory exceeds the allowed failure probability according renewed safety standards. Instead of analysing the Rhine-Meuse estuary as a whole, it was investigated which dike trajectories are most distinctive with respect to investments in dike reinforcements in the near future. It was found that these trajectories are 16-1, 16-2, 16-3 and 16-4. The trajectories make part of dike ring 16 'Alblasserwaard & Vijfheerenlanden' and are located in the so-called 'transition zone' where hydraulic loads acting on the dikes are determined by both influence of high water levels from the North Sea (being tide and storm surge amongst others) as determined by discharge of the rivers Rhine and Meuse.

The failure probabilities for these trajectories are largely determined by failure due to piping and failure due to overtopping/overflow. The failure probabilities for these mechanisms are calculated for the situation in 2015 and in 2100 where influence of climate change is taken into account. In case the failure probability for a dike section exceeded the maximum allowed failure probability given new safety standards from section 3.5, it is analysed up to what extend the dike should be heightened (Δh) or strengthened (ΔL). Based on this shortage on dike height and berm width, a cost analysis is performed. Furthermore, a qualitative analysis on the effects of measures for the entire Rhine-Meuse estuary system is made. From the findings of previous chapters, the following conclusions and recommendations are drawn.

7.1. CONCLUSIONS

7.1.1. EFFECTS OF 'SLUICES' ON FAILURE MECHANISM OVERTOPPING/OVERFLOW

In case the alternative strategy 'Sluices' is present in 2015, a significant reduction in hydraulic load levels (HBN's) is realized compared to the current strategy 'DP2015'. The hydraulic load levels per dike section are calculated, given the return periods that followed from the safety standards for this section. These safety standards were derived from the standards that were given for trajectories, including length-effects. On the Lek, a reduction in hydraulic load level of 0.8m (nearby Krimpen aan de Lek) to 0.3m (nearby Schoonhoven) is realized compared to the current situation. Upstream of Schoonhoven, the pumping stations in combination with sluices and a retention basin in the Eastern scheldt will not be effective in reducing hydraulic loads. Along the river Waal (Beneden Merwede) a reduction in hydraulic load level is realized up to the bifurcation nearby Gorinhem of 0.6 - 0.1m. The realized reduction in hydraulic loads leads to a situation where all dike sections meet the new safety standards that take effect from 2017.

The alternative strategy 'Sluices' is in *2100* still effective in lowering hydraulic load levels with respect to the 'DP2015' 2015 situation for a small part at the Lek, between Krimpen aan de Lek and Schoonhoven. The increase in Hydraulic load level is limited to an order of 0.10m for the entire stretch Krimpen aan de Lek - Schoonhoven and Krimpen aan de Lek - Dordrecht. Comparing 'Sluices' in 2100 with the safety standards, it is found that 5 of the 23 dike sections need to be heightened before 2100 (local bottom subsidence was not accounted for). When strategy 'DP2015' is compared to the same standards it is found that 10 of the 23 sections need to be heightened (also without bottom subsidence). The reduction in hydraulic load level in strategy 'Sluices' is largest for trajectory 16-2.

7.1.2. EFFECTS OF 'SLUICES' ON FAILURE MECHANISM PIPING

Piping is an important contributor to the overall failure probability of the dike trajectories within dike ring 16. The reduction in water levels - for piping the normative high water levels (MHW's) applied - also led to a significant reduction in failure probability. When strategy 'Sluices' is present in *2015*, a reduction in failure probability of factor between 1,000 (Krimpen aan de Lek) and 5 (nearby Schoonhoven) is realized. Upstream of Schoonhoven the effects are only slightly noticeable. On the Noord and the Beneden Merwede (extensions of the river Waal) a reduction in failure probability of factor between 250 (Krimpen aan de Lek) and 1.5 (between Dordrecht and Gorinchem) is realized. Again, more upstream the effects are only slightly noticeable.

In *2100* still a reduction in failure probability is found in the alternative 'Sluices'. The reduction is found over the stretches Krimpen aan de Lek - Schoonhoven and Krimpen aan de Lek - Dordrecht. The respective trajectory concerns 16-2. For the other trajectories, an increase in failure probability is found with a factor in the order of 1.5 to 5. In the 'DP2015' strategy, an increase in failure probability is found over the entire dike ring with factors in the order of 2 to 20. The largest increase is found in trajectory 16-2.

When the reduced failure probabilities for piping within 'Sluices' are compared to the new safety standards, it becomes clear that this strategy is effective enough in reducing failure probabilities due to piping for a large part of trajectory 16-2 up to 2100. For the other trajectories, the reduction in normative water levels is not enough to reduce the failure probabilities due to piping up to a level where the standards are met. Hence, still dike reinforcements are necessary for the remaining dike sections. For the dike sections that do not meet the safety standards, it is determined to what extend the length of additional piping berm should be increased (ΔL). A cost function is determined in which the costs for piping berms are a function of the shortage on berm width combined with initial costs for dike reinforcements.

Remarkable in the comparison of effects for piping with respect to height, is that the beneficial effects of piping for the alternative strategy reach further upstream for the mechanism than for overtopping/overflow. This is explained by the fact that the pumping stations and sluices are more effective in reducing more moderate water levels, which is beneficial for the reduction in failure probability of piping. For overtopping/overflow the failure probability is mainly determined by hydraulic load levels that have a return period equal to the applicable safety standard per dike section, which is often in the orders of 10,000 years. In case of such an event, extreme river discharge is the driving force for the loads and pumping stations become less effective.

7.1.3. COST REDUCTION IN 'SLUICES' WITH RESPECT TO NECESSARY DIKE REINFORCEMENTS

A NOMINAL SAVING OF €158 MILLION UP TO 2100 IS REALIZED WITHIN 'SLUICES' FOR DR16

In chapter 6 a cost calculation is made in order to get a first insight in the cost reduction for dike reinforcements of dike ring 16 within strategy 'Sluices' compared to strategy 'DP2015'. The nominal costs for 'DP2015' were calculated to be €1.719 billion, while the costs for strategy 'Sluices' are calculated at €1.561 billion, leading to a nominal saving of €158 million (9%). The costs are largely determined by high initial costs in case a dike reinforcement is executed. Cost drivers within these fixed costs are amongst others design works, equipment, demolition and replacement of infrastructure near or at the dike and land purchase and demolition of buildings. Besides that initial costs are high, also costs due to dike heightening with a robustness surcharge of 0.3m and reinforcements to comply with bottom subsidence, lead to a situation where the nominal costs for dike reinforcements between the strategies are in the same order.

A cost reduction of €446 million ‘Sluices’ is realized in present values

Looking at the costs in terms of net present values, a reduction of €446 million (37%) is realized within strategy ‘Sluices’ with respect to ‘DP2015’, in case the principles are followed as stated in subsection 6.3.4. Strategy ‘DP2015’ costs €1,211 million, whereas for ‘Sluices’ net present costs of €764 million are calculated. The difference in net present costs is clarified by a combination of a reduction in nominal costs (and thus net present costs) and the finding that reinforcements of trajectory 16-2 can be postponed by 37 years for both dike widening and dike heightening. It is noted that the cost calculation in terms of present values is very sensitive to applied principles and boundary conditions.

7.1.4. Remaining conclusions with respect to applied methodology

With respect to the applied methodology in this study, three conclusions are drawn. The first conclusion relates to the use of the Sellmeijer formula. In the calculation of piping, two different modes of the Sellmeijer formula are applied within VNK studies. This leads to inconsistent results. The inconsistency is found in situations where the bottom layer is differently schematized. In case the bottom consisted of one aquifer, the Sellmeijer ‘4-forces’ model was applied. In this situation the grains that were affected by piping were schematized as if they were embedded in the sand layer. In case the bottom consisted of two aquifers on top of each other, the Sellmeijer formula ‘2-forces’ model is applied, where it is assumed that the grain is not embedded in the sand layer, but laying at the surface. In this thesis, only the ‘2-forces’ model is applied, leading to larger computed failure probabilities than calculated in VNK for many dike sections.

The second conclusion is that for the piping mechanism the duration of an extreme event was not taken into account in the available formulas, while literature has shown that piping is a mechanism that is under the influence of duration. As conservative assumption, the water levels have been modelled as if they would be present for ever. In addition, knowledge of survived loads on dike trajectories make no part yet of the assessment for failure probability. It is known that the duration of high water level events is limited in the Rhine-Meuse estuary and that the use of survived loads on dike rings within the Rhine-Meuse estuary could reduce the calculated failure probabilities by a factor 2 to 20.

The third conclusion relates to the use of the ‘Blokkendoos’ to calculate costs for dike reinforcements. In the ‘Blokkendoos’ it is hard to define which cost drivers lead to high initial costs for dike reinforcements. It is advised to provide more transparency in the ‘Blokkendoos’ in order to check how costs are built up. Furthermore, the ‘Blokkendoos’ works with fixed costs per dike section for dike strengthening and reducing failure probabilities due to piping. This approach leaves no room for alternative measures like lowering the hydraulic loads (and thus reducing the necessary berm width). In this study, therefore an alternative approach is used to calculate costs for piping berms, as a function of necessary berm width.

7.2. Recommendations

The following recommendations are made:

7.2.1. Incorporate duration of high water events

As stated in the conclusions, duration of high water events have thus far been excluded in the assessment of dikes for piping. It is known that a critical time is needed for pipes to develop. If the duration of a high water event is lower than the critical duration for piping, dikes could be much safer than currently calculated in this thesis and the study of VNK (Vergouwe, 2014). Especially for trajectory 16-2, where the influence of the water levels at the North Sea is significant and the expected costs for dike reinforcements due to piping are high, including duration may lead to a significant cost reduction.

7.2.2. Make use of survived loads on piping

Related to the duration of high water events, also the use of survived loads on failure probabilities due to piping could lead to significant reduction in calculated failure probability. As shown in literature, the effects of including survived loads could lead to a reduction of factor 2 to 20 in the Rhine Meuse estuary. Again, in combination with the effects of strategy ‘Sluices’, this could lead to enough reduction in the failure probabilities to meet the proposed norms.

7.2.3. TRANSLATE EFFECTS OF MEASURES TO ENTIRE SYSTEM ON A QUANTITATIVE MANNER

In chapter 6, on a qualitative manner the effects of strategy 'Sluices' are translated to other trajectories that found to be distinctive in terms of costs according to literature. The calculated reduction in failure probability for dike ring 16, provides enough perspective to conduct more research in beneficial effects on the failure probability within the entire Rhine-Meuse estuary. It should be investigated how the reduction in hydraulic load levels also lead to a reduction in failure probability for the mechanisms 'macrostability' and 'erosion outer slope' which are not investigated in this research as this mechanisms played no significant role for the total failure probability in dike ring 16. Trajectories for which strategy 'Sluices' could be an opportunity in significant cost reduction are amongst others 15-2, 22-2 and 14-1. On the other hand, with the current location of the complexes in strategy 'Sluices', trajectories 14-2 and 19-1 will be exposed to much higher hydraulic loads than within strategy 'DP2015'. These trajectories are now located behind the Maeslant barrier, which will be removed in strategy 'Sluices'. An optimization in costs for dike reinforcements for these trajectories compared to costs to locate the locks, sluices and pumping stations more seawards should be found.

7.2.4. MAKE SURE A REALISTIC PUMING CONFIGURATION WITHIN 'SLUICES' IS APPLIED

On behalf of *ir.* Spaargaren, the configuration in the alternative strategy is set up with pumping stations that are in operation any time $h_{sluices,riverside} < h_{sluices,seaside}$ and $h_{sluices,seaside} > 1.0m + NAP$. With an expected sea level rise of 0.85m + NAP in 2100, this practically means that during each high tide the pumping stations are in operation. This configuration leads to very low moderate water levels upstream of the pumping stations and generates favorable water levels for the reduction in failure probability for the piping mechanism. It is questionable whether it is feasible to operate the pumping stations during each high water. It is therefore recommended to verify that a realistic configuration of operation mode of the pumping stations within the alternative strategy is used.

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A

CLASSIFICATION

A.1. SYSTEM OVERVIEW

From the databases information about the location of dike sections and hydraulic structures is extracted. Together with a QGIS database about the location of dike trajectories, it is possible to assign each section and structure to a specified trajectory. In figure A.1 an overview is given of the trajectories within the area of interest. In figures A.2a and A.2b a visualization is given about the location of dike sections and structures with their accompanying dike trajectories. The program QGIS is used to set up the final distribution and allocation of the sections and structures to trajectories.

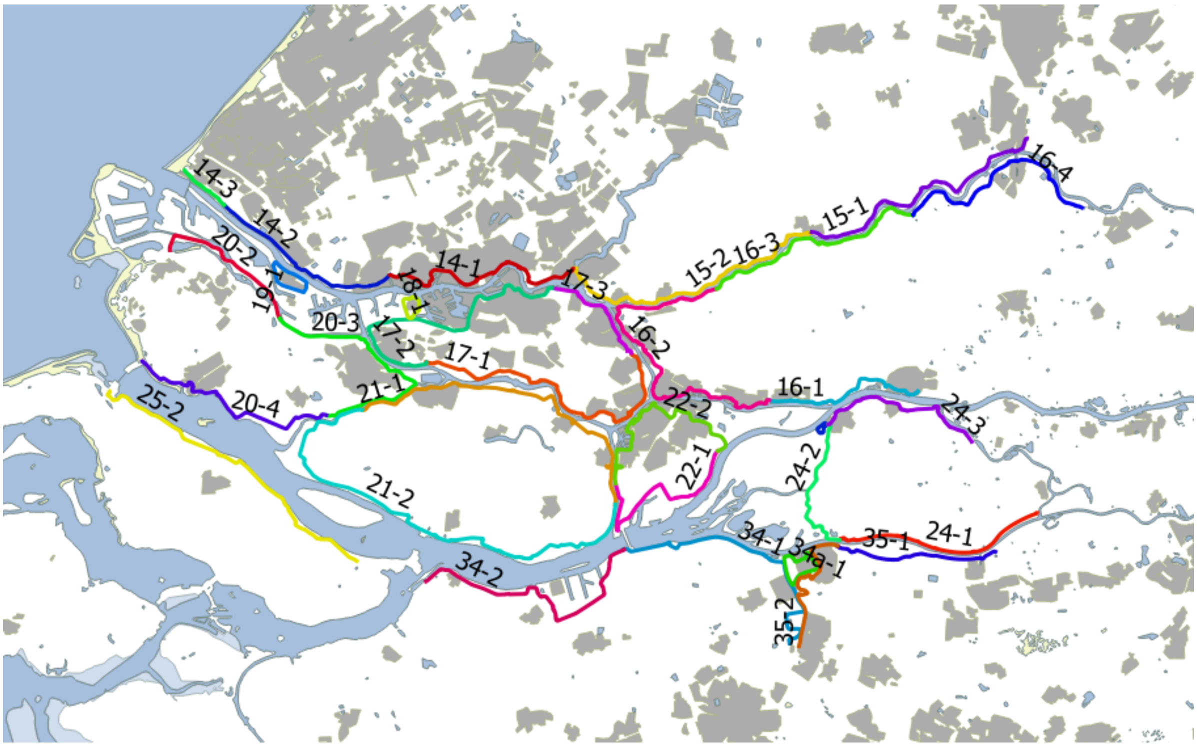
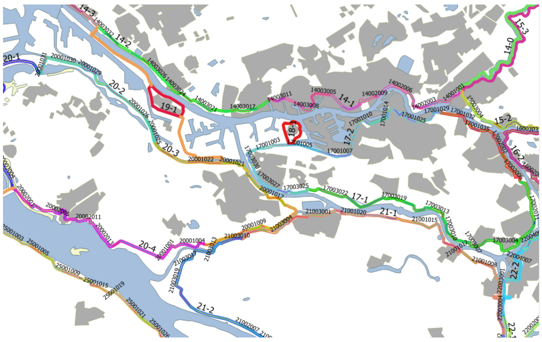
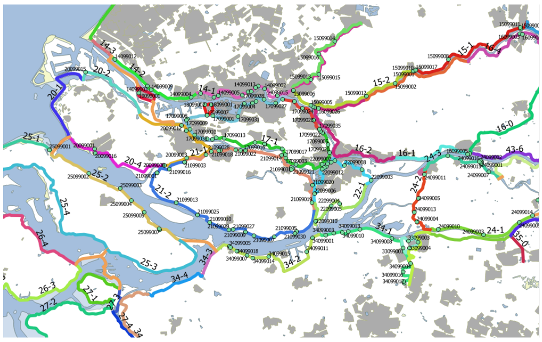


Figure A.1: Dike trajectories that are within the scope of this study

Regarding the structures a side note is made; Below figure A.2b shows all the structures, while not all of them are analysed in the VNK background reports. In line with the VNK analyses it is chosen to take the same structures into account as done in those reports and exclude the ones that earlier have to be found insignificant in the overall failure probability of a dike trajectory. It is remarked that this assumption is plausible for the 2015 scenario, but for the 2100 scenario the failure probability contribution of the excluded structures could or will be significant in the total failure probability.



(a) Sections



(b) Hydraulic structures

Figure A.2: Location of dike sections and hydraulic structures in dike trajectories according to new proposed standardizations

B

FRAGILITY

B.1. INTRODUCTION

This appendix describes step by step how failure probabilities for dike sections are calculated for the mechanism of piping by means of fragility. The fragility curve for piping $P(Z < 0|h)$ is a curve which expresses the failure probability P_f given water levels h . Combined with the probability density functions of water levels $f(h)$ the integration over the entire interval of water levels leads to the failure probability. In B.2 the composition of the fragility curves is elaborated and in B.3 the construction of probability density functions are stated. The final calculation of failure probabilities is explained in B.4.

B.2. COMPOSITION OF FRAGILITY CURVES FOR PIPING

In constructing the fragility curves, simplifications have been made and choices are made in order to compute realistic results. This section describes the steps undertaken for the construction of fragility curves and explains assumptions that have been made in the calculations.

B.2.1. VNK METHODOLOGY VERSUS APPLIED METHODOLOGY IN THIS STUDY

In order to compute the fragility curves, the dike profiles are extracted from databases of PC-Ring. These are identical to the used databases in VNK (Vergouwe, 2014). For VNK the processing of the databases is done via the software program PC-Ring. PC-Ring is an extensive statistical program that calculates failure probabilities of dike sections and dike rings, for several failure mechanisms (Steenbergen & Vrouwenvelder, 2003a). The program is capable of processing both the load and strength paramaters within the same calculation, making a full probabilistic calculation possible. Within PC-Ring one can manually determine the calculation method (for instance a Monte Carlo simulation or a FORM method) and analyse results afterwards. However, after a consultation with the software developer Steenbergen (2015), it became clear that the program is only capable of analysing failure probabilities of dike rings according to the *current* statistics. Hence it is impossible to calculate failure probabilities, taking into account level rise and different extreme river discharges.

As an alternative, a simplification of the calculation is made numerically of which the steps will be described next. At first the strength characteristics are calculated, resulting in a fragility curve per dike section. Next the occuring loads are translated to probability density functions, which enables to calculate the final failure probability.

B.2.2. COMPUTE FRAGILITY CURVE VIA REVISED SELLMEIJER FORMULA

In the database information of the geotechnical conditions is found in the table 'Stochasten_Sterkte_Data'. Here, for each dike section, up to 10 different piping scenarios are determined with different values. For each piping scenario, a single fragility curve is determined, by applying the formulas of section 2.5.4. Verification of the formula from (Schweckendiek & Calle, 2013) and the choice for the modelling of distribution function and standard deviation has taken place on 10 September 2015 in discussion with Raymond van der Meij, an expert of Deltares (Van der Meij, 2015). The chosen mean values and standard deviation are found in table B.1, including distribution type. Parameter h is used as variable from $h = 0m$ to $h = 10m$ with $\Delta h = 0.1m$.

The distributions are used for the computation of failure probabilities with the Monte Carlo method. The background of this method is described next.

Table B.1: Input parameters for computing fragility curves according to (Schweckendiek & Calle, 2013) and (Steenbergen & Vrouwenvelder, 2003b, p. 19)

Variable	Distribution	Mean value	Standard deviation
m_p	lognormal	1	0.08
h_b	normal	nom*	nom*
d	lognormal	nom*	nom*
L	lognormal	nom*	nom*
η	lognormal	0.25	0.0375
γ_s	normal	27	0.27
γ_w	deterministic	10	-
v	deterministic	1.33E-06	-
k	lognormal	nom*	nom*
g	deterministic	9.81	-
d_{70}	lognormal	nom*	nom*
d_{70m}	deterministic	2.08E-04	-
D	lognormal	nom*	nom*

In above table some values are stated with nom*, these values are computed from the input data as used in VNK, from the PC-Ring databases.

B.2.3. PROCESS PARTIAL PIPING SECTIONS

As described in Kanning (2012), the Dutch soil conditions have a high spatial variability in geotechnical properties and subsoil composition. A correlation length of about 300m is often found, indicating that the strength characteristics for separate cross sections with a spatial distance of 300m can be considered as independent from each other. Often dike sections have a length longer than 300m, so for the piping mechanism more cross sections are evaluated than one per dike section. In the database these are denoted as 'bodemvakken' or bottom sections, which are build up from one or more Piping scenarios (numbers 1 to 10), below table B.2 shows the build up of the seven bottom sections within dike section 16001001. For each of the 10 piping scenarios fragility curves are computed in case these were apparent in the database and next the fragility curve for each bottom section is constructed via the partial factors.

Table B.2: Structure of 7 bottom sections within dike section 16001001 with a percentage contribution per piping scenario

Vak	Bodemvak_ID	Pip1	Pip2	Pip3	Pip4	Pip5	Pip6	Pip7	Pip8	Pip9	Pip10
16001001	8097	0	0	85%	0	15%	0	0	0	0	0
16001001	8098	0	0	0	0	0	0	0	88%	0	12%
16001001	8099	0	0	88%	0	12%	0	0	0	0	0
16001001	8100	0	0	87%	0	13%	0	0	0	0	0
16001001	8101	0	0	88%	0	12%	0	0	0	0	0
16001001	8102	0	0	89%	0	11%	0	0	0	0	0
16001001	8103	0	0	91%	0	09%	0	0	0	0	0

B.2.4. CONSTRUCT FRAGILITY CURVE PER DIKE SECTION

In the last place, the normative fragility curve per *dike section* is determined by taking the maximum failure probability at each water level of the different *bottom sections*, see also figure B.1. Here, the normative fragility curve for dike section 16001001 is determined by taking the maximum failure probability of bottom sections 8097 to 8103 from table B.2 at each water level h. In the figure it looks like only the maximum of two lines is taken, but this has to do with the fact that the fragility curves for bottom sections 8097, 8099, 8100, 8101, 8102, 8103 are very similar to eachother, being built up from only piping scenario 3 and 5 with quite equal factors. This methodology is applied for the construction of all dike sections that are sensitive to piping in dike ring 16.

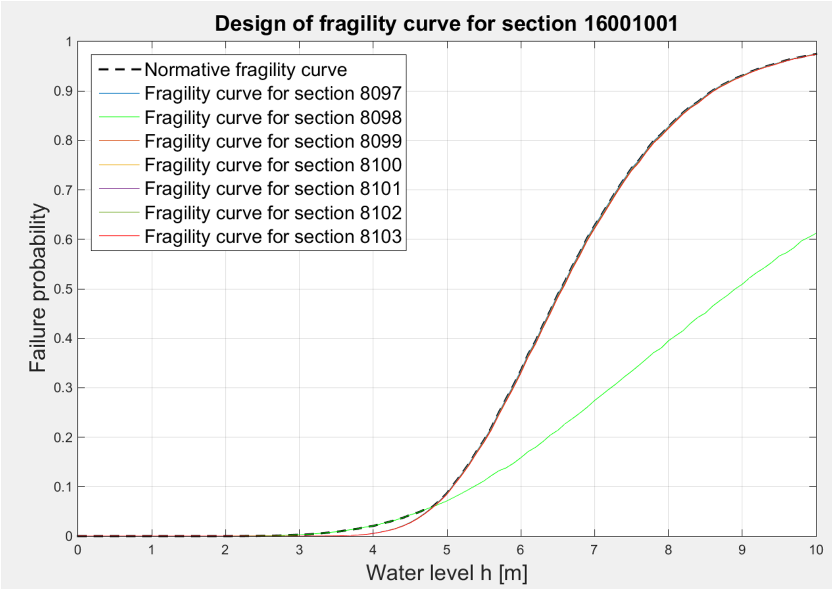


Figure B.1: Design of normative fragility curve for section 16001001

B.3. COMPOSITION OF PROBABILITY DENSITY FUNCTIONS

With the fragility curves described by the failure probabilities given water level h, the next step is to compute a probability density function $f(h)$ of h. The probability density function describes the probability density of certain water levels to occur over an interval of time [-/year]. The integral of the probability density function over a certain interval of h, states the probability [-/year] that a water level between this interval is reached.

B.3.1. LINK WATER STATIONS TO DIKE SECTIONS

The strength characteristics are modelled for each dike section at the location of the section. Each dike section is coupled to a water level station at which points the statistics for water levels are determined. In below figure B.2 it is shown how this link is made, in this figure for dike ring 22.

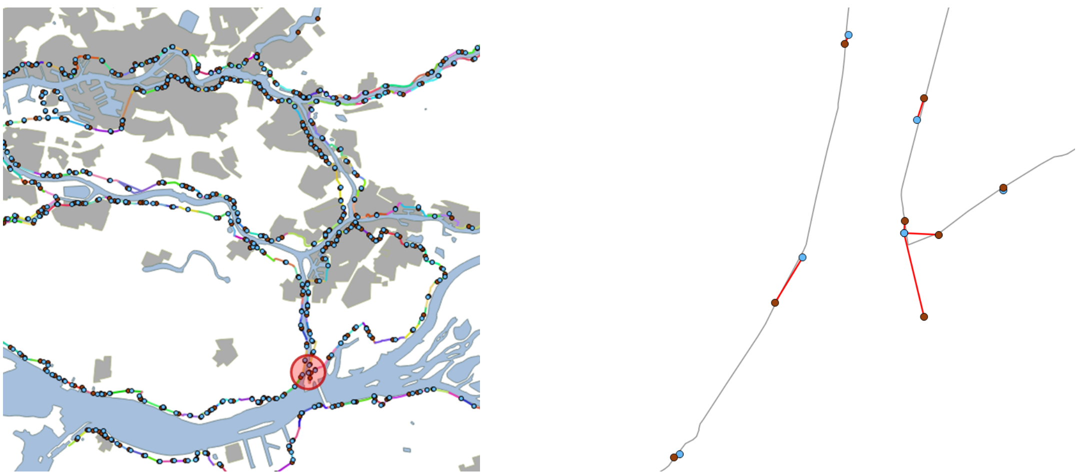


Figure B.2: Coupling of hydraulic data to dike sections, with in brown the dike section points, and in blue the hydraulic water stations

B.3.2. EXCEEDANCE FREQUENCY AND PROBABILITY OF WATER LEVEL H

For the water stations, water levels are computed with the program Hydra. For each water station, exceedance frequencies for at least 64 water levels with $\Delta h = 0.10m$ are computed. For water station 'Dkr 16 Boven Merwede km 959-960 Loc 6_121565_426779', the station coupled with dike section 16001001, the exceedance frequencies are stated in below figure:

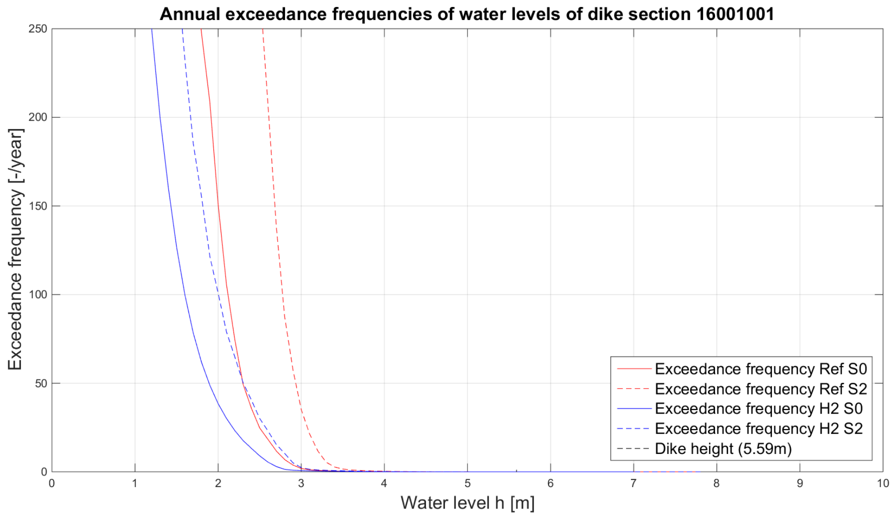


Figure B.3: Exceedance frequencies of normative water levels (MHW) for section 16001001, for the two strategies in 2015 and 2100

From the exceedance frequencies it is possible to compute the exceedance probabilities and later the non-exceedance probabilities. The derrivative of the non-exceedance probability function gives finally the probability density function. The exceedance probability can be found by the following formula **Referentie: document J.M. van Noortwijk**:

$$P(T \leq t) = 1 - exp(-f_{exceedance}) \tag{B.1}$$

Following for the non-exceedance probability:

$$P_{non-exceedance} = 1 - P(T \leq t) \tag{B.2}$$

Above function is also called the Probability distribution function with $F_x(X) = P(X \leq X)$. Below figures show the exceedance and non-exceedance probabilities for section 16001001:

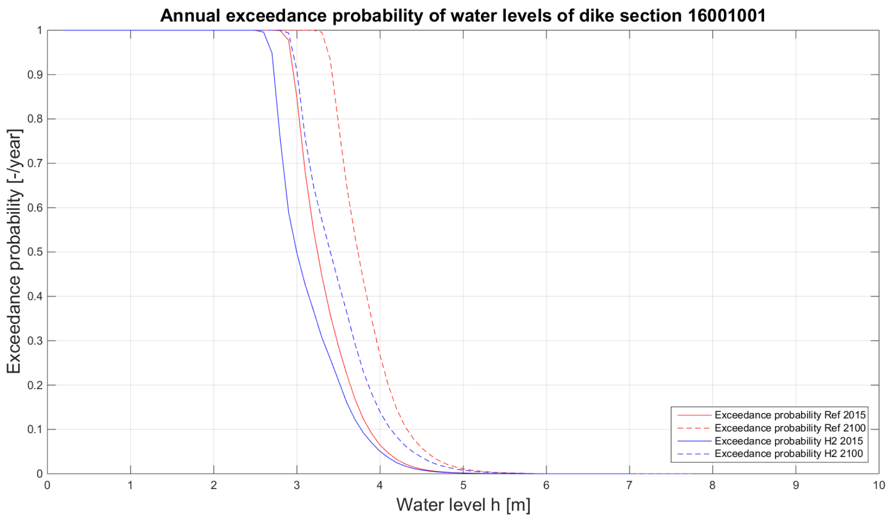


Figure B.4: Exceedance probabilities for section 16001001, for the two strategies in 2015 and 2100

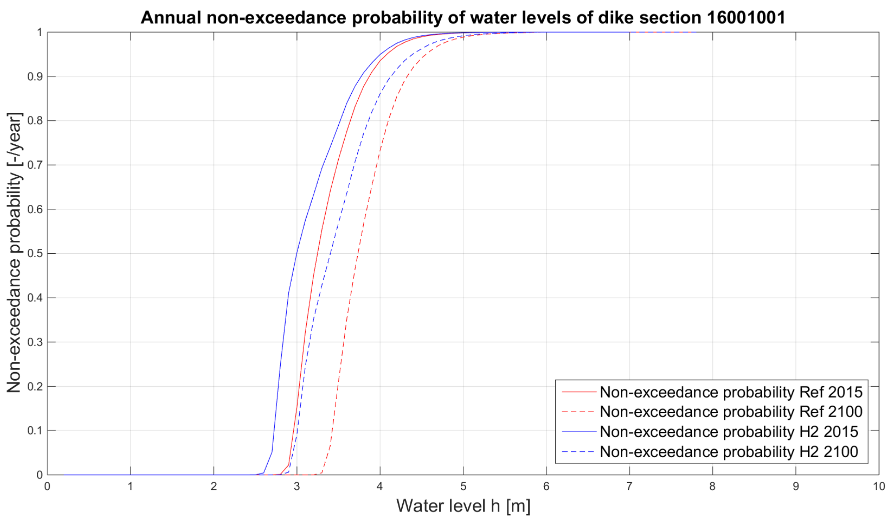


Figure B.5: Non-Exceedance probabilities for section 16001001, for the two strategies in 2015 and 2100

B.3.3. CONSTRUCTION OF PROBABILITY DENSITY FUNCTIONS

The derivative of the probability function gives the probability density function. It is defined as (CUR190, 1997, P 2-6):

$$f_h(H) = \frac{dF_h(H)}{dH}, H \in \mathbb{R} \tag{B.3}$$

The derivation of the probability function is done numerically, by means of the midpoint rule. It reads (Lecture notes computational modelling in flow and transportation):

$$\frac{y^{n+1} - y^{n-1}}{2\Delta h} = f(y^n) \tag{B.4}$$

By applying the midpoint-rule on the probability distribution function, finally the probability density function is found, as shown in figure 3.10. For the probability density function the following two formulas apply (CUR190, 1997, p. 2-6), saying that the probability density function may never have negative values (B.5) and the total integral the probability density function over the entire domain is equal to one (B.6):

$$f_h(H) \geq 0, H \in \mathbb{R} \tag{B.5}$$

$$\int_{-\infty}^{\infty} f(h)\,\mathrm{d}H = 1 \tag{B.6}$$

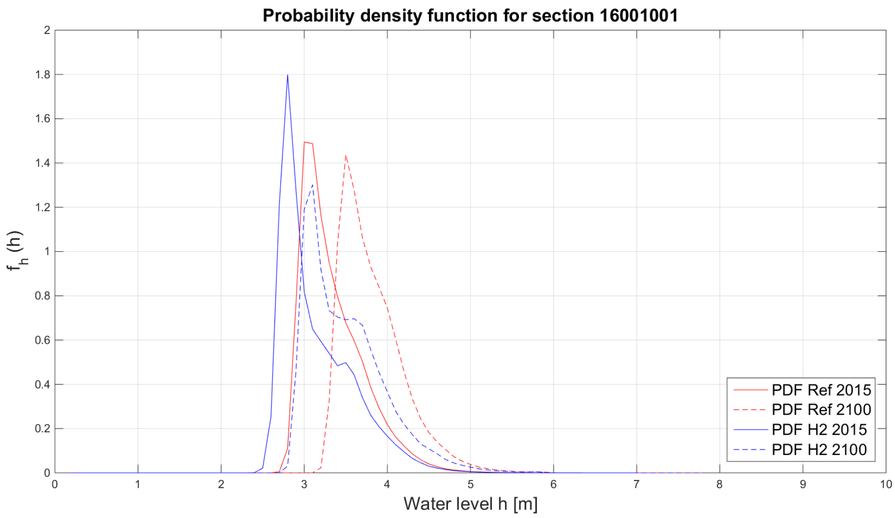


Figure B.6: Probability distribution function for section 16001001, for the two strategies in 2015 and 2100

POSSIBILITIES TO INCREASE THE ACCURACY OF THE PROBABILITY DENSITY FUNCTIONS

With respect to the load calculations, databases are used from T. Botterhuis as already mentioned. In the numerical computation of the probability density functions, rounding and truncation errors are aware, leading to a erratic pattern in some cases. Taking a smaller discretization in the space step Δh, would reduce this pattern. Furthermore in the summation of the probability density function with the fragility curves, more samples could also increase the accuracy of the failure domain. However, it would also lead to larger computation times in the Monte Carlo simulation. For this study the order of accuracy is assumed to be sufficient.

B.4. CALCULATION OF FAILURE PROBABILITY FOR A DIKE SECTION

The integration of the combined figure in which the probability density function is multiplied by the fragility curve over the range of water levels h, see equation B.7, leads finally to the overall failure probability of a dike section (figures B.7 and B.8):

$$P_f(z<0) = \int_{-\infty}^{\infty} \underbrace{P_f(Z<0|h)}_{\text{fragility curve}} \underbrace{f(h)}_{\text{pdf}}\,\mathrm{d}h \tag{B.7}$$

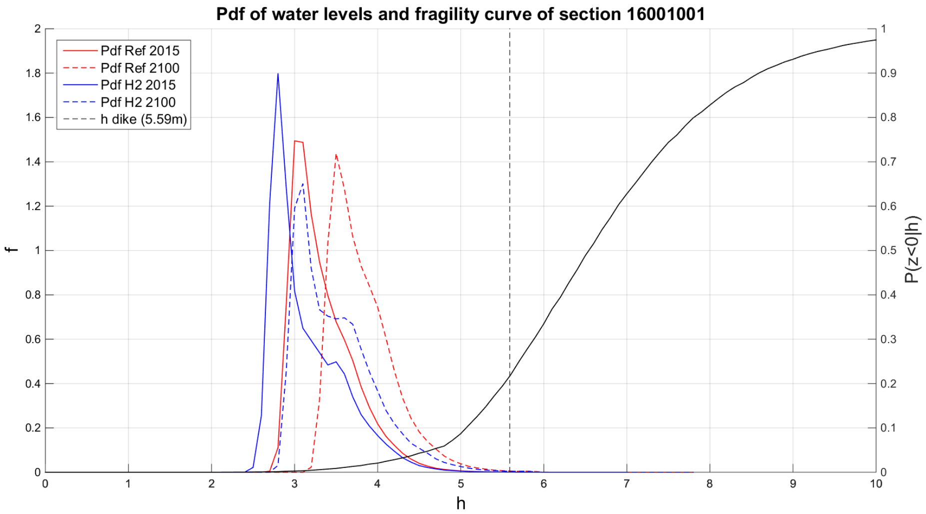


Figure B.7: Both pdf and fragility curve for section 16001001, for the two strategies in 2015 and 2100

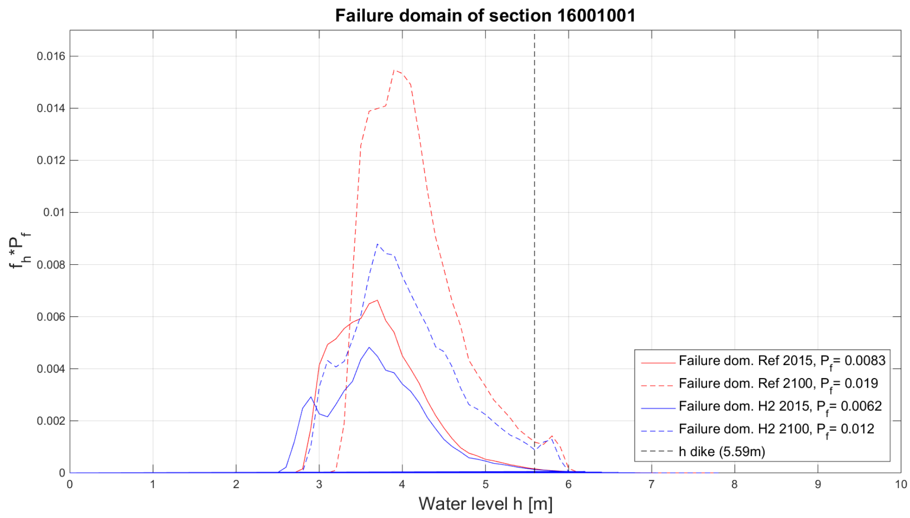


Figure B.8: Failure domain for section 16001001, for the two strategies in 2015 and 2100. The area under the graph is equal to the failure probability