Assessment of the consequences of higher safety standards for flood defences along rivers in The Netherlands

How to assess the reduction of the flood probability by different measures

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Challenge the future

Cover photo: Aerial photo of a flood on the Nederrijn River near Wijk bij Duurstede, February 1992
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Assessment of the consequences of higher safety standards for flood defences along rivers in The Netherlands

How to assess the reduction of the flood probability by different measures

By

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In partial fulfilment of the requirements for the degree of

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“De kennis van de voornaamste voorstellen tot rivierverbetering in vroeger jaren, alsmede hun voor- en nadeelen, blijft dan ook steeds belangrijk, en is haast onmisbaar, om grondig te kunnen oordeelen over de mogelijkheid tot afdoende verbetering onzer rivieren.”

“The knowledge of the main proposals for river improvements in earlier years, as well as their advantages and disadvantages, remains important and is almost indispensable to thoroughly judge the ability to adequately improve our rivers. “

– Cornelis Lely

Translated from “Strijd om de rivieren” (Heezik, 2006)

Preface

This thesis is submitted in partial fulfilment of the requirements for the degree of MSc in Civil Engineering at Delft University of Technology. The research was carried out in cooperation with HKV Consultants and supervised by Delft University of Technology. This report elaborates on how measures on river dike systems should be assessed and is aimed at scientists, engineers and anyone interested in the field of flood defences or river engineering.

I would like to thank my graduation committee for their guidance and support. Their knowledge and feedback was very helpful and it has steered me in the right direction. I would like to thank Prof.dr.ir. Matthijs Kok for chairing the committee and stimulating me to look further in depth as well in broader context. I would like to thank dr.ir. Saskia van Vuren for her unlimited enthusiasm and fair strictness and for giving me the possibility to look beyond the borders of my research. I would like to thank dr.ir. Erik Mosselman for his critical view and different theoretical angle on the subject and I would like to thank ir. David Kroekenstoel for the great care and commitment with which he has reviewed all my work.

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Jelle Jos van Zuijlen
Delft, June 2015

Executive summary

Since the major part of the Netherlands is, and has always been, vulnerable to floods, the Dutch people have an eventful history of flood protection. Since the large flood in 1953 a lot has happened in the regulations regarding the safety of flood defences. After completion of the first Delta Programme in 1960, several committees have been working on better and safer methods to design and measure the safety of dikes. In recent years the Room for the River (RfR) programme and Flood Risk in the Netherlands (FloRis) have introduced new ways to assess and improve the safety of river dikes. The latest developments regard the new Delta Programme 2015, which proposes new, in most cases stricter safety standards, based on the flood risk approach.

The newly proposed safety standards are based on the flood risk approach and are derived so that every person within embanked areas has a basic safety level of 10^{-5} per year. Also the societal financial risk and the risk of having large groups of casualties is taken into account. The main differences between the old and the new standards are:

- The new standards are based on probability of flooding instead of a design water level;
- The new standards are defined per dike reach, not per dike ring;
- The new standards have to be met per dike reach, not per dike section.

In 2050 all flood defences in the Netherlands will have to meet these safety standards. In the river area most dikes will have a safety standard of 1/10,000 or 1/30,000 per year, per dike reach of about 20-30 km.

FloRis calculated the present state of the dikes with PC-Ring, a probabilistic calculation model. The conclusion was that a lot of the dike sections do not meet the currently prevailing safety standards. They will also not meet the new standards in 2050 if no action is taken. The effect of climate change, which will lead to shorter return periods for extreme discharges, increases the difference between the present safety level of the dikes and the safety standards. This difference is called the design task.

The Delta Programme 2015 proposes a strategy for the improvement of the safety level of river dike systems. The climate change-induced part of the design task has to be resolved with spatial measures, which increase the flood conveyance capacity. The rest of the design task can be resolved with dike improvements.

Whether it is possible to resolve the climate change-induced part of the design task is not known yet. Current insights in the effectiveness of spatial measures focus merely on the water level reduction than on the actual reduction of the probability of failure. In FloRis an assessment has been made on the reduction of the probability of failure of dike ring 43 by the current RfR programme. The outcome of these calculations was that the reduction of the probability of failure by RfR is factor 1.3, which is insufficient to reach a significantly higher safety level. These calculations were made under the assumption that the effect of the measures is equal for all discharges. Since the effect will be maximal at the design flood and less for lower discharges, FloRis overestimated the effectiveness of the RfR programme.

Since there is no tool available to assess the effectiveness of measures, in terms of reduction of the probability of flooding, in reasonable detail without having to make complicated model calculations, a new tool has been developed during this research. This tool focuses on the statistics of failure of a dike rather than on the mechanical background of dike failure. The method is based on the theory that the probability of failure of a dike section can be built up out of three relations. The fragility curve, the probability of failure per water level, contains the strength properties of a dike section. The Q-h relation determines the relation between the discharge at Lobith and the local water level, and the Q-T relation determines the discharge statistics, i.e. the distribution of the hydraulic load over time. When these relations are combined one obtains the probability of failure per dike section. There are four failure mechanisms regarded (overflow/overtopping, macro stability, piping and damage and erosion of the outer slope) which all have their own fragility curve. By combining the individual probabilities of failure the probability of failure per dike reach is found.

The method has been tested by using input and output data from PC-Ring, and it was seen that the probability of failure as calculated with PC-Ring can be approximated with an average error of 1.13 per dike section and 1.20 for a reach of 16 km. The impact of climate change can be calculated with accuracy of 1.18.

This tool has then been used to calculate the effectiveness of several measures. The tested measures are three floodplain measures (side channel, excavation, dike repositioning), groyne lowering and dike improvements (raising, piping berm, combination). The measures are representative cases which are based on available model calculations, or based on engineering judgement. A spatial measure is simulated by changing the Q-h relation of a dike section. A dike improvement is simulated by changing the relevant fragility curve. Climate change is simulated by changing the Q-T relation.

The calculations show that spatial measures cause the largest reduction of the probability of failure of failure mechanism overflow/overtopping. When geotechnical failure mechanisms play a large role in a dike section or dike reach, spatial measures are not effective to reduce the total probability of failure, since the geotechnical properties are of more influence than the hydraulic load.

The effectiveness of spatial measures decreases after climate change, because climate change steepens and increases the probability density curve and a measure only shifts it towards lower water levels. Even if a set of spatial measures, which cause a reduction of the water level at MHW equal to the increase by climate change, is applied to a river stretch, this will not bring the probability of failure back to the original value.

Since the effectiveness of spatial measures has been overestimated, more measures will be needed to fulfil the climate-change induced task than was thought before. The preferred strategy of DP15 will therefore be more expensive and may not be feasible anymore. The recommended strategy is to fulfil the design task for geotechnical failure mechanisms by means of dike improvements. The design task for overflow/overtopping can then be fulfilled with a combination of spatial measures and dike improvements. Even though the effectiveness of spatial measures is less than was assumed, it is still recommended to combine spatial measures and dike improvements, since both types of measures contribute to the safety of the river system in another way, which can enhance each other.

Samenvatting

In een groot deel van Nederland is de kans op een overstroming erg groot. De Nederlanders hebben dan ook een rijke historie wat betreft het zichzelf beschermen tegen overstromingen. Naar aanleiding van de watersnoodramp in 1953 heeft de eerste Deltacommissie in 1960 de eerste regels en normen voor waterkeringen ingevoerd. In de jaren erna hebben verschillende andere commissies deze regels verbeterd. Programma's als Ruimte voor de Rivier (RvdR) en Veiligheid van Nederland in Kaart (VNK) hebben geleid tot nieuwe inzichten in hoe de overstromingskans kan worden verlaagd. Vorig jaar (2014) heeft het Delta Programma nieuwe, in veel gevallen strengere, normen voor waterkeringen voorgesteld.

De nieuwe normen voor waterkeringen zijn gebaseerd op de overstromingsrisicobenadering, en deze zijn zo afgeleid dat iedereen die binnendijks leeft een basisveiligheidsniveau heeft van 10^{-5} per jaar, dit is de kans dat iemand komt te overlijden als gevold van een overstroming. Ook het economische risico van een overstroming (MKBA) en het risico op grote groepen slachtoffers zijn hierin meegenomen. De belangrijkste verschillen met de huidige normen zijn als volgt:

- De nieuwe normen zijn gebaseerd op de overstromingsrisicobenadering en niet op het overschrijden van een waterstand;
- De nieuwe normen zijn gedefinieerd per normtraject, en niet per dijkring;
- Aan de nieuwe normen moet worden voldaan op trajectniveau, niet op vakniveau.

In 2050 moeten alle dijken en waterkeringen in Nederland aan deze normen voldoen. In het rivierengebied betekent dit dat de dijken een overstromingskans van 1/10.000 of 1/30.000 per jaar mogen hebben per dijktraject van ongeveer 20-30 km.

VNK heeft de overstromingskansen van alle dijkringen berekend met het probabilistische rekenmodel PC-Ring. Uit dit onderzoek bleek dat veel dijkvakken niet voldoen aan de huidige normen. Als er geen actie wordt ondernomen zullen de meeste dijken ook niet aan de nieuwe normen voldoen. Klimaatverandering zal ervoor zorgen dat het verschil tussen de huidige staat van de dijken en de normen nog groter wordt. Dit verschil wordt de ontwerpopgave genoemd.

Het Deltaprogramma voor 2015 (DP15) heeft een voorkeursstrategie (VKS) voor het verhogen van het veiligheidsniveau opgesteld. De klimaatopgave moet worden opgelost met rivierverruiming, en het deel van de ontwerpopgave dat volgt uit de nieuwe normen en achterstallig onderhoud moet worden opgelost met dijkversterking.

Huidige inzichten in de effectiviteit (= mate van verlaging van de faalkans) van rivierverruimende maatregelen hebben vooral gekeken naar de waterstandsverlaging dan naar de daadwerkelijke verlaging van de overstromingskans. VNK heeft een gevoeligheidsanalyse gedaan naar de faalkansverlaging door RvdR voor onder andere dijkring 43. De uitkomst hiervan was dat een reductiefactor van 1,3 RvdR de rivieren niet op een substantieel hoger veiligheidsniveau zou brengen. De berekeningen aan RvdR zijn gemaakt onder de aanname dat de waterstandsverlaging gelijk is voor alle afvoerniveaus. In werkelijk zal de waterstandsverlaging bij afvoeren lager dan MHW (Maatgevend hoogwater) afnemen, dus de effectiviteit van RvdR is op deze manier overschat door VNK.

Er is nog geen instrument beschikbaar waarmee de effectiviteit van maatregelen snel en met redelijke nauwkeurigheid kan worden afgeschat. Daarom is er voor dit onderzoek een speelinstrument ontwikkeld waarmee de verlaging van de faalkans van een riviertraject door verschillende maatregelen kan worden berekend. Dit instrument is gebaseerd op statistiek, en gaat er vanuit dat de faalkans van een dijkvak kan worden opgebouwd uit drie relaties: de 'fragility curve' beschrijft de kans op falen gegeven een waterstand, de Q-h relatie relateert de lokale waterstand *h* aan een debiet *Q* bij Lobith en de werklijn (Q-T relatie) beschrijft de kans op overschrijden van diezelfde rivierafvoer bij Lobith. Deze drie relaties bepalen de faalkans voor één van de faalmechanismen (overloop/golfoverslag, macro instabiliteit, zandmeevoerende wellen oftewel piping en erosie van het buitentalud). Alle faalkansen per dijkvak kunnen worden gecombineerd tot een faalkans voor een traject van meerdere dijkvakken.

Deze methode is geverifieerd aan de hand van rekenresultaten van VNK uit PC-Ring. De afwijking, uitgedrukt als de verhouding tussen beide uitkomsten, is gemiddeld 1,13 per dijkvak en 1,20 voor een traject van 16 km. Dit traject is gelegen op de Waal in normtraject 43-6. De impact van klimaatverandering kan ook redelijk nauwkeurig berekend worden; de afwijking is hier 1,18.

Met deze methode is de effectiviteit van verschillende maatregelen berekend. De beschouwde maatregelen zijn een nevengeul, uiterwaardvergraving, dijkteruglegging, kribverlaging en verschillende vormen van dijkversterking. Voor deze maatregelen zijn aan de hand van beschikbare modelresultaten en 'engineering judgement' representatieve varianten opgesteld. Ruimtelijke maatregelen worden gemodelleerd als een mutatie in de Q-h relatie, en dijkverbeteringen worden doorgevoerd door het verschuiven van de fragility curve. Klimaatverandering kan worden gesimuleerd door de werklijn aan te passen.

Uit de berekeningen volgt dat ruimtelijke maatregelen vooral effect hebben op de faalkans voor mechanisme overloop/golfoverslag. Wanneer de faalkans voor geotechnische faalmechanismen groot is hebben ruimtelijke maatregelen minder effect omdat de faalkans dan vooral bepaald wordt door de geotechnische eigenschappen van de dijk, en in mindere mate door de belasting.

De effectiviteit van ruimtelijke maatregelen neemt af na klimaatverandering. Klimaatverandering zorgt ervoor dat de kansdichtheid voor waterstanden waarbij falen kan optreden hoger worden, en bovendien dat de kansdichtheidskromme hier steiler wordt. Een ruimtelijke maatregel zorgt er slechts voor dat elk punt op de kansdichtheidskromme naar links (lagere waterstanden) verschuift, relatief aan de waterstandsverlaging. Zelfs als een maatregel de waterstand bij MHW net zoveel verlaagt als deze werd verhoogd door klimaatverandering is de kansdichtheid en daarmee de faalkans nog altijd hoger.

Omdat de effectiviteit van ruimtelijke maatregelen is overschat, zullen er meer maatregelen nodig zijn om de klimaatopgave op te lossen dan aanvankelijk werd gedacht. Dit kan ertoe leiden dat de VKS niet meer haalbaar of wenselijk is. De aanbevolen strategie is om de opgave voor geotechnische faalmechanismen op te lossen met dijkverbeteringen. De opgave voor hoogte kan dan worden opgelost met een combinatie van ruimtelijke maatregelen en dijkverbeteringen. Ook al zijn ruimtelijke maatregelen minder effectief dan werd gedacht, is het nog steeds aanbevolen wel ruimtelijke maatregelen te gebruiken, omdat deze verschillende voordelen kunnen hebben ten opzichte van dijkverbeteringen, en als beide maatregelen worden gecombineerd kunnen ze elkaar aanvullen.

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Symbol:	**Meaning:**	**[unit]**
a	Length-dependency per failure mechanism	[-]
b	Length of independent, equivalent dike sections	[m]
$F_R(\underline{X})$	Fragility curve	[-]
$F_X(\underline{X})$	Probability of non-exceeding of $\underline{X}$	[-]
$f_X(\underline{X})$	Probability density function of $\underline{X}$	[-]
h	Water level	[m+NAP]
$\underline{h}$	Specific value water level	[m+NAP]
IF	Impact factor	[-]
L_{reach}	Total length of considered dike reach	[m]
N	Factor for length effect	[-]
$P_{eis,i}$	Safety standard per dike section	[$year^{-1}$]
P_f	Probability of failure	[$year^{-1}$]
$P_{standard}$	Safety standard per dike reach	[$year^{-1}$]
Q	Discharge	[m^3/s]
R	Strength	[-]
RF	Reduction factor	[-]
S	Load	[-]
T	Return period	[year]
Z	Limit state function	[-]
ω	Contribution factor per failure mechanism	[-]

Abbreviations

In alphabetical order

Abbreviation:	**Meaning:**
DP15	Delta programme 2015
DT	Design task
ENW	Expertisenetwerk Waterveiligheid
FloRis	Flood Risk in the Netherlands (VNK)
FORM	First Order Reliability Method
HR2006	Hydraulische randvoorwaarden 2006 (Hydraulic boundary conditions 2006)
HWBP2	Hoogwater beschermingsprogramma deel 2 (Flood protection programme)
IF	Impact factor
MHW	Maatgevend hoog water (design flood)
MKBA	Maatschappelijke kosten-baten analyse
OI2014	Ontwerpinstrumentarium 2014 (Design rules 2014)
RF	Reduction factor
RfR	Room for the River
VNK	Veiligheid van Nederland in Kaart (FloRis)

1 Introduction

The vast part of the Netherlands is vulnerable to floods. The majority of the Dutch people lives in areas that are floodable, and the largest part of the Dutch economy is also established in these areas. Since the notorious storm surge of 1953, which gave cause to the establishment of the first safety standards, the knowledge has improved significantly. With this knowledge a transition is possible from safety standards based on an exceedance frequency towards a fully probabilistic approach. We are now able to assess both the probability and the consequences of a flood in detail and translate this to a safety level. The introduction of the flood risk approach will have severe effects on the standards for our flood defences as well as for the way in which they are tested and designed.

On Tuesday 16th September 2014, the annual 'Prinsjesdag', the Dutch government presented the outcome of the new Delta Programme for 2015 (DP15). It is the latest product of the everlasting struggle the Dutch lead against the water. Among other things the new Delta Programme contains a proposal to alter the safety standards for flood defences, based on the flood risk approach. The new, stricter, standards will be introduced in 2017. In 2050 all the flood defences in the Netherlands should comply with these standards.

The main rivers (Rhine branches and Meuse) are responsible for the largest part of the total flood risk of the Netherlands. Even though a lot of measures have been taken to improve the safety of the river dikes, still more than 300 km of these dikes do not meet the current safety standards. It is expected that this amount will increase with the introduction of the new standards. Together with higher peak discharges due to climate change and the increasing economic value and the dense population in the river area it is as clear as daylight that a lot of work has to be done to secure the safety of this area. There still remains one question: How?

1.1 Background information

1.1.1 Historical background of flood protection programmes in the Netherlands

After the large flood disaster in 1953 the Dutch government started working on measures and plans to prevent certain events from occurring again in the future. The first Delta Committee was formed and their work resulted in the first Delta programme. This plan prescribed the closure of the large estuaries which led to a shortening of the Dutch coastline of 700 km. Also the Delta Committee proposed a new way of looking at the safety of the Netherlands against flooding.

In 1960 the legal standards were introduced by the Delta Committee. These standards were based on economic risk and new insights in probabilistic techniques in hydraulic engineering. The way of thinking of the Delta Committee was based on probability of flooding and flood risk. Since they were not yet able to make detailed probabilistic design computations then, the new approach led to a design water level (basispeil) based on a frequency of exceeding of 10^{-4} per year for the coastline. This frequency was equal to the probability that a water level of N.A.P. + 5 m would occur at Hoek van Holland (Deltacommissie, 1960). Also for rivers and estuaries and for the other primary flood defences design water levels were given.

It took until 1977 before the regulations for river safety were further improved. The Becht Commission (Commission on river dikes) recommended that the safety requirements could be lowered from 1/3000 to 1/1250 per year. The commission proposed a modern integrated approach. In 1986 the dikes had to be raised again to a level equal to the former 1/3000 level, because new insight showed that the morphological effects in the river bed would cause higher water levels at the occurrence of the design flood. The Boertien I Commission (Commission on re-assessment of river dike reinforcement) proposed a different frequency distribution for extreme river discharges which resulted in an extreme discharge of 15,000 m^3/s at Lobith instead of 16,500 m^3/s. With this method the flood protection was held at the 1/1250 protection level (Weijers & Tonneijck, 2009).

In 1993 and 1995 two events of very high discharge occurred in the rivers Rhine and Meuse. These water levels led to near-disastrous floods. In the 1995 event 250,000 people and over a million animals had to be evacuated. This raised the awareness of the increasing danger of river floods. The high discharges also led to a raise of the extreme discharge (1/1250 per year) at Lobith to 16,000 m^3/s and 3800 m^3/s for the Meuse at Lith (Parmet, 2001). The discharge levels at Lobith and Lith are often used as starting point since this is the maximum amount of water that can enter the Dutch river systems.

Rather than just increasing the dike height to be able to handle this higher discharge the Dutch government decided to take a different approach. They decided that the water level at the peak discharge should be lowered instead, by increasing the flood conveyance capacity of the rivers. In order to reach this goal the Room for the River (RfR) programme was initiated. This programme contains more than 30 projects which contain primarily spatial measures. Spatial measures can be for example excavations in the floodplains, removal of hydraulic obstacles, dredging of the river bed or dike repositioning, all in order to give the river more room. These projects are to be finished in 2015 (Ruimte voor de Rivier, 2014).

1.1.2 Current safety standards

In The Netherlands the protection against flooding is defined by law. The level of protection is expressed as the probability of exceeding a threshold water level. The present safety approach was first defined in the Law on Flood Defences (Wet op de Waterkering) in 1996. This law defines for every dike ring the safety standard, presented as the probability of exceeding of the highest water level for which the flood defence should be designed. Areas with more economic risk and more inhabitants have a higher safety level. In 2009 the Law on Flood Defences was included in the Water Law, together with seven other laws. The Water Law regulates the management of surface and ground water and it improves the cohesion between water management and land use (Rijksoverheid, 2014).

In the Law on Flood Defences it is included that the water boards will assess the state of the flood defences every five years. To this end the design water levels are revised every same period, taking the latest records and insights into account. This results in the Hydraulic Boundary conditions. Water managers have to test their dike sections to the legal standards and corresponding boundary conditions and report the results to the government.

Because nowadays the knowledge on probabilistic calculation methods has improved, the current type of standards is old-fashioned. Besides this the risk has increased a lot since the old standards were established, because more people live in the areas that might be flooded and because the possible economic damage has increased significantly.

1.1.3 New safety standards

The first Delta Committee proposed a way of assessing the safety of dikes by means of a probabilistic method based on probabilities of flooding and flood risk, although they did not have the means to apply this. In 1992 the Expertise Network Flood risk (ENW, then TAW) started a programme to develop better calculation tools in order to make the quantification of probability of flooding and flood risk possible. Several studies helped develop the necessary calculation tools (Vergouwe, et al., 2014). After this the project Flood Risk in the Netherlands (FloRis, VNK in Dutch) started, which had as a goal to gain insight in the actual probability of flooding in the Netherlands, in the consequences of a flood and in the uncertainties that play a role in assessing this (Veiligheid Nederland in Kaart, 2005).

FloRis investigated the safety of the dike ring areas based on the actual probability of failure. It also functions as a basis for the new safety standards. The most important differences between the new and the current safety standards are:

- The new standards are based on probability of flooding instead of a design water level;
- The new standards are defined per dike reach, not per dike ring;
- The new standards have to be met per dike reach, not per dike section.

Furthermore the new standards are derived with the flood risk approach; both probability of flooding and the magnitude of the consequences define the safety standard. The standards are derived so that every individual within embanked areas has a maximum probability of being a casualty in a flood of 10^{-5} per year. This is called the basic safety level. Also the societal financial risk and the risk of having a large group of casualties is taken into account. (Ministerie van Infrastructuur en Milieu, 2014). The new safety standards will for most flood defences along the main rivers be stricter than the currently prevailing standards. For dike reaches in dike ring area 43 the new safety standards will be 1/10,000 or 1/30,000.

The newly proposed standards and assessment methods will be included in the law by 2017. After this a long series of dike improvements and spatial measures has to follow, ensuring that all dikes in the Netherlands meet the new standards by 2050. Currently most dikes do not meet these proposed standards so a lot of work has to be done to fulfil this task. More information about RfR, FloRis and the new standards is included in Appendix A.

1.2 Problem description

A large part of the river area does not meet the new safety standards. To ensure that the safety standards will be met everywhere in the Netherlands, a lot of work has to be done on the river systems. The difference between the present and the required safety state is called the design task. The Delta Programme 2015 prescribes the usage of “strong combinations” of traditional dike improvement and spatial measures. When the dikes are not strong enough, for example when they are unstable or when piping is a serious threat, this should be treated with dike improvement. When the design task results from higher peak discharges, as a result of climate change, spatial measures will be applied. It needs still to be specified to what extent this strategy is useable in terms of achieved reduction of the probability of failure by these spatial measures.

The Room for the River programme has obtained the objective regarding the water level reduction it intended. In FloRis an assessment has been made on the reduction of the probability of failure of dike ring 43 by the current RfR programme. The outcome of these calculations was that the reduction of the probability of failure by RfR is factor 1.3, which is insufficient to reach a significantly higher safety level (Vergouwe, et al., 2014). These calculations were made under the assumption that the effect of the measures is equal for all discharges. Since the effect will be maximal at the design flood and less for lower discharges, FloRis has overestimated the effectiveness of the RfR programme. An assessment needs to be done on the magnitude of this overestimation, by taking the water level reduction at lower discharges into account.

Before the preferred strategy can be translated into design rules, more insight is required in the effectiveness of spatial measures. Effectiveness is the reduction of the probability of failure of a dike section or dike reach induced by a measure. It is expressed as the ratio between the initial and the final probability of failure. When mentioning effectiveness other aspects of effectiveness, for example cost-effectiveness, are not taken into account.

However, at this moment no design tool is available that can be used to assess the effectiveness of measures in an easy way. If the effectiveness of a measure has to be calculated this takes a lot of modelling and extensive calculations, which require a lot of data and expertise, and take a lot of time. In order to be able to assess the effectiveness of different measures on the probability of failure of an arbitrary dike reach, it is useful to develop a tool that can make generic predictions on the effectiveness of these measures. This should be designed as a 'planning kit' (blokkendoos) in which one has several options to investigate different future scenarios. With this tool some insight can be achieved on how measures can be combined to reach the required safety level, and if the preferred strategy can be applied at all.

1.3 Objective and research questions

The objective of this thesis is the following:

> "To provide insight in the reduction of the flood probability by different measures, and how these measures can be used to reach the required safety level, by developing a rapid assessment tool based on the flood risk approach."

To reach this goal insight in the river system is needed. First of all the design task that is created by the new standards has to be defined. For this it is necessary to know the present probability of flooding and the proposed new safety standards, both expressed in terms of probability of failure per year. Information about this is documented in the FloRis results and the specification of the new safety standards.

To develop a well-founded advice on how the task has to be resolved it is necessary to gain knowledge of the potential of the RfR measures as well as traditional dike improvements. Instead of just assessing the water level reduction, the reduction of probability of failure should be calculated as well. In order to gain insight in the effectiveness of different measures a rapid assessment tool is developed. This tool calculates the probability of failure corresponding to the flood risk approach. The rapid assessment tool can also give insight in whether the effectiveness of spatial measures has been overestimated in previous studies like the FloRis programme.

The effectiveness of different measures on different failure mechanisms can lead to new insights in how measures can be combined. These insights are used to determine whether the strategy of DP15 can be used to reach the prescribed safety level, or that another strategy might be preferred.

These questions and objectives have been translated into four research questions:

1. **How can the difference between the current state and the new safety standards be assessed and transferred into a design task?**

2. **How can the effectiveness of different measures be assessed, according to the flood risk approach?**

3. **What is, for spatial measures and dike improvement measures, the effect on the total probability of flooding?**

4. **How can different measures be combined in order to reach the prescribed safety level?**

1.4 Scope and principles

There is a lot of work to be done on this subject. To make sure the goals are reached without falling into too much detail some boundaries have to be set.

The reference situation for the new standards is the situation in 2015-2020 when all the RfR measures and projects from the high water protection programme (HWBP2) have been completed. It must be noted that for the 2050 situation climate change is taken into account. The reference situation for the FloRis calculations however, is the situation before RfR and HWBP2. In order to be able to compare the calculations with the results of FloRis this will also be used as the reference situation for this research. Calculations for the reference situation are carried out with the discharge statistics as described in the Hydraulic Boundary conditions 2006 (HR2006). Calculations for climate change will be made with climate scenario W+2050 (KNMI, 2006), which is the scenario that is prescribed in the design rules for 2014 (OI2014). According to DP15 the design discharge will increase from $Q_{1/1250}$ = 16,000 m^3/s to $Q_{1/1250}$ = 17,000 m^3/s (Ministerie van Infrastructuur en Milieu, 2014)[1].

The total design task that follows from this is not univocal. It cannot be said that all the dikes will just meet the hydraulic boundary conditions that apply in the year 2050. Test rounds every 12 years will determine which dikes have to be improved (and which have priority), and which are still safe enough for the years to follow. For the sake of simplicity and uniformity the total design task will be defined as the difference between the required safety level in 2050 and the safety level that follows from the effect of climate change on the reference situation.

Since it is impossible to assess this for the whole Dutch river system within this research, focus will be on dike ring area 43. This dike ring area has high probability of failure for mechanisms piping

[1] Scenario W+2050 is different than the climate scenario used in DP15. Scenario W+ has a 1/1250 per year discharge of 18,000 m^3/s in 2050 and 20,000 m^3/s in 2100 (Deltares, 2011)

and macro stability (Vergouwe, et al., 2014) and therefore it is interesting to assess the effectiveness of different measures on this dike system. Calculations on measures will be done on a short model dike reach which is based on dike reach 43-6 on the Waal River.

This results in the following principles:

- The reference situation is the situation as used in FloRis: before RfR and HWBP2, and using discharge statistics as described in HR2006;
- Climate change will be taken into account following scenario W+2050, a physical maximum discharge (aftoppen) of 18,000 m^3/s will not be taken into account, since this is also not taken into account in the calculations with PC-Ring;
- The calculation methods as used in FloRis and the design rules from OI2014 (Rijkswaterstaat, 2014) will be used in order to guarantee consistency in the findings and enabling to compare the results with other researches;
- The total design task is the difference between the required safety level in 2050 and the safety level that follows from the effect of climate change on the reference situation when no action is taken until 2050;
- The magnitude of the water level reduction by measures for different discharges will be taken into account;
- The area of interest is the Waal river, especially the northern bank of reach 43-6 between Tiel and Gorinchem;
- Consequences will not be taken into account, only probability of failure will be assessed;
- Only primary flood defences which are part of a dike reach as defined by DP15 will be regarded;
- The new safety standards will not be validated, instead they are taken as given;
- Ground subsidence is not taken into account when assessing the composition of the design task;
- Measures regarding structures will not be taken into account.

1.5 Reading guide

Figure 1-1 shows the structure of this report. In chapter 1 and 2 the context of the research is described, and the problem is acknowledged. In chapter 3 the methodology is described which will be followed in order to find an answer to the research question. Chapter 4 defines the design task of the regarded dike systems. Chapter 5 elaborates the different measures that have been investigated, as well as some strategies in which they can be combined in order to fulfil the design task. The results of the calculations are presented and examined in chapter 6. Chapter 7 discusses the outcome of the research and provides answers to the research questions and objectives. Finally the conclusions are drawn in Chapter 8. Recommendations for further research will conclude this.

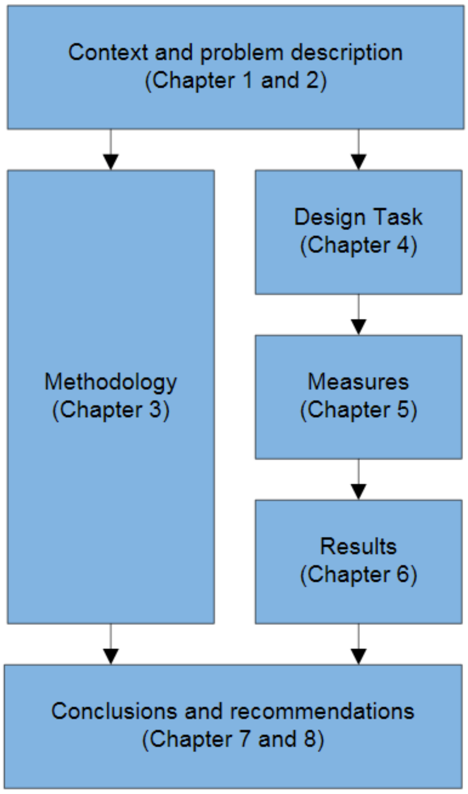


Figure 1-1: Structure of the report

2 Context

This chapter sets the context for this thesis. It gives a brief introduction to the failure of river dikes, by explaining some terminology and possible failure mechanisms of river dikes. This is followed by a brief description of the area of interest, the Waal River in dike ring area 43. Also the software package PC-Ring will be discussed.

2.1 Definitions

Before proceeding with the technical content of this report some definitions have to be established. The correct interpretation of these definitions is necessary to interpret the results correctly. Also, some definitions may slightly differ from the commonly used meanings.

Climate change
The way in which the discharge statistics change over time caused by climate change. In this thesis it is the difference between the discharge statistics as defined in HR2006 and the discharge statistics according to climate scenario W+2050. Climate change is dimensionless, but its effect can be expressed as the impact it has on the probability of flooding of a river system (see section 4.1).

Design task
The amount of work that has to be done before a dike section or dike reach meets with the safety standards, taking changes by climate change into account (see section 4.1).

Dike reach
A series of dike sections which make up part of a dike ring. The failure probability of dike reaches is defined as the safety standard.

Dike ring
A system of flood defences and high grounds, which surrounds a dike ring area and that provides protection against floods (Vergouwe, et al., 2014).

Dike section
A part of a flood defence with homogeneous strength and load properties (Vergouwe, et al., 2014).

Effectiveness
The reduction of the probability of failure of a dike section or dike reach induced by a measure. It is expressed as the ratio between the initial and the final probability of failure. When mentioning effectiveness other aspects of effectiveness, for example cost-effectiveness, are not taken into account.

Failure
Failure occurs when a system cannot function anymore, or when it cannot meet with the established criteria. The system cannot fulfil its primary function anymore. Failure of a flood defence means that it loses its water retaining function and that as a result thereof a flood occurs (Vergouwe, et al., 2014). For flood defences (and structures in general) distinction can be made between the ultimate limit state, which will lead to a flood, and the serviceability limit state, in which damage and disruption occur but no flood. In this report, when failure is referred to, this

means the ultimate limit state, unless explicitly stated otherwise. A dike will then break through or so much water will flow over it that flooding occurs (Rijksoverheid, 2014).

Failure frequency
The failure frequency is the unit in which the safety standards are defined. It describes the average number of failures of a system in a certain time span. In this case it is the probability of failure per year (Rijksoverheid, 2014).

Failure mechanism
The series of events that leads to failure. The failure mechanisms regarded in this thesis will be described in section 2.2 (Rijksoverheid, 2014).

Flood risk
The probability that an area is flooded, due to failure of the flood defence that is protecting it combined with the consequences of such an event. High consequences lead to high risk. (Rijksoverheid, 2014)

High waters
Situations where the discharge is so high that the floodplains start flowing along (around Q=7500m^3/s for the regarded dike reach). In case of mentioning 'low high waters' this indicates that the floodplains are only just starting to flow along.

Impact
The increase of the probability of failure of a river system, e.g. because of climate change. It is expressed as the ratio between the final and the initial probability of failure.

Measure
Procedure in which a change is made to a river system, with the purpose of reducing the probability of flooding of the system. Measures can include operations on the dikes, operations in the floodplains, dredging of the river bed, removal of hydraulic obstacles etc. In general there are two types of measures: spatial measures and dike improvements.

Measure – dike improvement
Dike improvements aim to reduce the probability of flooding of a river system by increasing the strength of the dike. This can be achieved by raising the crest height of the dike, or by improving the geotechnical strength of the dike. The latter can consist for example of the construction of a piping berm.

Measure – spatial measure
Spatial measures aim to reduce the probability of flooding of a river system by reducing the hydraulic load on the dikes. This can be achieved for example by excavations in the floodplains, the construction of a side channel, or reducing the hydraulic roughness of the system by lowering the groynes (if present).

MHW: Design flood (Maatgevend hoogwater)
The water level of discharge with an exceedance probability of 1/1250 per year is the official definition. In this research it is the water level that belongs to a discharge of 16,000 m^3/s in the reference situation and 17,000 m^3/s in 2050.

PC-Ring
This is a probabilistic model in which probabilities of failure can be calculated for different failure mechanisms for dikes, dunes and structures. These probabilities of failure can be combined to probabilities of failure per dike section, dike reach or dike ring. PC-Ring has more possibilities like calculating the probability of flooding scenarios to occur. FloRis calculations have been made with PC-Ring (Vergouwe, et al., 2014).

Probability of failure
Probability of failure is the probability that in a dike section or dike reach failure will occur in one year. It is factually the reciprocal value of the return period.

River area
The area of the Netherlands in which the main rivers flow. This contains the river Rhine, Meuse, Waal and IJssel.

Safety standard
This is a legally imposed requirement for the minimal safety level for a dike, expressed as the probability of failure per year for which the flood defence should be dimensioned.

2.2 Failure mechanisms

Dikes can fail in many different ways. The most well-known mechanism is overflow and wave overtopping, in which water actually flows over the crest of the dike. This failure mechanism is directly related to water level. But also failure mechanisms that are more stability-related can be a large threat to a river dike. These are also related to water level but failure can already occur at water levels far below the crest height. The failure mechanisms that are considered in this thesis are:

- Overflow and wave overtopping
- Macro instability
- Piping and heave
- Damage and erosion of the outer slope
- Failure of structures

These are the same as used in FloRis, in order to be able to compare the results. The failure mechanisms are described below, as well as the methods used to calculate the probability of failure (Steenbergen, et al., 2007).

2.2.1 Overflow and wave overtopping
This failure mechanism leads to collapsing of the dike because of large quantities of water flow over the dike. The inner slope will fail because the amount of water is larger than it can withstand. Failure can occur due to limited strength of the revetment of the inner slope or due to saturation.

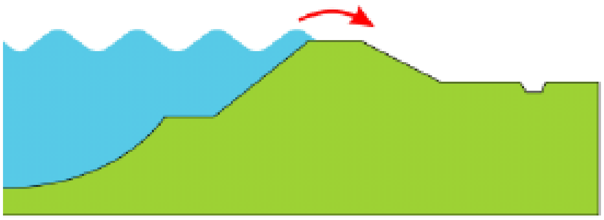


Figure 2-1: Failure mechanism overflow and wave overtopping (Vergouwe, et al., 2014)

Failure of revetment of the inner slope

In case of small waves (offshore directed wind) collapsing is induced by the partial mechanism overflow. In other cases the mechanism wave overtopping is dominant. The failure mechanism overtopping occurs if at a certain location the waves and the water level cause a water flow over the dike that is larger than the crest and inner slope can withstand. Erosion will occur, after which a breach can occur and the hinterland will be inundated. The critical discharge is calculated with the CIRIA formulae. CIRIA is a calculation method that calculates the strength of the dike and expresses it as a critical discharge, based on the strength of the grass revetment. This strength depends amongst others on the duration of the storm or flood during which water flows over the dike (Steenbergen, et al., 2007).

Saturation and sliding because of saturation

In the mechanism saturation the dike collapses because the inner slope gets saturated by overflowing or overtopping water and subsequently sliding occurs. These are two separate mechanisms which both have to occur to cause failure. Saturation occurs when the overflowing or overtopping discharge is higher than the critical discharge for saturation. This critical discharge is defined so that the inner slope will get saturated within the duration of the flood. The principle of the partial mechanism sliding is that the effective shear stress of the soil will be reduced when the ground gets saturated. This reduces the internal shear angle, so sliding will occur when the tangent of the inner slope of the dike is larger than the critical shear angle.

2.2.2 Macro instability

This failure mechanism occurs when a part of the dike is unstable and slides off, followed by dike failure. This can occur both on the outer and inner slope, but generally only the inner slope is taken into account. The calculations are done with a probabilistic stability analysis with computation model MPROSTAB.

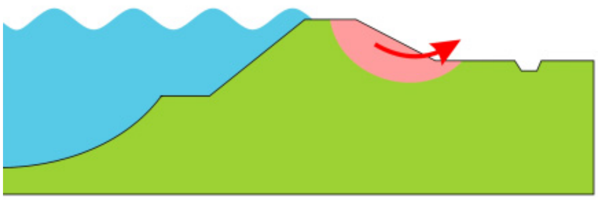


Figure 2-2: Failure mechanism macro stability (Vergouwe, et al., 2014)

Sliding is calculated with Bishop's slope stability analysis method. This method uses equilibrium of moments and equilibrium of vertical forces in the separate parts (Verruijt, 2001). After sliding it is assumed that failure also occurs. There is thus no residual strength. MPROSTAB is based on this method (Steenbergen, et al., 2007).

2.2.3 Piping and heave

This failure mechanism induces failure of the dike because the soil under it flows away. Water pressure will lift the closing layer (heave), after which pipes can occur. These pipes will flow out the sand particles which will lead to failure of the dike. Failure will thus only occur when both mechanisms occur. The guidelines from TAW-B (Calle & Weijers, 1994) and TR Zandmeevoerende wellen (Technische Adviescommissie voor de Waterkeringen, 1999) are used to describe this mechanism.

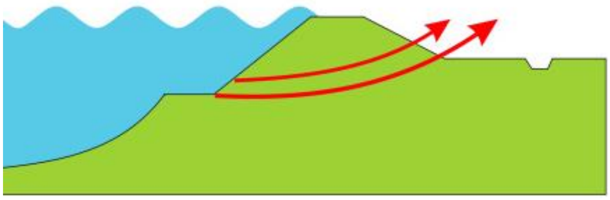


Figure 2-3: Failure mechanism piping and heave (Vergouwe, et al., 2014)

For the partial mechanism heave the water pressure will lift the water retaining layer. The maximum pressure can be expressed in terms of a critical water level difference. Heave will occur when the difference between the outer water level and the water level in the seepage ditch exceeds the critical water level difference.

For the partial mechanism piping, water transporting pipes will occur if the pressure of the outside water becomes too high. The maximum pressure the sand layer can bear can also be expressed as a critical water level difference. The dike will fail due to piping when the difference between the outer and inner water level, minus the vertical seepage length, exceeds the critical water level difference. If the difference is a little lower, pipes may also occur but they will not lead to failure. Several calculations can be used to determine the probability of failure due to piping, for example Sellmeijer's formula, which is used in FloRis. This failure mechanism will further be referred to simply as 'piping'.

2.2.4 Damage and erosion outer slope

With this failure mechanism the dike collapses because the revetment on the outside slope is damaged first by wave attack and after this erosion finds its way through the dike. Again this mechanism has two partial mechanisms; damage of the revetment and erosion of the inner dike.

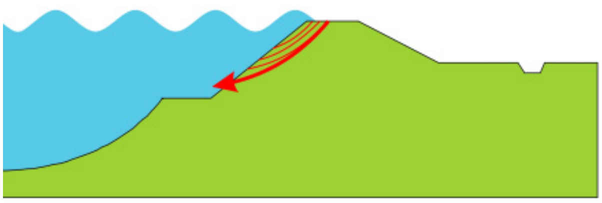


Figure 2-4: Failure mechanism damage and erosion outer slope (Vergouwe, et al., 2014)

For grass revetments both mechanisms can be combined. The dike fails if the storm duration is long enough to damage the revetment and flush the rest of the inner dike away. For rock revetments or asphalt the partial failure mechanisms should be assessed separately.

2.2.5 Failure of structures

Structures can fail in different ways, comparable to the failure mechanisms of dikes. The failure mechanisms for structures are:

- Wave overtopping or overflow
- Piping
- Failure in closing of the structure
- Structural failure

This research excludes measures on structures, so the failure mechanisms of structures are taken into account altogether.

2.3 Area of interest

It is not achievable to assess the whole river system within this research, so only a small part will be assessed. The area of interest lies on the southern boundary of dike ring area 43 (Figure 2-5). A short stretch of the Waal will be used to make detailed calculations (from now on: model dike reach). This section describes the location of the dike ring area and the model reach. Also the present safety state will be assessed.

Figure 2-5: Location of dike ring area 43 in the Netherlands

2.3.1 Dike ring area 43

Dike ring area 43: Betuwe, Tieler- en Culemborgerwaarden lies within the provinces of Gelderland and Zuid-Holland. It is surrounded by the Nederrijn and Lek in the North, the Pannerdensch kanaal and the Nederrijn in the East, the Waal and Boven-Merwede in the South and the Diefdijklinie in the West. The current safety standard for this dike ring is 1/1250 per year. The total length of the primary flood defences is about 170.8 km. The dike ring area covers 66,000 ha surface and it has 330,000 inhabitants. Some large infrastructure is present, for instance highways, railways and canals. The Waal River covers most of the southern part of this dike ring area. This thesis will focus on a stretch of the Waal River.

Safety level

The total dike ring area currently has a probability of failure higher than 1/100 per year (Vergouwe, et al., 2014). This is mostly due to the fact that the probability of failure for failure mechanism piping is very large for some sections. Chapter 4 will elaborate the safety level in more detail.

2.3.2 Model dike reach

The reach that is used for calculations is part of dike reach 43-6 in the Waal River. It exists of 16 dike sections, with different probabilities of failure and different failure mechanisms. An overview of the reach is shown in Figure 2-6.

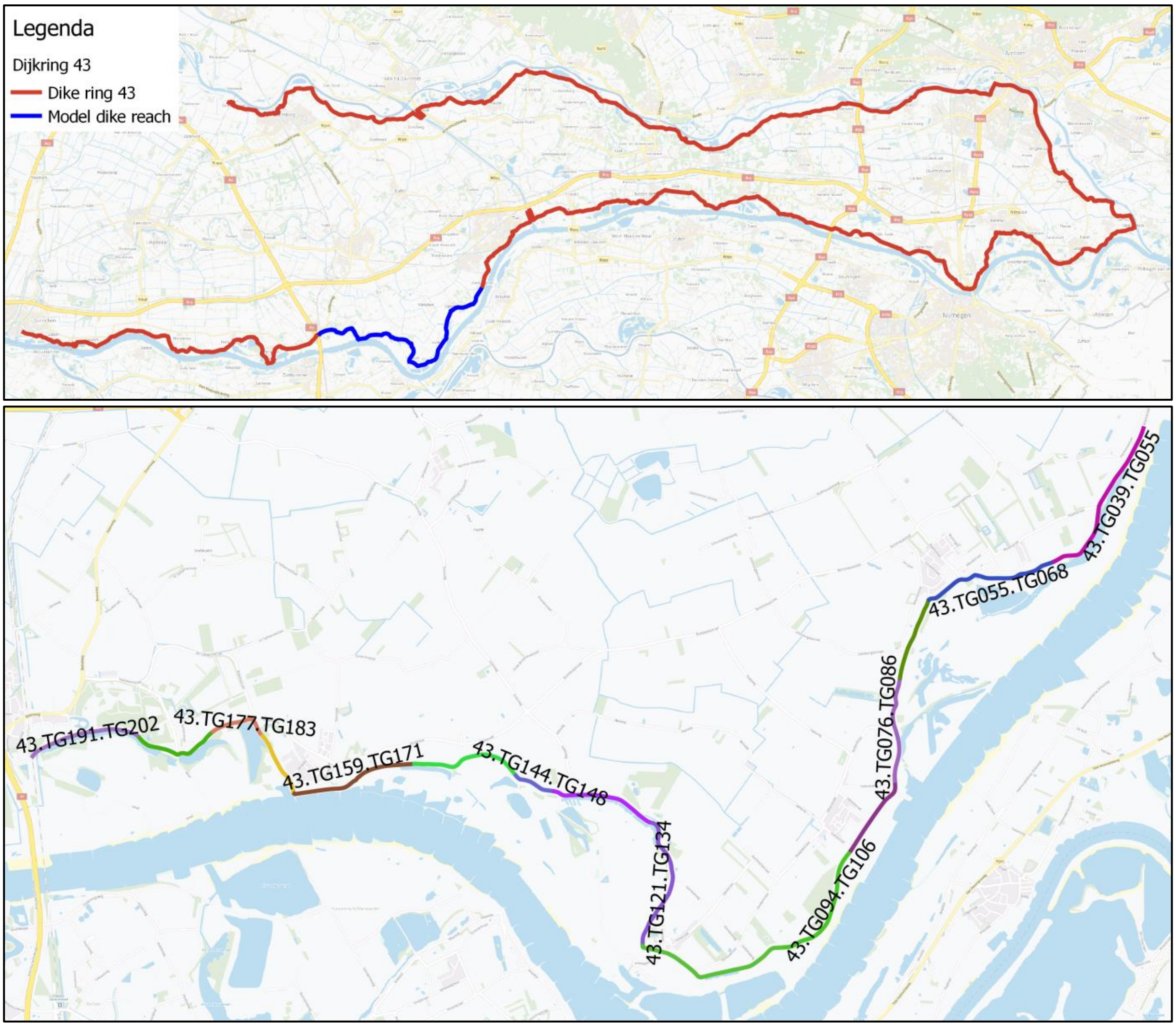


Figure 2-6: Upper picture: position of model dike reach within dike ring area 43, Lower picture: Overview of model dike reach (Source: Q-GIS)

The reach runs between Waardenburg (downstream) and Zwennewijnen (upstream) and is just over 16 km long. It contains several bends and the orientation of the winter dikes relative to the river axis also differs a lot. The probability of failure also differs per dike section and the dike sections have different failure mechanisms. The reach as such has been chosen for its variability, and it contains the sections with the highest probability of failure for overtopping, macro stability and piping of dike reach 43-6, see below.

Safety level

The safety level per dike section is shown below in Figure 2-7. The numbers have been calculated with PC-Ring. The total probability of failure of this dike reach is 1/156 per year. The figure shows that there are three sections with outstanding probability of failure. The dike reach has been selected so that there are some sections with high probabilities of failure in it. The reach contains the sections with the highest probability of failure for overtopping, piping and macro stability. The section left in the figure is also the section with the highest probability of failure of dike reach 43-6. The chosen reach contains no structures. This reduces the number of failure mechanisms that has to be considered and thus makes the calculations easier and less extensive.

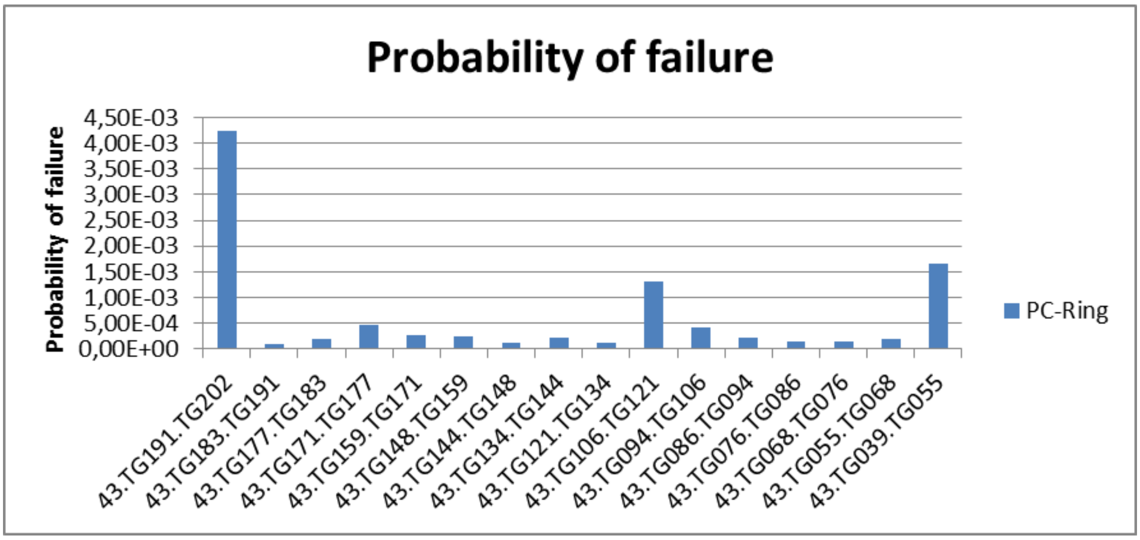


Figure 2-7: Safety level of the model dike reach

2.4 PC-Ring

PC-Ring is a probabilistic model which can calculate the probability of flooding of a dike ring area. The flood defence that is considered can be built up with dike sections, dunes and structures. The program calculates the probability that one element fails as a result of one of the failure mechanisms.

The input for a PC Ring calculation consists of extensive databases with information about the geometry of the dike sections, soil properties, water level statistics, discharge statistics, wind statistics and much more parameters that are needed to calculate the failure probability of a dike section or structure. The data is retrieved among others from dike examinations by water boards, flow calculations and soil measurements. Some failure mechanisms can be calculated within PC-Ring, but for example sliding of the inner slope is calculated outside PC-Ring with MPROSTAB. The results of these calculations are also input data for PC-Ring and this is incorporated in the total probability of failure.

Every dike section is schematized with one characteristic cross section. For this cross section the failure probability is calculated with a probabilistic method. Which probabilistic method is used can be selected. For this report all calculations are performed using the First Order Reliability Method

(FORM) calculation. The failure probability of the cross section is then translated to the failure probability of the dike section. All the probabilities of failure of all the dike sections and for all the separate failure mechanisms are eventually combined into the probability of failure for the whole dike ring. Dependencies between dike sections and failure mechanisms are taken into account. It is also possible to do this calculation for a part of a dike ring, for example a dike reach, by selecting only the dike sections that are considered.

PC-Ring is a calculation tool, not a design tool for measures. It is particularly suitable to calculate the present state of the dikes, or make small calculations on the effects of for example a water level reduction or a change in discharge statistics at Lobith or Lith (because of climate change). Also some optimization is possible, for example for the crest height. Since PC-Ring needs a lot of input data it is not a very handy tool to make design calculations with in an early stage.

The databases used in this research are the databases that have been used in the FloRis programme. These databases are based on the situation before RfR. Since the goal of FloRis was to determine the probability of failure of dike rings and dike reaches by means of the flood risk approach, some assumptions and simplifications have been made. For example failure mechanism piping is not considered for all dike sections. Instead a selection of 58 of all 158 dike sections in dike ring area 43, which are decisive for this failure mechanism, is made (Vergouwe, et al., 2014). This has been done to limit expenses and calculation time. This gives a proper representation of the probability of failure of the dike ring. However, when modifications are made to this calculation, for example when the climate scenario is changed, this may affect the accuracy of the calculation. This has to be taken into account.

2.4.1 Necessity of new tool

To assess the effectiveness of measures on the probability of failure of a flood defence, it is necessary to possess a tool in which these calculations can be made very quickly and easily, and with some precision. The Dutch government is working on a new set of design tools, but these will not be finished before 2017.

To be able to predict and assess the effectiveness of different measures generically, a new tool has been developed to be used in this research. With this tool it is able to estimate the effectiveness of measures on the probability of failure, when the present state of the dike or flood defence is known. The tool is based on the thought that the probability of failure is described by only three parameters. By altering any one of these, measures and external changes can be simulated and the impact on the probability of failure can be estimated.

The calculations that can be conducted with this tool are merely generic, so it is not a complete design tool. But it can give a quick insight in the type of measure that is most favourable for the considered situation.

3 Research methodology

In order to assess the effectiveness of measures a rapid assessment tool has been developed. This chapter describes the theory which this tool is based upon. Also a brief description of the use of the tool is given, and the calculation results are validated with PC-Ring calculation results.

3.1 Theoretical background

In order to understand the new method a brief introduction will be given on the statistical principles that are used and the input parameters and processes that play an important role.

3.1.1 Definitions

The following principles and functions are of importance for the used methodology (Kuijper, et al., 2002):

$Pf = P(failure)$ Probability of failure

Failure occurs when a series of events occurs that cause the flood defence to be unable to fulfil its primary task; retaining high water. (Partial) collapse will be the result.

$F_X(\underline{X}) = P(X \leq \underline{X})$ Probability of non-exceeding

The probability that a random variable X has a value equal to or lower than $\underline{X}$. $F_X(\underline{X})$ is also the probability distribution function.

$f_X(\underline{X}) = P(X = \underline{X}), X \in R$ Probability density function

The probability mass function for a continuous variable. This describes the probability that the variable X will attain the specific value $\underline{X}$. The probability density function is the derivative of the probability distribution function:

$$f_X(\underline{X}) = \frac{dF_X(\underline{X})}{dX}, \quad \underline{X} \in \mathbb{R}$$

Also, for some probability distributions the probability distribution function can only be presented as:

$$F_X(\underline{X}) = \int_{-\infty}^{\underline{X}} f_X(t)dt$$

$P(B \mid A)$ Conditional probability of failure

The probability that event B occurs, given that event A has occurred.

$P(A \cap B) = P(A)P(B \mid A)$ The probability that two events both occur

3.1.2 Failure and limit state

Failure occurs when the load on a dike exceeds the strength. This load can be for example the water level, and the strength in this case would be the crest height of the dike. If the water level exceeds the crest height water will flow over the dike and the dike no longer fulfils its water retaining function. This is an example of failure. It must be noted that this does not necessarily leads to destruction of the dike or occurrence of a breach.

The situation just before failure is called the limit state. This can be expressed with the so-called limit state function (or reliability function):

$$Z = R - S$$

In which:

- R = Strength (Résistance)
- S = Load (Sollicitation)

The situation where Z=0 is called the limit state. The probability of failure is the probability that R<S. The reliability is the probability that the limit state is not exceeded. This can be expressed as:

$$Pf = P(Z \leq 0) = P(S \geq R)$$

$$P(Z > 0) = 1 - Pf$$

3.1.3 Fragility curve

In hydraulic engineering, the fragility curve shows the performance of the flood defence as a function of the water level. For example, the probability of failure of a dike in relation to the local water level. Such a fragility curve for a dike section on the Waal River is given for the failure mechanism overflow and wave overtopping (Figure 3-1). The probability of failure can thus be described as:

$$F_R(\underline{h}) = Pf(\underline{h})$$

In which:

- $\underline{h}$ = Specific water level [m+NAP]

This is a <u>conditional</u> probability of failure. The condition is the water level. The probability of failure (Z≤0) can therefore be written as:

$$Pf(\underline{h}) = P(Z \leq 0 \mid \underline{h})$$

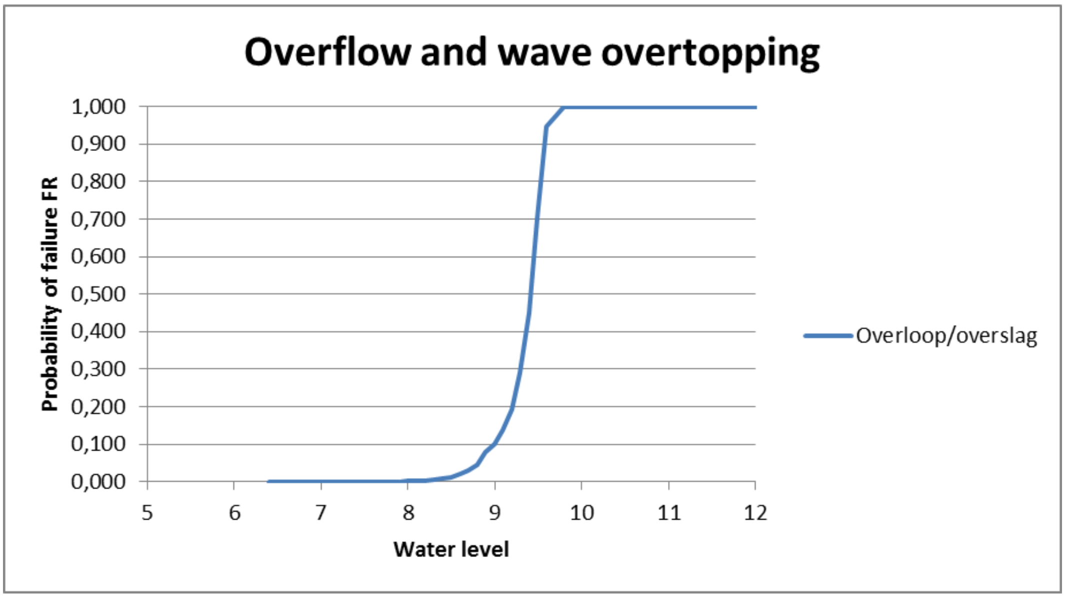


Figure 3-1: Fragility curve

These fragility curves represent the strength of a dike or dike section regarding a certain failure mechanism. For all dike sections and all failure mechanisms fragility curves can be derived. Together they represent the strength of a dike. The fragility curve in Figure 3-1 shows clearly that when the water level is higher, the probability of failure increases. This is the case since the mechanism overflow is solely dependent on the height of the dike. This specific mechanism consists of two parts: overflow and wave overtopping. The fragility curve is very steep around 9.5 m. This appears to be the crest height of the dike. The increasing probability of failure at water levels of 8 m and higher is an effect of the partial mechanism wave overtopping, which will occur at water levels slightly below the crest level of the dike, and is a result of wind statistics.

Fragility curves are retrieved from the PC-Ring software (how this is done is explained in Appendix B). The presented value is the cumulative probability of failure, so the probability that a certain failure mechanism occurs for a water level equal to or lower than the presented water level.

3.1.4 Discharge statistics

The occurrence of a discharge is a random variable and therefore it can be approached statistically. If a certain discharge has occurred in year one, this gives no information about the discharge levels of the next year. The discharge statistics are often expressed as the probability per year that the discharge is exceeded.

For the Netherlands the discharge statistics are often described as the discharge that enters the Netherlands near Lobith since this is the total discharge that enters the Dutch Rhine branch system. The discharge statistics are dependent of the climate scenario that is used. The discharge statistics used in PC-Ring for FloRis is presented in Figure 3-2. This is the discharge statistics as it is currently prevailing in the Netherlands (HR2006). It can be seen that the discharge with a return period of 1250 years lies at 16,000 m^3/s.

Discharge statistics as used in HR2006 are based on measurements of floods, statistical extrapolation of the return periods and flow model calculations. Many different discharge statistics are available, which are based on expected climate change (see section 4.1.2 Climate change).

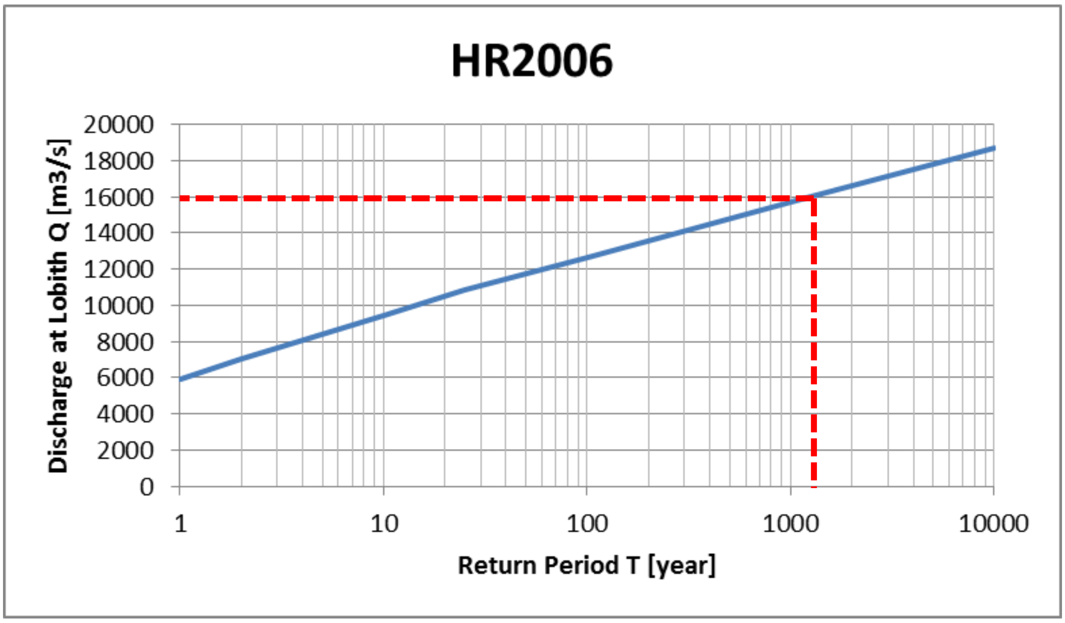


Figure 3-2: Discharge statistics used for FloRis

3.1.5 Water level statistics

Water level statistics are generally presented with a probability of exceeding the water level per year. An example of water level statistics is given below in Figure 3-3. The water level statistics are related to the discharge.

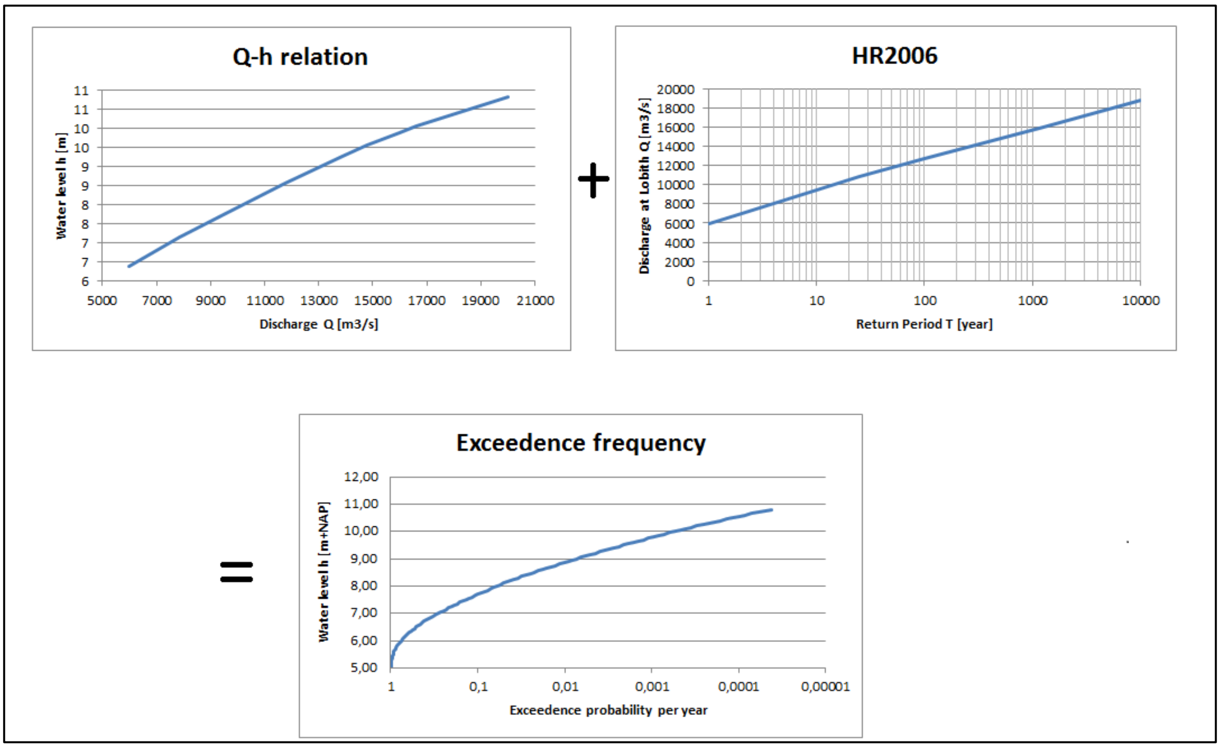


Figure 3-3: Relation between discharge, water level and return period

Water level statistics are, unlike discharge statistics, presented on a local scale. On every location along a river stretch the cross-section is different, so the water levels will vary over the length of a stretch for equal discharges. The water level can be presented as a function of the discharge, this is called the Q-h relation (with $Q=Q_{Lobith}$ and $h=h_{local}$). The water level then depends on the discharge at Lobith, since this is the location for which the discharge statistics are known. It also depends on the distribution of the total discharge over the separate river branches. The relation given in the top left graph in Figure 3-3 is an example of a local Q-h relation.

If one knows the applicable discharge statistics and the Q-h relation for the location, the water level statistics are easily retrieved. How this is done can be seen in Figure 3-3. For example, for the return period of 10 years the discharge is equal to 9460 m^3. From the upper left graph it can be read that the water level at this discharge is about 7.7 m. The same exercise is done for all return periods and water levels and the result is the lower graph.

The water level can also be expressed as the probability density function (Figure 3-4). The difference between the lowest graph in Figure 3-3 and the probability density is that the first figure describes the probability of *exceeding* a certain water level ($P(h > \underline{h})$) and the probability density describes the probability that a certain water level *occurs*. The relation between the exceedance frequency and the probability density can be described with:

$$f_h(\underline{h}) = \frac{dF_h(\underline{h})}{dh}$$

Where:

- $F_h(\underline{h})$ = Probability of non-exceedance ($P(h \leq \underline{h}) = 1 - P(h > \underline{h})$)

Note that $F_h(\underline{h})$ is the probability of non-exceedance of the water level and that $F_R(\underline{h})$ is the fragility curve!

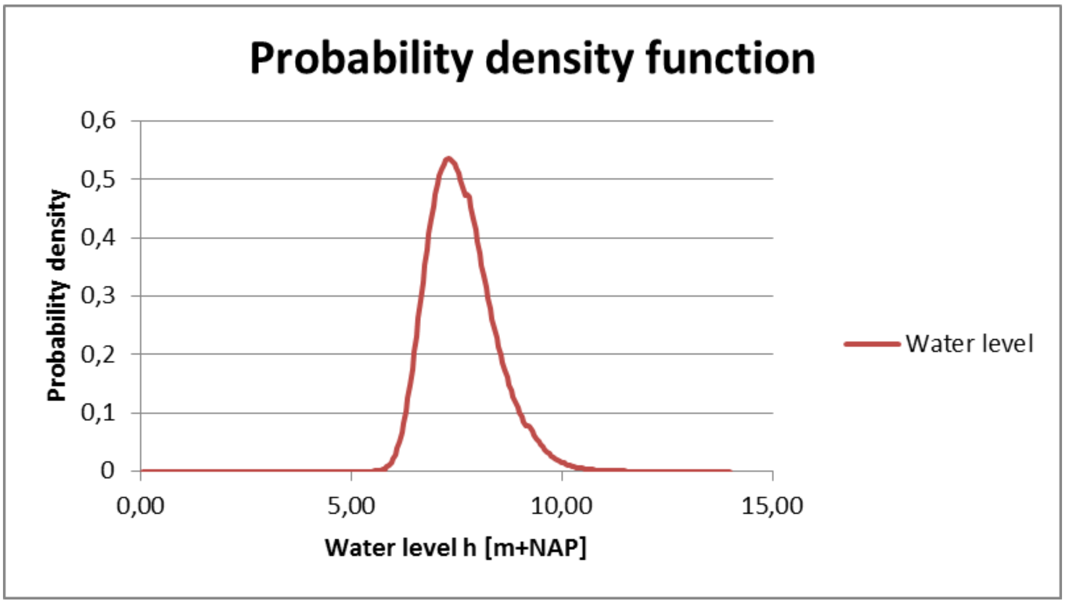


Figure 3-4: Probability density function

3.1.6 Relation

The probability of failure given a water level can be combined with the probability of occurring of the same water level. If this is integrated over the total range of water levels one gets the total probability of failure.

If the probabilities are correlated, combining these two probabilities can be done by multiplication. In this case the probabilities are correlated because they both relate to the water level. So for any given water level, the product of the conditional probability of failure and the probability of occurring of the condition (water level) give the total probability of failure. This can be described as follows:

$$P\left(Failure\ and\ water\ level\ h\right) = P(failure \cap h = \underline{h}) = P\left(Z \leq 0 \mid h = \underline{h}\right) \cdot P(h = \underline{h})$$

$P\left(Z \leq 0\right)$ is the probability of failure (see above) since the load is higher than the strength. This probability is given by the fragility curve (Figure 3-5). The probability of failure can thus be described as:

$$P\left(Z \leq 0 \mid h = \underline{h}\right) = Pf\,(\underline{h}) = F_R\left(\underline{h}\right)$$

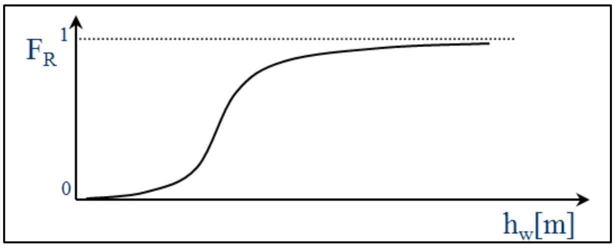


Figure 3-5: Fragility curve (Veiligheid Nederland in Kaart, 2007)

$P(h = \underline{h})$ is the probability that the water level is equal to a certain condition. This probability is given by the probability density function (Figure 3-6), so it can be described as:

$$P(h = \underline{h}) = f_h\left(\underline{h}\right)$$

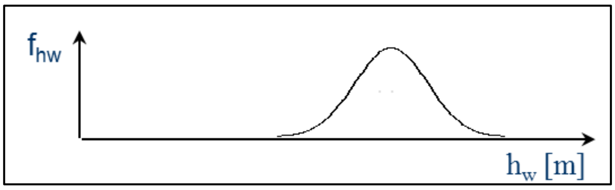


Figure 3-6: Probability density curve (Veiligheid Nederland in Kaart, 2007)

Together this gives for the probability of failure that:

$$P\left(Z \leq 0 \mid h = \underline{h}\right) \cdot P(h = \underline{h}) = F_R\left(\underline{h}\right) \cdot f_h\left(\underline{h}\right)$$

This is just the probability of failure for one water level. If the total probability of failure has to be known this should be integrated over all the water levels (Veiligheid Nederland in Kaart, 2007):

$$P_f = \int F_R(\underline{h}) * f_h(\underline{h})\, dh$$

This integral describes the surface beneath the graph in Figure 3-7 which equals the total probability of failure.

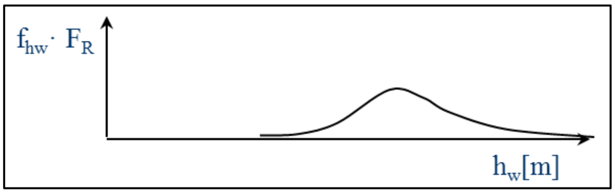


Figure 3-7: Product water level statistics and fragility curve (Veiligheid Nederland in Kaart, 2007)

So in fact there are three relations that determine the total probability of failure: the Q-T relation, the Q-h relation and the fragility curve. The water level statistics describe the load on the flood defence, and the fragility curve represents the Z-function. The strength is determined by the geometric and soil properties of the dike. If either one of these relations is altered the total probability of failure changes.

3.1.7 Alterations

Measures to the river system can be modelled by changing the fragility curve and the discharge. Spatial measures induce a change in the Q-h relation. Dike improvements change the strength of the dike and thus the fragility curve. Changes in the Q-h relation can be calculated with water flow calculations or in more detail with models like WAQUA. Fragility curves for mechanisms overflow and wave overtopping can be easily estimated and calculated since this will mostly cause a shift of the fragility curve towards the high water levels. Fragility curves for more complex mechanisms can be calculated with ground mechanical software packages like MPROSTAB.

The discharge statistics can also be altered by assuming different climate scenarios (see section 4.1.2). This will lead to a change of the Q-T relation and thus to a change in the h-T relation. Changing the discharge statistics will not affect the Q-h relation. In contrast to changes in the fragility curve or the Q-h relation, climate change is not something we can control or even influence.

Of course changing the climate scenario should be done with a certain caution. Climate scenarios are relatively uncertain since they are mostly based on extrapolations and assumptions. Choosing a climate scenario could lead to an over- or underestimation of the probability of failure. Therefore this uncertainty should always be taken into account when assessing the probability of failure.

3.2 Model build-up and validation

The new model calculates the probability of failure per dike section and per failure mechanism. For every calculation the Q-h relation, the prevailing fragility curve and the discharge statistics are required. These can be retrieved from the databases of FloRis by using PC-Ring.

Q-h relation

The Q-h relation, which describes the relation between $Q=Q_{Lobith}$ and $h=h_{local}$, is different for every location in a river. This is caused by geometrical differences. In the PC-Ring input databases every dike section has several locations for which the Q-h relation is known: hQ-stations. It is however mandatory to choose only one of these stations to be representative for the whole dike section. There is a distinction in Q-h relation along the river banks and in the river axis. Q-h relations from adjacent stations will have similarities, although nearby bank and axis stations may have large differences due to the presence of floodplains or groynes. The Q-h relations consist of 9 discharge levels, so in order to be able to make calculations a fit has to be made through these points.

Fragility curves

The fragility curves give the probability of failure per water level. These are different per dike section and also per failure mechanism. The fragility curves can be made with two different probabilities of failure: per year and per block duration (12.4 hours). Both are factually wrong but give a reasonable estimation of the actual probability of failure. For failure mechanism overflow/overtopping and macro stability the probability per block is used, for piping and damage and erosion outer slope the probability of failure per year is used. Why this gives good results is extensively described in Appendix B.

Discharge statistics

The FloRis calculations are made with the discharge statistics as described in section 4.1.2. The reference calculations are made with HR2006, which is the same discharge statistics as used in FloRis.

By combining the Q-h relation with the discharge statistics in the way described above the water level statistics can be retrieved. The fragility curves and water level statistics per dike section are discretized per water level between 0 m+NAP and 14 m+NAP with steps of 0.05.m, since the crest heights of all dike sections lay beneath 14 m+NAP. How the Q-h relation and fragility curves are retrieved from PC-Ring and how they are combined to the total probability of failure is described in Appendix B.

3.2.1 Model input and output

The calculations are made with a Matlab model. The Matlab code is included in Appendix E. Required data is automatically retrieved from Excel files, which contain per dike section the fragility curves and Q-h relations. The model offers a few choices to be made for the calculation:

- Dike section: The dike section that has to be calculated. It is also possible to compute a series of dike sections. The dike sections need to be specified beforehand in a database which contains information about the fragility curves and the Q-h relations;
- Failure mechanism: The model calculates one failure mechanism at a time. Choice can be made out of the failure mechanisms overflow/overtopping, macro instability, piping and heave and damage and erosion of the outer slope. These failure mechanisms have been described in section 2.2;
- Discharge statistics: Choice can be made between HR2006 and W+2050, see section 4.1.2;
- hQ-location: The calculation is made with the Q-h relation along the bank or in the river axis;
- Interpolation method: The method with which the Q-h relations are fitted. Choice is made between 2nd order fit, 3rd order fit and linear interpolation;
- Measure: The measures that can be chosen from are side channel, excavation of floodplains, repositioning of dikes, groyne lowering and dike improvement. It is also possible to calculate without a measure, this will give the probability of failure of the reference situation.

The recommended settings are:

- hQ-location = bank;
- Interpolation method = linear interpolation.

Why these settings give the most reliable results is explained in Appendix B.

The model gives output per dike section and per failure mechanism. This means that for a dike section with 3 failure mechanisms 3 different calculations have to be made. The output consists of a probability of failure, the difference between the model outcome and PC-Ring and two figures with plots of the fragility curve, Q-h relation, discharge statistics, probability density curve of the water level and the combined probability density curve of the probability of failure. In case of a spatial measure being calculated it shows extra plots for the Q-h relation, fragility curve and probability of failure. An example of these figures is shown in Figure 3-8.

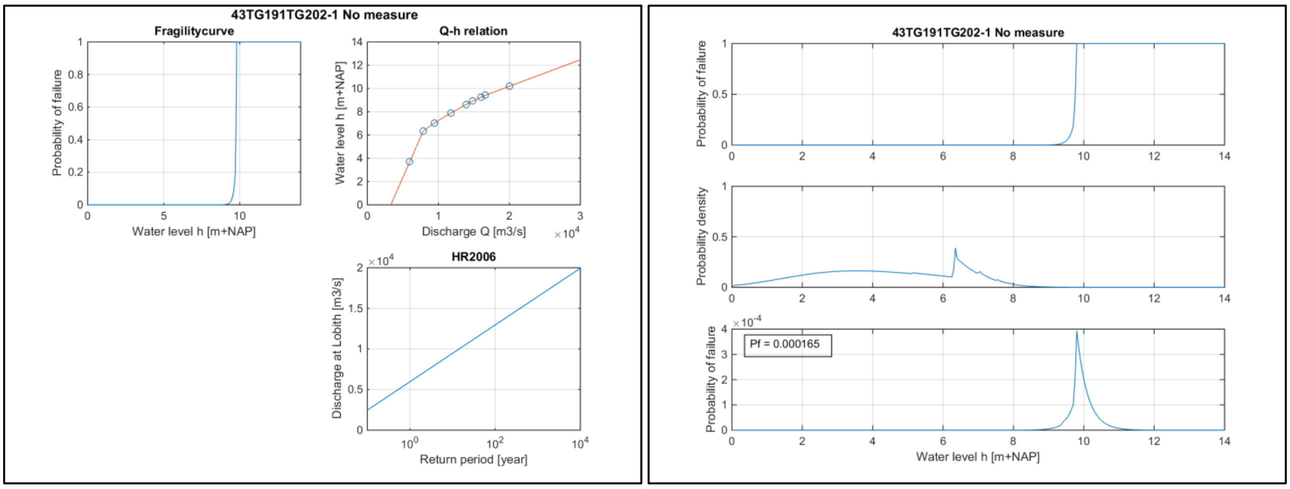


Figure 3-8: Output of Matlab model, dike section 43.TG191.TG202

The upper right plot in the left part of Figure 3-8 shows that there is a kink in the Q-h relation around Q = 7500 m^3/s. This kink is the locations where the floodplains start to flow. When the discharge increases, the water level increases less fast, due to the increase of width.

3.2.2 Accuracy of the reference calculation

In order to be able to assess the effectiveness of measures, the model first needs to give accurate outcome for the reference situation. The calculation results are validated by comparison with the outcome of the PC-Ring calculation. Since the data that has been used is retrieved from the FloRis data the outcome of the new model calculation should not differ too much from the PC-Ring calculations. The probability of failure per dike section for the calculation in PC-Ring and the calculation with the new model are compared in Figure 3-9.

Some sections show an overestimation and some show an underestimation of the total probability of failure. This is caused by the way in which the fragility curve is derived, and this differs per failure mechanism. For overflow/overtopping the probability of failure per block duration is used, which leads to an underestimation of on average 10%. Piping uses the probability per year which leads to a structural overestimation of about 30%. This can be seen in Figure 3-10 and Figure 3-11.

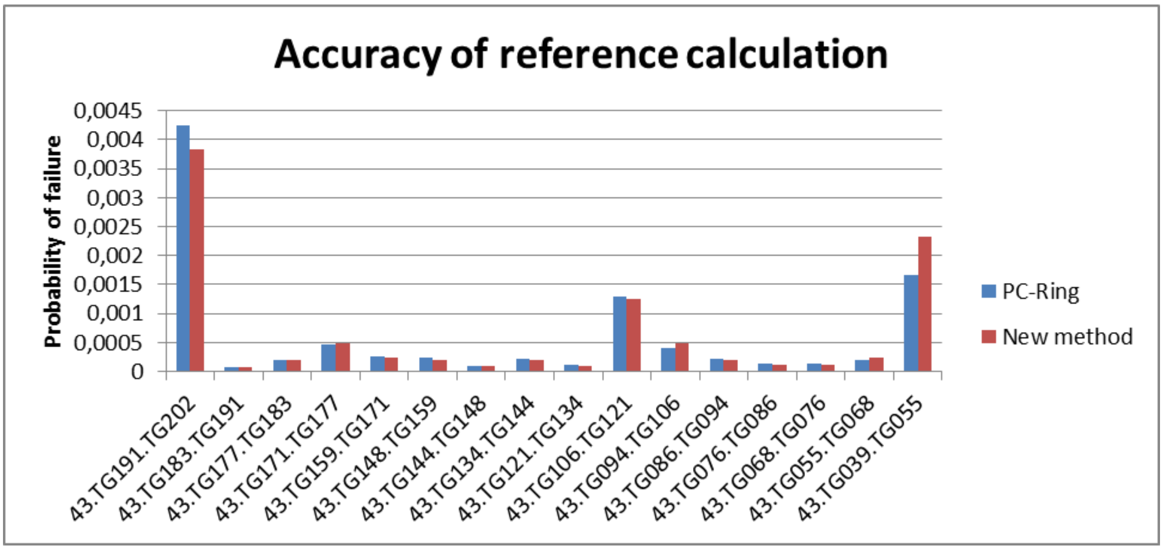


Figure 3-9: Accuracy of the model calculation for the reference situation

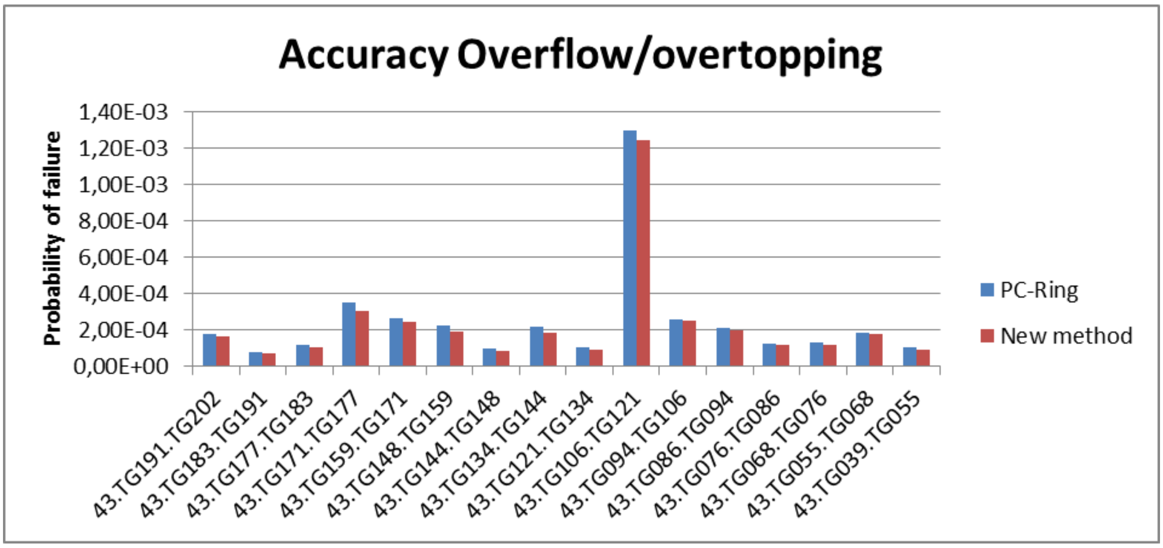


Figure 3-10: Accuracy of the probability of failure for overflow/overtopping

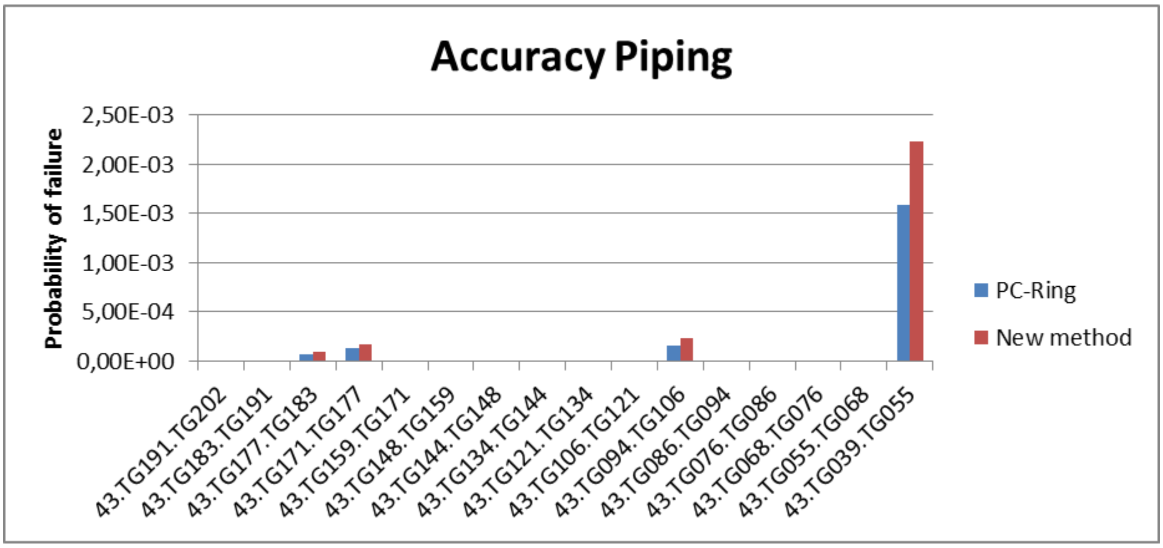


Figure 3-11: Accuracy of the probability of failure for piping

For all sections and failure mechanisms the error is shown in Table 3-1. The values between brackets have not been taken into account for the average, since the probability of failure is very small (order 10^{-10}), so the error is completely determined by round off and interpolation errors. The error is the ratio between the reference calculation and the calculation made with the rapid assessment tool. If the table shows a '-' sign this means that the failure mechanism does not play a role for that dike section.

Table 3-1: Accuracy of the calculations per failure mechanism and per section

Error	**Mechanism**				
Section	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.07	1.14	-	-	1.14
43.TG183.TG191	1.14	(3.11)	-	-	1.14
43.TG177.TG183	1.09	-	1.25	-	1.04
43.TG171.TG177	1.15	-	1.30	-	1.01
43.TG159.TG171	1.08	1.07	-	-	1.08
43.TG148.TG159	1.18	1.60	-	-	1.18
43.TG144.TG148	1.22	-	-	-	1.22
43.TG134.TG144	1.17	-	-	-	1.17
43.TG121.TG134	1.10	-	-	-	1.10
43.TG106.TG121	1.04	-	-	-	1.04
43.TG094.TG106	1.03	-	1.39	-	1.13
43.TG086.TG094	1.06	-	-	-	1.06
43.TG076.TG086	1.06	-	-	-	1.06
43.TG068.TG076	1.12	-	-	-	1.12
43.TG055.TG068	1.02	1.31	-	(4.72)	1.18
43.TG039.TG055	1.17	-	1.41	-	1.37
Average	**1.11**	**1.28**	**1.34**	**1.00**	**1.13**

All the values in the table are larger than 1 because only the absolute value of the error has been taken. This means that if the probability of failure calculated with the new method is higher than the reference calculation the reciprocal value is taken. This is done so that it is possible to calculate the average error of all the dike sections. It should always be monitored whether the error is an overestimation or an underestimation of the reference calculation. Generally overflow/overtopping and macro stability is underestimated and piping is overestimated.

3.2.3 Probability of failure per reach

PC-Ring uses a rather complicated method to combine the probabilities per section and per failure mechanism to the total probability of failure of a reach. This method takes dependency between failure mechanisms and between dike sections into account.

This method can be approximated quite well by using a relatively simple method. To calculate the total probability of failure of a reach first the total probability of failure per failure mechanism is calculated. The probability of failure of the whole reach is the sum of the total probabilities of failure of all the separate mechanisms.

The total probability of failure for overtopping is found under the assumption that the mechanism has full dependency between dike sections. This is a valid assumption since the water will first flow over the dike at the point with the lowest crest height. The probability of failure of the reach is equal to the weakest link in the chain. Therefore the total probability of failure for overtopping is the maximum probability of failure of the individual dike sections.

The other mechanisms are assumed to be completely independent. This is also a valid assumption since these mechanisms are based on the geotechnical properties of the dike. These properties are highly variable so it is safe to assume that it is different for all the dike sections. The total probability of failure of the reach is then the sum of all the individual probabilities of failure of the individual dike sections.

Table 3-2 shows the error that is induced when the probability of failure is calculated in this way. The second row shows the total probability of failure when the individual probabilities of failure are calculated with PC-Ring but are then combined with the method described above. This results in a difference of 16% with the more complicated combination method of PC-Ring itself. The lower row shows the probability of failure calculated with the new calculation method and combined with the method described above. This gives a total error of 20%.

Table 3-2: Probability of failure per reach

	Probability of failure	Error
PC-Ring original	1/156	-
PC-Ring	1/134	1.16
New method	1/130	1.20

3.2.4 Climate change

Climate change can be calculated with the new method by changing the discharge statistics. An assessment has been made on the accuracy of the calculation of the impact of climate change. This assessment is included in Appendix C. The impact of climate change on the probability of failure of a dike section or a reach can be estimated quite accurately with the new calculation method. The impact error (relative difference between the impact calculated by the new method and the impact calculated with PC-Ring) is for most dike sections and failure mechanisms in the order of 10%. The impact on the probability of failure of the whole reach is shown in Table 3-3.

Table 3-3: Impact of climate change on reach

Probability of failure	HR2006	W+2050	Impact
PC-Ring	1/156	1/75	2.07
New method	1/53	1/130	2.45

The error of the impact is 1.18. When the reference situation underestimates the probability of failure, the climate impact is overestimated. Also, when the reference situation overestimates the probability of failure, the impact is underestimated. When the error of the reference calculation is small, the error of the calculated impact will also remain small.

4 Design task

It has been shown that most of the dike reaches will not meet the new standards when these are introduced. The work that has to be done to reach the safety standards for a given year is called the design task. This chapter describes how this design task can be established. After this the magnitude of the design task will be assessed for the model dike reach as well as for the whole dike ring area 43.

4.1 Quantification of design task

In order to be able to assess the effectiveness of measures, it is necessary to express the design task numerically. To assess the magnitude of the total design task one should have knowledge about the present state, the new standards and changes (i.e. climate change) that are to be expected before the standards should be met.

4.1.1 Reduction and impact factor

The difference between the initial state and the final state can be described with the ratio of the failure probability. In case of a measure the probability of failure will decrease, then we speak of reduction. In case of climate change the probability of failure will increase, then we speak of impact. These can be described by the Reduction Factor (RF) and Impact Factor (IF). The impact factor is in fact the reduction factor needed to bring the dike back to its original state.

The reduction factor can be calculated as follows:

$$\frac{1}{RF} = \frac{Pf_{final}}{Pf_{initial}} \text{ or } RF = \frac{T_{final}}{T_{initial}} \text{ where } T_{final} = \frac{1}{Pf_{final}} \text{ and } T_{initial} = \frac{1}{Pf_{initial}}$$

where:

- RF = Reduction factor
- Pf_{final} = Probability of failure after implementation
- $Pf_{initial}$ = Probability of failure in the reference situation
- T_{final} = Return period corresponding to Pf_{final}
- $T_{initial}$ = Return period corresponding to $Pf_{initial}$

The impact factor is calculated with:

$$\frac{1}{IF} = \frac{Pf_{initial}}{Pf_{final}} \text{ or } IF = \frac{T_{initial}}{T_{final}}$$

where IF is the impact factor.

4.1.2 Climate change

Climate change makes up a part of the design task. Climate change between the reference situation and 2050 or 2100 causes a shift in the discharge statistics. It is commonly known that due to global warming the sea level is rising and a comparable effect is present in the rivers; it is expected that in the future the high discharges will occur more often. Higher discharges generally lead to higher water levels, which will increase the probability of failure.

Climate scenarios

The discharge statistics for the future are of course unknown, but a lot of calculations and predictions have been executed in this field. In the Delta programme it has been assumed that the design (1/1250 per year) discharge will increase from 16,000 m^3/s in 2015 to 17,000 m^3/s in 2050 and 18,000 m^3/s in 2100 (Ministerie van Infrastructuur en Milieu, 2014). Climate scenario W+2050, which is used in this research, assumes a design discharge of 18,000 m^3/s in 2050 and 20,000 m^3/s.

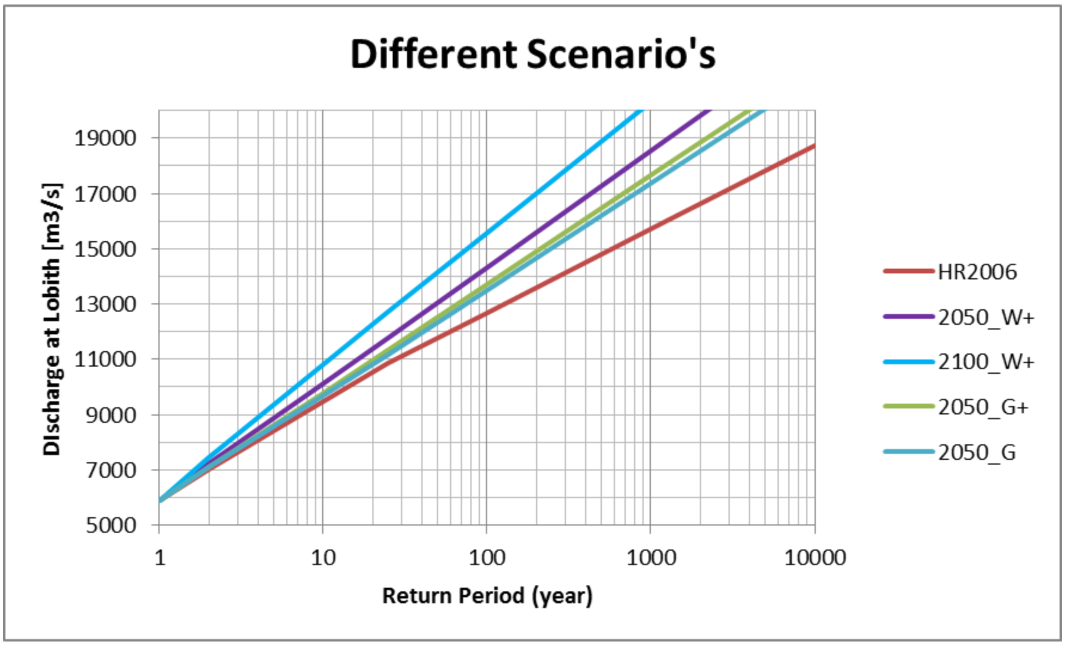


Figure 4-1: Different climate scenarios (Values: Hydra Zoet)

Figure 4-1 shows the discharge statistics for different climate scenarios. These discharge statistics have been retrieved by making a logarithmic fit through data from Hydra Zoet. The values may thus differ from the official discharge statistics. The Q-T relation can be described with the following expressions (Vrouwenvelder & Steenbergen, 2003):

$$
\begin{aligned}
q(T) &= 1620.7\ln(T) + 5893.3 && \textit{for } 1 < T \leq 2 \\
q(T) &= 1517.78\ln(T) + 5964.63 && \textit{for } 2 < T \leq 25 \\
q(T) &= 1316.43\ln(T) + 6612.61 && \textit{for } 25 < T \leq 10{,}000
\end{aligned}
$$

This is the expression for the line HR2006, which is the working line used for FloRis calculations and also in the calculations in this report for the reference situation and the situation before RfR. The other working lines represent different climate scenarios for 2050 and 2100. These can be described with similar expressions.

In this research use is made of climate scenario W+2050 (KNMI, 2006). This is represented by the purple line in Figure 4-1. The expression for this line is given in Appendix C. High discharges have a shorter return period, which will lead to higher probabilities of failure after climate change has occurred. The difference between the present state and the state after climate change is the climate change-induced design task. This can be represented with the impact factor IF.

In practice use is made of climate scenarios where the discharge is topped off around 18,000 m^3/s (aftoppen). This is related to the physical maximum discharge that can enter the Dutch Rhine system at Lobith. This effect cannot be accounted for in the PC-Ring software, and to make comparison possible, this is also not incorporated in the rapid assessment tool. It must be borne in mind that this may lead to large overestimations, since for some climate scenarios this discharge has a return period of about 500 years. This is a return period with a large contribution to the total probability of failure, so the overestimation of the probability of failure will be large for these scenarios.

4.1.3 Safety standards per dike reach

DP15 has proposed safety standards per dike reach. These safety standards have to be met in 2050. The safety standards for dike ring 43 are presented in Table 4-1.

Table 4-1: Safety standards for dike ring area 43 (Ministerie van Infrastructuur en Milieu, 2014)

Reach	Safety standard
43-1	1/30,000
43-2	1/10,000
43-3	1/30,000
43-4	1/30,000
43-5	1/30,000
43-6	1/30,000

A dike meets with the safety standard if the probability of flooding is within the right interval. The intervals that are used for the safety standards are shown in Table 4-2. These intervals are based on the fact that the probability of failure increases over time (because of climate change) and the safety state of the dikes is checked every 12 years (Werkgroep Deelprogramma Veiligheid, 2014).

Table 4-2: Division in intervals (Werkgroep Deelprogramma Veiligheid, 2014)

Class	Interval
100	1 – 170
300	170 – 550
1000	550 – 1700
3000	1700 – 5500
10000	5500 – 17000
30000	17000 – 55000
100000	55000 – 170000

4.1.4 Safety standard per dike section

The new safety standards can be translated into a maximum allowable probability of failure per dike section. This demanded probability of failure is based on the probability of failure of the whole dike reach, and the dependencies between several dike sections. The guidelines of OI2014 (Rijkswaterstaat, 2014) present a method to estimate the maximum probability of failure a dike section is allowed to have. OI2014 is a temporary guideline on how dikes should be designed, which is valid until the final design instrument for the new safety standards is completed.

The dike reaches in dike ring 43 have a safety standard of 1/30,000 or 1/10,000 (Table 4-3) (Ministerie van Infrastructuur en Milieu, 2014). This corresponds with a maximum probability of failure for this reach of 1/10,000 and 1/3000 respectively. OI2014 prescribes a factor 2 for the maximum acceptable probability of failure, but in this report a factor 3 is chosen, since this is being used in practice at the moment of writing. Also the contribution of each failure mechanism to the total probability of failure (ω) is given. The length of the dike reaches is given by (Vergouwe, et al., 2014). All the other parameters are prescribed by OI2014. The calculations shown below are for reach 43-6.

Overflow/ overtopping

The safety standard per dike section for the mechanism overtopping can be calculated with the following formula:

$$P_{eis,i} = \frac{P_{stamdard} \cdot \omega}{N}$$

in which:

- $P_{eis,i}$ = the safety standard per dike section
- $P_{standard}$ = the maximum acceptable probability of failure
- ω = the contribution factor for this failure mechanism
- N = a measure for the length-effect

There is no dependency between dike sections for this failure mechanism. The contribution factor is ω = 0.24. The factor N for the length effect is N = 1 for the whole upper river area. Together this leads for a safety standard per dike section of:

$$P_{eis,i} = \frac{1/10,000 \cdot 0.24}{1} = 2.40 \cdot 10^{-5}$$

Macro stability

The failure mechanisms macro stability and piping do have dependency between dike sections. This is also taken into account when the safety standard per section is calculated. This is done by the following formula:

$$P_{eis,i} = \frac{P_{\text{Standard}} \cdot \omega}{\left(1 + \frac{a \cdot L_{reach}}{b}\right)}$$

in which:

- a = length-dependency per failure mechanism
- b = length of independent, equivalent dike sections [m]
- L_{reach} = total length of the considered reach [m]

The values for macro stability are:

- ω = 0.04 [-]
- a = 0.33 [-]
- b = 50 [m]
- L_{reach} = 46,900 [m]

This leads to a safety standard per dike section of:

$$P_{eis,i} = \frac{1/10,000 \cdot 0.04}{\left(1 + \frac{0.033 \cdot 46,900}{50}\right)} = 1.25 \cdot 10^{-7}$$

Piping

The values for piping are:

- ω = 0.24 [-]
- a = 0.90 [-]
- b = 300 [m]

this leads to a safety standard per dike section of:

$$P_{eis,i} = \frac{1/10,000 \cdot 0.24}{\left(1 + \frac{0.90 \cdot 46,900}{300}\right)} = 1.69 \cdot 10^{-7}$$

Damage and erosion outer slope

The contribution factor for stability of the outer slope is ω = 0.1. With the same formula as for overtopping this leads to a safety standard of

$$P_{eis,i} = \frac{1/10,000 \cdot 0.10}{1} = 1.00 \cdot 10^{-5}$$

These safety standards per dike section can be translated into a design task per failure mechanism per dike section. The safety standards for all reaches of dike ring 43 is given in Table 4-3.

Table 4-3: Safety standard per dike section for all failure mechanisms

Reach	Safety standard	Length [km]	Overflow/ Overtopping	Macro stability	Piping	Damage and erosion Outer slope
43-1	1/30,000	15.8	2.40E-05	3.50E-07	4.96E-07	1.00E-05
43-2	1/10,000	34.0	7.20E-05	5.12E-07	6.99E-07	3.00E-05
43-3	1/30,000	25.3	2.40E-05	2.26E-07	3.12E-07	1.00E-05
43-4	1/30,000	26.4	2.40E-05	2.17E-07	2.99E-07	1.00E-05
43-5	1/30,000	22.4	2.40E-05	2.53E-07	3.52E-07	1.00E-05
43-6	1/30,000	46.9	2.40E-05	1.25E-07	1.69E-07	1.00E-05

4.1.5 Total design task

The reduction factor and the impact factor can also be used to describe the design task. The total design task (DT) consists of a climate change induced part and a standard-induced part. The standard-induced part is the relative difference between the safety level as stated in the standard and the safety level of the initial situation. The DT can be determined per dike section (and per failure mechanism) or for a dike reach. Figure 4-2 shows how the total design task is built up. The presented values are chosen arbitrarily and serve only illustrational purposes.

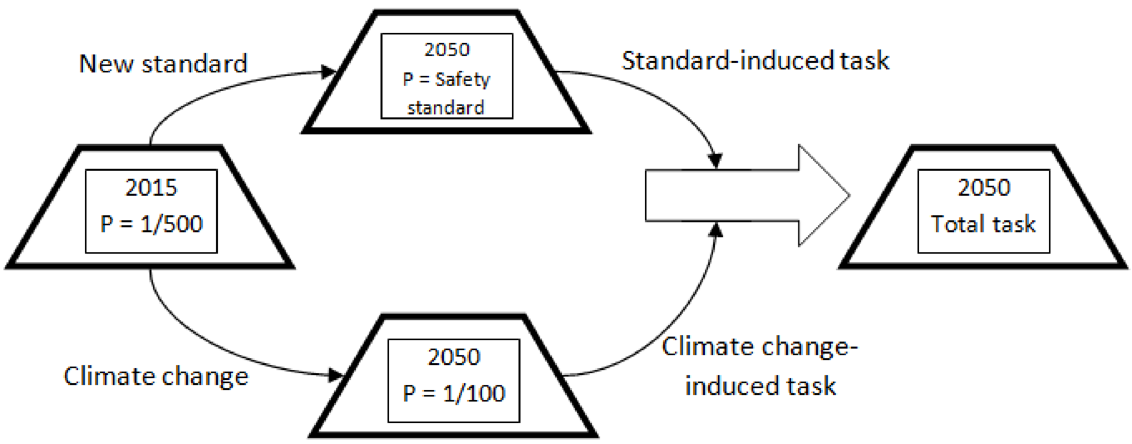


Figure 4-2: Total design task

The total task can be expressed as follows:

$$DT = RF_{standard} \cdot IF_{climate}$$

Since climate change is taken into account, the life time of the flood defence is taken into account when calculating the design task. In this research climate change is calculated for the year 2050, which is the year in which all the flood defences have to meet with the safety standards.

4.2 Dike ring area 43

This section elaborates the design task for the whole dike ring area 43. The reference state is the reference situation of this research, before RfR and HWBP2. The design task will first be elaborated per dike reach and after that per dike section.

4.2.1 Design task per reach

Present state
FloRis calculated the probability of failure for dike ring 43 in the present situation. This calculation was done using the software package PC-Ring. The same PC-Ring database (Projectbureau VNK2, 2014) has been used for the other calculations in this section, unless explicitly mentioned otherwise. The situation used in FloRis is the reference situation of this thesis. It is again noted that this is not the same as the reference situation of DP15.

The numbers shown in Table 4-4 are the results of the FloRis programme. The total probability of failure of this dike ring is in the interval of 1/100 per year (Table 4-2), which is a very high probability of failure. It can be seen that the failure mechanism piping has by far the greatest contribution to the total probability of failure. The contribution of damage and erosion outer slope and structural failure on the probability of failure is small compared to the other failure mechanisms. Table 4-5 shows the safety level per dike section for the reference situation. These values are also a result of the FloRis programme. The dike reaches have probabilities of failure between 1/100 and 1/500. The new standards for the reaches within this dike ring area are 1/10,000 or 1/30,000. The current state of the dikes in this area is thus not up to the new standard.

Climate change

Climate change leads to higher probabilities of failure in the considered section. In Table 4-4 the probability of failure for the different failure mechanisms is shown for the reference situation and for the year 2050.

Table 4-4: Probability of failure of dike ring 43 in the reference situation and after climate change (Projectbureau VNK2, 2014)

Mechanism	Reference	W+2050	$IF_{climate}$
Overflow/overtopping	1/767	1/182	4,2
Macro stability	1/132	1/75	1,8
Piping	1/40	1/27	1,5
Damage and erosion outer slope	1/15032	1/15373	1,0
Structures	1/4010	1/758	5,3
Total	**1/36**	**1/29**	**1,3**

The last column of Table 4-4 shows the total impact on the safety level caused by climate change. The probability of failure due to overflow/wave overtopping increases significantly. Failure mechanisms piping and stability also show some increase but the impact is smaller because the geotechnical construction of dikes are of more influence on the probability of failure than the water level.

Figure 4-4 shows the probability of failure of dike ring 43 per dike section in the reference situation and Figure 4-4 shows the probability of failure after climate change. For the reference situation it can be seen that in the North part of the dike ring area more dike sections have a higher safety level. This is because in the North the dikes have been improved recently (Vergouwe, et al., 2014) . The impact of climate change is also clearly visible, since in Figure 4-4 a lot of dike sections have a red or orange colour.

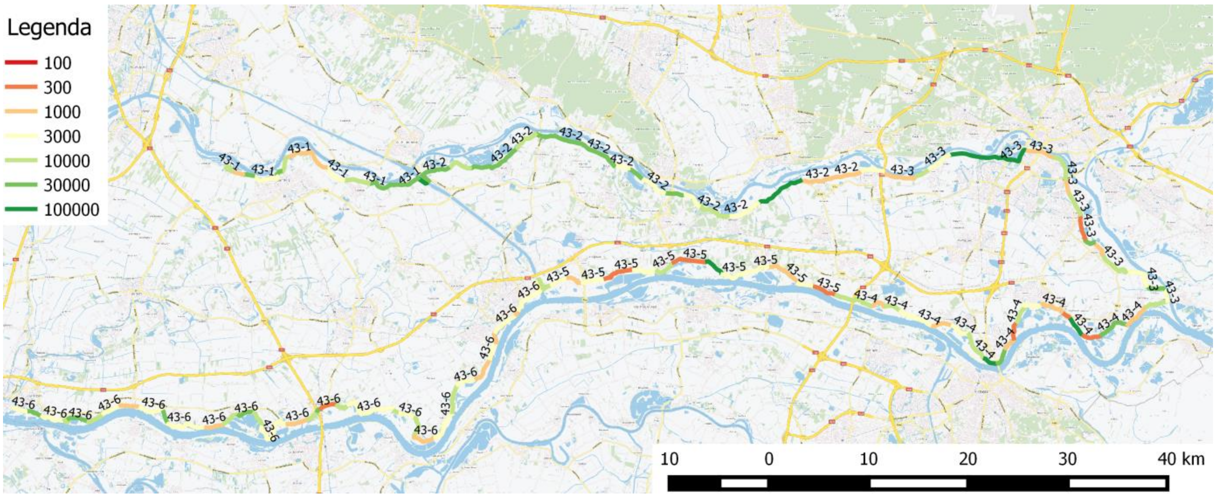


Figure 4-3: Probability of failure in reference situation

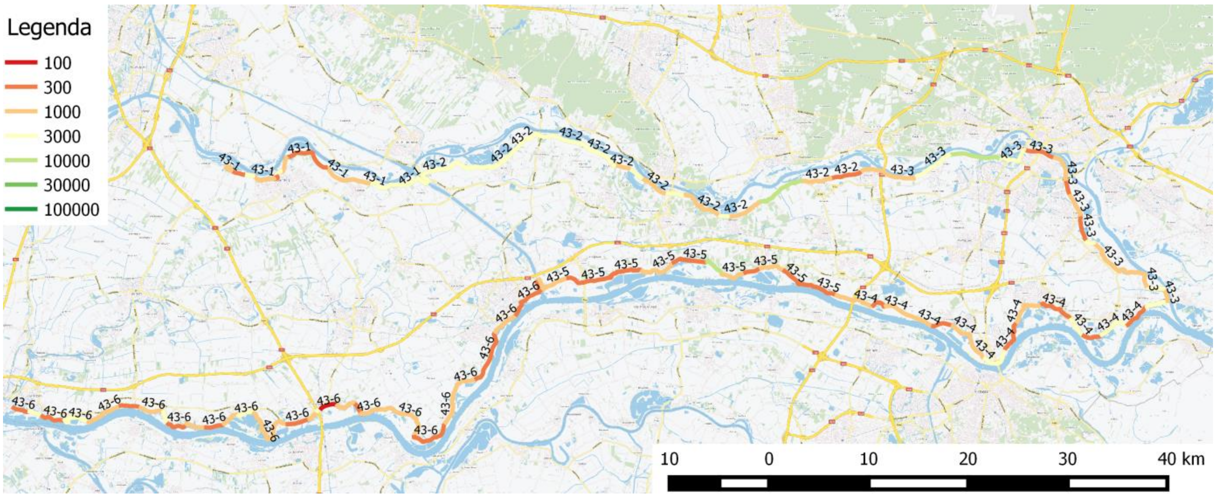


Figure 4-4: Probability of failure after climate change

Total design task

The total design task is the product of the climate change induced part and the standard-induced part. The new standards count per dike reach. The probability of failure in the reference situation (Werkgroep Deelprogramma Veiligheid, 2014), the probability of failure after climate change (PC-Ring calculation) and the required safety level per reach (Ministerie van Infrastructuur en Milieu, 2014) are shown in Table 4-5. This table shows the total design task, taking climate change and the new standards into account.

Table 4-5: Design task per reach for dike ring 43 (Ministerie van Infrastructuur en Milieu, 2014)

		Climate change induced part		Norm induced part		Total task
Reach	Reference	W+2050	$IF_{climate}$	Standard	$RF_{standard}$	Design task
43-1	1/500	1/130	3.85	1/30,000	60.00	231
43-2	1/500	1/170	2.94	1/10,000	20.00	59
43-3	1/300	1/110	2.73	1/30,000	100.00	273
43-4	1/100	1/60	1.67	1/30,000	300.00	500
43-5	1/100	1/60	1.67	1/30,000	300.00	500
43-6	1/100	1/60	1.67	1/30,000	300.00	500

Figure 4-5 also shows the design task per reach. It is seen that the whole southern river stretch (Waal River) has a design task of 500. That this design task is so high is due to the presence of sections with high probability of failure for geotechnical failure mechanisms.

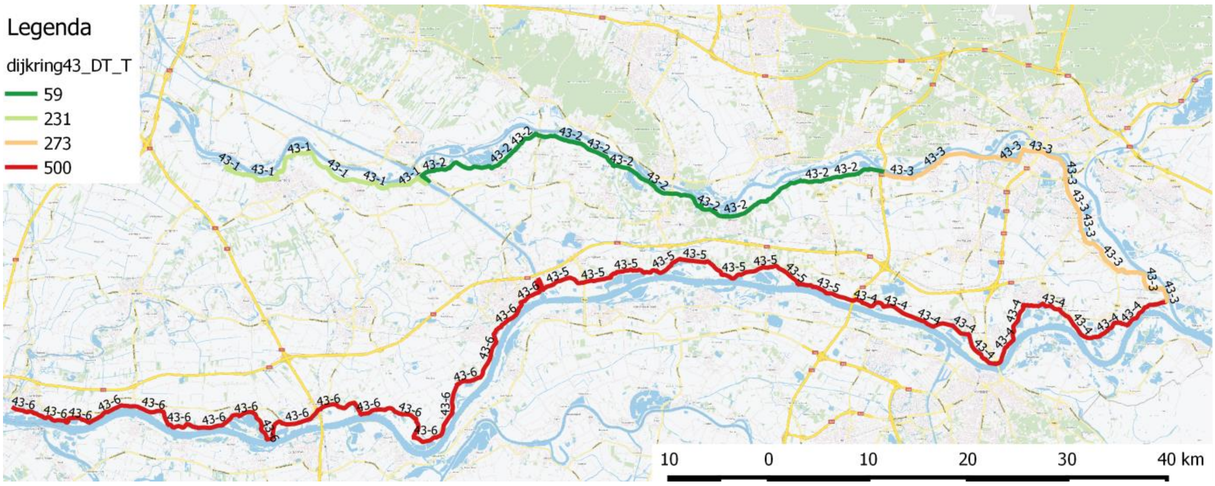


Figure 4-5: Design task per reach

Room for the River

In FloRis an assessment is made on the effectiveness of RfR. The probability of failure of the dike section is calculated under the assumption that for all dike sections the water level reduction is equal to the hydraulic target (in cm). It is also assumed that the water level reduction is equal for all discharges. This is physically incorrect so it will lead to an underestimation of the probability of failure and thus an overestimation of the effectiveness of RfR.

Table 4-6: Probability of failure of dike ring 43 after climate change and after climate change and RfR (Projectbureau VNK2, 2014)

Mechanism	Reference	After RfR	RF	W+2050	W+2050 & RfR	RF
Overflow/ overtopping	1/767	1/1326	1.7	1/182	1/272	1.5
Macro stability	1/132	1/196	1.5	1/75	1/106	1.4
Piping	1/41	1/59	1.4	1/27	1/37	1.3
Damage and erosion outer slope	1/15032	1/19399	1.3	1/15373	1/19763	1.0
Structures	1/4010	1/6754	1.7	1/758	1/1098	1.4
Total	**1/36**	**1/51**	**1.4**	**1/29**	**1/38**	**1.3**

In Table 4-6 the effect of the RfR measures on the probability of failure is presented for two climate scenarios: HR2006 and W+2050. For HR2006 the RfR measures are most effective for the failure mechanism overflow/overtopping since this has the most direct relation with the water level. The reduction factor is 1.7. For other failure mechanisms the geotechnical properties of the dike are more important. The total reduction factor is lower: 1.4. Since overtopping only contributes 3.9% to the total probability of failure, the effectiveness is very low. The probability of failure of the total dike ring remains very large.

For climate scenario W+2050 the effectiveness is also the highest for overflow/overtopping, the reduction factor is 1.5. This is slightly smaller than for HR2006. The total reduction factor by RfR is 1.3 including climate change. The impact of climate change is also 1.3 (Table 4-4), so the situation after climate change and RfR is approximately equal to the reference situation. This can also be seen in Table 4-6; the probabilities of failure are 1/38 and 1/36 respectively. However: FloRis has overestimated the effectiveness of RfR by assuming equal water level reduction for all discharges. If this is taken into account the probability of failure will be higher than in the reference situation, despite the RfR measure. The magnitude of this overestimation will be discussed in chapter 6.

Even though the probabilities of failure for most failure mechanisms are much higher for the situation after climate change and RfR, the total probability of failure is approximately equal. This is due to the fact that the probability of failure for piping and heave determines most of the probability of failure for this dike ring. Treating this failure mechanism will be very effective in decreasing the probability of failure.

4.2.2 Design task per dike section

The design task per dike section can best be expressed as design task per failure mechanism. The safety standards per dike section and per failure mechanism have been elaborated in section 4.1.4.

Figure 4-6 shows the design task per dike section for the mechanism overflow/overtopping. This is the only failure mechanism that is present in all the dike sections. The design task is the difference between the probability of failure for the mechanism overtopping with climate scenario W+2050 but without RfR and the safety standard per dike section. The probabilities of failure per dike section have been calculated with PC-Ring using the FloRis databases. The safety standards are given in Table 4-3.

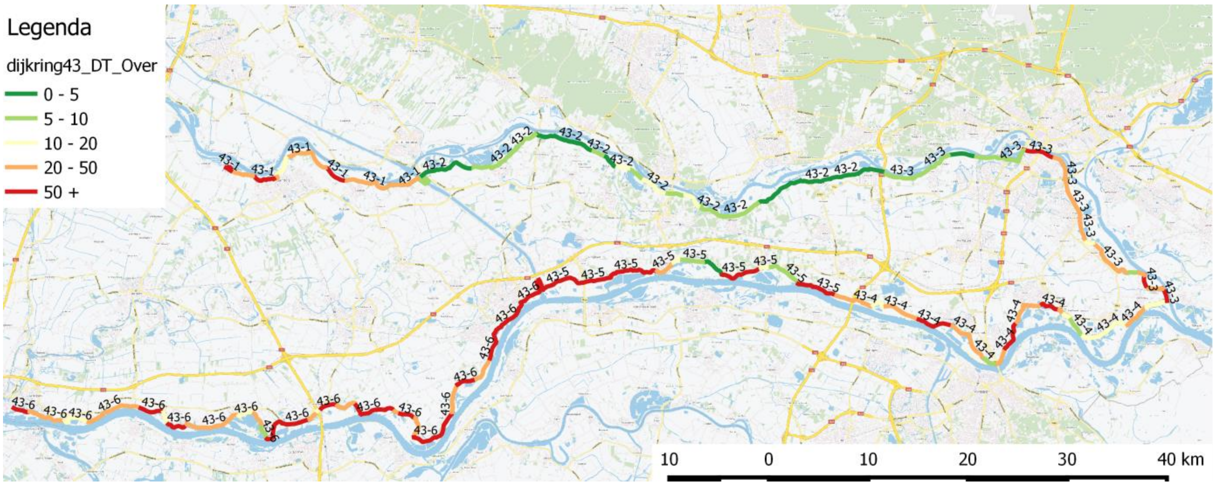


Figure 4-6: Design task per section for overtopping

The sections on the northern side of the dike ring have low design tasks, and thus relatively high safety levels. This is because these dikes have been recently improved. The whole southern stretch (Waal River) has design tasks of over 20, and more than half even higher than 50. This is only for failure mechanism overflow/overtopping. For other failure mechanisms certain figures can also be made, but since these mechanisms are not incorporated for all sections, and because the rest of this research will focus on a shorter stretch, this is left behind.

4.3 Model dike reach

In this section the design task of the model dike reach is treated. It will elaborate the present state and the state after climate change. The design task will be treated per dike section as well as for the whole model dike reach. The calculations have been made with the rapid assessment tool, unless explicitly mentioned otherwise.

4.3.1 Design task of the whole model reach

Present state

The total probability of failure of the model dike reach, calculated with the new method is 1/130 per year. In section 4.2.1 it was seen that the probability of failure of dike reach 43-6, in which the model dike reach is located, is 1/100 per year. The probability of failure of the model dike reach is thus in the same order.

Climate change

In section 3.2.4 the accuracy of calculations on climate change was assessed. It was seen here that the probability of failure after climate change is 1/54 per year. This means that the IF by climate change is 2.45. For the whole dike ring this was 1.3. This difference can be ascribed to influence of the length effect and because the model reach contains three of the sections with the highest probability for overtopping, piping and macro stability (see below).

Total task

The total design task for the model dike reach cannot be expressed unambiguously. Since it is only a part of a dike reach, there is no safety standard for this particular reach. Therefore the safety level of the whole dike reach will be used (1/30,000). The design task again exists of a norm-induced part and a climate change-induced part. This is shown in Table 4-7. The total design task is 566. For dike reach 43-6 this is 500 so this is in the same order.

Table 4-7: Total design task model dike reach

Reference	W+2050	$IF_{climate}$	Standard	$RF_{standard}$	Design task
1/130	1/53	2.45	1/30,000	231	566

4.3.2 Design task per dike section

Present state

Figure 4-7 shows the probability of failure per dike section and per failure mechanism. There are three dike sections that stand out in terms of probability of failure. The most left (downstream) downstream has the highest probability of failure for macro stability. This is also the weakest section in whole reach 43-6. The most right (upstream) section has the highest probability of failure mechanism piping of the whole dike reach. Section 43.TG106.TG121 has the highest probability of failure for overtopping of the reach. For most sections the probability of failure is almost entirely defined by one failure mechanism.

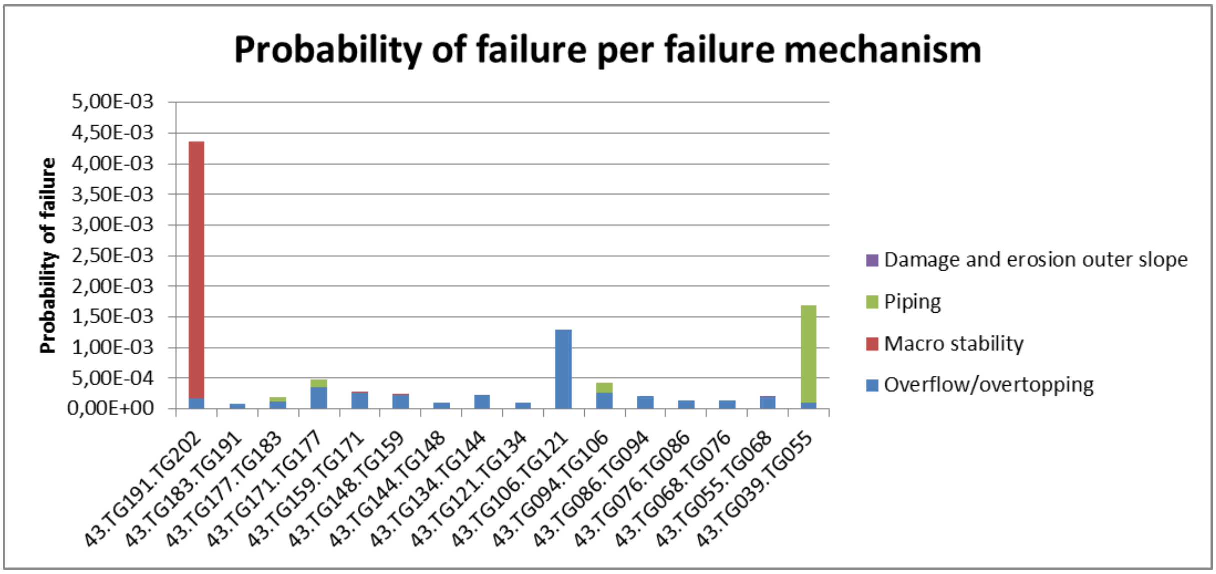


Figure 4-7: Probability of failure of the model dike reach

Table 4-8 shows the probabilities of failure per dike section and per failure mechanism for the reference situation. It should be noticed that the total probability of failure for the failure mechanism overflow/overtopping is equal to the maximum probability of failure of one section. For the other failure mechanisms the probability of failure is the sum of the individual probabilities of failure, but the total probability of failure is still mostly determined by one section. The probability of failure for macro stability of the second section is noted in parentheses, because this probability is very small (order 10^{-10}). This affects the impact factor so it is not taken into account.

Table 4-8: Probability of failure per dike section and per failure mechanism, reference situation

Probability of failure	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1/6060	1/273	-	-	1/261
43.TG183.TG191	1/14504	(0)	-	-	1/14504
43.TG177.TG183	1/9265	-	1/11212	-	1/5073
43.TG171.TG177	1/3254	-	1/5993	-	1/2109
43.TG159.TG171	1/4108	1/2287998020	-	-	1/4108
43.TG148.TG159	1/5238	1/1498762	-	-	1/5220
43.TG144.TG148	1/12174	-	-	-	1/12174
43.TG134.TG144	1/5343	-	-	-	1/5343
43.TG121.TG134	1/10563	-	-	-	1/10563
43.TG106.TG121	1/801	-	-	-	1/801
43.TG094.TG106	1/3976	-	1/4402	-	1/2089
43.TG086.TG094	1/5003	-	-	-	1/5003
43.TG076.TG086	1/8245	-	-	-	1/8245
43.TG068.TG076	1/8541	-	-	-	1/8541
43.TG055.TG068	1/5581	1/1355423	-	1/21098	1/4399
43.TG039.TG055	1/11241	-	1/449	-	1/432
Total	**1/801**	**1/273**	**1/369**	**1/21098**	**-**

Climate change

Table 4-9 shows the climate change-induced part of the design task per dike section and per failure mechanism. What is noticeable is that the impact for mechanism overflow/overtopping is much higher than for most of the other failure mechanisms. This is due to the fact that this failure mechanism is immediately affected by higher water levels. For the other failure mechanisms geotechnical composition of the dike is more important, so a higher water level has less effect on the probability of failure.

Table 4-9: Climate change-induced part of the design task

Section	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope
43.TG191.TG202	7.67	2.23	-	-
43.TG183.TG191	9.61	(-)	-	-
43.TG177.TG183	8.64	-	2.14	-
43.TG171.TG177	6.28	-	2.49	-
43.TG159.TG171	6.89	1.72	-	-
43.TG148.TG159	7.43	1.50	-	-
43.TG144.TG148	8.85	-	-	-
43.TG134.TG144	7.22	-	-	-
43.TG121.TG134	8.52	-	-	-
43.TG106.TG121	4.38	-	-	-
43.TG094.TG106	6.92	-	2.34	-
43.TG086.TG094	7.38	-	-	-
43.TG076.TG086	8.47	-	-	-
43.TG068.TG076	8.59	-	-	-
43.TG055.TG068	7.57	1.08	-	1.15
43.TG039.TG055	9.27	-	1.80	-

Total design task

The total DT is built up of the climate change-induced part and the standard-induced part. The standards per section have been determined in section 4.1.4. The total design task follows from these values and the present safety state and the climate change-induced design task that have been described above. The total design task per dike section is shown in Table 4-10. The red values indicate the sections with large probability of failure. It can be seen that the design task for these sections is much larger than for most other section. Therefore a lot of work needs to be done on these sections. What would be the best strategy to fulfil these design tasks is treated in the next chapters.

Table 4-10: Total design task per dike section and per failure mechanism

Section	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope
43.TG191.TG202	53	65314	-	-
43.TG183.TG191	28	-	-	-
43.TG177.TG183	39	-	1129	-
43.TG171.TG177	80	-	2463	-
43.TG159.TG171	70	-	-	-
43.TG148.TG159	59	8	-	-
43.TG144.TG148	30	-	-	-
43.TG134.TG144	56	-	-	-
43.TG121.TG134	34	-	-	-
43.TG106.TG121	227	-	-	-
43.TG094.TG106	72	-	3142	-
43.TG086.TG094	62	-	-	-
43.TG076.TG086	43	-	-	-
43.TG068.TG076	42	-	-	-
43.TG055.TG068	57	6	-	5
43.TG039.TG055	34	-	23645	-

5 Measures

Measures are taken with the intention of increasing the safety level of a dike and its underlying area. Measures can reduce the probability of failure in two ways; by increasing the strength of a dike by means of dike improvements, and by reducing the hydraulic load by increasing the flood conveyance capacity, thereby reducing the water level.

This chapter describes the characteristics of several spatial measures and dike improvements. Advantages or disadvantages of the measures will also be treated. Several strategies on how to combine these measures are also presented. After this four spatial measures are elaborated to be representative cases for calculations. Also some of the strategies are elaborated in more detail.

The following measures are being assessed:

- Side channel
- Excavation of floodplains
- Repositioning of dikes
- Lowering of groynes
- Dike improvement

The first three measures are floodplain measures. Floodplain measures become more effective at higher discharges. Lowering of groynes is more effective at lower discharges. Dike improvements do not reduce the hydraulic load, but they increase the strength of a dike section.

5.1 Spatial measures

Spatial measures are generally intended to increase the flood conveyance capacity of a river system. This can be done for example by increasing the volume of the system or by decreasing the hydraulic resistance. Increasing the volume of the system can be done by increasing the volume of the floodplains. The side channel, excavation of the floodplains and dike repositioning are examples of floodplain measures. Lowering the groynes is an example of how the hydraulic resistance can be decreased.

Floodplain measures increase the flood conveyance capacity of the floodplains. For the considered dike reach the floodplains start flowing along around 7500 m^3/s (see section 3.2.1). If the capacity of the floodplains is increased the water levels at discharges higher than 7500 m^3/s will be lower. The measure will be most effective at high discharges, but it will already play a role just after the floodplains will start flowing along.

The floodplain measures have been assessed using the web-based toolbox "Blokkendoos Rivieren" (Deltaprogramma, 2014). The background of these measures is described in Appendix D. The effect can be in the order of several centimetres up to several tens of centimetres (the design task for RfR was on average 30 cm), depending on the size and the location of the measure.

5.1.1 Side channel

Side channels, high water channels and green rivers are used when high discharges occur. In case of high water channels they generally start flowing around 12,000 m^3/s, but side channels may start flowing at lower discharges, for example at 7500 m^3/s, when the floodplains start flowing. This divides the flow over two river branches, thereby reducing the discharge in the primary branch. When this secondary channel lies within the winter bed it can be seen as an increase of the flood conveyance capacity of the floodplains. Side channels and high water channels generally lay within the floodplains, where green rivers are often branches outside the normal river track. Only measures that are taken in the floodplains are discussed here. The side channel can become active at the moment when the floodplains start being inundated, or at a higher discharge. The concept of a side channel is shown in Figure 5-1.

5.1.2 Excavation of floodplains

Excavation of the floodplains (Figure 5-1) directly increases the flood conveyance capacity of the floodplains, causing a lower water level at the upstream side of the measure. This measure will be effective from the moment the floodplains start flowing along, and will become more effective for higher discharges. The increase of the effect will however reduce for discharges up to MHW, since the relative increase of the perimeter is maximal for discharges where the floodplains are just starting to flow.

Figure 5-1: Side channel or excavation of floodplains (Ruimte voor de Rivier, 2014)

5.1.3 Dike repositioning

Dike repositioning means that the winter dikes are being placed backward, so that the floodplains become wider. This increases the capacity of the floodplain and it will lead to lower water levels starting from the discharge where the floodplains start flowing. The effect will increase for higher discharges. In practice, dike repositioning will in all cases be combined with dike improvement, since the new dike will be constructed stronger than the old dike, based on the prevailing safety standards. In this thesis however, these measures are regarded separately. Dike repositioning is displayed in Figure 5-2.

Figure 5-2: Dike repositioning (Ruimte voor de Rivier, 2014)

5.1.4 Lowering of groynes

This reduces the hydraulic resistance of a river. The flood conveyance capacity is increased so that more water can be discharged. This can be seen as an increase in the flow carrying area. Of course this measure is only applicable if groynes are present. This measure is effective from discharge level when the groynes are flooded. This is around 3000 m^3/s for the Waal river (DHV, 2009). The effect of this measure is in the order of several centimetres. Contrary to floodplain measures the maximum effect is not at MHW but at a lower water level. This makes it an interesting measure for dikes with a high probability of failure for geotechnical failure mechanisms.

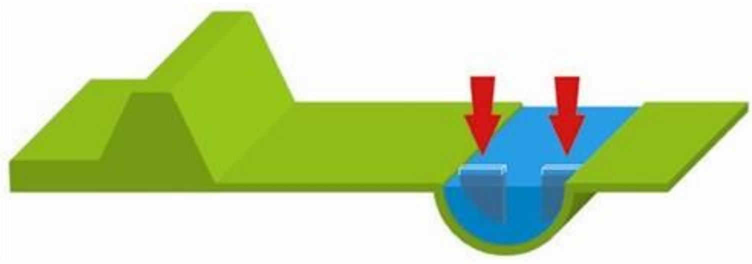


Figure 5-3: Lowering of groynes (Ruimte voor de Rivier, 2014)

5.2 Dike improvements

Instead of increasing the flood conveyance capacity and thereby the hydraulic load, the strength of dikes can also be improved. Dike improvements have generally no effect on the water level. The load remains the same so the strength should be improved in order to prevent failure. Dikes can be improved in different ways, having different results.

Dike raising

Dike raising decreases the probability of failure for mechanism overtopping and overflow. The fragility curve will shift to the right (or towards the higher water levels). This is shown in Figure 5-5.

Figure 5-4: Dike improvement (Ruimte voor de Rivier, 2014)

Piping berm

This measure makes the dike stronger or wider. It reduces the probability of failure for mechanisms such as stability and piping and heave. Failure mechanism piping can only occur if full pipes can occur. Whether this is possible depends on the geotechnical structure of the dike and the seepage length. By increasing the seepage length the probability of failure is reduced. The rule of thumb by Bligh states that for one metre water level difference a piping berm of 18 m is needed (Technische Adviescommissie voor de Waterkeringen, 1999). This has an effect on the fragility curve for this mechanism as shown in Figure 5-5.

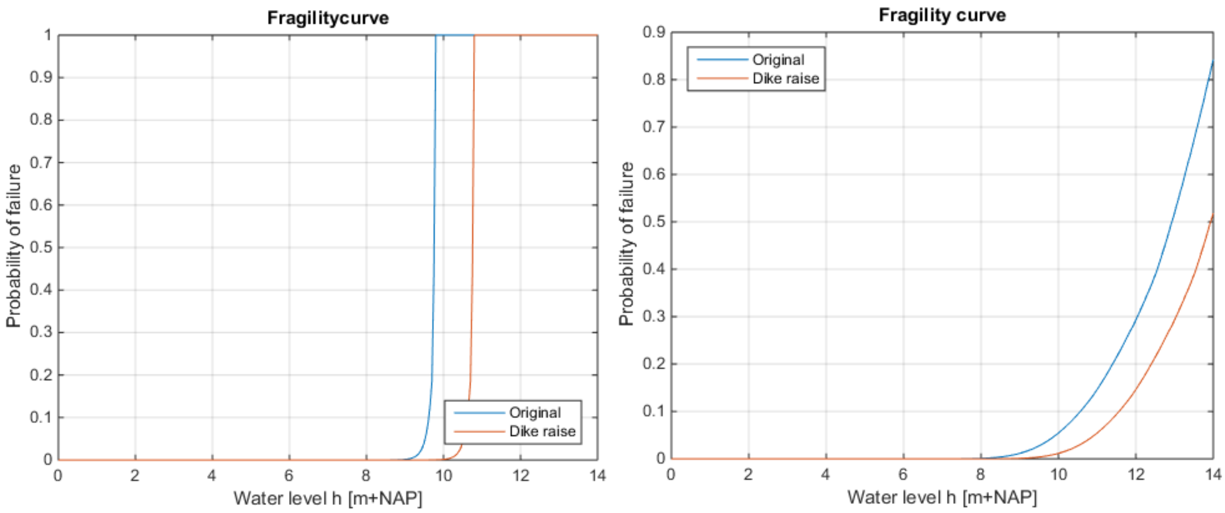


Figure 5-5: Fragility curve for overflow/wave overtopping after dike raising (left) and for piping after dike improvement (right)

5.3 Strategies

This total design task can be dealt with in several ways. Spatial measures alone will not be sufficient to reach the required safety state for most locations, since we have seen in section 4.3 that some sections have very large design tasks.

5.3.1 Preferred strategy Delta Programme 2015

The preferred strategy of the Delta programme aims on preventing the increase of water levels due to climate change by spatial measures. When spatial measures are insufficient the design task will be solved by means of dike improvements (Ministerie van Infrastructuur en Milieu, 2014).This method has much similarity with the old Room for the River programme.

In section 4.2 it was seen that the impact factor for climate change is 1.3. The reduction factor by the RfR programme is also 1.3. This implies that the strategy that DP15 prescribes could be feasible. It must however be borne in mind that the reference situation for the RfR programme and DP15 is different since this may affect the effectiveness of measures and thus the results. Also, the calculations made by FloRis overestimated the effectiveness of RfR. How large this overestimation is will be calculated in the next chapter, but this may also influence one's opinion on which strategy should be used.

5.3.2 Only dike improvements

Probably the cheapest strategy would be to use dike improvements everywhere. Increasing the height or the width of a dike is generally cheaper than creating a big spatial measure. Dike improvements are also very failure mechanism-specific; if overflow/overtopping is the dominant failure mechanism dikes are raised, if piping is the dominant failure mechanism a piping berm is constructed. This strategy also has its limitations since in some situations a combination of spatial measures is desirable or even required.

5.3.3 Weakest links

If a dike reach has one or a few very weak dike sections, the safety level can be improved a lot by treating this weak links. If this is combined with spatial measures the effectiveness may be very high.

5.4 Implementation

Floodplain measures and lowering of groynes affect the Q-h location of a dike section it is used on. This happens on the location of the measure itself, but also upstream. The effect is at its maximum at the most upstream end of the measure and it decreases in upstream direction.

Instead of calculating the exact effect of the different measures, for all of the types of measures one representative variant is drafted. These measures are not elaborated numerically, but the values are based on an analysis of the Blokkendoos Rivieren for different measures (Appendix D). For all these measures the water level reduction at MHW is defined. This is also the only information the 'Blokkendoos' gives us. The water level reduction at lower discharges has to be estimated. The variants that are derived in this way are expected to give reasonable outcome which can be used to compare different measures generically. Spatial integration is not taken into account.

5.4.1 Q-h relations

To be able to elaborate the Q-h relation per dike section, it is assumed that the water level reduction at discharges lower than MHW is relative to the water level reduction at MHW. This relation is equal for all upstream dike sections. If the effect on the Q-h relation and the water level reduction at MHW is known the new Q-h relation can be determined for every dike section. Since all the spatial measures affect the water level a bit differently this relation is different for different measures.

Side channel

The side channel starts flowing along at 9000 m^3/s. Side channels are generally large measures, therefore the side channel will store a lot of water when discharges increase, until the point where the side channel is completely flowing along and the water level will start increasing equal to the original Q-h relation. Starting from 9000 m^3/s the water level reduction will rapidly increase. The increase of the water level reduction will decline for higher discharges, until the maximum water level effect is reached at 17,000 m^3/s. For discharges higher than MHW the water level reduction will stay the same, this counts for all the measures.

Excavation

Excavations will be effective from the moment the floodplains start flowing along (Q = 7500 m^3/s). Water will flow into the excavated area at discharges where the floodplains are just flowing along, so the water level reduction increases very fast starting at this discharge level. For higher discharges the water level reduction will still increase, but slower. This is similar to the side channel, but the maximum water level reduction will generally be lower.

Dike repositioning

Dike repositioning will be effective from the moment the floodplains start flowing along, and its effect will increase for higher discharges. It is assumed here that this increase is linear, since the flood conveyance capacity is increased for all water levels and discharges. This is unlike the side channel and the excavation where the extra space created by the measure will at some point be fully used after which the water level will increase parallel to the original Q-h relation.

The effect of these three measures on the Q-h relation can be seen in Figure 5-6. The effect in this picture is exaggerated for illustrational purposes. The floodplains start flowing around 7500 m^3/s for this section (43.TG191.TG202, the most downstream section in the model dike reach). This can be seen by the kink in the blue original Q-h relation. The water level reduction at MHW for each measure is elaborated below.

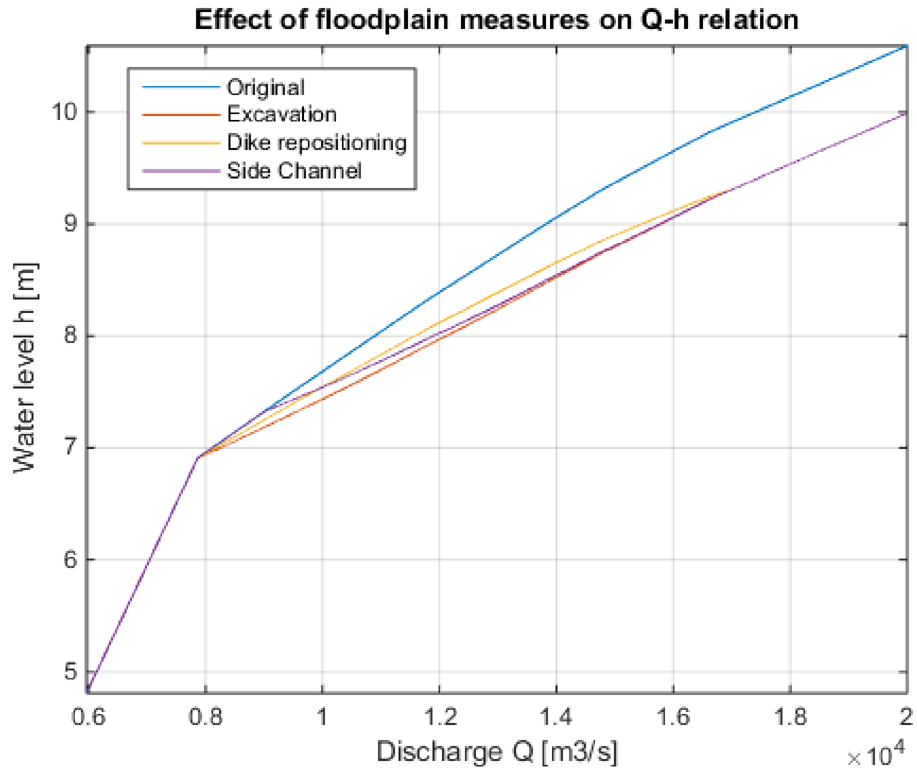


Figure 5-6: Effect of floodplain measures on Q-h relation (based on Q-h relation of section 43.TG191.TG202)

Lowering of groynes

Groyne lowering is not taken into account in the Blokkendoos, but since it could be of interest for flood defences where piping is a big problem, the effect of this measure will still be assessed. For the effect of this measure on the Q-h relation some assumptions need to be made. First of all it is assumed that all of the dike sections that are regarded contain groynes. All the groynes will be flooded at a discharge of 3000 m^3/s. From this discharge (and the corresponding water level) the effectiveness will increase linearly with the discharge. The water level lowering effect is maximal at Q = 5000 m^3/s. The reduction of the water level is then 15 cm. This is a reasonable estimate for the Waal since this has been calculated for the same river stretch (DHV, 2009). The water level lowering will linearly decrease for higher discharges until the water level effect is 10 cm for the maximum discharge of 17,000 m^3/s (Huthoff, et al., 2011). This effect on the Q-h relation is valid for all dike sections. The Q-h relation is shown in Figure 5-7.

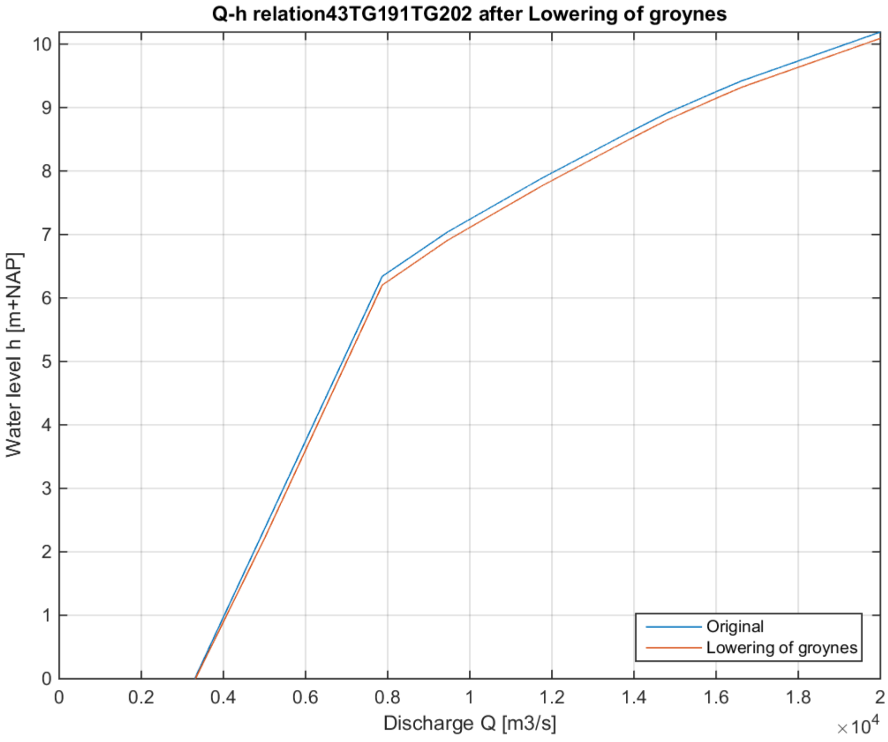


Figure 5-7: Effect of groyne lowering on Q-h relation

5.4.2 Water line

The water line is the water level over the whole reach at a constant discharge. For the original situation this is derived from the Q-h relations for all the separate dike sections. For different discharge levels the water level of the original situation is given in Figure 5-8. This water line is for the bank measurement locations.

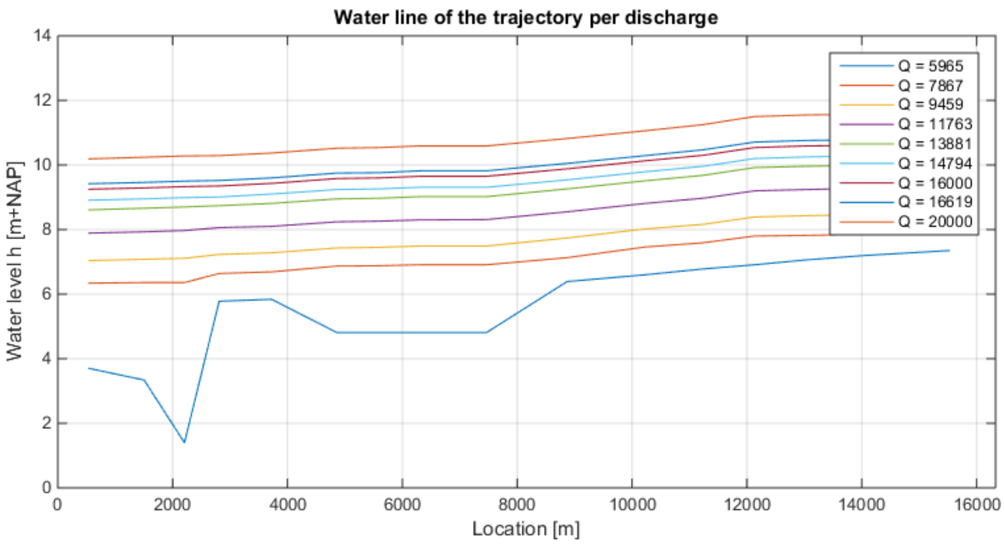


Figure 5-8: Water line for different discharges in reference situation

It can be seen that for the lowest discharge (the lowest blue line) the water level at different points is very different. This is a physical effect. The sections that show very low water levels will not have flooded floodplains at this discharge, for example due to the presence of summer dikes.

5.4.3 Water level reduction

Based on the analysis in Appendix D for all the types of floodplain measures a representative average variant is elaborated in order to be able to make some calculations on the effectiveness of these measures for all discharge levels. All measures have a water line based on one of the real measures. The reach of the effect is in the order of 50 km, which agrees with the measures in the Blokkendoos. The length of the regarded dike reach is only 16 km, but in reality there will also be measures which extend beyond the limits of the dike reach they are primarily designed for.

The maximum water level reduction by the side channel is 20 cm, this is at the most downstream end of the model dike stretch. The water level reduction by the excavation is 6 cm and the repositioning of dikes causes 8 cm water level reduction. This is thus the water level reduction at Q = 17,000 m^3/s. The water level reduction for groyne lowering at this discharge is 10 cm, as has been described above.

For the measures side channel, excavation and repositioning of dikes it is assumed that the measure lies just outside of the model dike reach. The most upstream point of the measure, where the water level reduction is the highest, coincides with the most downstream point of the model reach. In practice the water level reduction increases over the length of the measure. Measures are generally several kilometres long. Since the model dike reach is only 16 km it is chosen that the measures lay just outside the model dike reach. Also the downstream water level rise due to a sudden decrease of the flood conveyance capacity is not taken into account due to this choice.

For the lowering of groynes it is assumed that the effect on the water line and on the Q-h relation is equal for all dike sections. This is a reasonable assumption if it is also assumed that for several sections downstream of the model reach also groyne lowering is applied. The water level reduction will increase in upstream direction over the river stretch where groynes are lowered. This effect will reach a maximum (equilibrium) value at some point, after which the effect stays approximately the same for upstream sections. It is assumed that for the whole model dike reach this equilibrium is reached.

Furthermore it counts for all measures that spatial integration is not taken into account. It is thus assumed that the floodplains and the surroundings offer enough space to construct the measures in such a way that the proposed effect is reached. Also no bifurcations or confluences are taken into account so the water level effect graph will be smooth.

Figure 5-9 shows the water level effect of all the considered measures at a discharge of 20,000 m^3/s. The maximum discharge is 17,000 m^3/s, but for all the measures it is assumed that the absolute water level lowering for discharges higher than 17,000 m^3/s remains constant. The figure thus shows the water level effect relative to the reference situation at MHW.

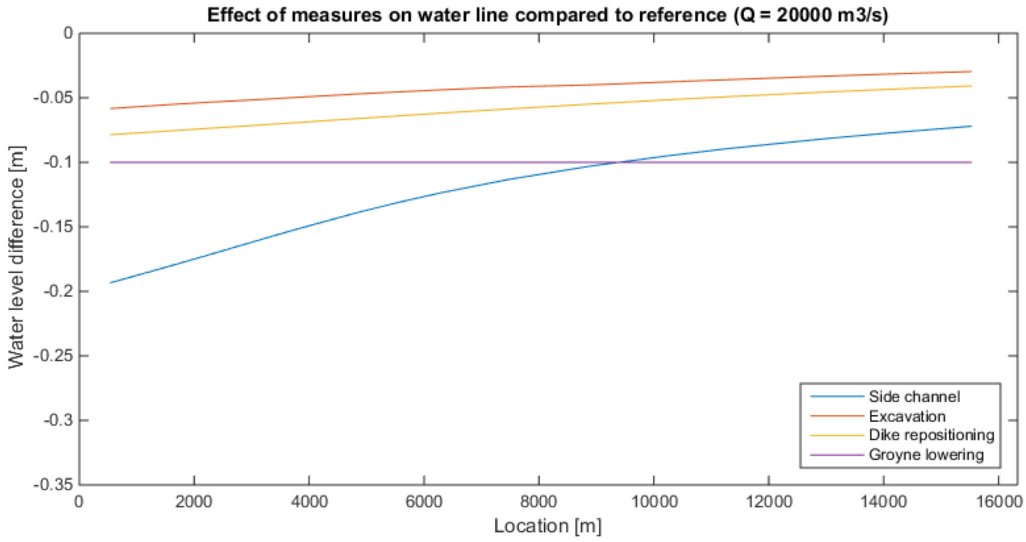


Figure 5-9: Water level effect at Q = 20000 m^3/s

5.4.4 Combination

With the water level reduction at MHW per dike section and the relative Q-h relations for every measure, the new Q-h relations for all the dike section locations (16 in total) can be calculated. This is done by inserting the maximum water level reduction per location in a Matlab script. With the correct measure and the original Q-h relation this script will generate a new Q-h relation for after the measure per location. This script is shown in Appendix E.

With the new Q-h relations the water lines for different discharge levels can be drawn. For a discharge of 9459 m^3/s (relatively low flood) and for 14,794 m^3/s (moderate flood) these lines are drawn in Figure 5-10 and Figure 5-11. The figures show that all measures have a smaller water level reduction for lower discharges, except for the groyne lowering. This measure is for low discharges much more effective than the other measures. Therefore it is expected that it is more effective for geotechnical failure mechanisms. The result of this on the probability of failure will be discussed in chapter 6 Results.

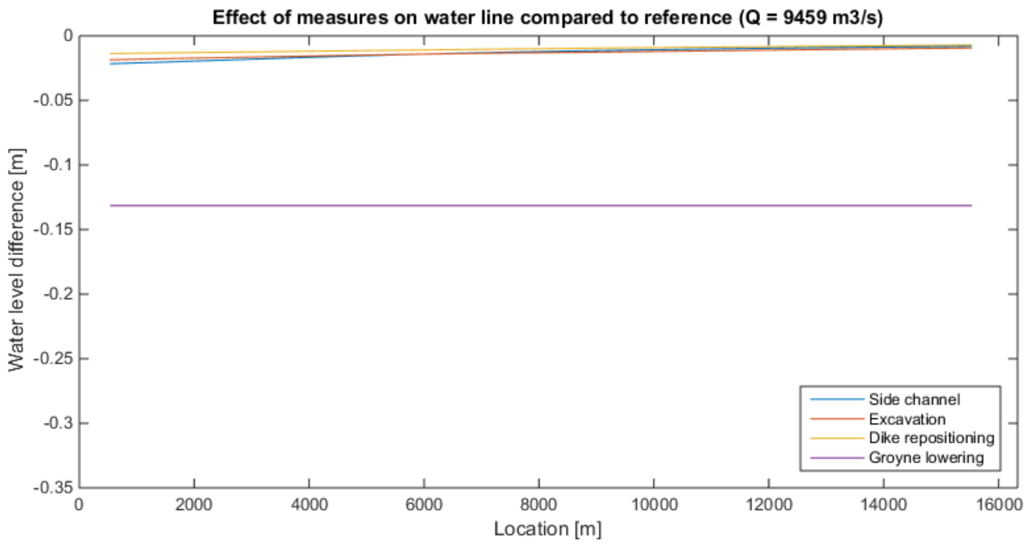


Figure 5-10: Water level effect at Q = 9459 m^3/s

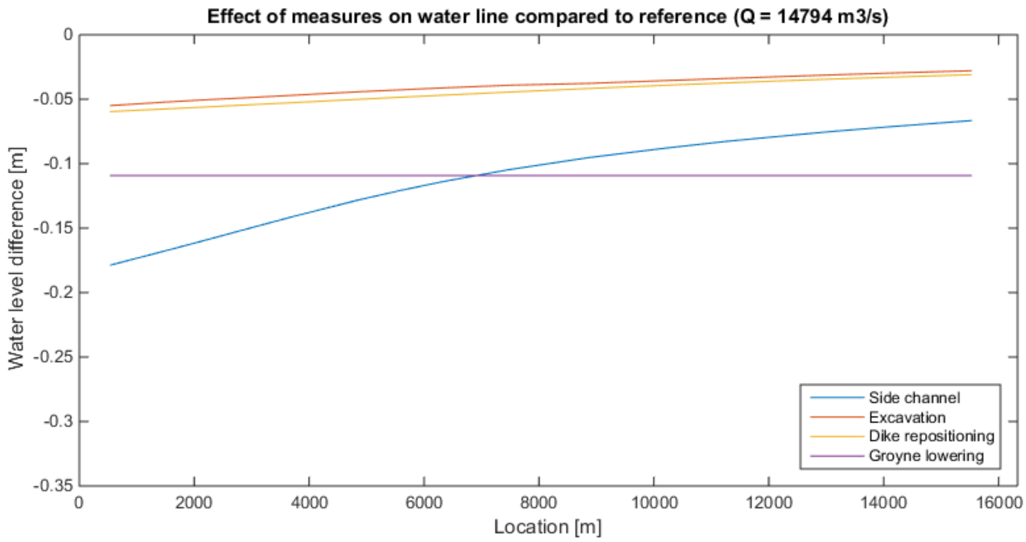


Figure 5-11: Water level effect at Q = 14794 m^3/s

5.4.5 Dike improvements

The effect of dike improvement on the fragility curves for several failure mechanisms has been shown in section 5.2. Dike improvements are constructed over the whole reach, and the amount of improvement is equal over the whole reach.

The effectiveness of dike improvement is difficult to assess since it contains different operations. Dike improvement can be either dike raising, construction of a piping berm or a combination of these. The effect of a dike raise has different effect on the fragility curve for overflow than for piping. The fragility curve for piping is not (or hardly) affected by the dike height, but only by the width of the dike. Also, a piping berm has no effect on the probability of failure for overflow/overtopping. Furthermore also the probability of failure for macro stability will be affected by these interventions, but there is no direct relation for this.

First of all the effectiveness of dike raising of 25, 50, 75 and 100 cm will be assessed for the failure mechanism overflow/overtopping. To assess the effect of pure raising of the dike only the effectiveness on the probability of failure for overtopping will be assessed. The probable effectiveness of dike raising on macro stability and piping will not be regarded, since these failure mechanisms are treated in another way.

In order to simulate the construction of a berm of 4.5, 9, 13.5 and 18 m, the fragility curve will be shifted to the right (higher water levels) with 25, 50, 75 and 100 cm. The piping berm will be constructed in such a way that all the sections meet the requirements for macro stability. Since a berm enhances the moment equilibrium this is a reasonable assumption. Height is not added to the dike so the probability of failure for overflow/overtopping will not change.

Since dike improvement will in practice never be only raising or a piping berm, some combinations should be regarded. Since a higher dike will also lead to a wider dike the following combinations are regarded:

- Combination 1: 0.25 m rise and 4.5 m berm
- Combination 2: 0.5 m rise and 9.0 m berm

- Combination 3: 0.75 m rise and 13.5 m berm
- Combination 4: 1.0 m rise and 18,0 m berm

The probability of failure for macro stability will be reduced to the safety standard per section, thereby assuming that dike improvements will always be constructed in such a way.

5.5 Implementation of strategies

The measures that have been described above should eventually be used to give insight in the applicability of the three strategies that are treated in this research. This section describes how the rapid assessment tool and the results of the calculations on the effectiveness of measures can be used to gain insight in the applicability of the strategies.

5.5.1 Preferred strategy Delta Programme 2015

For the preferred strategy it is of importance to investigate whether spatial measures can be used to fulfil the climate change-induced design task. In chapter 4 it was seen that this design task is very high for failure mechanism overflow/overtopping. The effectiveness of the measures should be calculated for climate scenario W+2050. The effectiveness of the measures on the different failure mechanism will provide insight in the applicability of this strategy.

5.5.2 Only dike improvements

Different amounts and combinations of dike improvements can give insight in whether the design task per dike section and per failure mechanism can be fulfilled by means of dike improvement.

5.5.3 Weakest links

For the weakest links calculation the probability of failure of the three weakest links will be reduced to the prevailing safety level per dike section (defined in 4.1.4 Safety standard per dike section). The three sections that are the weakest links are the sections with the highest probability of failure for overflow/overtopping, macro stability and piping. The probability of failure of these sections is reduced to the safety standard per section. The result of this is shown in Figure 5-12.

The reduction factor induced by treating the weakest links is as much as 9.13. The new probability of failure of this reach is 1/1191. All the spatial measures are combined with this in order to assess the effectiveness of the strategy. The same assessment is done with climate scenario W+2050.

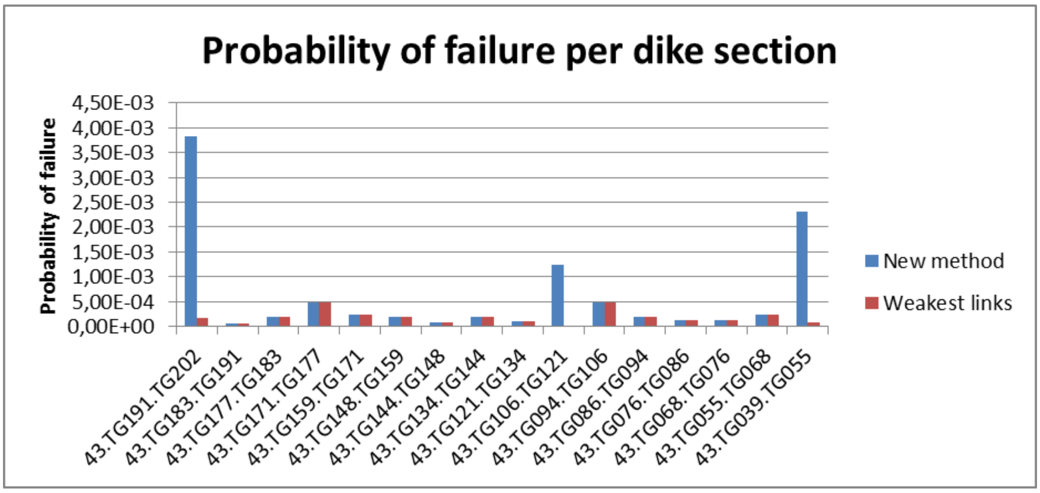


Figure 5-12: Weakest links treatment

6 Results

The Q-h relations as they have been derived in the previous chapter and in Appendix D have been used to calculate the probability of failure after the implementation of measures. This section presents the results and analyses the effectiveness of all the elaborated measures, both spatial measures and dike improvements. The effectiveness per failure mechanism, per dike section and per dike reach will be analysed. The effectiveness will be assessed for HR2006 as well as for W+2050. After this the different strategies to combine the measures to fulfil the design task will be discussed. Also the presumed overestimation by FloRis will be calculated.

6.1 Side channel

The side channel that is considered has a maximum water level reduction of 20 cm. The side channel starts flowing along at 9000 m^3/s. Figure 6-1 shows how the side channel affects the probability of failure. The probability density for high water levels reduces and this reduces the peak in the probability of failure graph (lower graph in the right part of Figure 6-1). The effect of the side channel on the other failure mechanisms is included in Appendix F.

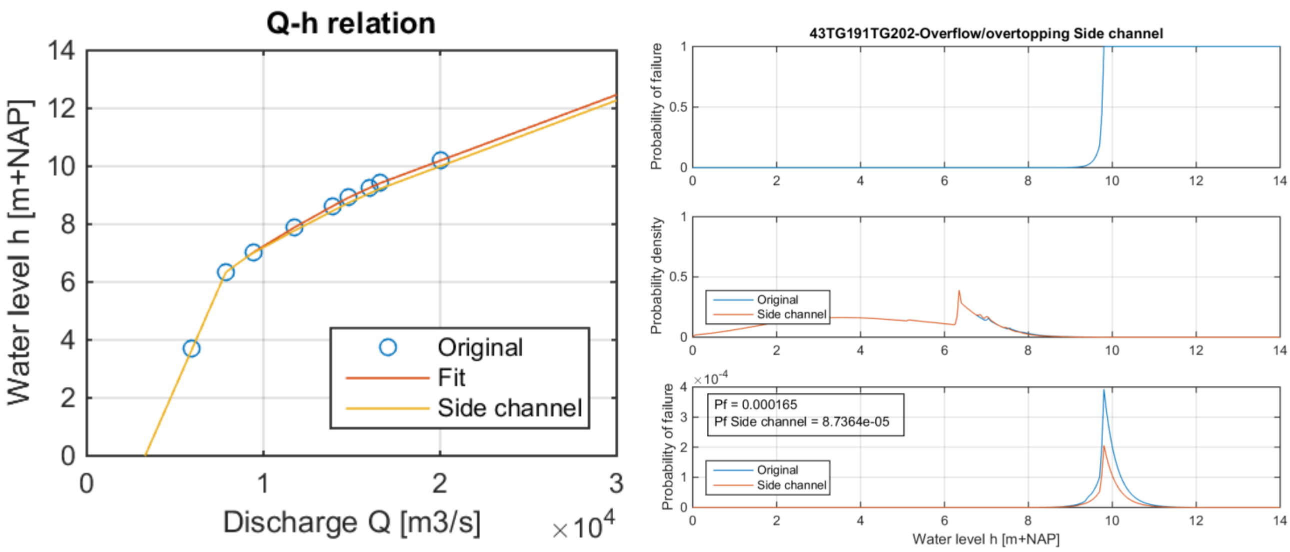


Figure 6-1: Q-h relation with side channel for dike section 43.TG191.TG202 (left) and effect of side channel on probability of failure (right)

6.1.1 HR2006

Table 6-1 shows the reduction factor per failure mechanism and per dike section. The effect per section generally reduces in upstream direction (top-down in the table) and this is also the case for most failure mechanisms. Some sections however seem to differ from this pattern. For example the first section has a reduction factor that is much lower than the next sections. This is caused by the high probability of failure for macro stability. The other sections that differ from this pattern also have a high contribution of geotechnical failure mechanisms to the total probability of failure. The table shows that the measure is less effective for geotechnical failure mechanisms than for overtopping. Even in the most upstream sections the reduction factor for overflow/overtopping is pretty high, even higher than the reduction for most of the sections with piping and macro stability on the downstream end.

Table 6-1: Effectiveness of side channel on failure mechanisms, HR2006

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.89	1.25	-	-	1.27
43.TG183.TG191	1.80	(1.72)	-	-	1.80
43.TG177.TG183	1.75	-	1.18	-	1.43
43.TG171.TG177	1.67	-	1.24	-	1.49
43.TG159.TG171	1.65	1.08	-	-	1.65
43.TG148.TG159	1.58	1.05	-	-	1.58
43.TG144.TG148	1.51	-	-	-	1.51
43.TG134.TG144	1.48	-	-	-	1.48
43.TG121.TG134	1.43	-	-	-	1.43
43.TG106.TG121	1.30	-	-	-	1.30
43.TG094.TG106	1.37	-	1.11	-	1.24
43.TG086.TG094	1.34	-	-	-	1.34
43.TG076.TG086	1.32	-	-	-	1.32
43.TG068.TG076	1.30	-	-	-	1.30
43.TG055.TG068	1.28	1.00	-	1.01	1.21
43.TG039.TG055	1.27	-	1.05	-	1.06
Total	**1.30**	**1.25**	**1.07**	**1.01**	**1.30**

The side channel is very effective at high discharges (Figure 5-9 up to Figure 5-11). This results in a high reduction factor for the probability of failure for overflow/overtopping. At discharges around 8000 m^3/s, where the floodplains just start flowing, the water level reduction is however very small. Since the geotechnical failure mechanisms already start to contribute to the probability of failure at these water levels the effectiveness on these failure mechanisms is small, as is also seen in the table.

Figure 6-2 shows per section the reduction of the probability of failure for overflow/overtopping and the safety standard per section. For some sections on the downstream side the probability of failure is almost reduced to the required level. The more we look upstream the water level effect becomes less and the measure is also less effective. This corresponds with Table 6-1. Even though this measure causes a significant water level reduction this is not sufficient to reach the safety standard for any section. This is even more so for geotechnical failure mechanisms (Appendix F).

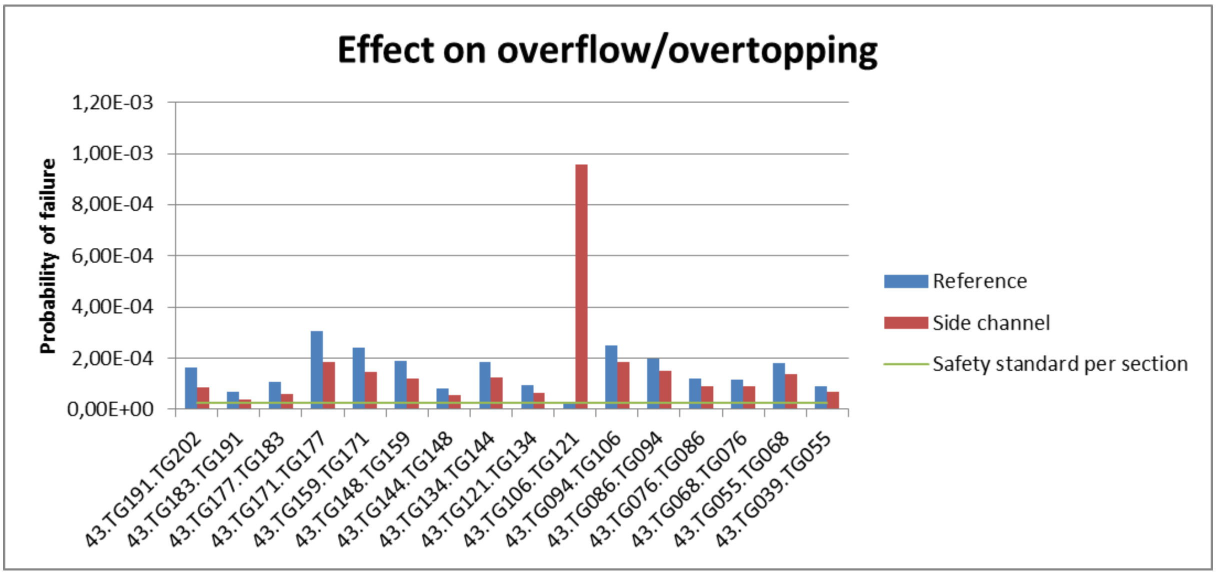


Figure 6-2: Effectiveness of side channel on overflow/overtopping

6.1.2 W+2050

The effectiveness of this measure for climate scenario W+2050 is shown in Table 6-2. When this is compared to Table 6-1 it becomes clear that the effectiveness for overflow/overtopping is less after climate change. Especially the downstream dike sections show large differences. For other failure mechanisms this is different. Especially more upstream dike sections show larger reduction factors after climate change. The total effectiveness of macro stability is however smaller than for HR2006, since this mechanism is mostly dominated by the first dike section, which has a lower reduction factor for W+2050.

Table 6-2: Effectiveness of side channel on failure mechanisms, W+2050

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.58	1.23	-	-	1.27
43.TG183.TG191	1.53	(1.50)	-	-	1.8
43.TG177.TG183	1.50	-	1.19	-	1.43
43.TG171.TG177	1.46	-	1.24	-	1.49
43.TG159.TG171	1.43	1.13	-	-	1.65
43.TG148.TG159	1.39	1.07	-	-	1.58
43.TG144.TG148	1.36	-	-	-	1.51
43.TG134.TG144	1.34	-	-	-	1.48
43.TG121.TG134	1.30	-	-	-	1.43
43.TG106.TG121	1.21	-	-	-	1.3
43.TG094.TG106	1.25	-	1.12	-	1.24
43.TG086.TG094	1.24	-	-	-	1.34
43.TG076.TG086	1.22	-	-	-	1.32
43.TG068.TG076	1.21	-	-	-	1.3
43.TG055.TG068	1.20	1.01	-	1.01	1.21
43.TG039.TG055	1.19	-	1.06	-	1.06
Total	**1.21**	**1.23**	**1.08**	**1.01**	

6.1.3 Total reduction

Figure 6-3 shows the total probability of failure of the whole dike model reach before and after climate change and with and without the side channel. Also the contribution of the different failure mechanisms can be seen. For the situation with HR2006 the total reduction factor for the model dike reach for this measure is 1.19, so a reduction of the probability of failure of about 20%. In the situation with climate scenario W+2050 the total reduction factor is 1.18.

The total probability of failure for this reach is 1/130 per year (and 1/53 per year after climate change) and the safety standard for the whole dike reach (so not this partial reach!) is 1/30,000. A 20% reduction will thus not be sufficient to reach the proposed safety level, or even counteract climate change. The regarded dike reach is highly dominated by geotechnical failure mechanisms like piping and macro stability, as can also be seen in Figure 6-3. Since this measure is not very effective for these failure mechanisms, the total reduction factor is moderate.

One more thing that catches the eye is the relatively large increase of the probability of failure for overflow/overtopping after climate change. This has already been mentioned in section 4.3.2. The increase of the probability of failure due to climate change is much larger than the decrease due to the measure.

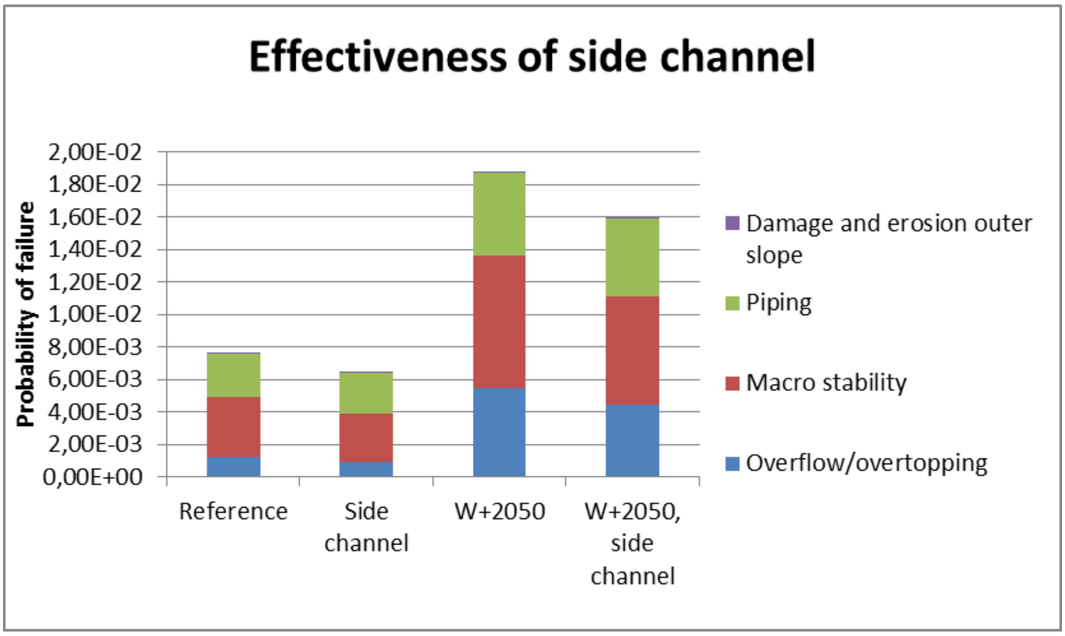


Figure 6-3: Effectiveness side channel on the total probability of failure

6.1.4 Inflow moment

The side channel now starts flowing at Q = 9000 m^3/s, which is comparable with a high water channel. The effectiveness of the measure would change if the moment of inflow would be at another discharge. To assess the sensitivity of this discharge the calculation for the side channel is made for two other moments of inflow: Q = 7867 m^3/s (inflow floodplains) and Q = 12,000 m^3/s. the results are shown in Table 6-3. It seems that the measure is indeed a little more effective when the side channel starts flowing at the same moment as the floodplains. And if this happens at a much higher discharge the total reduction factor for the reach gets significantly lower.

Table 6-3: Effect of inflow moment on reduction factor for reach

Inflow moment [m^3/s]	Reduction factor
9000	1.19
7867	1.22
12000	1.10

The moment of inflow is not very important for the mechanism overflow/overtopping, since the fragility curve is very steep and the probability of failure graph is a very narrow peak which occurs at high water levels. However, since this reach is dominated by geotechnical failure mechanisms, it is recommended that the side channel starts flowing at the lowest possible discharge in order to get the maximum reduction. This is in this case the same moment as the floodplains start flowing. If for example the summer dikes are lowered the side channel could even start flowing at lower water levels. At these water levels no water reaches the winter dike yet, so it is unnecessary to do so. This could also have negative side effects regarding navigation or sedimentation.

6.2 Excavation

Excavation of the floodplains gives a maximum water level reduction of 6 cm. This is much less than the maximum reduction caused by the side channel. For discharges around 9500 m^3/s the water level reduction is almost equal to the reduction due to the side channel, but for higher discharged the difference increases (Figure 5-9 to Figure 5-11).

Figure 6-4 shows how the excavation affects the probability of failure for overflow/overtopping. This is comparable to the side channel, only the water level reduction is clearly smaller. The water level reduction due to excavation increases fast for low water levels, the effect of this is however very small due to the small effect at MHW. Figure 6-4 clearly shows that the reduction of the probability of failure is also smaller. For other failure mechanisms this is shown in Appendix F.

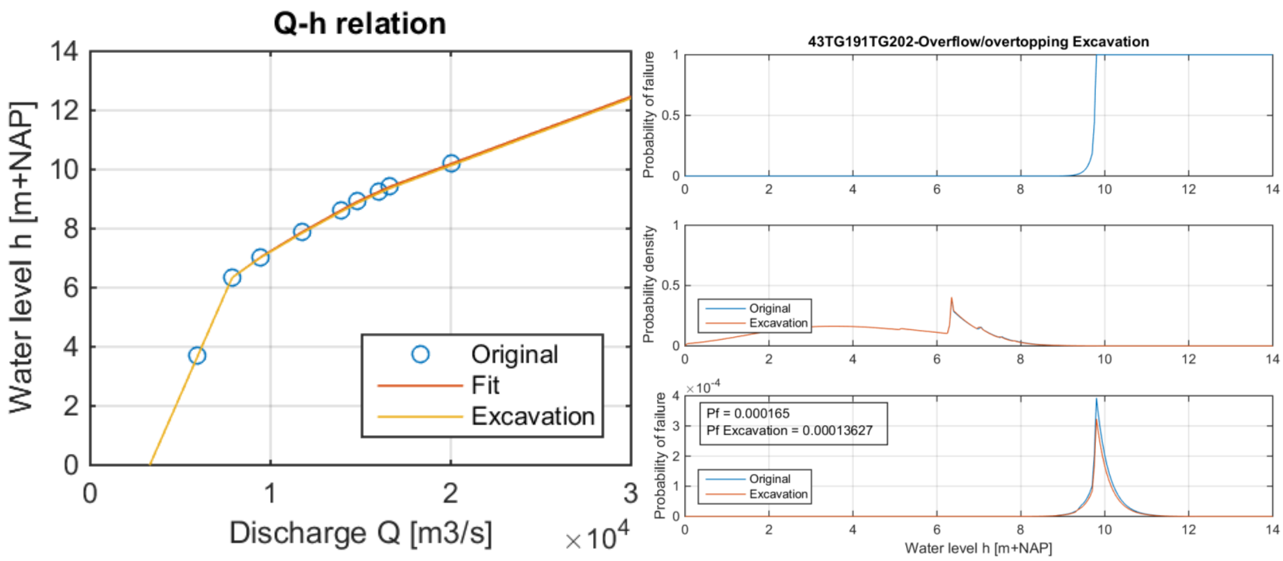


Figure 6-4: Q-h relation with excavation (left) and effect of excavation on probability of failure (right)

6.2.1 HR2006

Table 6-4 shows the effectiveness per failure mechanism and per dike section. Even though the values are lower, it is comparable with the side channel. The effectiveness for geotechnical failure mechanisms is smaller than the effectiveness for overflow/overtopping, especially further upstream.

Figure 6-5 shows that the measure is also not sufficient to reach the safety standard per dike section. This was to be expected since the effect is smaller than with the side channel.

Table 6-4: Effectiveness of excavation on failure mechanisms, HR2006

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.21	1.08	-	-	1.09
43.TG183.TG191	1.20	1.18	-	-	1.20
43.TG177.TG183	1.19	-	1.06	-	1.13
43.TG171.TG177	1.18	-	1.08	-	1.14
43.TG159.TG171	1.18	1.03	-	-	1.18
43.TG148.TG159	1.17	1.02	-	-	1.17
43.TG144.TG148	1.15	-	-	-	1.15
43.TG134.TG144	1.15	-	-	-	1.15
43.TG121.TG134	1.14	-	-	-	1.14
43.TG106.TG121	1.11	-	-	-	1.11
43.TG094.TG106	1.13	-	1.05	-	1.09
43.TG086.TG094	1.13	-	-	-	1.13
43.TG076.TG086	1.12	-	-	-	1.12
43.TG068.TG076	1.11	-	-	-	1.11
43.TG055.TG068	1.11	1.00	-	1.01	1.08
43.TG039.TG055	1.10	-	1.03	-	1.03
Total	**1.11**	**1.08**	**1.03**	**1.01**	

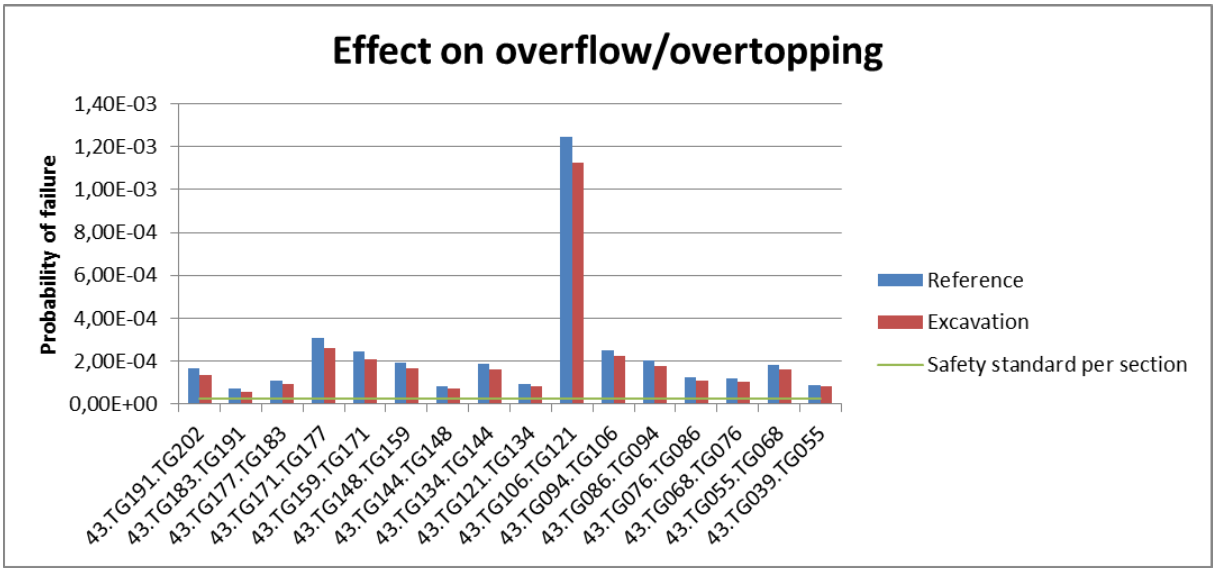


Figure 6-5: Effect of excavation on overflow/overtopping

6.2.2 W+2050

Table 6-5 shows the effectiveness of the excavation after climate change. The effectiveness of overflow/overtopping again decreases, where some of the sections show larger reductions for the other failure mechanisms after climate change.

Table 6-5: Effectiveness of excavation on failure mechanisms, W+2050

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.15	1.07	-	-	1.08
43.TG183.TG191	1.14	1.13	-	-	1.14
43.TG177.TG183	1.13	-	1.06	-	1.12
43.TG171.TG177	1.13	-	1.07	-	1.12
43.TG159.TG171	1.12	1.05	-	-	1.12
43.TG148.TG159	1.12	1.03	-	-	1.12
43.TG144.TG148	1.11	-	-	-	1.11
43.TG134.TG144	1.11	-	-	-	1.11
43.TG121.TG134	1.10	-	-	-	1.10
43.TG106.TG121	1.08	-	-	-	1.08
43.TG094.TG106	1.09	-	1.05	-	1.08
43.TG086.TG094	1.09	-	-	-	1.09
43.TG076.TG086	1.08	-	-	-	1.08
43.TG068.TG076	1.08	-	-	-	1.08
43.TG055.TG068	1.08	1.00	-	1.01	1.07
43.TG039.TG055	1.07	-	1.03	-	1.03
Total	**1.08**	**1.07**	**1.03**	**1.01**	

6.2.3 Total reduction

The total effectiveness of this measure on the model dike reach is factor 1.07 for the HR2006 situation. For the situation with W+2050 the reduction factor is 1.06. The effectiveness again declines after climate change. Also the effect is rather small compared to the side channel and it is not sufficient to fulfil the design task at all. Figure 6-6 shows that despite the small reduction factor the largest part of the reduction comes from geotechnical failure mechanisms.

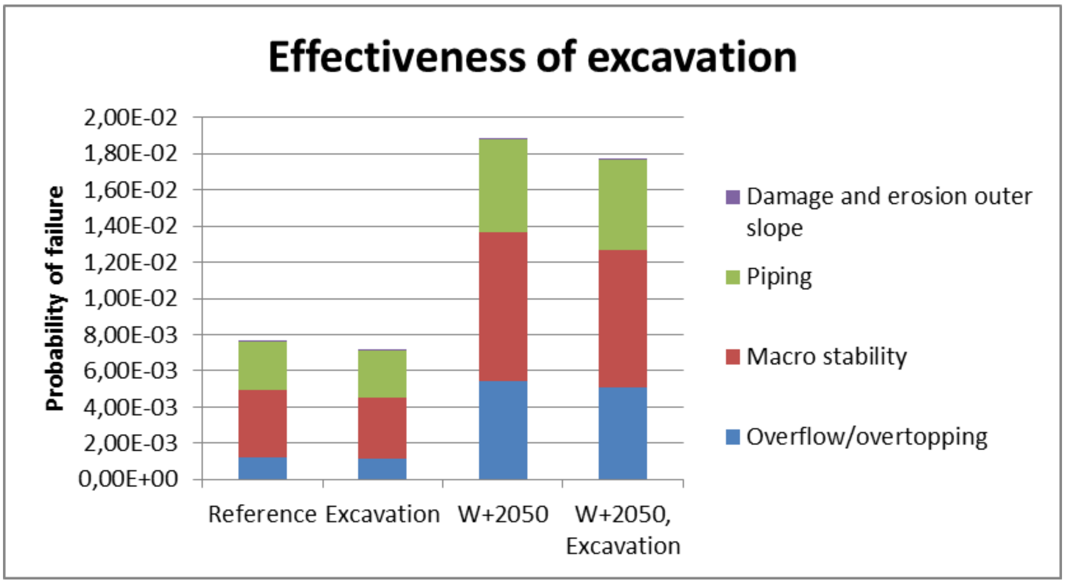


Figure 6-6: Effectiveness excavation on the total probability of failure

6.3 Repositioning of dikes

The repositioning of dikes has a maximum effect of 8 cm. Figure 6-7 shows how the measure affects the probability of failure for overflow/overtopping. This is comparable to the other floodplain measures. For the other failure mechanisms this is included in Appendix F.

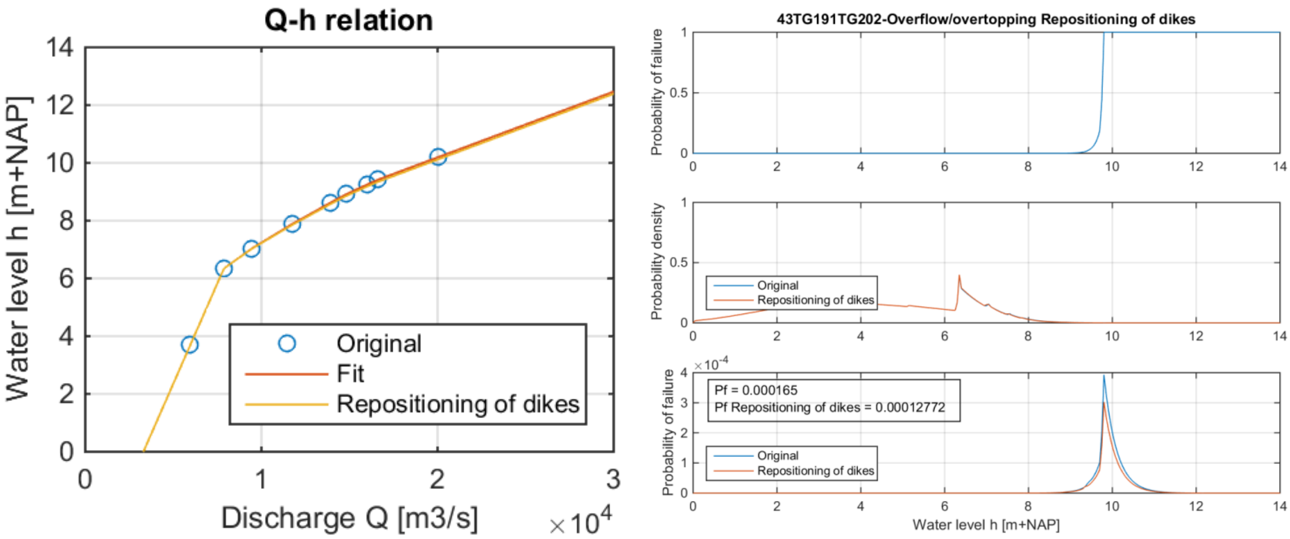


Figure 6-7: Q-h relation with dike repositioning (left) and effect of dike repositioning on probability of failure (right)

6.3.1 HR2006

Table 6-6 shows that the reduction of the failure mechanism overflow/overtopping is appreciably larger than for the geotechnical failure mechanisms. The effectiveness on overtopping is larger than for the excavation, since the water level reduction at MHW is larger. The effectiveness for the geotechnical failure mechanisms is in the same order as for the excavation, even though the water level effect at MHW is larger than with excavation.

Table 6-6: Effectiveness of dike repositioning on failure mechanisms, HR2006

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.29	1.08	-	-	1.08
43.TG183.TG191	1.28	1.24	-	-	1.28
43.TG177.TG183	1.27	-	1.06	-	1.17
43.TG171.TG177	1.24	-	1.08	-	1.18
43.TG159.TG171	1.25	1.03	-	-	1.25
43.TG148.TG159	1.24	1.02	-	-	1.24
43.TG144.TG148	1.22	-	-	-	1.22
43.TG134.TG144	1.21	-	-	-	1.21
43.TG121.TG134	1.20	-	-	-	1.20
43.TG106.TG121	1.13	-	-	-	1.13
43.TG094.TG106	1.19	-	1.05	-	1.12
43.TG086.TG094	1.18	-	-	-	1.18
43.TG076.TG086	1.17	-	-	-	1.17
43.TG068.TG076	1.16	-	-	-	1.16
43.TG055.TG068	1.15	1.00	-	1.00	1.12
43.TG039.TG055	1.14	-	1.02	-	1.03
Total	**1.13**	**1.08**	**1.03**	**1.00**	**1.13**

The explanation is found in the Q-h relations (Figure 5-6). The effect of repositioning of dikes increases linearly from the moment the floodplains start flowing, and the effect of the excavation increases very fast for lower discharges and then diverges to the maximum value. In Figure 5-9 until Figure 5-11 it was seen that the difference with the excavation measure only becomes significant for high discharges. For low water levels the water level reduction is even a little smaller than for the excavation, so the reduction of the probability of failure of geotechnical failure mechanisms should be approximately equal to the excavation, which is indeed the case. The effectiveness per dike section for overflow/overtopping is shown in Figure 6-8.

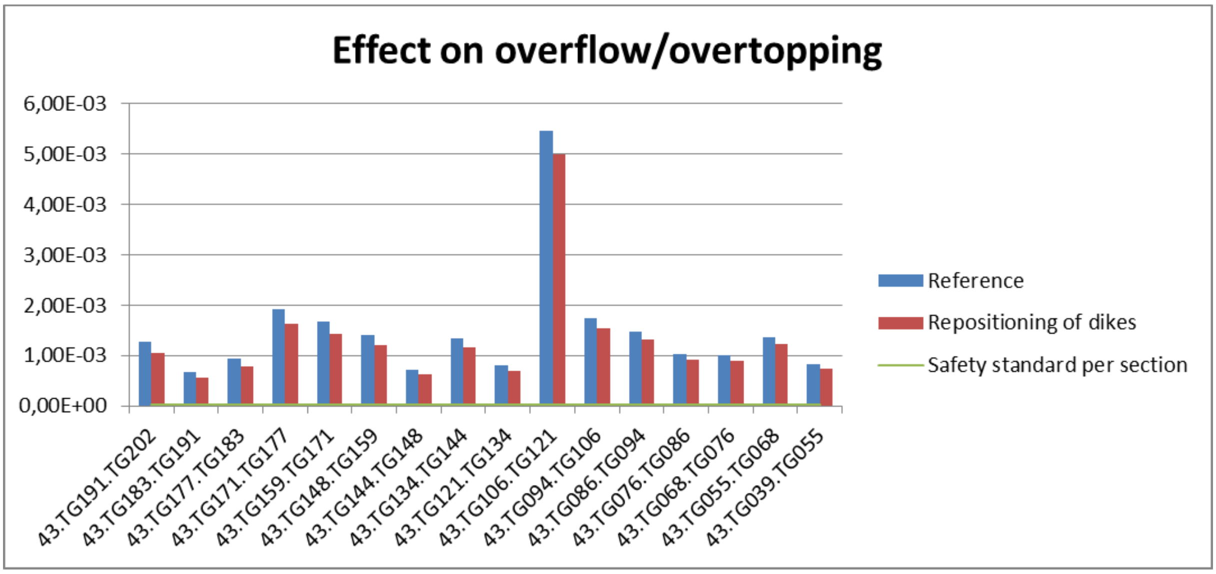


Figure 6-8: Effectiveness of repositioning of dikes on overflow/overtopping

6.3.2 W+2050

Table 6-8 shows the effectiveness of dike repositioning on the failure mechanisms for climate scenario W+2050. The effect is comparable with the side channel and the excavation.

Table 6-7: Effectiveness of dike repositioning on failure mechanisms, W+2050

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.20	1.07	-	-	1.08
43.TG183.TG191	1.19	1.18	-	-	1.19
43.TG177.TG183	1.19	-	1.07	-	1.17
43.TG171.TG177	1.18	-	1.08	-	1.16
43.TG159.TG171	1.18	1.05	-	-	1.18
43.TG148.TG159	1.17	1.03	-	-	1.17
43.TG144.TG148	1.16	-	-	-	1.16
43.TG134.TG144	1.15	-	-	-	1.15
43.TG121.TG134	1.15	-	-	-	1.15
43.TG106.TG121	1.09	-	-	-	1.09
43.TG094.TG106	1.13	-	1.05	-	1.11
43.TG086.TG094	1.12	-	-	-	1.12
43.TG076.TG086	1.12	-	-	-	1.12
43.TG068.TG076	1.11	-	-	-	1.11
43.TG055.TG068	1.11	1.00	-	1.00	1.10
43.TG039.TG055	1.10	-	1.03	-	1.04
Total	**1.09**	**1.07**	**1.03**	**1.00**	

6.3.3 Total reduction

The reduction factor for the whole reach is 1.07 for both climate scenarios. This is the same as the excavation measure, even though this measure has more effect on the mechanism overflow overtopping. The large influence of geotechnical failure mechanisms on the probability of failure of the reach causes this (Figure 6-9).

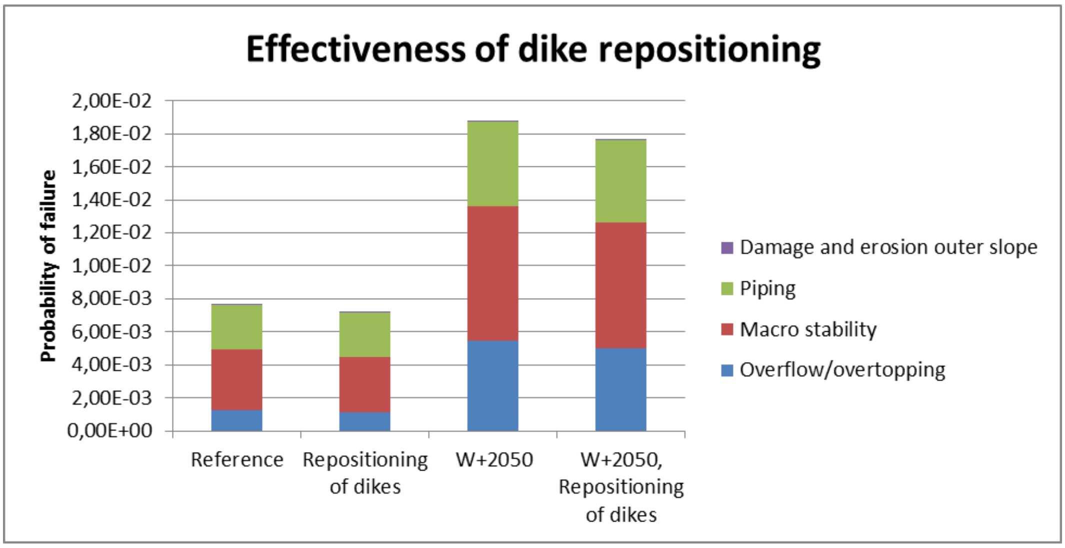


Figure 6-9: Effectiveness dike repositioning and weakest links

6.4 Lowering of groynes

Lowering of groynes has a different effect on the Q-h relation then the floodplain measures, and therefore it will also have different effect on the probability of failure. The reduction of the water level is larger for low water levels so the effectiveness on geotechnical failure mechanisms is higher than for the floodplain measures. This can be seen in the middle graphs in Figure 6-10.

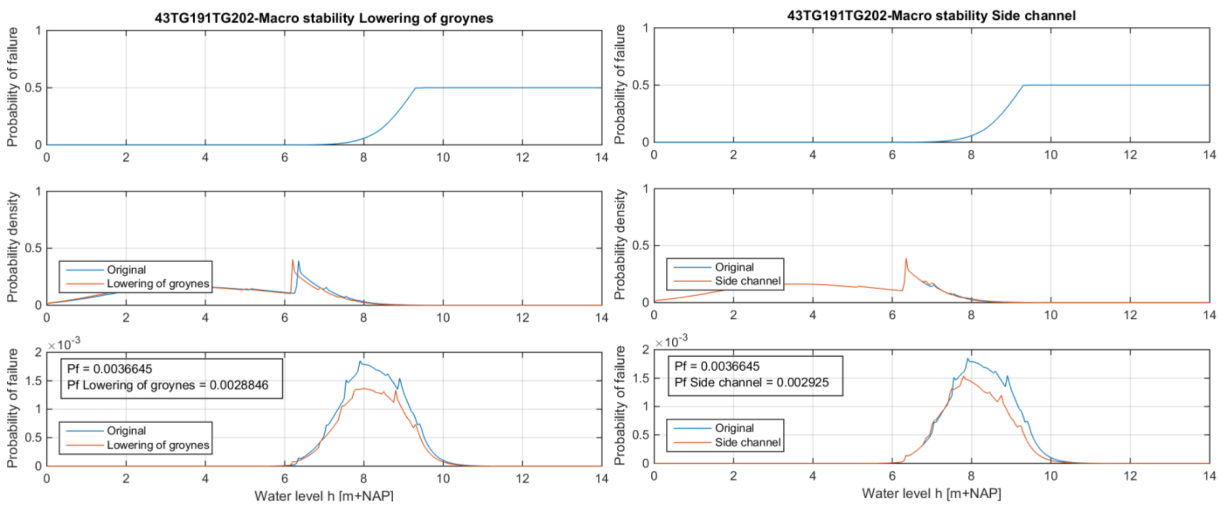


Figure 6-10: Effect of groyne lowering and side channel on macro stability

6.4.1 HR2006

Table 6-8 shows the effectiveness of groyne lowering per failure mechanism and per section.

Table 6-8: Effectiveness of groyne lowering on failure mechanisms, HR2006

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.39	1.27	-	-	1.28
43.TG183.TG191	1.39	1.36	-	-	1.39
43.TG177.TG183	1.39	-	1.25	-	1.32
43.TG171.TG177	1.38	-	1.30	-	1.35
43.TG159.TG171	1.39	1.21	-	-	1.39
43.TG148.TG159	1.39	1.17	-	-	1.39
43.TG144.TG148	1.38	-	-	-	1.38
43.TG134.TG144	1.38	-	-	-	1.38
43.TG121.TG134	1.38	-	-	-	1.38
43.TG106.TG121	1.32	-	-	-	1.32
43.TG094.TG106	1.39	-	1.29	-	1.34
43.TG086.TG094	1.39	-	-	-	1.39
43.TG076.TG086	1.38	-	-	-	1.38
43.TG068.TG076	1.38	-	-	-	1.38
43.TG055.TG068	1.38	1.07	-	1.13	1.32
43.TG039.TG055	1.39	-	1.26	-	1.26
Total	**1.32**	**1.27**	**1.26**	**1.13**	

The effectiveness for geotechnical failure mechanisms is higher than for all of the considered the floodplain measures. The effectiveness on macro stability is equal to the effectiveness of the side channel on this mechanism, even though the water level effect at MHW is only half as big. The effectiveness on piping is even larger. This proofs that a measure with large water level reduction for low discharges is more effective for geotechnical failure mechanisms. It should be mentioned that comparing these measures might give wrong insight since the water level reduction at MHW by the floodplain measures and the groyne lowering is not equal.

The effectiveness for this measure would normally increase over the length of the stretch over which the groynes are lowered. In this research it is assumed that the effect is equal over the whole stretch. The values in Table 6-8 should thus be assessed per section and not per reach.

6.4.2 W+2050

Table 6-9 shows the effectiveness of groyne lowering on the failure mechanisms for climate scenario W+2050. The effectiveness for all failure mechanisms has decreased significantly. For the other measures it was seen that the effectiveness for geotechnical failure mechanisms hardly decreased, or even shown a slight increase.

Table 6-9: Effectiveness of groyne lowering on failure mechanisms, W+2050

Reduction factor	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.27	1.20	-	-	1.21
43.TG183.TG191	1.27	1.25	-	-	1.27
43.TG177.TG183	1.27	-	1.20	-	1.25
43.TG171.TG177	1.26	-	1.22	-	1.26
43.TG159.TG171	1.27	1.19	-	-	1.27
43.TG148.TG159	1.27	1.15	-	-	1.27
43.TG144.TG148	1.26	-	-	-	1.26
43.TG134.TG144	1.27	-	-	-	1.27
43.TG121.TG134	1.27	-	-	-	1.27
43.TG106.TG121	1.22	-	-	-	1.22
43.TG094.TG106	1.27	-	1.22	-	1.26
43.TG086.TG094	1.27	-	-	-	1.27
43.TG076.TG086	1.26	-	-	-	1.26
43.TG068.TG076	1.26	-	-	-	1.26
43.TG055.TG068	1.26	1.07	-	1.12	1.26
43.TG039.TG055	1.27	-	1.20	-	1.21
Total	**1.22**	**1.20**	**1.21**	**1.12**	

6.4.3 Total reduction

The total effectiveness that is calculated for the reach is 1.28 for the situation with HR2006. This is higher than for the other measures, since lowering the groynes is effective on geotechnical failure mechanisms which determine the probability of failure in this reach. After climate change the effectiveness decreases to 1.21. This decrease in the effectiveness is remarkable, because it shows that the effectiveness of geotechnical failure mechanisms has a large influence on the total reduction factor. This makes sense since the failure probability for these mechanisms is very large in the model dike reach.

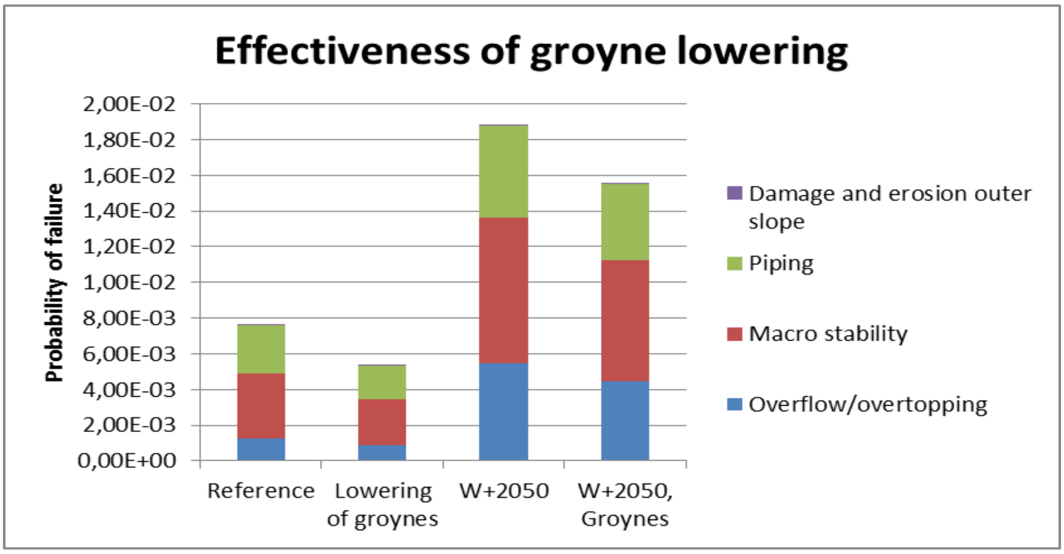


Figure 6-11: Effectiveness groyne lowering and weakest links

6.5 Dike improvement

In dike improvements distinction is made between raising the dike and applying a piping berm. These are first assessed separated and also combinations of these measures have been calculated. The dike improvements have not been calculated for the situation in 2050, since the climate change-induced part of the design task has to be resolved my means of spatial measures. The design task induced by the new safety standards and the shortcomings of the dike due to weak spots or overdue maintenance has to be resolved by dike improvements.

6.5.1 Dike raising

Calculations have been made for dike raising with 25 cm, 50 cm, 75 cm and 100 cm. It was assumed that only the probability of failure for mechanism overflow/overtopping was affected. The effectiveness on the probability of failure for overflow/overtopping is shown in Table 6-10.

Figure 6-12 also shows the effectiveness of dike raising on the probability of failure for overtopping. A dike raise of 25 cm seems insufficient to reduce the probability of failure of any of the sections to below the safety standard per section. A rise of 0.5 m reduces almost half of the sections below the safety standard. With a rise of 0.75 there are only two sections left from which the probability of failure is too high, and raising the dike with 1.0 m is almost sufficient; there is only one section left which doesn't meet the safety standard per section for overflow/overtopping.

Table 6-10: Effectiveness of different amounts of dike raising on overflow/overtopping

Section	25 cm	50 cm	75 cm	100 cm
43.TG191.TG202	2.28	5.23	12.00	27.59
43.TG183.TG191	2.26	5.12	11.61	26.39
43.TG177.TG183	2.26	5.12	11.61	26.35
43.TG171.TG177	2.22	4.98	11.26	25.64
43.TG159.TG171	2.27	5.19	11.90	27.32
43.TG148.TG159	2.29	5.25	12.05	27.70
43.TG144.TG148	2.23	5.00	11.26	25.40
43.TG134.TG144	2.24	5.06	11.50	26.21
43.TG121.TG134	2.23	4.98	11.16	25.11
43.TG106.TG121	1.96	4.29	9.72	22.19
43.TG094.TG106	2.30	5.28	12.16	27.99
43.TG086.TG094	2.27	5.18	11.80	26.88
43.TG076.TG086	2.25	5.08	11.45	25.84
43.TG068.TG076	2.25	5.08	11.47	25.89
43.TG055.TG068	2.24	5.05	11.39	25.70
43.TG039.TG055	2.27	5.18	11.82	27.03
Total	**1.96**	**4.29**	**9.72**	**22.19**

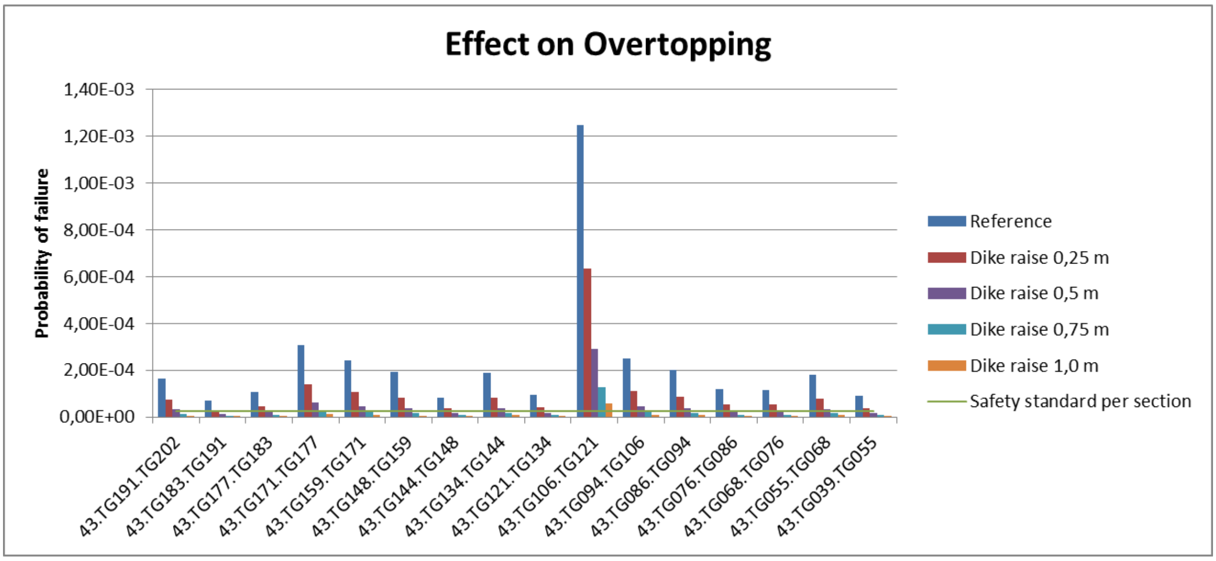


Figure 6-12: Effect of dike raising on overflow/overtopping

However, if dike raising does not affect the probability of failure for all the other failure mechanisms, the total reduction factor will still be low. This is shown in Table 6-11. Even though a dike raise of 1.0 m provides reduction factors up to 28 for overtopping, the total reduction is limited due to the dominance of the geotechnical failure mechanisms.

Table 6-11: Reduction factors for dike raising

Dike raise [cm]	Reduction factor
25	1.09
50	1.14
75	1.17
100	1.18

6.5.2 Piping berm

In order to simulate the construction of a berm of 4.5, 9, 13.5 and 18 m, the fragility curve is shifted to the higher water levels with 25, 50, 75 and 100 cm. It is assumed that the piping berm is constructed in such a way that all the sections meet the requirements for macro stability. Since a berm enhances the moment equilibrium this is a reasonable assumption. Height is not added to the dike so the probability of failure for overflow/overtopping will not change.

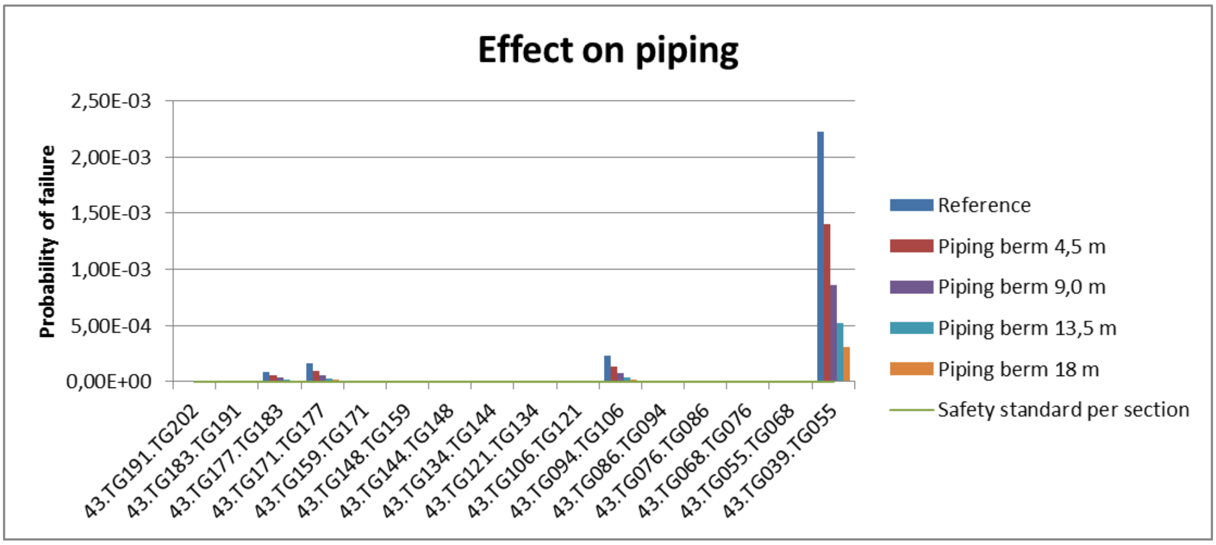


Figure 6-13: Effect of piping berm on piping

Figure 6-13 shows the effectiveness of piping berms in the probability of failure for piping. Most sections show no values because these sections are not recorded in FloRis. The reduction of the probability of failure for piping is significant, yet a berm of 18.0 m is not sufficient to reduce the probability of failure of any of the dike sections below the safety standard per section. The berm of 18.0 m has a reduction factor of 7.3 for the section with the highest probability of failure. Since the berm also affects the probability of failure of macro stability the total reduction factor for the whole reach is severe. This is shown in Table 6-12.

Table 6-12: Reduction factors for piping berm

Berm [m]	Reduction factor
4.5	2.57
9.0	3.30
13.5	4.02
18.0	4.64

6.5.3 Combinations

Since a higher dike will also lead to a wider dike the following combinations are regarded:

- Combination 1: 0.25 m rise and 4.5 m berm
- Combination 2: 0.5 m rise and 9.0 m berm
- Combination 3: 0.75 m rise and 13.5 m berm
- Combination 4: 1.0 m rise and 18.0 m berm

The probability of failure for macro stability will again be reduced to the safety standard per section.

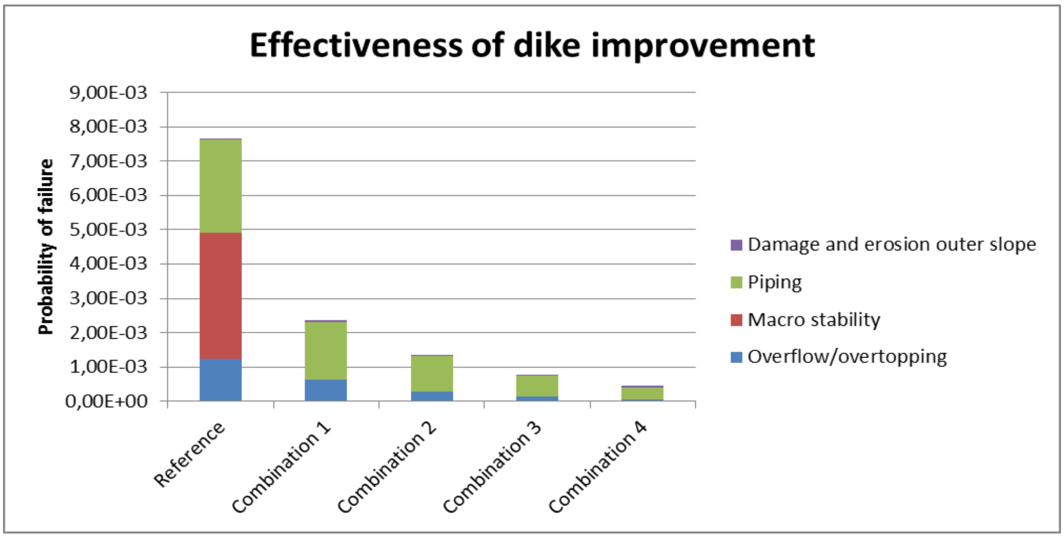


Figure 6-14: Effectiveness of dike improvement per failure mechanism

Figure 6-14 shows the effectiveness of the mentioned combinations of dike improvement. The reduction factors are shown in Table 6-13. The fact that the probability of failure for macro stability is highly reduced plays an important role in the total reduction factor. For combination 4 the residual probability of failure is almost entirely determined by piping. The reduction factors are significant in comparison with the reductions that the considered spatial measures give. However, it is not yet enough to reach the safety standard for the model dike reach (1/30,000).

Table 6-13: Reduction factors for dike improvement

Combination	Reduction factor	Probability of failure
1	3.24	1/422
2	5.62	1/733
3	9.74	1/1269
4	16.64	1/2170

6.6 FloRis calculation

The FloRis project made calculations on the effectiveness of the whole RfR project. For these calculations it was assumed that the water level reduction at MHW is equal to the hydraulic task imposed by RfR. This water level reduction was inserted in PC-Ring as a reduction of the water level via parameter ‘modelfout in locale waterstand’ for all locations. In this way the water level effect is equal for all discharges. In the foregoing it was seen that the water level reducing effect for lower discharges will be smaller though. The FloRis calculations thus overestimated the water level effect for most of the discharges, leading to an overestimation of the effectiveness of the RfR project.

To assess the impact of this overestimation two calculations have been made. First of all the effect of the floodplain measures has been calculated in PC-Ring in a comparable way as FloRis calculated the effect of RfR. For all dike sections the local water level reduction is entered in PC-Ring. This is the effect at MHW for the measure. The water level reduction is then equal for all discharges, so PC-Ring overestimates the effect of the separate measures.

This calculation is made for the floodplain measures. Table 6-14 shows the effectiveness as calculated with PC-Ring and with the new method, presented as a reduction factor of the probability of failure. This shows that the calculation made with the method of FloRis indeed gives an overestimation of the reduction factor. The overestimation is systematic, the reduction calculated by FloRis is factor 2 larger, when looking at the return period in years. The difference between both methods is not only due to overestimation by the FloRis approach, but also model errors play a role. It is however not expected that this will have large impact on the results shown in the table.

Table 6-14: Comparison reduction factor PC-Ring calculation and new method

Measure	Effectiveness PC-Ring	Effectiveness new method
Side channel	1.36	1.19
Excavation	1.11	1.07
Dike repositioning	1.15	1.07

The second calculation is made with the new method. The effectiveness of the RfR programme in the considered reach is calculated. The water level reduction at MHW is equal to the water level reduction caused by RfR, so these are the same values as used in the PC-Ring calculations. For lower discharges the effectiveness declines linearly to zero on the moment the floodplains start flowing. This leads to a higher probability of failure than PC-Ring and thus lower effectiveness of the RfR programme. This is shown in Table 6-15. The values show that the PC-Ring calculation highly overestimated the effectiveness of RfR for this reach. The overestimation of the reduction of the probability of failure is again approximately factor 2 when looking at the return period in years.

Table 6-15: Effectiveness of RfR for the reach

Method	Reduction
PC-Ring	1.53
New method	1.24

For this reach the effectiveness following PC-Ring is larger than for the whole dike ring (section 4 Design task). It is thus advisable that a calculation with the new method is made for the whole dike ring in order to see how much the total overestimation by FloRis is. Nevertheless this outcome leads to the conclusion that the effectiveness of RfR is indeed overestimated by FloRis.

6.7 Strategies

In chapter 5 different strategies have been explained. This section will elaborate the results and the applicability of these strategies.

6.7.1 Preferred strategy of DP15

The Delta programme aims on fulfilling the climate change-induced design task by means of spatial measures. The climate change-induced part of the design task (or the impact factor IF) is 2.45. All of the considered spatial measures have a reduction factor that is a lot smaller, both for the situation with HR2006 and with W+2050. This means that plurality of measures is required in this relatively short dike reach.

It has been seen in section 4.2.1 that the reduction factor due to the old RfR programme is approximately 1.3. For the whole dike ring area 43 the IF due to climate change was 1.3 as well. This supports the strategy by DP15 that climate change can be counteracted by means of spatial measures. However, in section 6.6 it has been proven that the calculation made by FloRis overestimated the effectiveness of RfR. This means there will be more measures needed to get the required effect than was expected before.

6.7.2 Weakest links

The model dike reach has a very high probability of failure for geotechnical failure mechanisms. This causes the reduction factors of the considered spatial measures to be low, since they have small effect on these failure mechanisms. If the weakest links are treated separately, the effectiveness of these measures increases.

Reducing the probability of failure of the three weakest links to the safety standards causes a reduction factor of 9.13, which was determined in section 5.5.3. When the same weakest links are treated for the situation after climate change, the reduction factor is 6.02. The total probability of failure is then reduced from 1/53 to 1/320 per year.

Table 6-16 and Table 6-17 show the reduction factor caused by different measures combined with the weakest links treatment. The tables show the values for the reference situation and after climate change respectively. In both tables the first column shows the reduction factor of the measure without treating the weakest links. The second column shows the reduction factor of the measure combined with the weakest links. The third column shows the part of the total reduction factor that is caused by the measure. The most right column shows the probability of failure after the measure and the weakest links treatment have been implemented.

Table 6-16: Reduction factor with and without weakest links (HR2006)

Measure	Without weakest links	With weakest links	After weakest links	Probability of failure
Side channel	1.19	11.87	1.30	1/1547
Excavation	1.07	10.03	1.09	1/1308
Dike repositioning	1.07	10.22	1.12	1/1332
Lowering groynes	1.28	11.94	1.31	1/1557

Table 6-17: Reduction factor with and without weakest links (W+2050)

Measure	Without weakest links	With weakest links	After weakest links	Probability of failure
Side channel	1.18	7.79	1.29	1/414
Excavation	1.06	6.62	1.10	1/352
Dike repositioning	1.07	6.81	1.13	1/362
Lowering groynes	1.21	7.49	1.24	1/398

The tables show that the reduction factor of the measure increases if the weakest links have been treated. If the values are compared to the results per measure that have been presented earlier in this chapter, it is seen that the effectiveness of the measures approaches the effectiveness of the reduction factor for the failure mechanism overflow/overtopping. Treating the weakest links has thus reduced the probability of failure of the geotechnical failure mechanisms and made overflow/overtopping the dominant failure mechanism. This can also be seen in Figure 6-15.

The figure also shows the effect of the side channel in combination with the weakest links (3rd and 6th column). Especially the 5th and 6th column show that the side channel is especially effective on the failure mechanism overflow/overtopping, which is in accordance with the results that have been presented earlier.

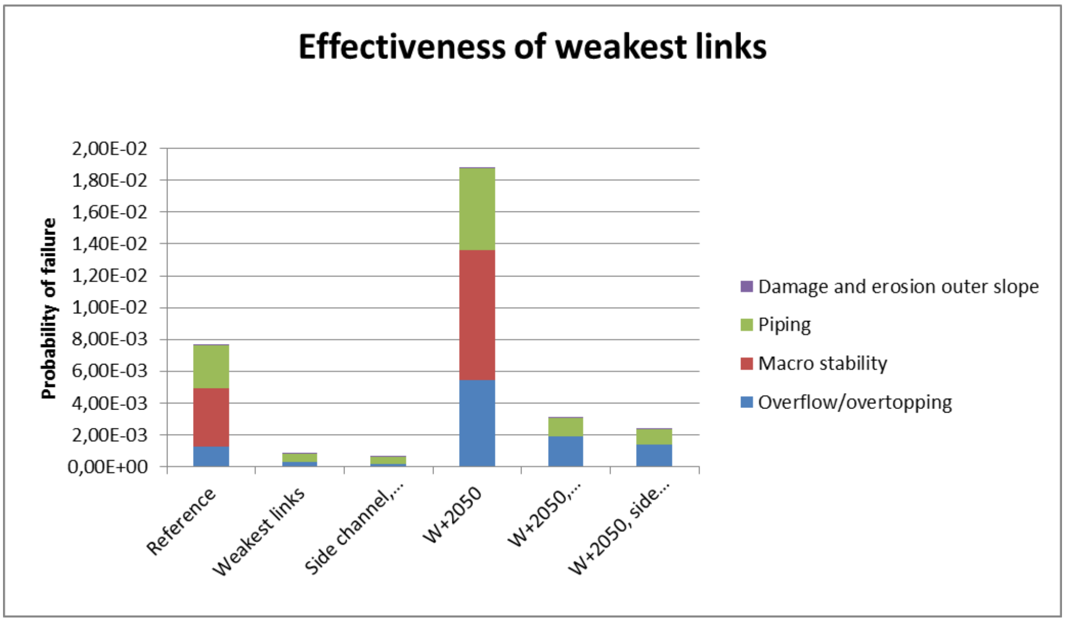


Figure 6-15: Effectiveness of weakest links

6.7.3 Only dike improvements

The safety standard-induced design task for the model dike reach has been determined in section 4.3.1, and it is as much as 231. The total design task per dike section varies between 30 and 80 for overtopping, with one outlier of 226. For the other failure mechanisms the total design task is up to several thousand. The dike improvements that have been regarded show reductions of up to 30 for overtopping. Also Figure 6-12 showed that raising all the dike section with 1.0 m reduces the probability of failure of almost all of the dike sections to the required level.

The dike improvements that have been regarded are not effective enough to reduce the probability of failure of the geotechnical failure mechanisms to the required level. Since some sections have very high design task for these mechanisms, they will have to be treated with more effective dike improvements than only applying a piping berm. There are calculation models available in which this can be calculated.

7 Discussion

This section will briefly discuss the measures and strategies and their relation to the design task. First of all the effectiveness of the different measures will be compared. After this the effect of climate change on the probability of failure and the effectiveness of the regarded measures to counteract these are discussed. The different strategies will be weighed against each other and finally the findings will be put in a broader context.

7.1 Effectiveness

Figure 7-1 shows the effectiveness of all the measures on the probability of failure of the different failure mechanisms for the whole reach. For dike improvement combination 2 is chosen (arbitrarily). Raising the dikes is the most effective measure for the mechanism overtopping. This makes perfect sense since the dike is raised with 50 cm and the maximum water level reduction is only 20 cm (locally). Therefore the comparison between spatial measures and dike improvements is not really fair.

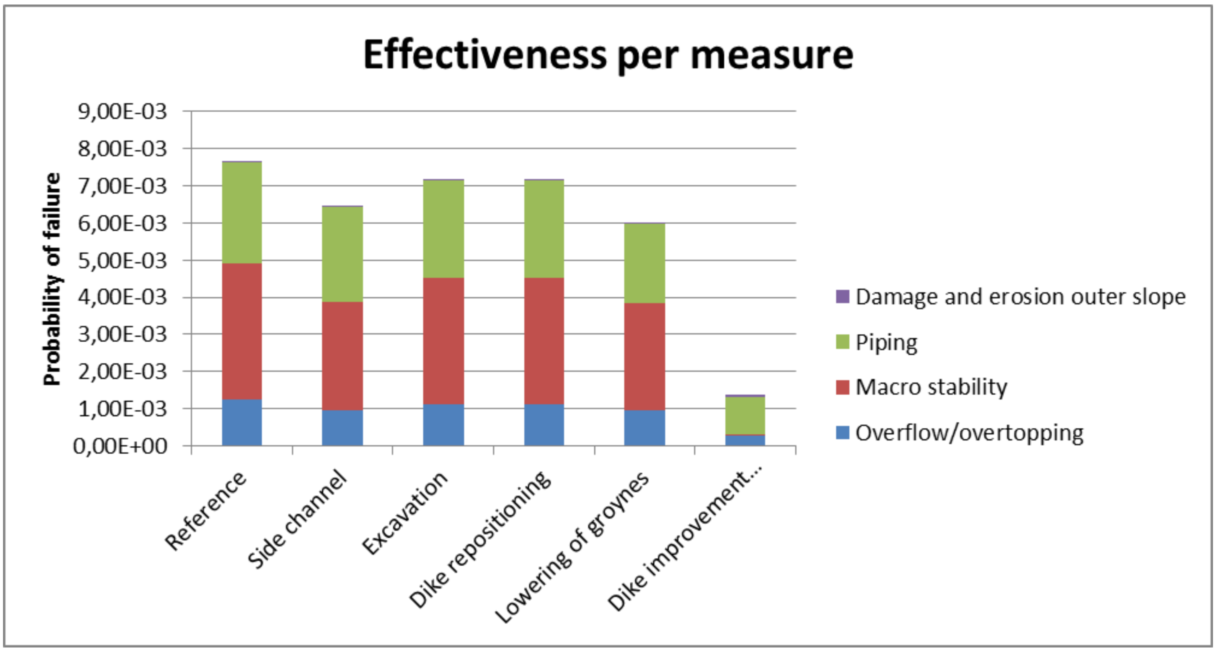


Figure 7-1: Effectiveness of all measures, HR2006

The total reduction factor of all the spatial measures is limited. The highest reduction factor is found with the lowering of groynes. The reduction by this measure is 1.21 after climate change. The lowering of groynes is thus insufficient to reduce the climate change-induced design task ($IF_{climate}$=2.45).

All the considered spatial measures have the highest reduction factor for the failure mechanism overflow/overtopping. The side channel shows the largest values, which can locally be up to 1.9. This is not sufficient to reach the design task for overflow/overtopping for any section, let alone the total design task of the model dike reach. The regarded measures can contribute to reducing the total design task though. When a combination of measures would be used this may have severe effect on the design task.

The presence of dike sections with very high probability of failure for geotechnical failure mechanisms dominates this dike reach. The spatial measures that are considered are not very effective to reduce this probability of failure. The reduction of the water level with 20 cm is not sufficient to reduce the probability of failure significantly. This can be explained with the fragility curve for piping and macro stability; slightly lower water levels (in the high domain, say Q between 10,000 m^3/s and 17,000 m^3/s) still have a high probability of failure. The fragility curve is very flat. Since the probability of failure for the geotechnical failure mechanisms is very large, small reduction factors still mean a significant reduction of the probability of failure (Figure 7-1). But the probability of failure still remains high.

The effect of all the regarded spatial measures decreases fast in upstream direction. This affects the reduction of the probability of failure of geotechnical failure mechanisms more than the mechanism overflow/overtopping. Moreover, if the water level reduction at MHW is small, the reduction at lower discharge will be almost negligible. It has been often mentioned that the water level reduction at low discharges is very important for geotechnical mechanisms. In general it can be concluded that the considered spatial measures are more effective in reducing the probability of failure for overflow/overtopping than for other failure mechanisms.

7.1.1 Dike improvement

Dike improvement is by far the most effective measure. This is partly because for dike raising the raise is much higher than the water level reduction caused by the measures, so it is more effective on the failure mechanism overtopping. Also dike improvement can be applied very targeted, the measure reduces the probability of failure for the mechanisms that are needed. The measure can be used locally, which makes it very effective in reducing the probability of failure. This is also the principle of the weakest links strategy.

7.2 Climate change

Climate change is responsible for a part of the design task. Even though this will in most cases be very small in comparison to the design task that is caused by the difference between the new standards and the current safety state of the dikes, it is not negligible. The climate change-induced task differs per failure mechanism, but it is the largest for overflow/overtopping, as has been seen in section 4.3.2. The climate change-induced design task for overtopping can be up to factor 9.6. The total impact of climate change on the model dike reach (IF) is 2.45, which is a lot lower than the impact climate change has on overflow/overtopping.

The total reduction factor of the considered spatial measures is between 1.07 and 1.28 for HR2006, and even up to 1.65 for a single dike section. A single one of these measures would thus not be sufficient to fulfil the climate change-induced design task. The reduction factor for each of the measures is shown in Table 7-1.

Table 7-1: Reduction factor per measure after climate change

Measure	HR2006	RF_HR2006	W+2050	RF_W+2050
No measure	1/130	-	1/53	-
Side channel	1/155	1.19	1/63	1.18
Excavation	1/139	1.07	1/56	1.06
Repositioning of dikes	1/139	1.07	1/57	1.07
Groyne lowering	1/166	1.28	1/64	1.21

Table 7-1 shows that the effectiveness of the spatial measures reduces after climate change. The explanation for this has to be found in the probability density curve of the water level. Figure 7-2 shows the probability density curve of one of the dike sections in the regarded dike reach.

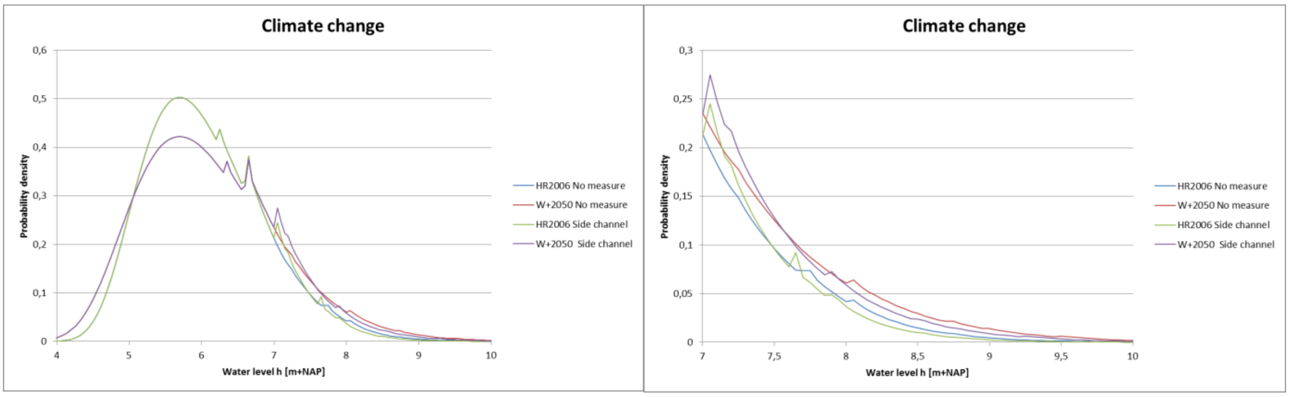


Figure 7-2: Influence of measures and climate change on probability density (section 43.TG171.TG177)

The figure contains four curves; the probability density curve of the water level with and without the measure side channel, both for the two involved climate scenarios. Note that the side channel in this image is different than the side channel that has been regarded above. It has been modelled so that the reduction of the water level at MHW is equal to the increase in water level due to climate change. This increase is the difference between the water level at Q = 16,000 m^3/s and Q = 17,000 m^3/s. For this dike section this is 25 cm. The water level reduction caused by the side channel at Q = 17,000 m^3/s is thus also 25 cm.

The left figure shows the effect of climate change on the whole probability density curve. The climate change smears out the probability density curve: the peak is lower but both lower and higher water levels occur more often. It is also seen that due to the side channel, the probability density at water levels between 7 and 10 m+NAP slightly decreases (the green and purple line). It can be seen that the measure is not effective enough to reduce the probability density in 2050 (purple line) to the level of the reference situation (blue line). Since the probability density of these water levels is higher, this will lead to a higher probability of failure than in the reference situation. These water levels play an important role for geotechnical failure mechanisms.

The impact of climate change on the probability of failure for overflow/overtopping is factor 6.18. The reduction of this probability of failure caused by the measure is 2.20 for the situation with HR2006. After climate change the reduction factor is smaller: 1.73. This measure will thus not be able to reduce the probability of failure more than the climate change has increased it.

To explain the theory behind this, a graphic has been made in Figure 7-3. This figure shows what happens in the right tail of the probability density curve. The blue line indicates the reference situation, without a measure or climate change. The red line is the probability density curve after climate change. The increase in water level at MHW is 25 cm (note that also the MHW discharge has changed!). Since the probability density curve is 'smeared out' by climate change, the probability density of these water levels is not only higher, the probability density curve is also steeper.

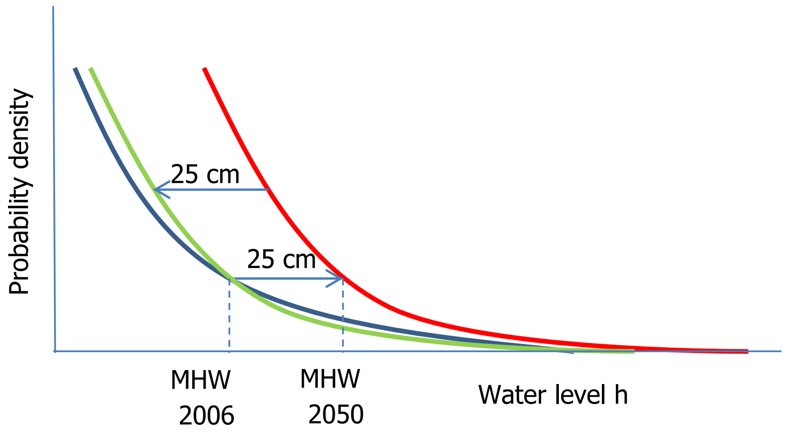


Figure 7-3: Theory behind climate change

Now when a measure is applied that reduces the water level, this practically means that any point on the probability density curve shifts to the left, relative to the water level reduction at that discharge. For Figure 7-3 and for this explanation it is assumed that the water level reduction is equal at all water levels[2].

The green curve is the situation after the measure has been applied. It is seen that the green line lies above the probability density curve of the reference situation for all water levels beneath MHW. This means that the probability of failure will still be higher after the measure was installed, even though the water level reduction at MHW was equal to the increase due to climate change!

This example shows that the increase of the probability of failure due to climate change is much larger than the decrease due to the measure. This means that if a number of spatial measures, which cause a reduction of the water level at MHW equal to the increase by climate change, is applied to a river stretch, this will not bring the probability of failure back to the original value. This insight is of great importance when the strategies are evaluated.

7.3 Strategies

In chapter 5 three strategies were established to be regarded within this thesis; the preferred strategy by DP15, the weakest links method and only using dike improvements. The applicability of these strategies will be discussed here based on the findings of this research.

[2] Note: this is an overestimation of the effectiveness of this measure. One of the main findings of this report is that this is not representative for the practical reality. This assumption has thus only been done for illustrational purposes.

7.3.1 DP15

The preferred strategy of DP15 aims on fulfilling the climate change-induced design task by means of spatial measures. When the model dike reach is regarded the climate change-induced design task is factor 2.45. The regarded measures are all small and have thus a small reduction factor. Even the side channel, which causes a water level reduction of 20 cm, cannot fulfil the climate change-induced design task. This means that larger measures, or a combination of measures has to be used in order to reach this goal, even for this relatively short dike reach. This does not have to be a problem, since the strategy already assumes that multiple measures have to be applied in order to reach the climate change-induced design task.

In section 4 Design task it was seen that climate change has an impact factor of 1.3 on dike ring area 43 for the year 2050 with climate scenario W+. The effect of RfR estimated by FloRis was 1.4, which is approximately equal to the climate change induced task. This would support the strategy of DP15. However, it has been shown above that the calculations as made by FloRis give an overestimation of the effectiveness of the measures. This overestimation is approximately factor 2 (section 6.6). Above that it was also shown that a set of measures that causes a reduction of the water level at MHW equal to the increase of the water level due to climate change, will not bring the river system back to the original safety state. These two findings support the hypothesis that the effectiveness of spatial measures on the probability of flooding has been overestimated.

This will have consequences on the applicability of the preferred strategy of DP15. It is of course possible to fulfil the climate change-induced design task by means of spatial measures, but the amount or the size of the spatial measures will be higher than was expected. The effect of this on the costs of this strategy has to be determined in further research.

7.3.2 Weakest links

The weakest links strategy was not intended to fulfil the total design task. It is merely a strategy to increase the safety level of a dike reach without taking large spatial measures. This strategy is proven to be effective for the model dike reach. Reduction factors above factor 10 can be achieved in combination with one of the regarded spatial measures.

One thing that was notable was that the effectiveness of the spatial measures increased after treating the weakest links. This means that spatial measures are more effective on dike reaches where geotechnical failure mechanisms play no role. This is in accordance with the findings in chapter 6, which showed that the highest reduction factors where reached for the failure mechanism overflow/overtopping. The weakest links strategy is thus a good way to enhance the effectiveness of spatial measures.

7.3.3 Only dike improvements

Dike improvements can in all cases be used to increase the safety level of a river system. Raising dikes is very effective for overtopping; in the model dike reach all sections (except for one) would meet the safety standard with 1.0 m dike raising. Improving the dikes for geotechnical failure mechanisms is more complicated. It is also not really feasible to model this with a shift of the fragility curve. This should be calculated with a geotechnical calculation model.

Even though the strategy of dike improvements is approved in the Netherlands, it also has its downsides. It is not desirable that dikes are raised infinitely if higher discharges occur. If this is done the water levels will also increase. If a dike breach would occur during a flood this would mean that a lot more water will enter the hinterland. This should always be taken into account and this is also the reason why increasing the flood conveyance capacity is basically a good idea.

7.3.4 Recapitulation

None of the three regarded strategies can be marked as the most suitable one to reduce the failure probability of the dikes of the main rivers in the Netherlands. The strategies all have advantages and disadvantages. The preferred strategy of DP15 can be used, even though the amount of spatial measures that will have to be combined in order to fulfil the climate change-induced part of the design task will be larger than expected. This will inevitably lead to higher costs, which may lead to the use of another strategy after all.

It has been noticed that the measures are more effective after the weakest links have been treated, and that the reduction factor of spatial measures is the highest for failure mechanism overflow/overtopping, so spatial measures should be used on dike reaches on which this failure mechanism is dominant. The climate change-induced design task for this failure mechanism is however very high, so a lot of measures would be required.

It would be a good strategy if the design task for geotechnical failure mechanisms is fulfilled with dike improvements. The design task for overflow/overtopping can be fulfilled with a combination of spatial measures and dike raising. With this strategy the design task is not divided in a climate change-induced and a safety standard-induced part, but the total design task is resolved with measures that are most effective per failure mechanism.

7.4 Broader context

In order to give some advice of the generic applicability of the findings in this report, the results should be put in a broader context. This section briefly describes what would happen if the regarded measures would be applied at another river system in the Netherlands, or when these measures would be slightly modified, for example by changing the location of the measure.

7.4.1 Location of the measure

If floodplain measures are considered locally the reduction factor is much higher than for the whole reach. The effect is maximal at the downstream end. If the location of the measure could be changed this would be a good way to optimize the effectiveness of the measure for a reach. In this way the maximum reduction factor coincides with the weakest section, which gives the highest reduction of the probability of failure, both locally and for the total reach.

This is however a matter of spatial integration. A spatial measure often needs, as the term suggests, a lot of space to be implemented. For this reason it might not always be possible to optimize the location of the measure.

7.4.2 Another dike system

If the considered spatial measures, or at least measures with a comparable effect on the water level, were to be implemented at another location, the reduction of the probability of failure would be different than for the model dike reach.

If the dike system on which a measure was to be applied would completely meet the safety standards the reduction factor would be largely determined by the reduction factor for the failure mechanism overflow/overtopping. This could bring the dike system to a higher safety level, but only slightly higher, since the maximal reduction factor for overflow/overtopping was 1.9, which was seen at the regarded side channel. This effect has also been seen with the weakest links calculations. The reduction factor increased after the weak links were treated.

7.4.3 Opposite dikes

A positive side effect of spatial measures that hasn't been mentioned is that the water level reduction will also have influence on the probability of failure of the dikes on the other side of the river. This can be seen as a benefit, since this also reduced the design task of this other dike. This may be helpful when a set of measures is composed to reduce the total probability of failure of a river system.

7.4.4 Cost-benefit

Finally the costs of spatial measures and dike improvements should be compared. Dike improvements will probably be cheaper than spatial measures, since it involves less ground movement, there are no (or hardly any) investments in land surface needed and the activities are generally less complicated.

On the other hand spatial measures may offer some extra benefits. As mentioned above the measure is effective on both banks of the river. Besides that a spatial measure can be used to improve the quality of the area. Spatial measures can be combined for example with nature projects or even recreation. And spatial measures may be a solution in situations where raising a dike is not desirable for aesthetical reasons.

When the benefits of spatial measures are carefully weighed against the costs, it may turn out that for some situations spatial measures will in fact be feasible or even desirable.

8 Conclusion

This chapter summarizes the conclusions of this research. The research questions are answered and the conclusions will be presented in relation to the objectives of this research.

8.1 Objective and research questions

The objective of this thesis is the following:

> "To provide insight in the reduction of the flood probability by different measures, and how these measures can be used to reach the required safety level, by developing a rapid assessment tool based on the flood risk approach."

In order to reach this goal a tool has been developed with which calculations have been made on the effectiveness of spatial- and dike improving measures. The development and use of the tool have led to insights about the tool itself and the effectiveness of the considered measures and the applicability of strategies. These insights will be discussed by answering the research questions.

8.1.1 How can the difference between the current state and the new safety standards be assessed and transferred into a design task?

The design task can be expressed as the reduction factor, the ratio between the present and the ultimate (desired) state, which is required to reduce the probability of failure to the desired level. It consists of a safety standard-induced part and a climate change-induced part. The standard-induced part is the difference between the reference state and the state that the new safety standards prescribe. In this thesis the climate change induced part is the difference between the reference situation and climate scenario W+2050. The design task can be assessed per dike reach (the new safety standards re specified per reach) or per dike section. Since the unit 'reduction factor' is relative, attention should always be paid to the reference situation. A high reduction factor on low probability of failure is probably less effective on the total probability of failure than a small reduction factor on very high probability of failure. However, it does give an idea of how sensitive a failure mechanism is for a measure.

8.1.2 How can the effectiveness of different measures be assessed, according to the flood risk approach?

To assess the effectiveness of different measures a rapid assessment tool has been developed, which focuses on the statistics of dike failure rather than the mechanics of a failure mechanism. By combining the discharge statistics, the Q-h relation (with $Q=Q_{Lobith}$ and $h=h_{local}$) and the fragility curve (strength) of a dike section the probability of failure can be calculated. This has been verified by calculating the probability of failure with the tool, using the statistics from FloRis.

By changing either one of the relations, changes to the river system can be simulated. Changing the discharge statistics simulates climate change, changing the Q-h relation simulates a spatial measure and changing one of the fragility curves simulates dike improvements. Calculating the new probability of failure after changing one or more of these relations will give insight in the impact on, or the reduction of the probability of failure of the system. Several of these changes have been executed and the effects of it have been calculated and analysed. Four spatial measures have been elaborated based on the analysis of existing or previously designed measures.

8.1.3 What is, for spatial measures and dike improvement measures, the effect on the total probability of flooding?

The new rapid assessment tool has been used to calculate the effectiveness of four spatial measures: a side channel, excavation of the floodplains, repositioning of dikes and lowering of groynes. For all of these measures a representative variant has been elaborated, which have been tested on a model dike reach which is based on a dike reach on the Waal River. Also some dike improvements have been regarded: raising the dikes, applying a piping berm and several combinations of these.

The effectiveness of the considered spatial measures is moderate; the reduction factors lay between 1.07 and 1.28 for the total model dike reach. The effectiveness per dike section and per failure mechanism is maximum 1.9, for overflow/overtopping. The water level reduction is 20 cm here. The effectiveness of measures decreases after climate change, since due to climate change the probability density curve is steeper and higher for the water levels that determine the probability of failure (Q = 10,000 m^3/s until Q = 17,000 m^3/s).

For all the considered spatial measures the reduction factor for failure mechanism overflow/overtopping is the largest. Effectiveness on geotechnical failure mechanisms is almost negligible for smaller measures. The reason for this is twofold: the low water level reduction upstream of the measure and the high initial probability of failure for geotechnical failure mechanisms. For dike improvements the effectiveness increases with the magnitude of the measure. Dike raising of 1 m is shown to be sufficient to reduce the probability of failure for overtopping of almost all the sections in the considered reach to the required safety level for this failure mechanism.

The reduction factor of spatial measures is optimized if the dike reach it is applied on is dominated by the failure mechanism overflow/overtopping. The reduction factor thus increases when the weakest links are treated.

8.1.4 How can different measures be combined in order to reach the prescribed safety level?

The effectiveness of spatial measures in RfR has been overestimated with factor 2 by FloRis. It is expected that this is also the case on larger scale. This means it would require a lot more measures to fulfil the climate change-induced design task than was expected. This would make the preferred strategy of DP15 more expensive than expected.

Even though the spatial measures that have been regarded in this research are not effective enough to reduce the probability of failure significantly, it is still recommended to use a combination of dike improvements and spatial measures. In practice not only one spatial measure will be used, but a set of measures will be applied to reduce the probability of flooding, so the reduction factor will be higher than the single measures that have been regarded in this report. However it is not advisable to use only spatial measures to fulfil the climate change-induced design task. The recommended strategy is to use dike improvements to fulfil the design task for geotechnical failure mechanisms. The total design task for overtopping can then be fulfilled with a combination of spatial measures and raising the dikes at the weakest (lowest) locations. In this way the reduction factor of the spatial measures is optimized, the costs remain relatively low and there is still an improvement of the environment thanks to the use of spatial measures.

8.2 Main conclusions

The main conclusions of this research follow from the answers that have been given on the research questions in the previous section. They are divided in four categories:

Available calculation methods:

- Currently existing programmes, like PC-Ring, are not designed to assess the effectiveness of (a set of) measures, but merely to calculate the probability of failure of flood defences in the present state;
- The method that FloRis has used to assess the effectiveness of the RfR programme leads to an overestimation of the effectiveness of the measures of about factor 2 for the considered model dike reach. It is expected that the effectiveness of the total RfR programme is also overestimated;

New calculation method:

- The probability of failure of a dike section for a certain failure mechanism can be computed by combining the discharge statistics, the Q-h relation (where $Q = Q_{Lobith}$ and $h = h_{local}$) and the fragility curve. With this method and the FloRis databases results from PC-Ring computations with an average error (ratio) of 1.13 per dike section and a total error of 1.20 for the considered dike reach;

Effectiveness of measures:

- Spatial measures have the highest reduction factor for failure mechanism overflow/overtopping. When geotechnical failure mechanisms play a large role in a dike section or dike reach, spatial measures are not effective to reduce the total probability of failure, since the geotechnical properties are of more influence than the hydraulic load;
- If a set of spatial measures, which cause a reduction of the water level at MHW equal to the increase by climate change, is applied to a river stretch, this will not bring the probability of failure back to the original value, because climate change steepens and increases the probability density curve and a measure only shifts it towards lower water levels. The probability density will still be higher than in the original situation;
- The effectiveness of spatial measures decreases after climate change;

Application of measures:

- Since the effectiveness of spatial measures has been overestimated, more measures will be needed to fulfil the climate-change induced task than was thought before. The preferred strategy of DP15 will therefore be more expensive and might not be feasible anymore;
- The recommended strategy is to fulfil the design task for geotechnical failure mechanisms by means of dike improvements. The total design task for overflow/overtopping can be fulfilled with a combination of spatial measures and dike improvements;
- It is necessary to combine spatial measures and dike improvements, since both types of measures contribute to the safety of the river system in another way, which can enhance each other.

These insights may provide a guideline for the way in which the river area can be improved to reach the new safety standards that have been proposed by DP15. It can be used as a starting point for more detailed research into the effectiveness of measures. The conclusions give an idea on how the new calculation method can be used in practice and how the results have to be interpreted. This can also be used as a handlebar to formulate a strategy for the improvement of the safety level of the Dutch river system.

8.3 Extra conclusions

Next to the main conclusion some other findings have been done in this research. These are summarized below:

New calculation method:

- By altering the discharge statistics the impact of climate change can be calculated with the new tool, the accuracy of this is 1.18;
- By altering the Q-h relation the effectiveness of a spatial measure can be calculated;
- By altering the fragility curve the effectiveness of dike improvements can be calculated;
- The fragility curve for failure mechanism overflow/overtopping and macro stability can be approached with the conditional probability of failure per 12.4 hours (block duration) with an average error of 1.11 for overtopping and 1.24 for macro stability. Since the probability of failure per block duration is used this is an underestimation;
- The fragility curve for failure mechanism piping can be approached with the conditional probability of failure per year with an average error of 1.34. Since the probability of failure per year is used this is an overestimation;

Effectiveness of measures:

- The effectiveness of floodplain measures can be optimized when the reduction of the water level increases fast for water levels where the floodplains are just starting to flow (around 8000 m^3/s for the considered dike reach). In this way the reduction of the probability of failure of geotechnical failure mechanisms is the largest. For this reason it is important that, for example, a side channel starts to flow along at low high waters, and not at high discharges such as is common practice with high water channels (which generally start flowing around Q = 12,000 m^3/s). The increase in effectiveness of the measure can be about 10%;
- Dike improvements are in all cases more effective than spatial measures, since they are more effective for geotechnical failure mechanisms, which dominate the probability of failure of the model dike reach;
- Lowering of groynes is more effective at lower high waters than most of the floodplain measures. Therefore this measure is more effective to reduce the probability of failure for geotechnical failure mechanisms. However, the reduction of groyne lowering on the probability of failure for piping and macro stability will in most cases be insufficient to reach the safety standard per dike section, since the probability of failure in separate dike sections can be very high due to geotechnical shortcomings;
- Floodplain measures are most effective just upstream of the measure. The effectiveness quickly declines in upstream direction. This is especially the case for geotechnical failure mechanisms;

- The location on which a measure is implemented, and its size, may have great impact on the effectiveness;
- If the probability of failure is determined by one or several dike sections that have a very large probability of failure for one failure mechanism, treating this sections separately has very high influence on the probability of failure;
- Small reduction factors can still lead to large reduction of the probability of failure, if in the reference situation the probability of failure is very high;
- If there are sections with very high probability of failure for one mechanism it is not easy to say something about the effectiveness for a reach;
- In the analysis of the Blokkendoos (Appendix D) it was concluded that there is no apparent relation between the size of the measure, the effect of the measure and the reach of the measure. This emphasizes that all measures are unique and therefore it is hard to parameterize measures based on those details.

Application of measures:

- The weakest links strategy is not sufficient to fulfil the design task, but it can be a valuable strategy to postpone the necessity of large scale operations, since it can reduce the probability of failure of the model dike reach with factor 10 and higher;
- In order to optimize the effectiveness of spatial measures, it is advisable to treat the weak links of a dike reach first;
- Spatial measures will at all times have to be combined with dike improvements. It is practically impossible to fulfil the whole design task with spatial measures. This is due to the fact that some sections have a very high probability of failure for geotechnical failure mechanisms, which are more determined by the geotechnical construction of the dike than the hydraulic load, and to the fact that most spatial measures have a small reduction factor.

8.4 Recommendations

If one intends to utilize or improve the new calculation method there are a few aspects that deserve extra attention.

Assumptions

For this research some assumptions have been done that may influence the outcome. For example the maximum physical discharge that can enter the Dutch Rhine branches (aftoppen) of 18,000 m^3/s has not been taken into account. This leads to an overestimation of the calculated probability of failure, since discharges that can never occur are still taken into account. For the calculations with HR2006 this overestimation will be negligible since the return period for this discharge is very long. If calculations would be made for other climate scenarios, especially scenarios for the year 2100 the return period of this discharge reduces to approximately 500 years, thereby increasing the probability density and thus the contribution to the total probability of failure of extremely high discharges. The overestimation caused by this can be severe, so it is recommended that the physical maximum is taken into account in further research.

Also the reference situation of this thesis is different from the reference situation of DP15. In the reference situation for DP15 a lot of weak dike sections have already been improved in HWBP2 and the water levels are already reduced by the old RfR programme. This may lead to slightly different conclusions regarding the most efficient strategy. It is therefore recommended to incorporate this in further research.

New rules for calculations on piping are being developed at the moment of writing. Changes in the calculation include incorporating failure mechanism heave in the calculation, and the adaption of several parameters to the latest insights. The new method may lead to either higher or lower probabilities of failure for this failure mechanism, which is case-dependent. It is recommended that the most up-to-date calculations are always used when calculations are made with a comparable rapid assessment tool.

The climate scenario W+2050 that is used in this research is the same as prescribed in OI2014, which is different from the climate scenario DP15 uses. Also the Q-T relation that has been used is not the official discharge statistics, but it is based on a fit. This may have led to an overestimation of the impact of climate change. This will however not influence the conclusions. Other climate scenarios will lead to either higher or lower probabilities of failure. The choice for a certain climate scenario is a political aspect, but the effects of using another scenario have to be taken in to account when making calculations.

General recommendations:

The costs of different measures have not been taken into account in a quantitative way in this research. Even though it has been stated that spatial measures may have several advantages compared to dike improvements, it is expected that the strategy of DP15 will not be economically feasible. It is therefore recommended that a proper cost-benefit analysis will be executed, where it is taken into account that the effectiveness of the measures is less than has been expected before. This may result in new insight in the economic feasibility of the strategy of DP15.

The measures that have been calculated in this research were only individual measures with a relatively small water level reduction. The effectiveness of a set of measures will obviously have more effect on the total probability of failure. It is recommended that the effectiveness of a set of measures will be calculated in further research, so that the effectiveness of combined measures can be weighed against the effectiveness of a single measure or dike improvements.

It may be wise to search for another unit than the reduction factor, because it is an ambiguous unit, which makes it hard to see whether a measure is effective or not. A large reduction factor of a low probability of failure is less effective than a small reduction of a very high probability of failure. It is always necessary to take the reference state into account. However, it does give insight in how sensitive a failure mechanism is to a measure.

It is recommended to make some calculations on the effectiveness of measures for more dike reaches, using the rapid assessment tool. This will give more insight in the magnitude of the overestimation of the effectiveness. Since it takes some time to extract all the necessary data from PC-Ring and implement this in the calculation method this is outside the scope of this thesis.

When using the method:

It is always necessary to validate the calculations that are made. This can be done by comparing the results to calculations or other data. If the reference calculation gives different values than it should, calculations on the effectiveness of measures or on the impact of climate change are unreliable. In this research validation has been done by using the same data as PC-Ring and sharpen the model to get the smallest error. If another data set is used for the calculations than it is compared with the outcomes will probably differ more but it will still give insight in the value of the calculations.

When improving the method:
The fragility curves can be derived from PC-Ring in a different way, which makes it possible to approach the outcome of PC-Ring even better. To do this the probability of failure per 12.4 hours (block duration) should be transferred into the probability of failure per year. For this the duration of the blocks should be taken into account, and the size and shape of the hydrograph. This is explained in Appendix B.

The measures have been elaborated at a high level of abstraction. If these measures would be elaborated numerically the Q-h relation change will be more precise, which will lead to more precise calculations. To elaborate the measures information is required about the geometry of the river; width, depth, width of the floodplains and reasonable estimates for the roughness and the gradient. With the formula of Chézy it is then possible to calculate the water level for a discharge. The measure can be parameterized in this way, and the effectiveness can be calculated. The upstream effect at different discharge levels can be elaborated by calculating the course of the backwater curve with the formulae by Bélanger and Bresse (Vriend, et al., 2011) . This would give more precise results than with the assumption that the Q-h relation is relative to the water level reduction at MHW.

Instead of parameterizing measures the effectiveness of measures can also be calculated by calculating the Q-h relations for all locations with a WAQUA or Sobek model. This can be used as input data for the rapid assessment tool to calculate the effectiveness or the results can be used to validate the calculations on the effectiveness of different measures.

References

Biesboer, F., 2012. Het dossier Ruimte voor de Rivier. *De Ingenieur,* pp. 21-35.

Calle, E. & Weijers, J., 1994. *Technisch rapport voor controle op het mechanisme piping,* Delft: TAW.

Deltacommissie, 1960. *Rapport Deltacommissie Eindverslag en Interimadviezen,* Den haag: Deltacommissie.

Deltaprogramma, 2014. *Blokkendoos rivieren.* [Online]
Available at: http://www.deltaportaal.nl/blokkendoos/
[Geopend 3 4 2015].

Deltares, 2011. *Verkenning van mogelijke fysieke en sociaaleconomische ontwikkelingen in de 21ste eeuw op basis van KNMI'06 en WLO-Scenario's, voor gebruik in het Deltaprogramma 2011-2012,* Delft: Deltares.

DHV, 2009. *Morfologische effecten Kribverlaging Midden-Waal,* Utrecht: Programmadirectie Ruimte voor de Rivier.

Duits, M. et al., 1998. *Probabilistische Belastingmodellen en Rekentechnieken ,* Lelystad: HKV Lijn in water.

Heezik, A. v., 2006. *Strijd om de rivieren.* Den Haag: HNT Historische producties Den Haag in samenwerking met Rijkswaterstaat.

Huthoff, F., Paarlberg, A., Barneveld, H. & van der Wal, M., 2011. *Rivierkundig onderzoek WaalSamen,* Lelystad: HKV Lijn in water.

KNMI, 2006. *KNMI Climate Change Scenarios 2006 for the Netherlands.* De Bilt: KNMI.

Kroekenstoel, D., 2014. *Brondocument Blokkendoos Rivieren,* Utrecht: Deltaprogramma Rivieren.

Kuijper, H., Vrijling, J., Roos, A. & Ale, B., 2002. *Probability in Civil Engineering.* 1 red. Gouda: CUR.

Ministerie van Infrastructuur en Milieu, 2014. *Deltaprogramma 2015,* Den Haag: Ministerie van Infrastructuur en Milieu.

Ministerie van Infrastructuur en Milieu, 2014. *Synthesedocument Rivieren ,* Utrecht: Ministerie van Infrastructuur en Milieu.

Ministerie van Infrastructuur en Milieu, 2014. *Synthesedocument veiligheid,* Utrecht: Deltaprogramma, Deelprogramma veiligheid.

Parmet, B., 2001. *Analyse van de maatgevende afvoer van de Rijn te Lobith,* Arnhem: Rijkswaterstaat.

Programmabureau Ruimte voor de Rivier van Rijkswaterstaat, 2014. *23e voortgangsrapportage Ruimte voor de Rivier,* Utrecht: Programmabureau Ruimte voor de Rivier van Rijkswaterstaat.

Projectbureau Ruimte voor de Rivier, 2006. *Planologische Kernbeslissing deel 4,* Utrecht: Projectbureau Ruimte voor de Rivier.

Projectbureau VNK2, 2011. *De methode van VNK2 nader verklaard,* Den Haag: Projectbureau VNK2.

Projectbureau VNK2, 2014. *Database Dijkring43_basissom,* Den Haag: VNK2.

Projectbureau VNK2, 2014. *Database Dijkring43_referentie,* Den Haag: VNK2.

Rijksoverheid, 2014. *Helpdesk water.* [Online]
Available at: http://www.helpdeskwater.nl/onderwerpen/wetgeving-beleid/waterwet
[Geopend 06 oktober 2014].

Rijkswaterstaat, 2014. *Handreiking ontwerpen met overstromingskansen,* Utrecht: Rijkswaterstaat.

Ruimte voor de Rivier, 2014. *Ruimte voor de Rivier.* [Online]
Available at: http://www.ruimtevoorderivier.nl/
[Geopend september 2014].

Silva, W., Klijn, F. & Dijkman, J., 2001. *Room for the Rhine branches in the Netherlands what research has taught us,* Delft: WL Delft hydraulics.

Steenbergen, H., Vrouwenvelder, A. & Koster, T., 2007. *Theoriehandleiding PC-Ring Versie 5.0 Deel A: Mechanismebeschrijvingen,* Delft: TNO.

Technische Adviescommissie voor de Waterkeringen, 1999. *Technisch Rapport Zandmeevoerende Wellen,* Den Haag: TAW.

UNESCO-IHE, 2013. *Samenwerken aan maatwerk, een slimme combinatie van succes en inhoud,* Den Haag : Rijkswaterstaat ruimte voor de rivier.

Van Rooijen, E., 2014. *Effectiveness of Room for the River Intervention works,* Delft: TU Delft.

Veiligheid Nederland in Kaart, 2005. *Hoofdrapport Veiligheid Nederland in kaart,* Utrecht: Rijkswaterstaat.

Veiligheid Nederland in Kaart, 2007. *Het ontwerppunt,* Utrecht: VNK2.

Vergouwe, R., Huting, R. & Van Der Scheer, P., 2014. *Veiligheid Nederland in Kaart 2 Overstromingsrisico dijkring 43 Betuwe, Tieler- en Culemborgerwaarden,* Den Haag: Ministerie van Infrastructuur en Milieu.

Verruijt, A., 2001. *Grondmechanica.* Delft: VSSD Delft.

Vriend, H. d. et al., 2011. *Collegedictaat rivierwaterbouwkunde.* Delft: TU Delft.

Vrouwenvelder, A. & Steenbergen, H., 2003. *Theoriehandleiding PC-Ring Versie 4.0 Deel B: Statistische modellen,* Delft: TNO.

Vrouwenvelder, A. & Steenbergen, H., 2003. *Theoriehandleiding PC-Ring Versie 4.0 Deel C: Rekentechnieken,* Delft: TNO.

Weijers, J. & Tonneijck, M., 2009. *Lecture notes CT5314 Flood Defences.* Delft: TU Delft.

Werkgroep Deelprogramma Veiligheid, 2014. *Technisch-inhoudelijke uitwerking van eisen aan primaire waterkeringen (DPV 2.2),* Utrecht: Deelprogramma veiligheid.

Wouden, F. v. d. & Grashoff, P., 2009. *PC-Ring 5.3.0 Gebruikershandleiding,* sl: Demis BV/QQQ Delft/TNO Bouw.

Zijlema, M., 2012. *Computational modelling of flow and transport.* Delft: TU Delft .

Appendices

A. Background information

A.1 Room for the River

Room for the river consists of 34 projects to give the rivers more room. The goal is to reduce water levels by using spatial measures. The following describes the motivation for Room for the River as well as the method. The results of Room for the River are assessed based on the FloRis approach. For illustration purposes dike ring 43 is taken as an example. This dike ring has a lot of Room for the River projects and it stands a good example of a dike ring along the main rivers.

A.1.1 Cause and principle

The high waters in 1993 and 1995 raised the awareness that some intervention was needed in the river area. The high discharge values that had occurred relatively short consecutively added new high values to the discharge measurements of the Rhine. Together with the canalization of the Oberrhein upstream this led to a higher peak discharge. The 1/1250 value for the discharge was calculated to be 16,000 m^3/s for the Rhine river at Lobith. This value, estimated in 2001 for the new Hydraulic Boundary Conditions (HR2001) was 1000 m^3/s higher than the value that had been used so far. This shift in discharges also led to higher design water levels. With this higher design water level and the insight that the river branches did not meet the safety level of 1/1250 years became more obvious. Some measures were required to improve the safety of the rivers.

In order to increase the safety level of the rivers the Room for the River programme was initiated. This programme focussed on the lowering of the maximum water level by giving the river more room instead of applying more traditional dike improvements. Room for the River initially consisted of a series of 39 projects to give the rivers more room.

The goal is to reach the required safety level along the Rhine branches and the lower parts of the river Meuse before 2015. For the Rhine this means that a discharge of 16,000 m^3/s should be possible within the required safety level as expressed in the Law on Flood Defences. This is 1000 m^3/s more than the current capacity and this difference has to be reached by giving the river more room, resulting in lower water levels. For the river Meuse the needed flood conveyance capacity is 3800 m3/s (Programmabureau Ruimte voor de Rivier van Rijkswaterstaat, 2014). For every measure a hydraulic target has been defined as the minimal lowering of the water level in case of the extreme discharge (Projectbureau Ruimte voor de Rivier, 2006).

Room for the River aims to increase the safety of over 4 million inhabitants in the river area. Besides this the programme also aims on improving the quality of the environment by creating appealing urban and rural areas. Also more recreational area is created and the economy will receive a boost by the measures (UNESCO-IHE, 2013).

A.1.2 Method

To reach the goals that have been set, Room for the River contains 34 projects where the river is given more room. These projects are all based on nine different measures. Some of these measures actually create more room, for example by deepening the main channel and by moving the dikes. But also the flow can be improved by lowering the groynes or removing hydraulic obstacles. In Figure A-1 the complete set of Room for the River measures is presented. If it is not possible or too expensive to create more space, traditional dike improvements are used (Ruimte voor de Rivier, 2014).

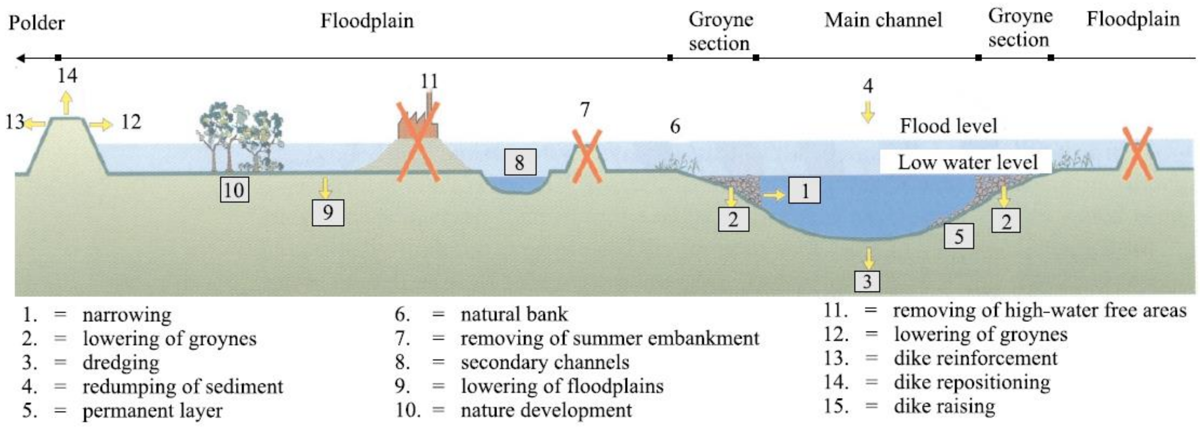


Figure A-1: Room for the River measures (Silva, et al., 2001)

A.1.3 Results

Most of the measures only have effect in case of high water. Due to the RfR works at some locations the water levels may be lowered up to 70 cm. Also the goals for the maximum discharge in both the Rhine branches and in the Meuse are reached (Programmabureau Ruimte voor de Rivier van Rijkswaterstaat, 2014).

The project is also successful in the way that it stays within budget. Most of the projects are also finished within time. One of the success factors is that there is a lot of cooperation with municipalities, provinces and other stakeholders. They also cooperated in forming the plans so they are very much involved in the execution of the project.

Just lowering the water level alone will not lead to a flood-proof country though. For example in the Betuwe Area the water levels in case of high discharge can be up to 6m, so a small reduction of the water level will not solve this problem. Also near the sea the water level is governed by tidal effects. For all locations, but these locations in particular, it holds that weak dikes still need to be improved.

To assess the effect of RfR the FloRis programme assumed that the water level lowering as a result of the RFR measures is equal to the difference between the hydraulic boundary conditions HR1996 and HR2006. This effect can be seen for dike ring 43 in Figure A-2. This lowering of the water level leads to a reduction of the total probability of failure with a factor 1.5 to 3 according to FloRis.

Figure A-2: water level reduction due to RfR according to FloRis (Van Rooijen, 2014)

Figure A-3: Reduction factor of total flood probability according to FloRis (Van Rooijen, 2014)

Just lowering the water level alone will not lead to a flood-proof country though. For example in the Betuwe Area the water levels in case of high discharge can be up to 6m, so a small reduction of the water level will not solve this problem. Also near the sea the water level is governed by tidal effects. For all locations, but these locations in particular, it holds that weak dikes still need to be improved.

In order to be safe, we should not just look at the probability of a certain discharge or water level occurring. There are other failure mechanisms that can be normative in relation to the failure probability of a dike. For example the failure mechanism piping is independent of the water level and so it can already occur at a water level well below the maximum (design) water level. FloRis has investigated the effect of the RFR measures on the total failure probability of the dikes. For a lot of dikes the safety level seems to be between 1/100 and 1/500 per year, which is not even close to the required 1/1250 per year (Biesboer, 2012).

To illustrate this, Table A-1 shows the influence of RFR on different failure mechanisms in dike ring 43. It can be seen that the RFR measures cause a big reduction on the probability of most of the failure probabilities. However, reduction of the water level does not bring the dike ring area to a higher safety level, even though the total failure probability is reduced with 30%. The economic risk is also lowered with about 30%. So in order to reach the required safety level additional measures are required (Vergouwe, et al., 2014). The reduction factor of the total flood risk due to Room for the River is shown in Figure A-3 for dike ring area 43.

Table A-1: Probability of failure per mechanism and probability of flooding for dike ring 43 (Vergouwe, et al., 2014)

Type kering	Faalmechanisme	Faalkans (per jaar)		
		Oorspronkelijk	Ruimte voor de Rivier	Reductie factor
Dijk	Overloop en golfoverslag	1/770	1/1.330	1,7
	Macrostabiliteit binnenwaarts	1/130	1/200	1,5
	Opbarsten en piping	>1/100	>1/100	1,5
	Beschadiging bekleding en erosie dijklichaam	1/15.000	1/19.400	1,3
Kunstwerk	Overloop en golfoverslag	1/6.000	1/12.500	2,1
	Niet sluiten	1/12.700	1/14.900	1,2
	Onderloopsheid en achterloopsheid (piping)	1/983.000	<1/1.000.000	2,2
	Constructief falen	1/621.000	<1/1.000.000	1,9
Overstromingskans		>1/100	>1/100	1,3

A.2 FloRis

In 1992 the ENW (Expertise Network Water safety) has initiated a series of researches in order to be able to quantify the probability of flooding and thereby the flood risk. Several studies have improved the calculation methods. After this the FloRis project started in 2001.

FloRis stands for "Flood Risk and Safety in the Netherlands" (in Dutch: Veiligheid van Nederland in Kaart, VNK). This research was carried out by "Rijkswaterstaat" in combination with the waterschappen (water authorities) and the province boards and was finished in 2005. The goal of the research was to gain insight in the consequences of flooding in the Netherlands and the probability that they will occur.

The research aimed on calculating the consequences of flooding in greater detail. Also the flood risk has been determined with a new method. The essence of the method is that different failure mechanisms all contribute to the probability of flooding and not just the occurrence of an extreme water level. The chance of occurrence of all these mechanisms combined is the probability of flooding of the whole dike ring area. The research mapped 16 of the 53 dike ring areas in the Netherlands, thereby providing a good representation of the safety of the Netherlands.

FloRis showed that in case of a flooding, in the most likely scenarios not the whole dike ring will flood, leaving the economic damage and the number of casualties far below the maximum. In the dike rings that surround the rivers it is more likely that the whole dike ring will flood. Also, a flooding is more likely to occur due to mechanisms piping and heave instead of an extreme water level. Also dysfunction of water retaining structures as an effect of human failure is a realistic scenario, which can be easily prevented by better documentation of emergency instructions and proper practice.

One of the recommendations was to investigate all of the 53 dike rings in detail to establish a complete image of the flood safety in the Netherlands. Also FloRis started using a method to evaluate the cost and benefits of flood defence projects. Also the probabilistic method contains a lot of uncertainties, for example on the mechanism of piping. FloRis advises to do more research on such topics to improve the methods for future use.

FloRis has just been one link in a series of studies on the safety of the Netherlands. The outcome of the research does not stand by itself: further investigation is necessary to come to a proper image of the flood safety of The Netherlands. During the research the opportunities of the flood risk approach became clear and generally accepted (Veiligheid Nederland in Kaart, 2005).

A.2.1 VNK2 (FloRis 2)

As a follow up the VNK2 project has started in 2006 as an initiative from the ministry of infrastructure and environment, the Unie van Waterschappen (Union of water boards) and the "Interprovinciaal overleg". This programme analyses for 58 dike ring area's the flood risk, expressed in terms of economic damage and numbers of casualties. Just like in FloRis the methods used in VNK2 are different from the current legal standards. The results of VNK2 give base to support measures and strategies in the context of multiple layer safety. Also the insights can help in developing new legal tools for the assessment of flood defences and to prioritize reinforcement measures from HWBP (high water protection programme). Furthermore it is the foundation for the new legal standards.

VNK2 points out the weakest links or the ones that carry the greatest risk. By this VNK2 makes it possible to invest more effective and more cost efficient in flood defences.

A.2.2 Method

The probability of failure of a flood defence is determined by the total probability of all combinations of loads and strengths under which a flood defence will collapse. The probabilistic method allows us to combine this while also taking the uncertainties regarding the values of the loads and strength properties into account.

The method of VNK2 first analyses the probabilities and consequences separately. In the next step these are combined giving the overall flood risk of the considered dike ring area.

The steps of the consequence track of the method are shown in Figure A-4. It is impossible to assess the consequences of a flooding for every location of a dike. Therefore the first step is to divide the dike ring area in sections with equal consequences. For these sections the pattern of flooding and the economic damage in case of a flood are roughly equal, independent of the exact location of the breach within this section.

For every section a calculation of the flooding is performed for different load conditions such as the water level and the duration of the high water, since the consequences will differ with these conditions. Also a sensitivity analysis is executed in the width of the breach, strength of secondary flood defences and the duration of the high water. In VNK2 the moment of the breaching is set at the moment when the hydraulic load is at its maximum. This is not necessarily the case so this would lead to either an overestimation or an underestimation of the consequences.

After this different scenarios are defined. Since it is possible that more than one section will fail the consequences of multiple breaches has to be assessed. If the possibility of multiple breaches is taken into account the number of scenarios is very high (2^n-1, n being the number of sections). If it is taken into account that a breach will decrease the hydraulic load on the rest of the dike ring, the number of scenarios equals the number of sections. In VNK2 a maximum of 13 sections is used in order to limit the number of calculations to 8.191.

For every one of these scenarios the consequences are calculated based on the flooding calculations. After this the economic damage and the number of fatalities is calculated. The economic damage of a flood is dependant of e.g. the water depth, the area inundated and the kind of land use. The number of casualties can be diminished by preventive evacuation; this effect is taken into account in the calculations. The scenarios of evacuation depend on the time between the prediction and the actual disaster and if the evacuation proceeds as planned.

Single floods are combined to gain information about multiple breaches. Water depths are added, but the sum may never exceed the worst case scenario. For flow and water level rising velocities no sum is made because this would lead to unrealistic values. The local maximum is used here instead.

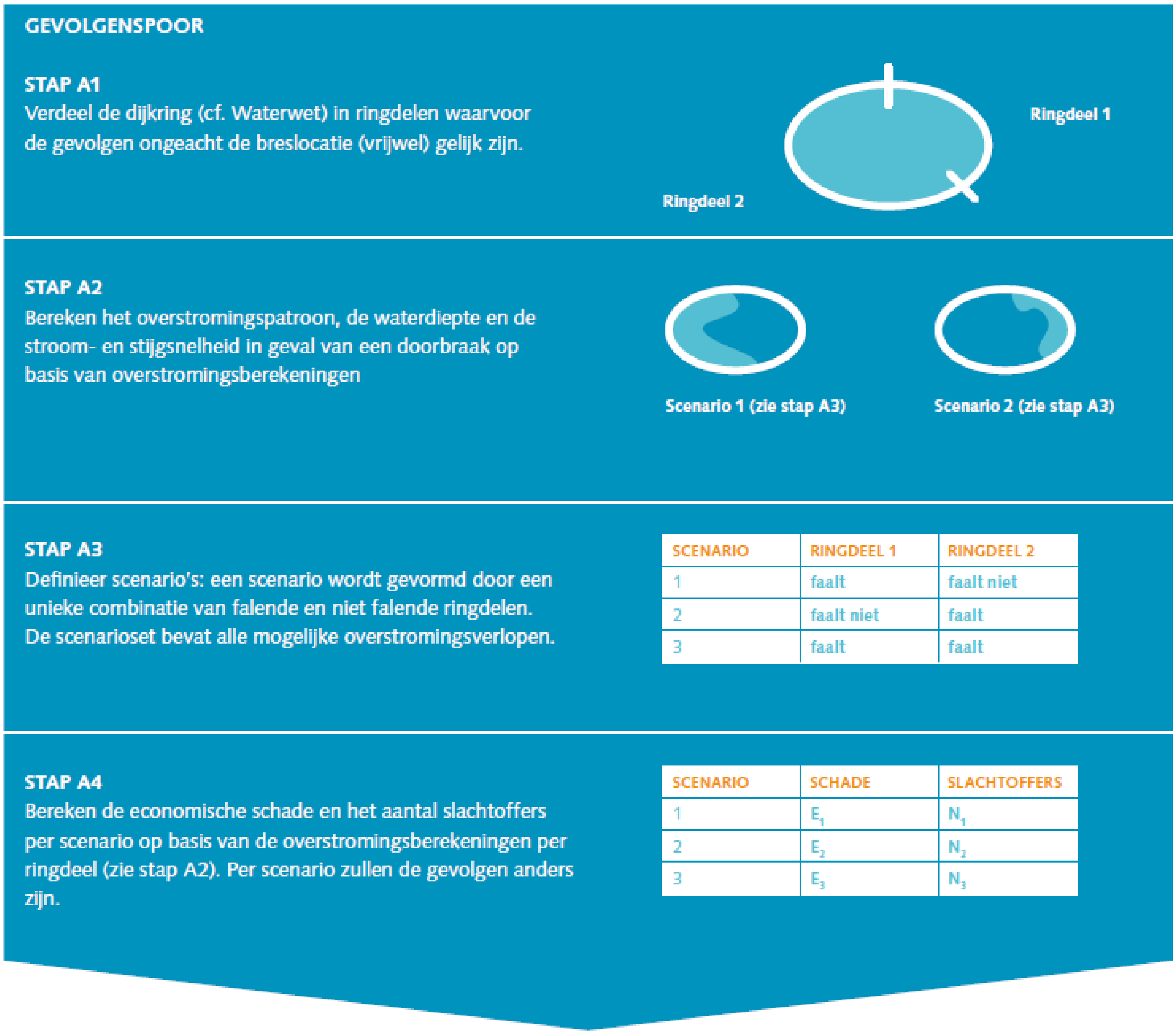


Figure A-4: Determination of consequences (Projectbureau VNK2, 2011)

The probability track starts with dividing the sections in elements like dike parts and structures where the loads and strengths may be considered homogeneous. For every element the probability of failure is calculated. The separation is made on basis of changes in load or strength properties, flood defences of different categories, boundary of a dike ring section or water board area and the presence of structures. A structure counts as a single element and for the calculation of risk there is no difference between a dike and a structure.

Every element and every failure mechanism is schematized to describe the properties that are relevant for determining the probability of failure. Based on the calculated probabilities of failure per element and per failure mechanism a combined probability for all elements and mechanisms is made: this is the probability of flooding of a dike ring. It is at least equal to the maximum failure probability of one element and at most equal to the sum of all failure probabilities of all the elements. The probability of flooding of a dike ring is equal to the probability that any element in the dike ring fails.

Some mechanisms have a high spatial correlation, but for example piping can have very much spatial difference since it is largely dependent on the ground properties. The probability of failure of these mechanisms increases with the length of the dike. This is called the length effect.

Not all failure mechanisms from the legal test are taken into account in VNK2. For example micro-instability and heave are not taken into account. Also damage of the slope caused by flow and waves is not taken into account. These mechanisms are not induced by high water and they will not directly lead to a breach.

Hydraulic load consists of a water level and wave attack. The legal test uses Hydraulic Boundaries (HR2006) and VNK2 uses TMR2006 (Thermometerrandvoorwaarden 2006). Especially in the river area these boundary conditions have some differences, of about 20 cm in water level and up to 30-50 cm in the Vechtdelta area. Also in VNK2 not only the test level is taken in to account but the whole water level distribution (Figure A-5), since a flood defence can also fail at a water level below or above the test level. Emergency measures are not considered.

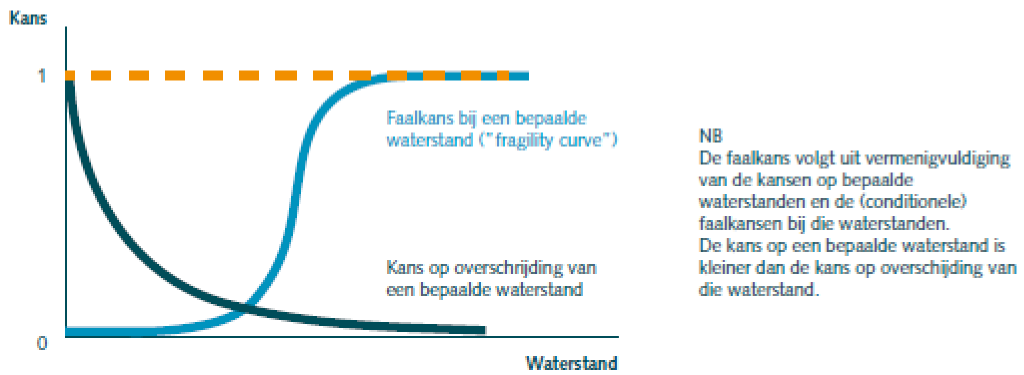


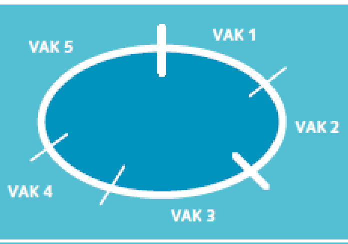
Figure A-5: Probability of failure (Projectbureau VNK2, 2011)

The last step is to calculate the chance a certain scenario will indeed occur. A scenario probability is calculated based on the failure probabilities per element. Per section the consequence is independent of the location (which element) of the failure. The chance on a certain scenario is equal to the chance that the concerned section fails anywhere, while the other sections remain intact. These scenarios represent all the possible floods in the dike ring area, so the sum of the scenario probabilities is the probability of flooding of the whole dike ring area.

KANSENSPOOR

STAP B1

Verdeel de dijkring (cf. Waterwet) in vakken waarin de sterkte-eigenschappen en belastingen homogeen zijn. De grens van een ringdeel valt samen met een vakgrens.



STAP B2

Bereken per vak een faalkans voor de verschillende faalmechanismen. Uit de combinatie van de kansen per faal-mechanisme per vak volgt de kans op een overstroming ergens in het dijkringgebied. Bij het combineren van de faalkansen wordt rekening gehouden met afhankelijkheden tussen faalmechanismen en vakken.

VAK	FAALKANS PER FAALMECHANISME		FAALKANS PER VAK
	Overloop	Piping	
1	$Kans_{over,1}$	$Kans_{pip,1}$	$Kans_1$
2	$Kans_{over,2}$	$Kans_{pip,2}$	$Kans_2$
3	$Kans_{over,3}$	$Kans_{pip,3}$	$Kans_3$
4	$Kans_{over,4}$	$Kans_{pip,4}$	$Kans_4$
5	$Kans_{over,5}$	$Kans_{pip,5}$	$Kans_5$
Gecombineerd	$Kans_{over}$	$Kans_{pip}$	Overstromingskans

STAP B3

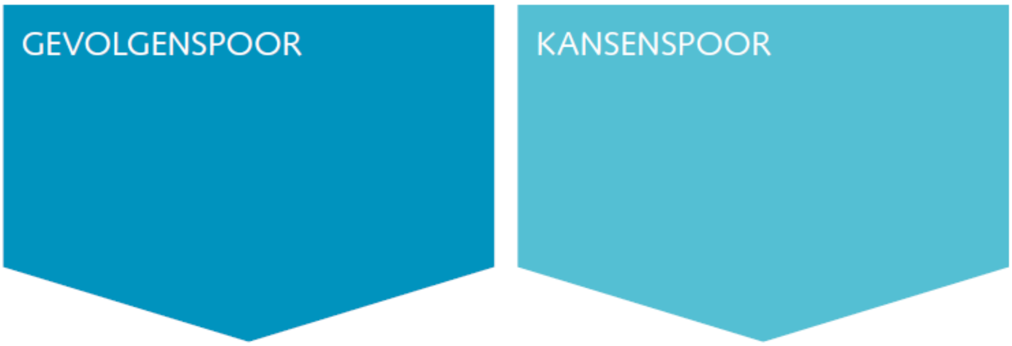
Bepaal de kansen op het optreden van de scenario's (zie stap A3). Dit wordt gedaan door op basis van de kansen per vak te berekenen wat de kans is dat er in bijv. ringdelen 1 en 2 tegelijk een bres optreedt. De scenariokansen zijn nodig om de koppeling tussen kansen en gevolgen te kunnen maken.

SCENARIO	SCENARIOKANS
1	$Scenariokans_1$
2	$Scenariokans_2$
3	$Scenariokans_3$
4	Overstromingskans

Omdat de scenarioset alle mogelijke overstromingsverlopen omvat, is de som van de scenariokansen gelijk aan de eerder berekende kans op een overstroming ergens in het dijkringgebied.

Figure A-6: Determination of probabilities (Projectbureau VNK2, 2011)

The flood risk is calculated based on the probabilities and consequences per scenario. The sum of all the scenarios is the total flood risk, since all the scenarios together characterize all the possible floods within the area.



RISICOBEREKENING
Bereken op basis van de scenariokansen en de gevolgen per scenario de verwachtingswaarde van de schade en de verwachtingswaarde van het aantal slachtoffers. Een verwachtingswaarde is een gewogen gemiddelde van alle mogelijke uitkomsten, met als gewichten de kansen op die waarden. Op basis van de scenariokansen en de gevolgen per scenario kunnen ook het groepsrisico (FN-curve), de economische schadecurve (FS-curve), het lokaal individueel risico (LIR) en het plaatsgebonden risico (PR) worden bepaald.

SCENARIO	SCENARIOKANS X SCHADE	SCENARIOKANS X SLACHTOFFERS
1	$\text{Scenariokans}_1 \times E_1$	$\text{Scenariokans}_1 \times N_1$
2	$\text{Scenariokans}_2 \times E_2$	$\text{Scenariokans}_2 \times N_2$
3	$\text{Scenariokans}_3 \times E_3$	$\text{Scenariokans}_3 \times N_3$
Som	Verwachtingswaarde schade	Verwachtingswaarde slachtofferaantal

Figure A-7: Determination of total flood risk

A.2.3 Calculation method

The probability of failure is calculated with a so called limit-state function, also a Z-function. This function describes the difference between strength and load.

$Z = R - S$

R = Resistance

S = Solicitation (load)

When $Z<0$ the load exceeds the strength and failure will occur. For most strength and load properties the exact values are unknown so they have a probability of occurrence. For some combinations failure will occur. The probability of failure of an element is determined by the sum of all the probabilities that go with combinations of values where $Z<0$. The calculations per failure mechanism are calculated with a FORM calculation.

The failure probabilities per element and per failure mechanism have to be combined in order to gain information about the total probability of failure for a dike ring. The values are not simply added but also dependency between different properties or loads is taken into account. A dike ring is considered a series system which fails when one of the links fails. This gives the following boundaries for failure probabilities:

Lower boundary: $P_{combi} = \mathrm{Max}(P_i)$

Upper boundary: $P_{combi} = \mathrm{SOM}(P_i)$

A.2.4 Results

The results show that the strength of flood defences varies widely and also the consequences can be different per area. Also the location of a dike failure can have a great influence on the gravity of the implications. Some dike ring area's will fill up as if it were a bowl, where other dike rings may have only little consequence on one location and a far greater impact on another location Figure A-8. If a dike ring is longer (consists of more individual dike sections) the chance of a flooding occurring at a certain location gets bigger. This is known as the length effect. If the required data is collected properly it is very well possible to provide insight to the effects of measures.

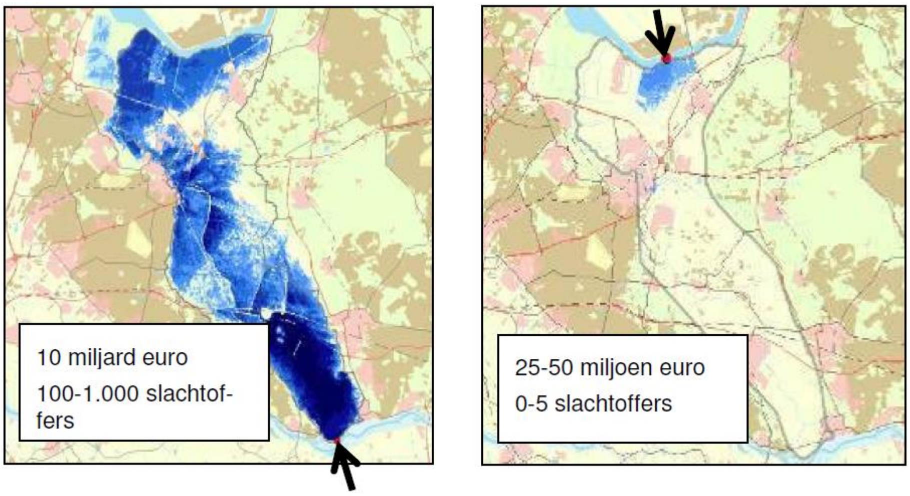


Figure A-8: Different consequences for different breach locations (Werkgroep Deelprogramma Veiligheid, 2014)

A.3 Delta Programme 2015

In the Delta Programme of 2015 five delta decisions are proposed to improve the flood safety policy of the Netherlands and the fresh water management. The delta decisions are as follows:

- Decision Flood safety: a new approach to protect people and economy against floods;
- Decision Fresh water: new approach to reduce water shortage and optimizing the usage of fresh water for economy and utility use;
- Decision spatial adaptation: new approach for robust and climate proof development in urban areas;
- Decision Rhine-Meuse delta: structuring choices for flood safety in the Rhine-Meuse delta;
- Decision Ijsselmeer area: structuring choices for flood safety and fresh water in the Ijsselmeer area.

34.7% of the flood defences along the main rivers, the big lakes and the sea do not meet the current safety standards. Within the Netherlands there is a lot of difference between the safety levels, and especially in the river area the safety levels are too low. Besides that, the current standards are old-fashioned. There are more people and more economic value that needs protection. Also the knowledge has improved since the legal standards were established. Due to climate change the expected water levels keep increasing. All these problems are water safety tasks that the Delta programme aims to solve.

The vast part of the Netherlands is vulnerable to floods. The majority of the Dutch people lives in areas that are floodable, and the largest part of our economy is also established in these areas.

Knowledge that was missed after the 1953 flood is now available, and with this a transition towards the flood risk approach is possible. We can now asses both the probability and the consequences of a flood in detail and translate this to a safety level. The introduction of the flood risk approach will have severe effect on the standards for our flood defences as well as for the way they are tested and designed. The new standards should be introduced in 2017 and in 2050 all the flood defences in the Netherlands should meet these. Since already 34.7% of the flood defences in the Netherlands don't meet the current standards this is a great task.

A.3.1 Decision water safety

The delta decision on water safety proposes to implement the flood risk approach in the legal system. This will ensure that the requirements for flood defences will have a direct relation with the possible consequences. The new policy will ensure that for every single person in embanked area a safety level of 10^{-5} will apply. This means that the chance that a person will die as a consequence of a flood should be below 1 in 100,000 per year. In areas where the consequences are larger, for example very densely populated areas or areas where the economic consequences are very large, a higher safety level will be used. This minimum safety level per person is a new approach, and it will reduce the difference in individual risk as it is now. After implementation of the new approach the economic risk decreases with factor 20 and the group risk decreases with factor 45.

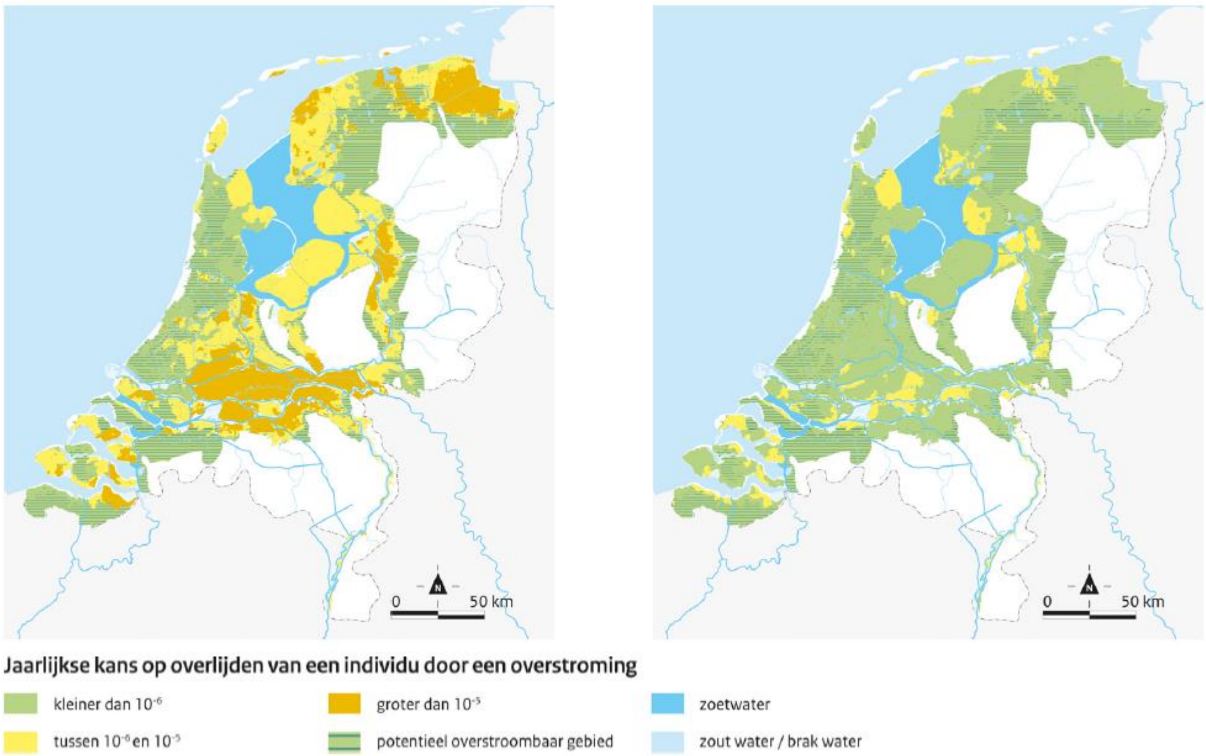


Figure A-9: LIR in the reference situation (left) and in 2050 (right) (Ministerie van Infrastructuur en Milieu, 2014)

In the current water safety policy the standards are defined per dike ring area. However, the consequences of a flood are highly correlated with the actual location of the breach (Figure A-8). In the new approach the division in dike ring approach will not be used anymore. Instead a division in dike sections will be applied and the safety standards will be defined per dike section, so investments can be better balanced. The safety levels that are proposed are: 1:300, 1:1000, 1:3000, 1:10,000, 1:30,000 and 1:100,000. Sub-programmes per area and the national goals of the water safety policy have led to the specification of the safety standards. These standards are presented in Appendix E. The flood defences that already meet the requirements will be maintained, the rest will be improved before 2050. This will be done by means of dike improvements and spatial measures as well, and in some cases Delta dikes (with a very small probability of failure) will be the best solution.

Flood risk can be treated on different levels, or safety layers. This is the principle of multi-layer safety. The three layers are flood prevention measures, spatial adaptation to reduce consequence and crisis management. The delta decision water safety and spatial adaptation aim to meet this principle. The water safety decision is based on layer one and the spatial adaptation decision is based on layer two. Layer three is taken care of by water authorities and other authorities.

Figure A-10: Multiple layer safety

Prevention remains the main issue. The safety level should be reached by reducing the probability of flooding by building strong dikes and flood defences and by giving the rivers enough room to be able to safely discharge the design discharge. Only in specific cases where it is very expensive or socially undesirable to reach the required safety levels by means of these measures, a so-called 'smart combination' with level two measures is made in order to reach the safety level. If this is the case, specific agreements are made upon the division of task, responsibility and cost. The area-oriented approach of the Delta programme makes it easy for provinces and municipalities to work together with flood defence management and the national government. If instead of first layer measures a combination of measures is proposed it is necessary to be approved by the minister before it can be executed.

The measures that have to be taken for water safety are recorded in the Delta plan water safety. The preferred strategies for the different areas give the direction for programming of the measures. The execution of flood risk measures can be divided in two categories; regular dike improvement and more complex spatial measures and 'smart combinations'. The dike improvement measures are prioritized and programmed within the High water protection plan (HWBP) which is a part of the Delta plan water safety. In the Delta plan water safety a preliminary program of river measures is given. In the Delta Programme 2016 this will be elaborated in further detail.

The implementation of the new water safety policy proceeds in three steps. The flood risk approach is already being used in the prioritizing of the measures for the High Water Protection Programme (HWBP). The new water safety policy and corresponding safety standards should be included in the law in 2017. Both Rijkswaterstaat and the water boards have a positive position towards the new approach. Also the test instrumentation has to be adapted to the new approach. New tools for the testing and design of flood defences are being developed and will be introduced in two steps. In 2017 it will be possible to register flood defences that do not meet the new standards for the HWBP. In 2019 a detailed assessment tool will be available. In the new test instruments the effect of foreland and other present elements on the reduction of the hydraulic load and thereby on the flood risk.

It is also of great importance that crisis management, especially preventive evacuation, is always in order. The involved authorities work together for a waterproof evacuation plan, and among citizens awareness is created about self-reliance.

A.3.2 Preferred Strategy Rivers

Presently the river area has the greatest flood risk in the Netherlands. More than 300 km of river dikes do not meet the current standards. A lot of river dikes are sensitive for the failure mechanism piping. Also the economic development in the river area has been large the past decennia, which increases the need for a higher safety level in the river area. Climate change causes the discharges to increase on the long term. For the Rhine the expected normative discharge in 2050 is 17,000 m3/s.

When the dikes are not strong enough, for example when they are instable or when piping is a serious threat, this should be treated with dike improvement. When the task results from higher peak discharges, spatial adaption will be applied. A strong combination of both spatial measures and dike improvements is necessary to create a robust river area.

The Delta plan water safety holds all programmed and scheduled measures, researches and pilots that contribute to the tasks for water safety in the Delta programme. With the results of these researches the proposals for delta decisions and preferred strategies are followed. For water safety among other research is done on the development of new test instrumentation, discharge distribution over the Rhine branches and the usage of 'smart combinations' in several areas (Ministerie van Infrastructuur en Milieu, 2014).

A.4 Background new standards

This appendix discusses the background of the standards. This is based on Technisch-inhoudelijke uitwerking DPV versie 2.2 (Werkgroep Deelprogramma Veiligheid, 2014)

A.4.1 Current system

In the current system the floodable part of the Netherlands is divided in dike ring areas. A dike ring area is an area that is protected against open water by a system of flood defences. The sea, main rivers and big lakes are counted as open water. Flood defences that provide direct or indirect protection against open water are called primary flood defences. There are three categories:

- a-defences directly retain open water and are part of a dike ring,
- b-defences provide indirect protection against open water; (closure) dams and moving flood defences,
- c-defences are part of a dike ring but not directly retaining open water. They separate dike rings from one another or separate dike rings from backwater.

Different categories of flood defences have different standards. The Waterlaw defines the safety standard for a-defences as:

"... The safety level indicated as the average probability of exceeding – per year – of the highest water level for which the open water retaining flood defence should be designed, taking other factors that determine the water retaining capacity." (Werkgroep Deelprogramma Veiligheid, 2014).

For the entire dike ring area the same safety level holds. For b-defences, also referred to as connecting defences, the current demands hold that the defence should have the same safety level as the highest safety level as adjacent or underlying a- and c-defences. For c-defences hydraulic boundary conditions are set that assume an overlying flood defence has failed. When a c-defence separates two dike ring area with different safety level, the highest value is normative.

A.4.2 Characterization of the new standards

The fact that in the current system the standards are set per dike ring area, and the fact that the consequences are only roughly incorporated in the definition of safety levels lead to a limited effectiveness of this method. Also the interaction between dike ring areas is disregarded.

By improving the coherence between standards for flood defences, the flood risk approach offers perspective to increase the effectiveness of the flood defence system. To reach this the following aspects should at least be taken into account:

- More differentiation in safety levels of dike rings and reaches: adapt the safety level to differences in consequences within a dike ring area;
- Standards for b-defences should be adapted to the underlying area: take the reducing effect of water bodies between the defences and the actual flood risk of the underlying area into account
- The position of c-defences can change: The standards for dike sections are no longer uniform for the whole dike ring area so the function of c-defences will also be different.

A.4.3 Exceeding to flooding

The current standards are expressed as a probability of exceeding extreme conditions for which a flood defence should be designed. The new standards will be expressed in terms of probability of flooding, the probability of a breach in a dike reach that actually leads to inundation of the underlying area.

The probability of exceeding determines the load on the flood defence. The reliability of the defence is assessed separately per dike section (dijkvak), and the different mechanisms are assessed separately.

The probability of flooding is the probability that the load on flood defence is higher than the strength, which causes failure and thereby the creation of a breach of a flow over the defence. This calculation includes the probabilities of failure of all the failure mechanisms and the reach as a whole. The assessment of reliability of the flood defence under extreme conditions is captured in one single number: the probability of flooding. The probability of flooding occurring in an area is obtained by combining the probabilities of flooding of all the dike reaches that may cause flooding of this particular area.

The standards are future oriented since it is not a reaction on a disaster (as was the case in 1953) but we are adapting the existing system to future changes in the hydraulic conditions. Not only discharges and water levels will rise in the future, also the economic value of the protected area grows. The new standards give requirements that should be met in 2050.

Since the probability of flooding will increase over the years due to climate change and land subsidence, and the potential damage grows due to economic growth, the investments made in a flood defence cannot be seen as a one-time decision, but it must be seen as an exercise that should be repeated. When in the current situation it is found that a dike (section) does not meet the standards, it can take up to 10 or even 20 years until the dike is actually improved. All this time the safety level of this dike was too low, which is a dangerous and risky situation. From this point of view it would be better if the decision to improve the dike would be made before the safety standard is exceeded.

The middle probability value lies somewhere between the maximum allowable probability and the design probability. At the moment the middle probability value is exceeded it gives enough time to improve the flood defence before the maximum probability of failure is reached.

The principles on which the new safety standards are based are the following:

- Every person in an embanked area should have the same level of basic safety: the probability of getting killed in a flood should not be higher than 1 in 100,000 per year (Local Individual Risk, LIR),
- Prevent social disruption: Extra protection for areas where big economic damage or large numbers of casualties are expected,
- Protect important infrastructure: Facilities like utilities and hospitals are important after a disaster, therefore they should be protected.

The basis safety level is elaborated with the local individual risk (LIR). It is determined per dike reach what the maximum probability of flooding may be so that the LIR value of 10^{-5} is not exceeded. Also the economic growth and the increasing flood probability due to climate change and land subsidence has been taken into account to establish standards that will still satisfy in 2050.

The consequences of a flood are determined by simulating flood scenarios. The same scenarios as have been used in WV21 and VNK2 were used, updated and refined if necessary. The flood scenarios assume that regional flood defences do not fail, so they are related to the failure of a-defences. The scenarios show that the dike ring area's in the river area can have deep water levels when flooded.

A.4.4 Division of reaches

Reaches are distinguished by the level of threat, difference in consequences, flooded area and the length of a reach. Significant differences in the consequences between two dike parts give reason to distinguish two dike reaches.

In the current system the division is made up by dike ring areas. For the whole dike ring the standards are equal. However, consequences of floods are highly dependent of the location of the breach or failure within the dike ring. This is for example the case in Dike ring 45 Gelderse Vallei, as can be seen in Figure A-8. When a breach occurs in the south the consequences are worse than when it would happen in the north. The economical optimal safety level is thus in the south larger than in the north. Because the new water safety standard is based on the flood risk approach, the standards are expressed in terms of allowable flood probability, dependent on the consequences. This means in practice that for dike reaches within the same dike ring different standards can apply.

The division in dike reaches is based on the division in dike sections as used in VNK2. This division is too detailed though, which would result in a specification that is too much scattered. When the division is scaled up a level, local coincidences lose importance and a much more organized image is created. Also the length of different reaches should not differ too much, for the same reason. For the analyses the reference situation is defined as the expected probability of flooding (2015/2020) after completion of the projects RfR and HWBP2. The probability of flooding per dike reach in the reference situation is shown in Figure A-11.

Figure A-11: Probability of flooding in the reference situation (Werkgroep Deelprogramma Veiligheid, 2014)

Local Individual Risk (LIR) is the probability per year to die in a flood, thereby taking the possibility of evacuation into account. It is defined as the product of the probability of flooding, percentage of people that stay after evacuation and mortality. Based on flood scenarios and estimated evacuation percentages the LIR can be mapped. Figure A-12 shows the LIR-values for the reference situation. It can be seen that a large part of the Netherlands has a higher value than the indicated value of 10^{-5}. The river area is one of the locations where the higher LIR values occur.

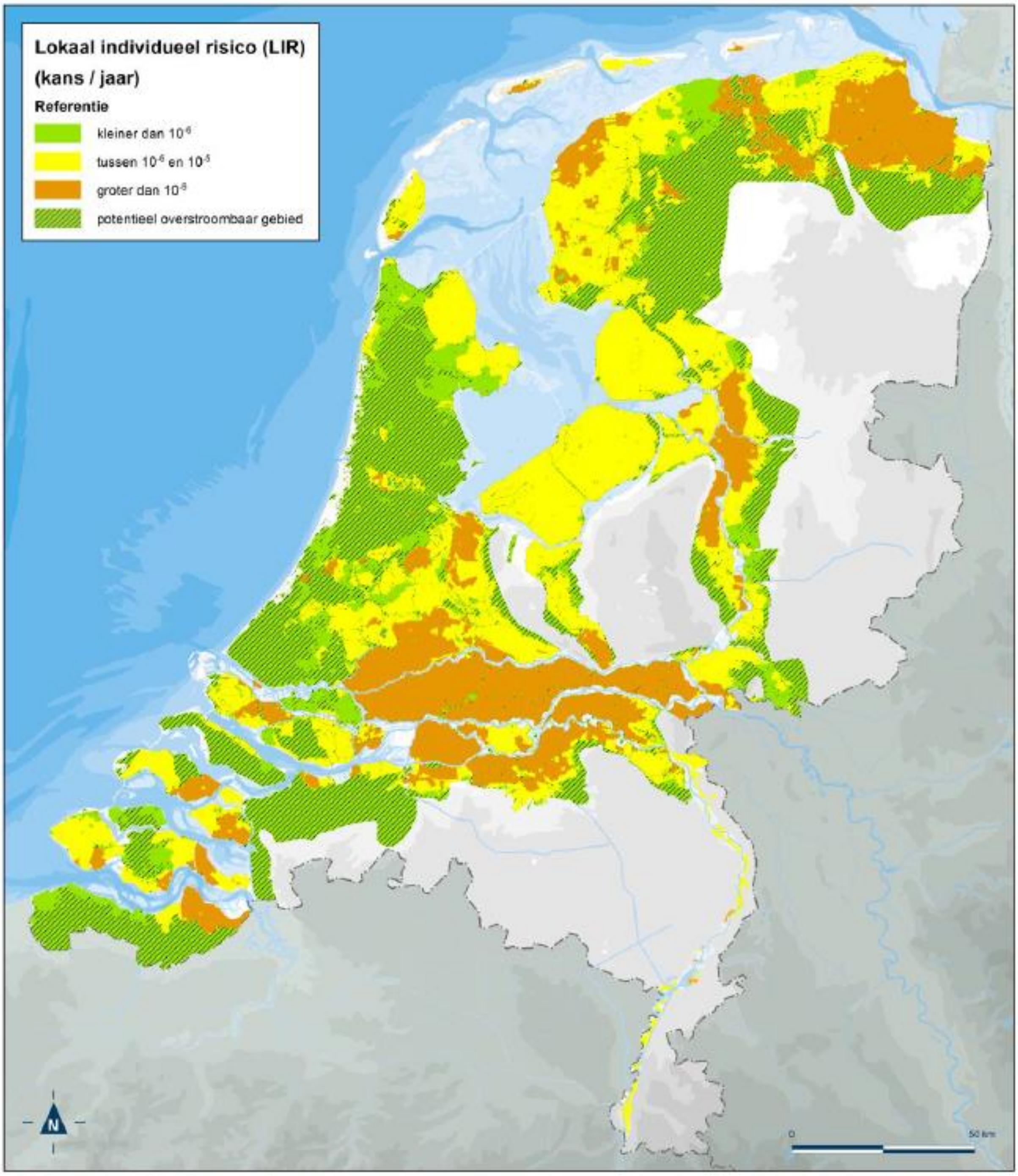


Figure A-12: LIR in reference situation (Werkgroep Deelprogramma Veiligheid, 2014)

The economic risk is the product of flood probability and economic damage caused by a flood. The damage has been calculated based on flood scenarios, using damage functions for different land use. The economic risk is calculated for the year 2050, since this is the year the new standards shall be met. High values for economic risk can be caused by either large damage, high probability of flooding, or both. Big economic risks are found in the river area, Flevoland and the southwest delta of Holland. The economic risk in 2050 with flood probability as in the reference situation is presented in Figure A-13.

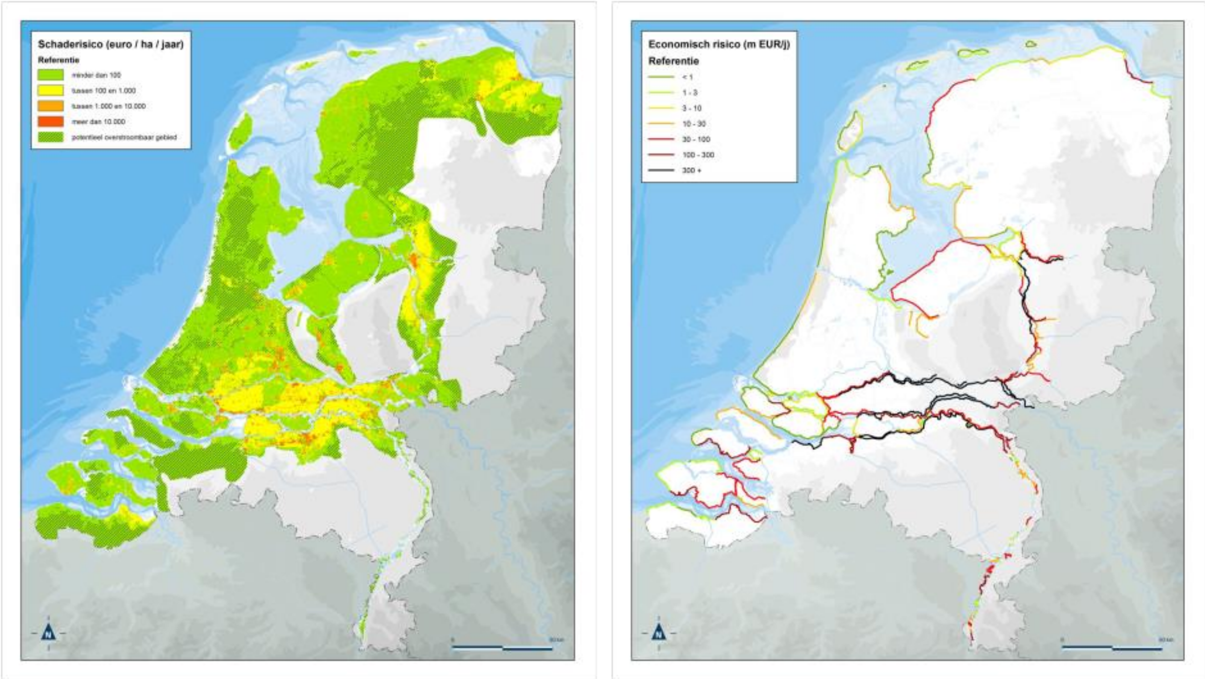


Figure A-13: Economic risk in 2050 with probability of flooding as in the reference situation per area (left) and per reach (right) (Werkgroep Deelprogramma Veiligheid, 2014)

From social point of view it is important to look at the probability of large groups of casualties in a flood. A flood with a lot of casualties has a larger impact on society than a small one. This group risk depends on the probability of flooding, the number of expected casualties and the dependency between flood probabilities in different reaches. There are different scenarios of flooding within a dike ring but also multiple dike rings can flood during one event. The probability of this occurring is important for the total group risk of the Netherlands.

For the river area a method is developed to take both correlation in hydraulic load and system working into account. When there is a breach in a dike section, downstream of this location the hydraulic load decreases. In the river area system working is important because the amount of water in the rivers is limited. Several analyses have been executed with and without the effects of system working into account. The calculation with system work is more realistic but also more sensitive to uncertainties. The situation without system work is more conservative and thus leads to overestimation of the calculated values. Despite all of this, the safety standards for group risk are calculated without system work, so there is an overestimation.

Group risk is presented by the FN curve (Figure A-14), which represents the probability that there are more than N casualties. The surface underneath the graph is equal to the expected number of casualties per year.

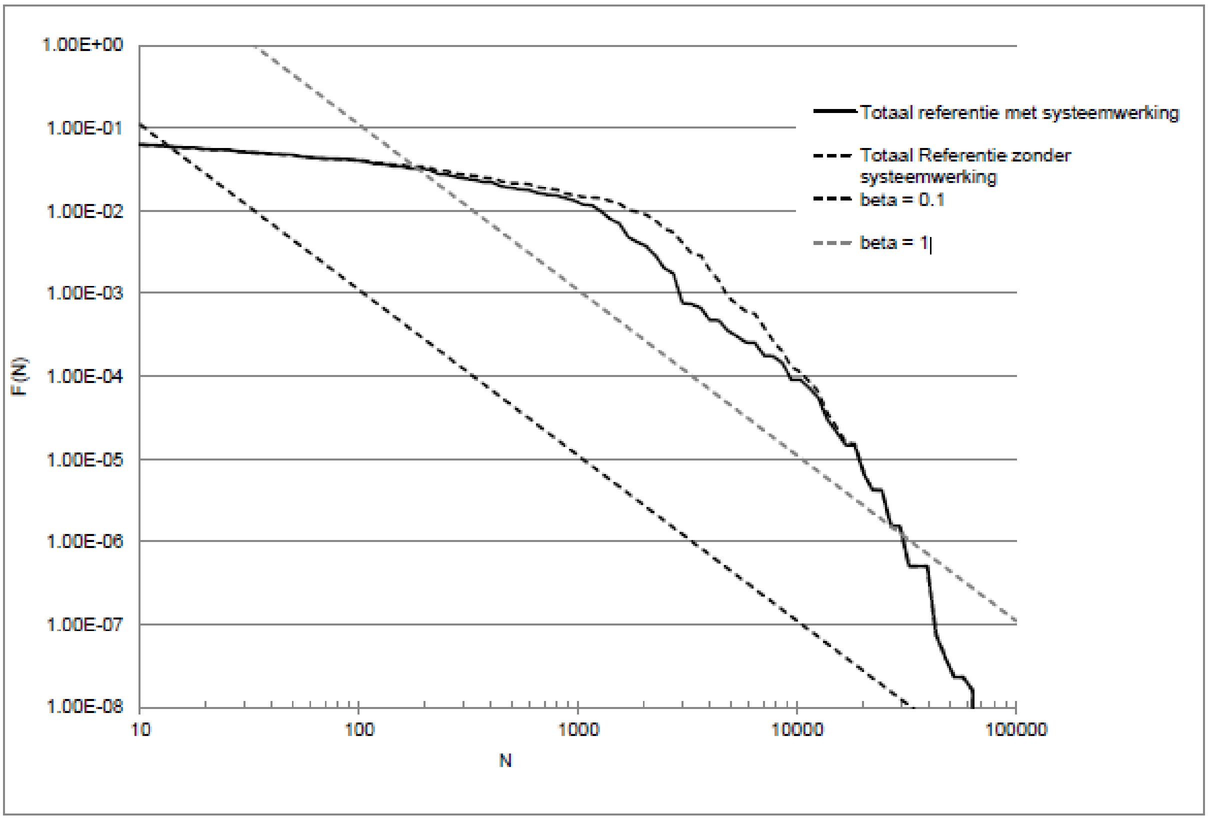


Figure A-14: FN-curve for the reference situation (Werkgroep Deelprogramma Veiligheid, 2014)

A.4.5 New standards

The main principle for the water safety policy is that everybody should have a maximum Local Individual Risk of 10^{-5}, this is the probability of dying due to a flood. This basis safety level can be reached by improving places with a large individual risk. If the flood area of a reach has an overlap with another reach the LIR value consists of more contributions of reaches.

The basis safety level gives for every reach the standards so that for every adjacent area the LIR standard is reached. This does not always mean that a reduction of the probability of flooding is necessary. In some cases it is also possible to achieve the safety level by measures of layer 2 and 3. So the LIR value of 10^{-5} does count for everybody, but not everywhere. This makes it possible to exclude areas with very high demands for basic safety from the equation. Figure A-15 gives all the values for maximum probability of flooding per reach, based on the requirements for basic safety.

The requirements for basic safety are derived from the distribution of the LIR values over the floodable area, where the area with the maximum LIR value is decisive. The LIR value is composed by the probability of flooding, percentage of evacuation and mortality. All these composites have certain uncertainties that are all found back in the LIR value. Above that, the LIR value is very much sensitive to local variations.

Figure A-15: Required safety level from LIR (Werkgroep Deelprogramma Veiligheid, 2014)

The economic optimum safety level depends on the economic damage after a flood and the cost required to reduce the probability of flooding (Figure A-16). For every reach the flood damage in 2050 is calculated as well as the cost to increase the safety level with factor 10. The highest safety levels are in the river area and in Rijnmond-drechtsteden and Flevoland.

The group risk doesn't count per area, but it is a national achievement, accomplished by the total flood defence system. This can be interpreted in several ways. There are two reduce the group risk, reducing the probability or reducing the consequences. The safety standards are probabilities of flooding per reach and so the reduction of flood probability is the aim. The group risk in the reference situation is mostly contributed to by the central river area and the southern coast area, since the probabilities of flooding are very high here.

Figure A-16: Required safety level from economic optimum (Werkgroep Deelprogramma Veiligheid, 2014)

By combining the standards based on LIR, economic value and group risk man can establish a possible safety standard. The highest standards are normative for every reach. The proposed safety level is presented in Figure A-17.

Figure A-17: Combined probability of flooding standard (Werkgroep Deelprogramma Veiligheid, 2014)

B. Validation of calculation method

When changes are made to a dike system, for example when a measure is taken or when the boundary conditions change, a tool is required to calculate the impact on the probability of failure. Software like PC-ring or Hydra can make these calculations but they require a lot of input data, and thus the total calculation process takes a lot of time. In order to be able to make faster calculations an alternative calculation model is made. This calculation makes use of PC-Ring data and PC-Ring can be used to validate the results. This method makes it possible to assess the effects of measures and climate change without going into too much detail.

This memo describes how the required data can be retrieved from PC-Ring and how the calculation can be performed. Also the method will be validated by calculating the probability of failure for different dike sections and comparing the result with the results of PC-Ring. This reference calculation should give comparable results. If this is not the case it is very hard to draw valuable conclusions If the method is validated the calculation can also be done with data sets other than those from PC-Ring.

B.1 Method

From the theoretical basis it is known that the probability of failure of a dike or a dike section can be built up by the fragility curve and the water level statistics. The fragility curve shows the conditional probability of failure, where the condition is the water level. The water level statistics indicate the frequency of occurring of (high) water levels. When these are combined this gives the total probability of failure of the dike. This is illustrated in Figure B-1. The upper graph shows the water level statistics (the probability density of the water level). The middle graph shows the probability curve (the probability that a dike fails, given the water level). The lower graph shows the combination of the first two graphs. The red hatched area indicates the total probability of failure, this is equal to the area under the curve. This is thus a summation of all the separate probabilities that the dike fails at a given water level, provided the probability that this water level occurs. The red star indicates the design point. This is the point with the highest probability density, hence the most likely failure situation.

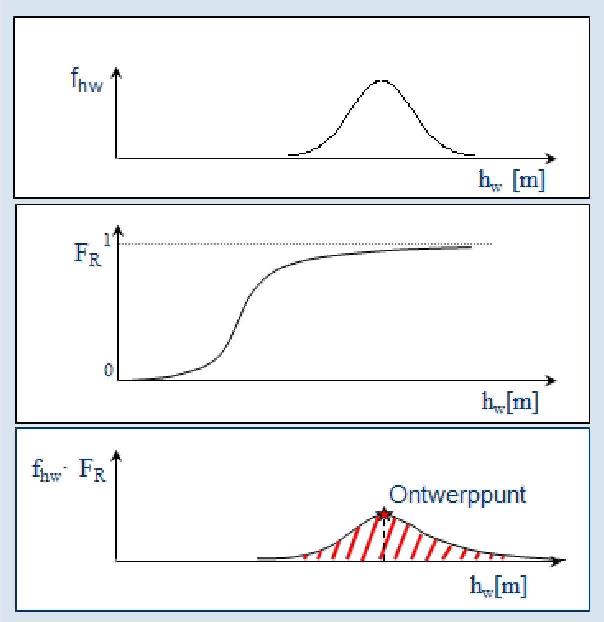


Figure B-1: Theory of the probability of failure

B.1.1 Derivation of the fragility curve

The fragility curve is retrieved from PC-Ring by making a design calculation. To make this calculation first the discharge is set to be constant in time on 5893.3 m^3/s. This value has an exceedance frequency of 1 per year value of the discharge ($Q_{1/year}$), based on the currently prevailing discharge statistics (TMR2006). The model now has a constant discharge, with a corresponding constant water level which is different for every location. This is of course the water level that has a probability of exceeding of 1 per year ($h_{1/year}$).

The input is a range of water levels. These are entered as values for the water level error (modelfout in locale waterstand). The range chosen is between the toe of the dike and approximately one meter above the rest of the dike. The local water level error values represent a water level which is relative to the 1 per year water level the model uses. For example, a model error value of -0.5 means that the water level used for the calculation is 0.5 m lower than the value the model would use in a normal calculation. The interface of the design calculation tool in PC-Ring is shown in Figure B-2. The program will now calculate the probability of failure for all water levels between $h_{1/year}$-1m and $h_{1/year}$+4m, with intervals of 0.05m.

The output exists of a table with the given values of the water levels and the probability of failure for this water level. This probability of failure is given in four different values: the upper and lower boundaries of the probability of failure, the probability of failure per block of 12 hours and the probability of failure per year. These values can differ a lot and so care should be taken in selecting the appropriate probability of failure for the calculations. In fact, since the discharge statistics have been modified, none of these values are exactly correct and some modification will be required. This will be elaborated in detail in B.4 Method of deriving fragility curve. The fragility curve can be plotted from these values and is shown in Figure B-3. From now on the fragility curve for overflow/overtopping and macro stability will be made with the values for probability of failure per 12 hours. For piping and stability inner slope use is made of the probability of failure per year.

Berekeningsmethode ontwerp- of gevoeligheidsberekeningen

	MHW controle	Overloop/golfoverslag
OntwerpMethode	Tabel met betrouwbaarheidsindex	Tabel met betrouwbaarheidsindex
Stochast	MHW hoogte	Modelfout in locale waterstand
Minimum waarde	0,00	-1,00
Maximum waarde	14,00	4,00
StapGrootte	0,05	0,05
Eerste waarde iteratieproces	0,00	0,00
Tweede waarde iteratieproces	0,00	0,00
Betrouwbaarheidsindex	0,000	0,000

Figure B-2: Design calculation function in PC-Ring

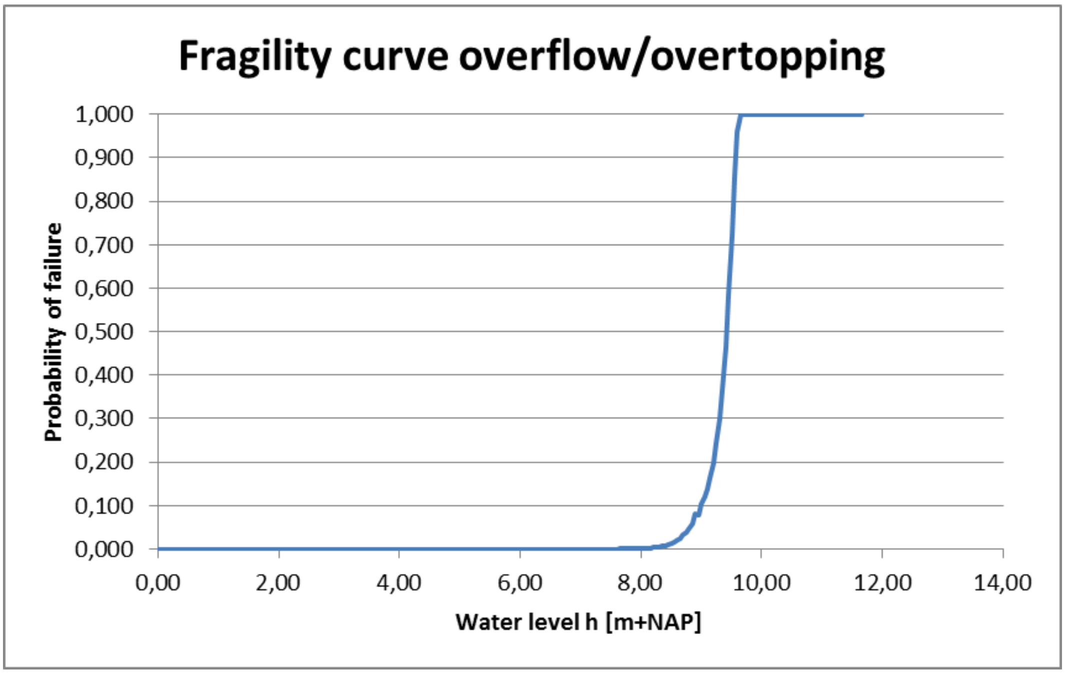


Figure B-3: Fragility curve for failure mechanism overflow/overtopping

B.1.2 Water level statistics

The probability density curve of the water level is not as easily retrieved from PC-Ring. To retrieve this use is made of the PC-Ring input data. The PC-Ring database contains a lot of hQ-stations. These are locations where the Q-h relation is measured or modelled. Before a calculation is made the user has to specify which station is used in the probabilistic calculation. Since adjacent stations will have slight differences due to differences in the river geometry at that specific point, it can be expected that different stations will lead to a slightly different probability of failure. PC ring contains stations at the river bank and at the river axis.

The PC-Ring input for the Q-h relation exists of only 9 discharge levels and the corresponding water levels. In PC-Ring the Q-h relation is retrieved by a 3rd order fit through WAQUA output data, and some correction for assumptions and to match the Q-h relations to the known MHW (Wouden & Grashoff, 2009). The 9 points that are shown in the PC-Ring input file are extracted out of this fit. If another fit is made through these points the retrieved expression will most certainly not be exactly equal to the PC-Ring input data. The Q-h relations used in this calculation are thus introducing an error. The magnitude of this error depends on the shape of the Q-h relation. If this is smooth the relation can be approached relatively accurate by a 2nd or 3rd order polynomial. If the relation has a "kink" the use of a 2nd order polynomial will introduce a large error. A 3rd order polynomial would give better results. Linear interpolation is also a good option. For the first explorative calculations a 2nd order fit will be used since this is most easily retrieved.

The Q-h relation can be used to calculate the Q for every water level that is considered. It is a logical choice to use the same water levels as for the fragility curve. In this way errors induced by interpolation and round off are prevented. With the discharge the return period can be calculated by using the expression for the discharge statistics:

$$Q(T) = a \ln(T) + b$$

The values for a and b are coefficients that are different for different intervals of T (Table B-1). Figure B-4 shows the process of combining the Q-h relation with the discharge statistics.

Table B-1: Values of a and b for HR2006

	0 < T ≤ 2	2 < T ≤ 25	25 < T ≤ 10,000
a	1620.7	1517.78	1316.43
b	5893.3	5964.63	6612.61

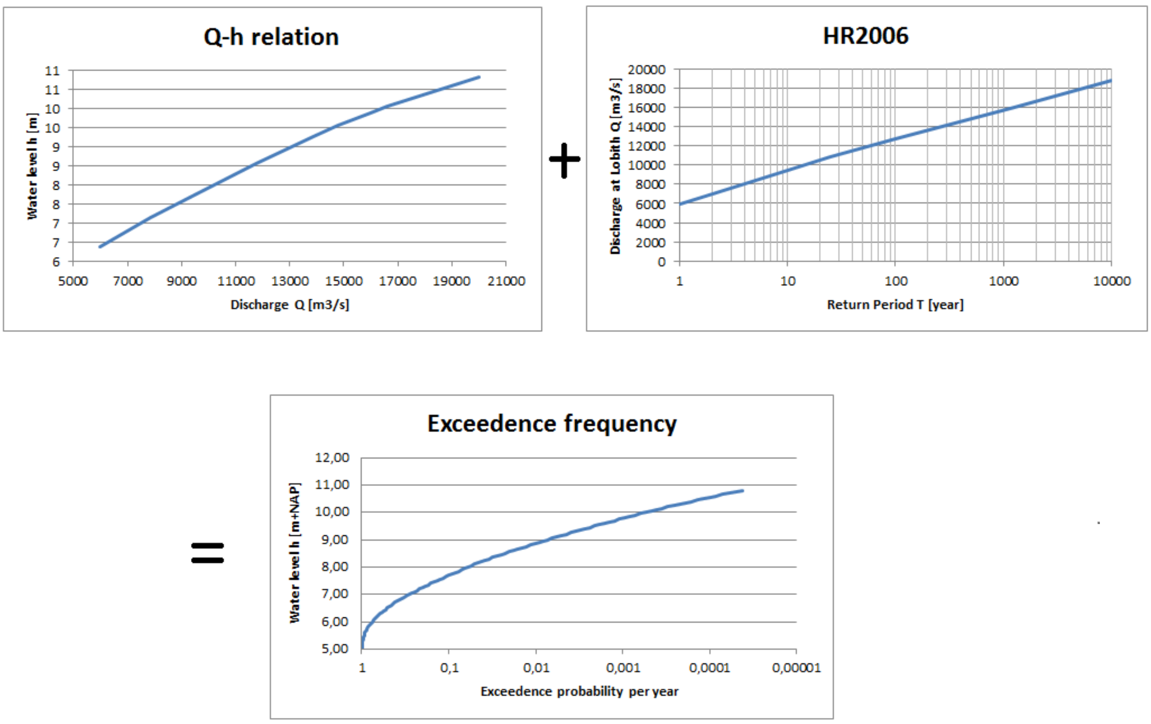


Figure B-4: Combination of Q-h relation and discharge statistics

When the return period T is known it is possible to calculate the probability of occurring per year. The Poisson distribution is used to calculate this:

$$P(Q > \underline{Q}) = 1 - e^{\left(-\frac{1}{T}\right)}$$

This is the probability that a certain discharge is exceeded within a period of one year. It can be seen that the probability that the discharge which has a return period of T=1 is exceeded, is not equal to 1. This is because this discharge occurs on average once per year, but it is very well possible that there is a year in which it is not exceeded. The probability is $P\left(Q > Q_{1/year}\right) = 1 - e^{-1} = 0.63$.

The probability of exceeding is easily translated in the probability of non-exceeding. The probability of non-exceeding shows the cumulative probability density curve. The probability density curve, which is the one that is needed, is the derivative of the cumulative density function. Since this is not available as a continuous function (although it is possible to fit one) the derivative is calculated per location by means of a numerical scheme. In this analysis this is done with the following numerical scheme:

$$f_1 = \frac{3F_2 - 4F_1 + F_0}{2\Delta h} \quad \text{BDF method}$$

There are other numerical schemes which could be used, like one of the Euler schemes, but the BDF (Backward Differential Formula) has the lowest truncation error (Zijlema, 2012). The differentiation of the water level statistics gives the probability density of the water level (Figure B-5).

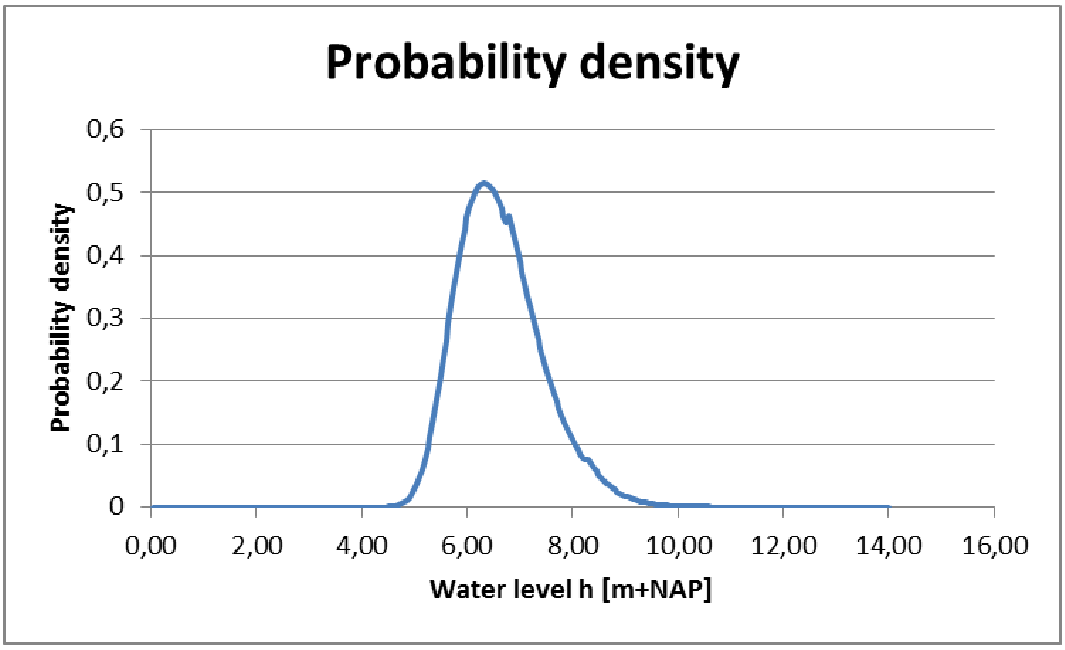


Figure B-5: Probability density curve of the water level

B.1.3 Combination

Now that for every water level the associated probability of failure and the probability of occurring are known it is possible to compute the total probability of failure. Figure B-1 shows that the product of both these functions gives an expression for the total probability of failure. This probability of failure is equal to the surface under the graph. Since no numerical expression is available the integral is approximated with a (right) Riemann sum.

B.2 Fit for Q-h relation

For the fitting of the Q-h relation different methods can be used. 3rd order polynomial fitting, 2nd order polynomial fitting and linear interpolation will be assessed. Since PC-Ring uses this the 3rd order polynomial should give the best results.

Table B-2 gives the error per section and the average error for the failure mechanism overflow/overtopping. The 3rd order fit indeed seems to give the best results, although the errors of interpolation and 2nd order are also very small.

Table B-2: Average error for different fits, overflow/overtopping

	2nd order		3rd order		Interpolation	
Section	Pf	Error	Pf	Error	Pf	Error
43TG191TG202	1.66E-04	1.065	1.67E-04	1.059	1.65E-04	1.070
43TG183TG191	7.09E-05	1.103	7.22E-05	1.085	6.89E-05	1.135
43TG177TG183	1.15E-04	1.019	1.13E-04	1.044	1.08E-04	1.089
43TG171TG177	3.01E-04	1.171	3.09E-04	1.139	3.07E-04	1.146
43TG159TG171	2.42E-04	1.092	2.48E-04	1.063	2.43E-04	1.084
43TG148TG159	1.88E-04	1.197	1.89E-04	1.193	1.91E-04	1.178
43TG144TG148	8.19E-05	1.225	8.52E-05	1.178	8.21E-05	1.221
43TG134TG144	1.86E-04	1.181	1.86E-04	1.179	1.87E-04	1.173
43TG121TG134	9.54E-05	1.091	9.84E-05	1.058	9.47E-05	1.099
43TG106TG121	1.15E-03	1.125	1.25E-03	1.038	1.25E-03	1.039
43TG094TG106	2.49E-04	1.040	2.55E-04	1.015	2.51E-04	1.030
43TG086TG094	2.02E-04	1.050	2.06E-04	1.028	2.00E-04	1.059
43TG076TG086	1.28E-04	1.004	1.27E-04	1.012	1.21E-04	1.057
43TG068TG076	1.25E-04	1.051	1.24E-04	1.063	1.17E-04	1.123
43TG055TG068	1.85E-04	1.012	1.90E-04	1.042	1.79E-04	1.019
43TG039TG055	9.87E-05	1.056	9.46E-05	1.102	8.90E-05	1.172
Average		1.093		1.081		1.106

Table B-3 gives the average error and error per section for failure mechanism macro stability. 3rd order gives again the lowest average probability of failure, although the differences are again small.

Table B-3: Average error for different fits, macro stability

	2nd order		3rd order		Interpolation	
Section	Pf	Error	Pf	Error	Pf	Error
43TG191TG202	3.99E-03	1.049	3.97E-03	1.055	3.66E-03	1.142
43TG183TG191	1.97E-10	3.223	1.97E-10	3.214	2.04E-10	3.107
43TG159TG171	4.49E-10	1.036	4.43E-10	1.051	4.37E-10	1.066
43TG148TG159	6.88E-07	1.553	7.13E-07	1.499	6.67E-07	1.601
43TG055TG068	7.24E-07	1.287	7.40E-07	1.315	7.38E-07	1.312
Average		1.630		1.627		1.646

Table B-4: Average error for different fits, piping

	2nd order		3rd order		Interpolation	
Section	Pf	Error	Pf	Error	Pf	Error
43TG177TG183	9.41E-05	1.316	9.47E-05	1.324	8.92E-05	1.248
43TG171TG177	1.75E-04	1.367	1.72E-04	1.346	1.67E-04	1.304
43TG094TG106	2.42E-04	1.480	2.36E-04	1.449	2.27E-04	1.392
43TG039TG055	2.39E-03	1.512	2.22E-03	1.405	2.23E-03	1.408
Average		1.419		1.381		1.338

Table B-4 gives these values for failure mechanism piping. Now interpolation gives the best values. The error of 2nd order is the largest here, even though the differences remain small. It seems that all fits give reasonable results.

However, the 3rd and 2nd order fits show some strange fits for several dike sections, like in Figure B-6. This may lead to larger errors, seem the fits make no sense from a physical point of view. For this reason linear interpolation will be used. From this point onwards all calculations are made with linear interpolation unless explicitly noted.

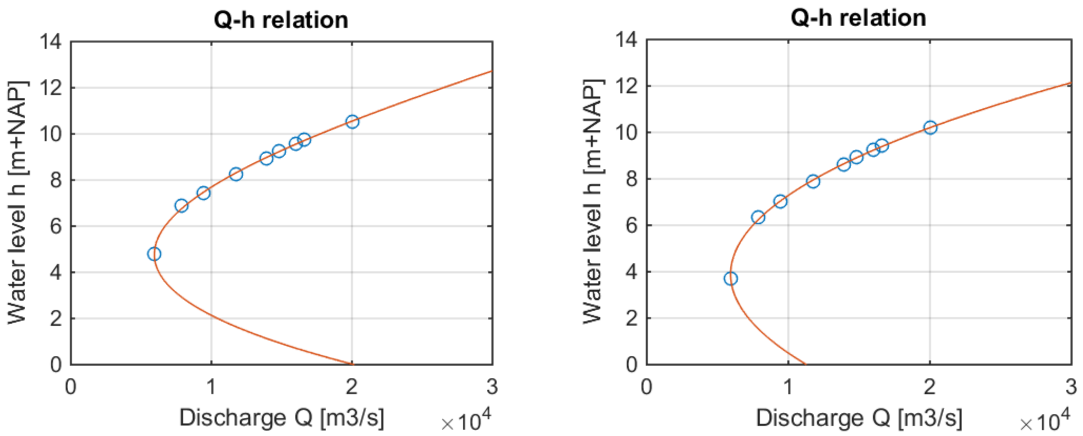


Figure B-6: Q-h relation fit 3rd order (left) and 2nd order (right)

B.3 Accuracy for different hQ-stations

In principle the calculation with the Q-h relation for the measurement location along the bank should be more accurate than the one with the axis station. Whether this is true for all dike sections and failure mechanisms will be assessed below. Also the accuracy of the calculation with the axis station will be assessed, since in practice not always the Q-h relation along the bank will be known. In this situation the probability of failure will have to be estimated by using the Q-h relation on the closest available location.

Table B-5 shows the error per dike section for both locations for the failure mechanism overflow/overtopping. It shows that for some dike sections the bank calculation is indeed more accurate, but for other sections the axis calculation is more accurate. The differences are always small but the average error of the bank calculation is smaller. There seems to be no connection with the difference in the Q-h relations, for example when the bank relation has a kink.

Table B-5: Average error for different hQ stations, overflow/overtopping

	Bank		Axis	
Section	Pf	Error	Pf	Error
43TG191TG202	1.65E-04	1.070	1.27E-04	1.391
43TG183TG191	6.89E-05	1.135	7.19E-05	1.088
43TG177TG183	1.08E-04	1.089	1.31E-04	1.117
43TG171TG177	3.07E-04	1.146	3.56E-04	1.012
43TG159TG171	2.43E-04	1.084	2.96E-04	1.121
43TG148TG159	1.91E-04	1.178	1.42E-04	1.586
43TG144TG148	8.21E-05	1.221	7.07E-05	1.419
43TG134TG144	1.87E-04	1.173	1.36E-04	1.614
43TG121TG134	9.47E-05	1.099	1.03E-04	1.010
43TG106TG121	1.25E-03	1.039	1.21E-03	1.069
43TG094TG106	2.51E-04	1.030	2.80E-04	1.081
43TG086TG094	2.00E-04	1.059	2.09E-04	1.014
43TG076TG086	1.21E-04	1.057	1.01E-04	1.268
43TG068TG076	1.17E-04	1.123	1.17E-04	1.123
43TG055TG068	1.79E-04	1.019	1.68E-04	1.087
43TG039TG055	8.90E-05	1.172	1.04E-04	1.007
Average		1.106		1.188

Table B-6 and Table B-7 show the average error for the failure mechanisms macro stability and piping. The errors for the axis calculation are somewhat larger than the bank calculation. It is safe to say that the bank calculations are a more accurate, even though the difference is not shattering.

It is best to use the Q-h relations for the bank locations in this context. However, if the Q-h relations differ slightly this does not lead to large differences. So if one has data from another source, this will give reasonably accurate results as well.

Table B-6: Average error for different hQ stations, macro stability

	Bank		Axis	
Section	Pf	Error	Pf	Error
43TG191TG202	3.33E-03	1.255	3.66E-03	1.142
43TG183TG191	2.14E-10	2.960	2.04E-10	3.107
43TG159TG171	4.78E-10	1.026	4.37E-10	1.066
43TG148TG159	7.34E-07	1.456	6.67E-07	1.601
43TG055TG068	7.08E-07	1.259	7.38E-07	1.312
		1.591		1.646

Table B-7: Average error for different hQ stations, piping

	Bank		Axis	
Section	Pf	Error	Pf	Error
43TG177TG183	8.92E-05	1.248	1.12E-04	1.571
43TG171TG177	1.67E-04	1.304	1.73E-04	1.354
43TG094TG106	2.27E-04	1.392	2.37E-04	1.454
43TG039TG055	2.23E-03	1.408	2.50E-03	1.582
		1.338		1.491

B.4 Method of deriving fragility curve

The calculation of the probability of failure of a reach in PC-Ring takes several steps. The calculation takes fluctuation in time and space of several stochastic variables into account. The calculation is first made for a time period (block) of one block duration (12,4h), for one cross section. Afterwards this is converted into the probability of failure for the time span that is regarded, for example one year, and to the length of the considered dike section. The calculations are made per failure mechanism and also per wind direction. And some mechanisms even have several partial mechanisms. All these separate calculations have to be combined into one probability of failure for the whole reach. This can be summarized in the following steps (Vrouwenvelder & Steenbergen, 2003):

1. Calculation of probability of failure per cross section, block, partial mechanism and wind direction;
2. Combine partial failure mechanisms;
3. Process probability of the wind direction;
4. Calculate Pf for whole dike section;
5. Sum over the wind directions;
6. Calculate probability of failure for total period;
7. Combine failure mechanisms;
8. Combine dike sections.

Step one is the probabilistic calculation, the other steps are merely combination of other results.

Step 6 is the most important step in deriving the fragility curve out of PC-Ring. The fragility curve should give the probability of failure per year per water level. However, if the fragility curve is derived in the way described in B.1.1 Derivation of the fragility curve, the probability of failure per year gives an overestimation.

PC-Ring calculates the probability of failure per year based on 352 tides. This is based on the winter half year. Within this half year there are six hydrographs, of which the peak discharge is determined by the Q-T relation. The course of the hydrographs is described with equidistant blocks (Borges-Castanheta model) of which the width is one or several block durations (note: this are different blocks than the blocks mentioned above!). The width of the blocks is determined per dike section. One hydrograph will have a shape comparable to Figure B-7. The middle (highest) column is the peak discharge. All the columns have a corresponding probability of failure. The combination of all these probabilities of failure (the product, since it is independent) gives the total combination for the regarded time span.

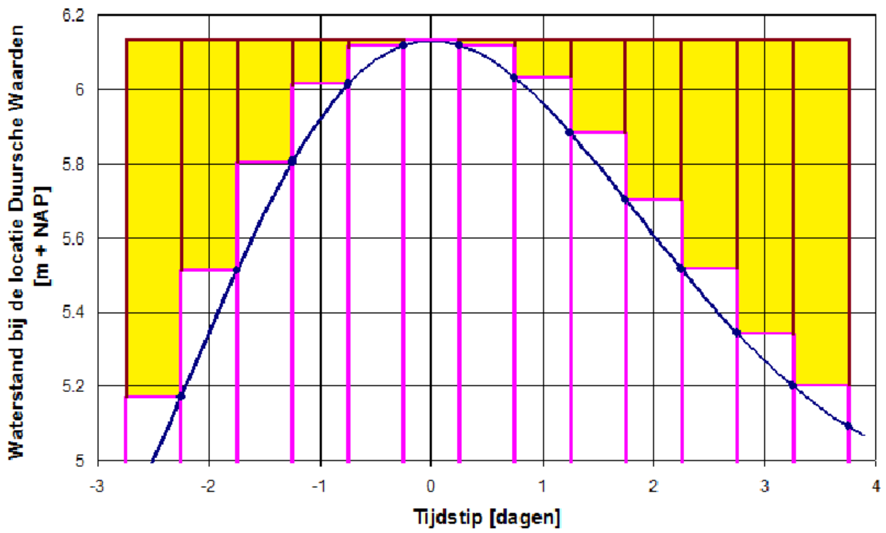


Figure B-7: Example hydrograph according to Borges-Castanheta model (Duits, et al., 1998)

Now if the fragility curve is derived as described above, the Q-T is taken out of the calculation. In other words: the discharge is always equal to the imposed discharge (or the discharge corresponding to the imposed water level). All the columns in Figure B-7 would have the same height and therefore the same probability of failure. The probabilities of failure for all the blocks (except the peak discharge) will thus be overestimated. If these probabilities per block are then combined into a yearly probability of failure this will lead to a large overestimation of the probability of failure. This overestimation is made for all the imposed water levels.

Overtopping

For overtopping especially this overestimation is very big. This can be seen from the shape of the fragility curve (Figure B-8). The fragility curve is very steep, since the probability of failure is 0 if the water level is significantly lower than the dike crest. If the water level is a little bit lower than the crest the probability of failure will be a little larger since wave overtopping will play a role. If the water level exceeds the crest height the probability of failure is 1, since the water will flow over the dike. The fragility curve rises from 0 to 1 in a very narrow band of water levels. If the block model is regarded this means the following. If the peak water level is equal to the crest height the probability of failure is approximately 1. The columns to the left and to the right have a lower water level so a lower probability of failure. Only several blocks more to the side of the hydrograph the probability of failure will be 0. So only the middle few blocks contribute to the probability of failure. If the peak water level is below the crest height the total probability of failure will approach 0 very quickly.

Now if the yearly probability of failure is taken the probability of failure for all the blocks is equal to the probability of failure of the highest block. Since there are only a few blocks that actually contribute to the probability of failure this leads to a major overestimation. This can be up to 15 times the actual probability of failure (Table B-8). Since there are very few blocks that contribute to the probability of failure per year it is a better estimation to take the probability of failure per block instead. Then only the probability of the peak discharge is taken into account, the contribution of lower water levels (and thus the contribution of wave overtopping) is then neglected. This causes a structural underestimation of 10% of the probability of failure (Table B-8). Note: if the water level is (much) higher than the crest height, much more blocks will contribute, but since the probability of failure of most of these blocks will be 1 this has no influence when they are neglected (multiplying with 1).

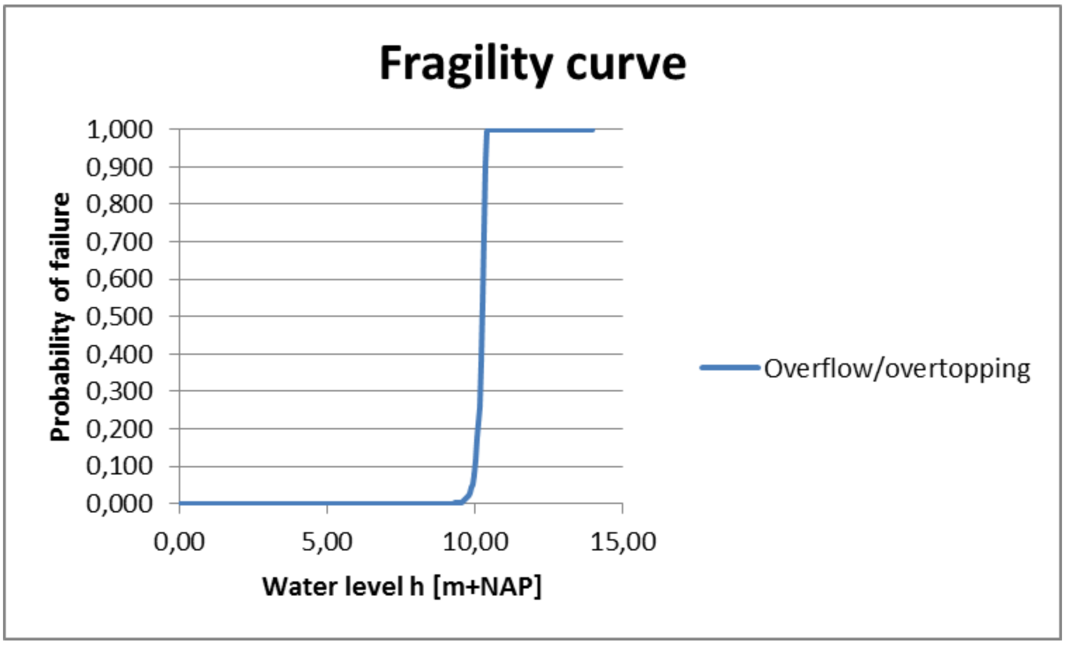


Figure B-8: Fragility curve overflow/overtopping

Piping

Piping on the other hand has a much gentler fragility curve (Figure B-9). The probability of failure increases gradually for higher water levels. This means that the adjacent columns in the block model have a contribution to the probability of failure that is only slightly different. In this case it will not be acceptable to take the peak discharge block and neglect the rest. In fact, it gives a better approximation to take the probability of failure per year, since all the blocks only give a small overestimation. In this case the overestimation is on average 30%, which is acceptable.

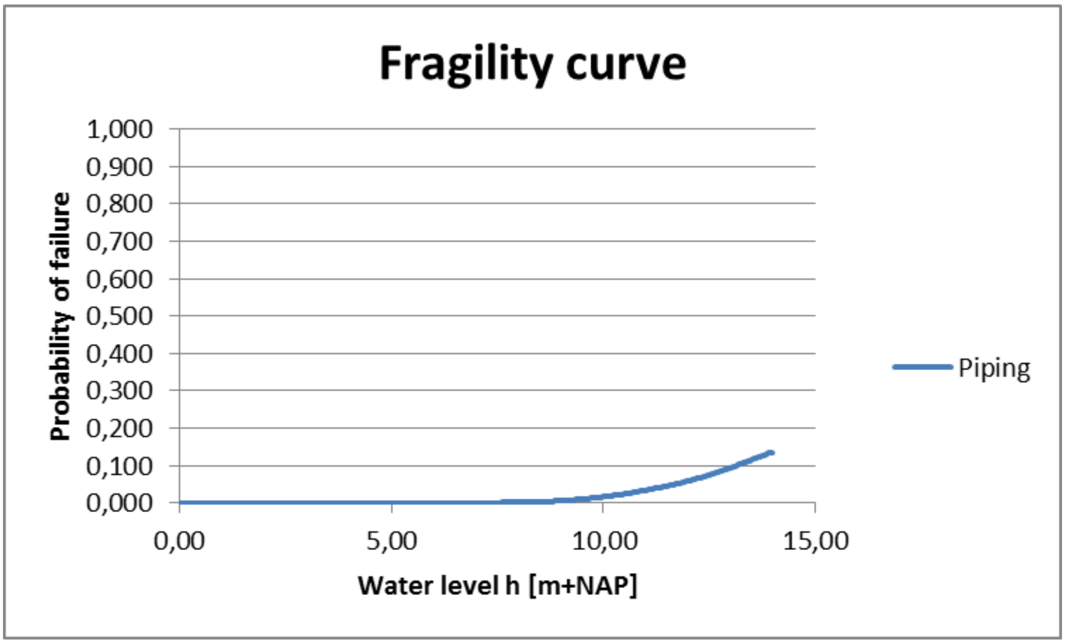


Figure B-9: Fragility curve piping

Table B-8: Effect of using different value for fragility curve, overtopping

	Pf_block duration		Pf_year	
Section	Pf	Error	Pf	Error
43TG191TG202	1.65E-04	1.070	0.00060306	3.415
43TG183TG191	6.89E-05	1.135	0.00038998	4.983
43TG177TG183	1.08E-04	1.089	0.00060599	5.157
43TG171TG177	3.07E-04	1.146	0.00212301	6.025
43TG159TG171	2.43E-04	1.084	0.0011067	4.194
43TG148TG159	1.91E-04	1.178	0.0006699	2.977
43TG144TG148	8.21E-05	1.221	0.0008569	8.544
43TG134TG144	1.87E-04	1.173	0.00132525	6.037
43TG121TG134	9.47E-05	1.099	0.00146567	14.08
43TG106TG121	1.25E-03	1.039	0.00366411	2.826
43TG094TG106	2.51E-04	1.030	0.00060262	2.327
43TG086TG094	2.00E-04	1.059	0.00042549	2.009
43TG076TG086	1.21E-04	1.057	0.00036291	2.830
43TG068TG076	1.17E-04	1.123	0.00022033	1.675
43TG055TG068	1.79E-04	1.019	0.00052318	2.864
43TG039TG055	8.90E-05	1.172	0.00011622	1.114
Average		**1.106**		**4.441**

Other mechanisms

For macro stability the probability of failure per 12 hours is approximately equal to the probability of failure per year. This failure mechanism is not time dependent. The 12 hour value gives slightly more accurate values. However, some of these results are overestimated and others are underestimated. This makes macro stability the most uncertain failure mechanism. Damage and erosion of the outer slope gives better results for the failure probability per year.

B.5 Total accuracy

Table B-9: Accuracy of the calculations per failure mechanism and per section

Error					
Section	Overflow/ overtopping	Macro stability	Piping	Damage and erosion outer slope	Section
43.TG191.TG202	1.07	1.14	-	-	1.14
43.TG183.TG191	1.14	(3.11)	-	-	1.14
43.TG177.TG183	1.09	-	1.25	-	1.04
43.TG171.TG177	1.15	-	1.30	-	1.01
43.TG159.TG171	1.08	1.07	-	-	1.08
43.TG148.TG159	1.18	1.60	-	-	1.18
43.TG144.TG148	1.22	-	-	-	1.22
43.TG134.TG144	1.17	-	-	-	1.17
43.TG121.TG134	1.10	-	-	-	1.10
43.TG106.TG121	1.04	-	-	-	1.04
43.TG094.TG106	1.03	-	1.39	-	1.13
43.TG086.TG094	1.06	-	-	-	1.06
43.TG076.TG086	1.06	-	-	-	1.06
43.TG068.TG076	1.12	-	-	-	1.12
43.TG055.TG068	1.02	1.31	-	(4.72)	1.18
43.TG039.TG055	1.17	-	1.41	-	1.37
Average	**1.11**	**1.28**	**1.34**	**1.00**	**1.13**

Table 3-1 shows the error for all the failure mechanisms and per dike section. The values between brackets have not been considered for the average error, since the probability of failure is very low (10^{-10}) so the error is almost completely determined by round-off errors and interpolation.

B.6 Conclusions

It is very well possible to calculate the probability of failure without using PC-Ring. Most of the sections assessed give an error of a factor 1 to 1.5. This is not 100% accurate but it is good enough to use these results for the assessment of effectiveness of measures or impact of climate change or other external influences. There are a lot of factors that influence the accuracy of the calculation, but they all have very limited effect.

The fit that is used for the Q-h relation is linear interpolation. This gives reasonable results and it gives no fits that are physically impossible. Also it is possible to do the calculations with a Q-h relation that is slightly different, for example with the axis data instead of the bank data.

The most influential parameter is the fragility curve. If this is not elaborated correctly it may lead to large errors. Caution is thus always required when a fragility curve is used to make calculations.

B.7 Recommendations

The fragility curves could be elaborated in more detail in order to get even better results for the reference calculation. In this case the probability of failure per block (tide) should be translated into the probability of failure per year, taking the Q-T relation and the width of the blocks PC-Ring calculates into account

This analysis shows that there are a lot of factors influencing the accuracy of the calculation. Therefore it is recommended that if this method is used to calculate the impact of changes on the probability of failure of the dike section, to always use another calculation tool to compare the results for the reference situation. If the results for the reference situation are in the same order of the results PC-Ring gives, this can be used as a good starting point for making some exploratory calculations. If the reference situation has an error of more than factor 2 a lot of caution is required when making calculations, for example for measures. In these cases the accuracy of the relative effectiveness (or impact) cannot be guaranteed.

C. Climate change

The calculation method can also be used to calculate the impact of climate change on the probability of failure. The parameters for the Q-T relation can be altered so that the discharge statistics represent one of the climate scenarios. The same can be done in PC-Ring. In principle this should give approximately the same result, especially when the PC-Ring and the alternate calculation show comparable results for the reference calculation (HR2006). With both methods the probability of failure of the reach has been calculated for the reference situation and for climate scenario W+2050. The parameters for both Q-T relations are given in Table C-1 and Table C-2.

Table C-1: Parameters for HR2006

	$0 < T \leq 2$	$2 < T \leq 25$	$25 < T \leq 10{,}000$
a	1620.7	1517.78	1316.43
b	5893.3	5964.63	6612.61

Table C-2: Parameters for W+2050

	$0 < T \leq 2$	$2 < T \leq 25$	$25 < T \leq 10{,}000$
a	1929.11	1805.86	1831.24
b	5893.34	5978.76	5897.07

Figure C-1 shows the Q-T relations for the reference situation (HR2006) and climate scenario W+2050. Since the purple line shows that high water levels occur more often the probability of failure will increase with this climate scenario. The physical maximum discharge is not taken into account.

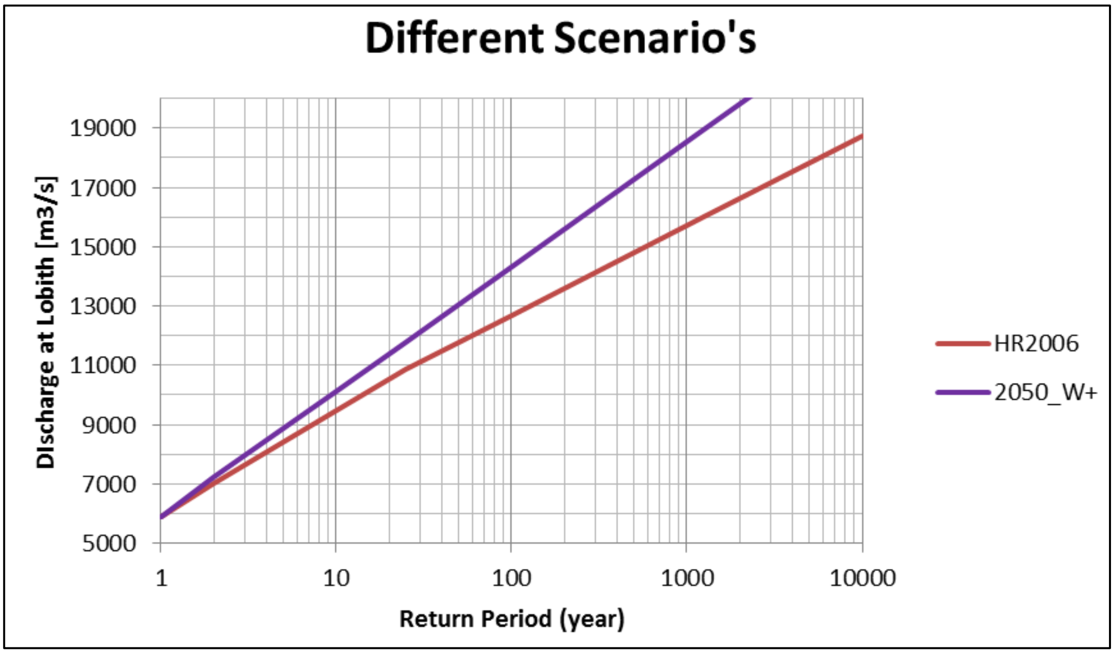


Figure C-1: Two different Q-T relations: HR2006 and W+2050

From the results two things can be derived. First of all the accuracy of the calculation for other climate scenarios, thus the error between the PC-Ring outcome and the new calculation outcome.

And secondly the difference in the impact. Since this study aims on predicting the impact of measures and climate change on the probability of failure, it is a logical choice to compare the impact factor of both calculation methods.

Table C-3 shows the error between the calculation with PC-Ring and the calculation with the new method per dike section and per failure mechanism. With both calculation methods calculations are made for HR2006 discharge statistics and W+2050 discharge statistics. What is interesting is that for almost all calculations the error between the two methods is smaller for W+2050. This suggests that the climate impact of the PC-Ring calculations is smaller than the climate impact calculated with the new method for mechanisms overtopping and macro stability, since the new methods underestimated the probability of failure for these mechanisms. Piping was initially overestimated so for this mechanism the PC-Ring calculations have more climate impact than the new calculation method.

Table C-3: Error for HR2006 and W+2050

Error	**Overflow overtopping**		**Macro stability**		**Piping**	
Section	Error HR2006	Error W+2050	Error HR2006	Error W+2050	Error HR2006	Error W+2050
43.TG191.TG202	1.07	1.02	1.14	1.09	-	-
43.TG183.TG191	1.14	1.01	3.11	1.16	-	-
43.TG177.TG183	1.09	1.01	-	-	1.25	1.19
43.TG171.TG177	1.15	1.03	-	-	1.30	1.24
43.TG159.TG171	1.08	1.01	1.07	1.06	-	-
43.TG148.TG159	1.18	1.06	1.60	1.25	-	-
43.TG144.TG148	1.22	1.05	-	-	-	-
43.TG134.TG144	1.17	1.02	-	-	-	-
43.TG121.TG134	1.10	1.05	-	-	-	-
43.TG106.TG121	1.04	1.00	-	-	-	-
43.TG094.TG106	1.03	1.03	-	-	1.39	1.26
43.TG086.TG094	1.06	1.01	-	-	-	-
43.TG076.TG086	1.06	1.02	-	-	-	-
43.TG068.TG076	1.12	1.02	-	-	-	-
43.TG055.TG068	1.02	1.04	1.31	1.36	-	-
43.TG039.TG055	1.17	1.07	-	-	1.41	1.42
Average	1.11	1.03	1.65	1.18	1.34	1.28

This can also be seen in Table C-4. Underestimation of the probability of failure in the reference situation leads to an overestimation of the climate impact. Also, overestimation of the probability of failure in the reference calculation gives a lower climate impact. Furthermore the table shows the error in the calculated impact. The error for the calculation with HR2006 and the error on the impact (relative difference between the impact calculated by the new method and the impact calculated with PC-Ring) have been plotted in Figure C-2 for failure mechanism overflow/overtopping. It shows that a small error in the reference calculation will lead to a small error in the calculated impact.

Table C-4: Comparison of calculation methods

Impact	Overtopping			Macro stability			Piping		
Section	PC-Ring	New method	Error	PC-Ring	New method	Error	PC-Ring	New method	Error
43.TG191.TG202	7.03	7.67	1.09	2.12	2.23	1.05	-	-	-
43.TG183.TG191	8.55	9.61	1.12	2.89	7.76	2.68	-	-	-
43.TG177.TG183	8.02	8.64	1.08	-	-	-	2.24	2.14	0.96
43.TG171.TG177	5.65	6.28	1.11	-	-	-	2.62	2.49	0.95
43.TG159.TG171	6.28	6.89	1.10	1.72	1.72	1.00	-	-	-
43.TG148.TG159	6.70	7.43	1.11	1.17	1.50	1.28	-	-	-
43.TG144.TG148	7.58	8.85	1.17	-	-	-	-	-	-
43.TG134.TG144	6.31	7.22	1.15	-	-	-	-	-	-
43.TG121.TG134	7.35	8.52	1.16	-	-	-	-	-	-
43.TG106.TG121	4.23	4.38	1.03	-	-	-	-	-	-
43.TG094.TG106	6.55	6.92	1.06	-	-	-	2.57	2.34	0.91
43.TG086.TG094	6.91	7.38	1.07	-	-	-	-	-	-
43.TG076.TG086	7.84	8.47	1.08	-	-	-	-	-	-
43.TG068.TG076	7.83	8.59	1.10	-	-	-	-	-	-
43.TG055.TG068	7.13	7.57	1.06	1.04	1.08	1.03	-	-	-
43.TG039.TG055	8.49	9.27	1.09	-	-	-	1.78	1.80	1.01

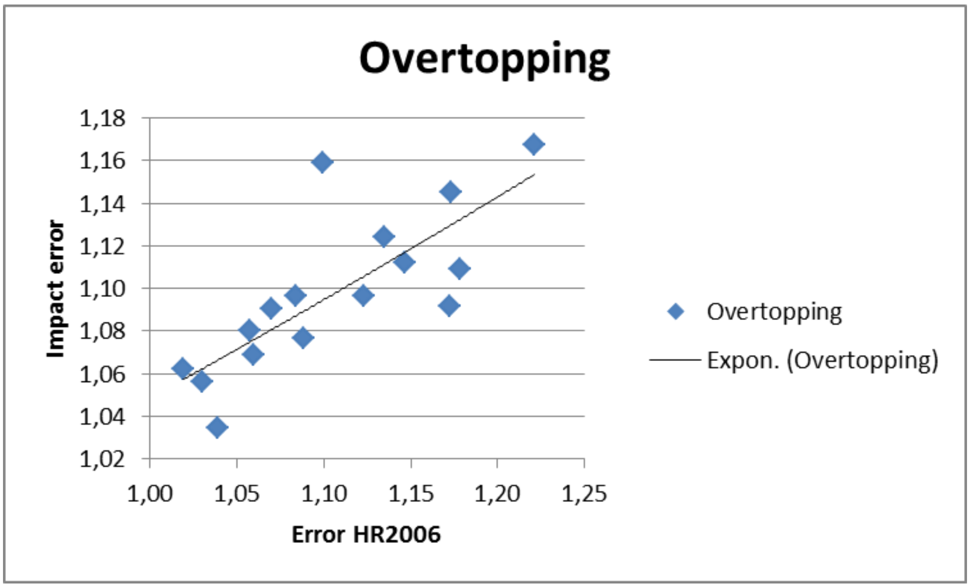


Figure C-2: Impact error vs error in the reference calculation

C.1 Per reach

The impact of climate change on the probability of failure is also calculated. The results are shown in Table C-5. The impact of climate change on the reach is overestimated. The probability of failure of the reference calculation was underestimated.

Table C-5: Impact of climate change on reach

	Probability of failure	Impact
PC-Ring HR2006	1/156	
New method HR2006	1/130	
PC-Ring W+2050	1/75	2.07
New method W+2050	1/53	2.45

D. Analysis Blokkendoos Rivieren

The ‘blokkendoos rivieren’ (Deltaprogramma, 2014) (planning kit) is a web-based application which contains the estimated effects of several measures which are planned to be taken in the Dutch river area. It is developed to be able to compose a set of measures or to form a strategy on how to organize the river area. It contains almost 400 river measures of different categories, all pre-calculated and elaborated to a level on which it can be used for assessments. The blokkendoos simply distracts the calculated water level lowering of a selected measure from the design task imposed by climate change. The blokkendoos assumes that water level lowering due to multiple measures can be added up.

The projects are selected and (re-)elaborated by the Delta Programme Rivers and originating from different studies. The reference situation is 2015: after completion of RfR and HWBP-2. For the determination of the climate change induces design task one can look at the years 2050 and 2100 and there are different Delta scenarios available (Deltares, 2011).

This analysis will look at the effect of some of the measures which are implemented for the Waal River. The categories of measures that will be looked at are:

1. Floodplain measures
2. Lowering of groynes
3. Dike improvements

For these categories several elaborated measures will be analysed. The assessed scenario will be W+2050, which is equal to delta scenarios Steam and Warm. For all the measures the effect on the water level is assessed, in order to gain insight in the difference in effects of measures. Also a brief look is taken into the costs of the project, in order to be able to also assess the cost efficiency of different measures in a later stage. For this analysis mainly measures in the reach between Tiel and Gorinchem will be assessed (43-6) since this is the area of interest of the whole research. The effect a measure has on the mean high water level will be used to assess the effect it will have on lower water levels. The design task presented by the blokkendoos is based on a discharge of 16,000 m3/s for 2015, 17,000 m3/s for 2050 and 18,000 m3/s for 2100.

For all the measures the following will be assessed. First of all the size and location of the project. This can be expressed in km of river stretch over which the project expands. The reach of the project tells something about over how many kilometres the measure affects the water level. The maximum reduction is necessary to gain insight in the effectiveness of the measure at high discharges. Also the magnitude of the project, expressed in monetary investments and ground movement.

D.1 Floodplain measures

Floodplain measures increase the flood conveyance capacity of the floodplains. Therefore these measures will not be effective during low discharges. For the Waal River the floodplains start flowing along around 5000 m3/s. Measures in the floodplains can include a secondary channel or green river, lowering of floodplains by means of excavation, removing of hydraulic obstacles like high water free areas or repositioning of dikes. The measure will be most effective at high discharges, but it will already play a large role just after the floodplains will start flowing along. Removal of hydraulic obstacles will not be assessed since this has very small effect compared to the other floodplain measures.

D.1.1 Side channel

Secondary channels, high water channels and green rivers are used when high discharges occur. This divides the flow over two river branches, thereby reducing the discharge in the primary branch. When this secondary channel lies within the winter bed it can be seen as an increase of the flood conveyance capacity of the floodplains. Side channels generally lay within the floodplains, where green rivers and high water channels are often branches outside the normal river track. This analysis will focus on measures in the floodplains only.

Side channel "Sleeuwijk"

Near Sleeuwijk a side channel is planned. This measure is schematized and calculated on behalf of the Delta Programme Rivers (DPR). The calculations are calculated with the hydraulic model of DPR. (Kroekenstoel, 2014)

Table D-1: Project details "Sleeuwijk"

Floodplain measure	Side channel Sleeuwijk
Project size	4 km
Project location	Km 953-957
Project reach	45 km
Maximum MHW reduction	0.23 m
Investment costs	232.7 M€

Figure D-1 and Figure D-2 show the reduction of the design task that is accomplished by measure "Sleeuwijk". The measure is effective until about 40 km upstream from the measure. It can be seen that at the downstream side of the measure a rise in water level is caused. This is because of backwater since the side channel flows back into the river and thus a reduction of flow profile occurs, causing the water level to rise. At the upstream end of the side channel the opposite happens; the flow profile expands, causing a reduction in water level. This reduction has effect up to 40 km upstream. The sudden increase of the water level at 914 km is caused because this is the model boundary.

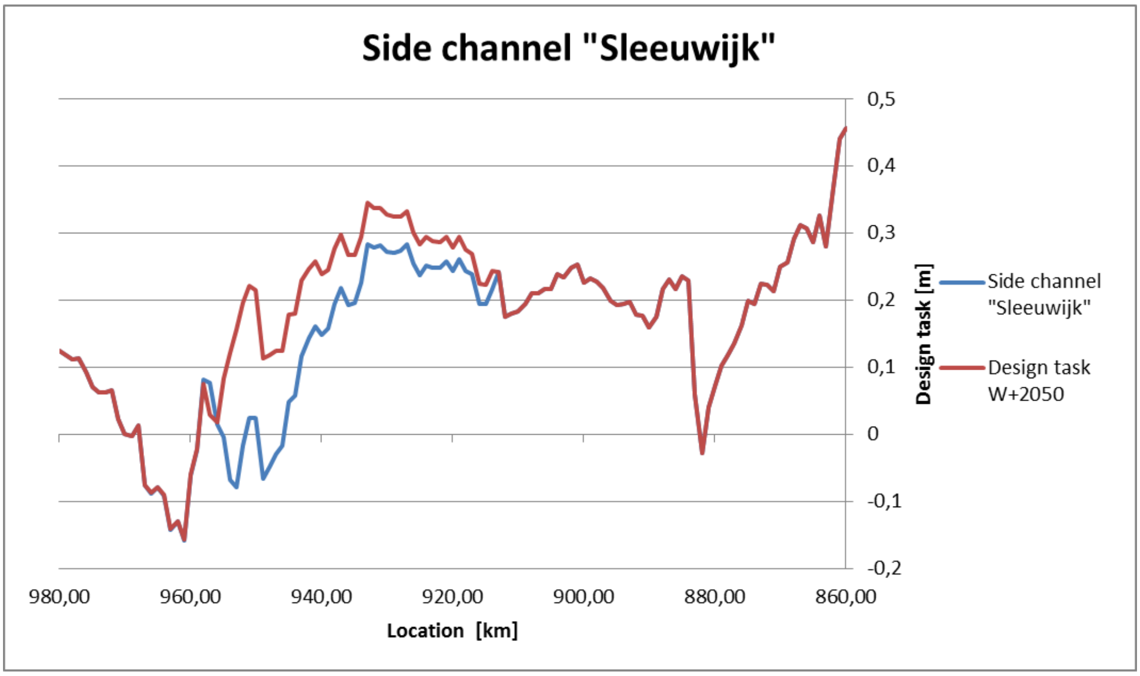


Figure D-1: Sleeuwijk, reduction of design task

Figure D-2: Sleeuwijk, water level effect

Side Channel "Kop van Heerewaarden Rood"

This side channel lies within the floodplains of the Waal River. Since it is only a small project, the water level lowering effect is only small as well. It also shows an increase in the water level at the downstream side and a kink at the upstream side. This upstream kink is again caused by the model boundary. The maximum water level lowering is 0.04 m and the measure is effective over 38 km. This measure is elaborated for the WaalWeelde project, and it has been calculated with a recent hydraulic model.

Table D-2: Project details "Kop van Heerewaarden Rood"

Floodplain measure	Side channel "Kop van Heerewaarden Rood"
Project size	<1 km
Project location	Km 924
Project reach	38 km
Maximum MHW reduction	0.04 m
Investment costs	15.2 M€

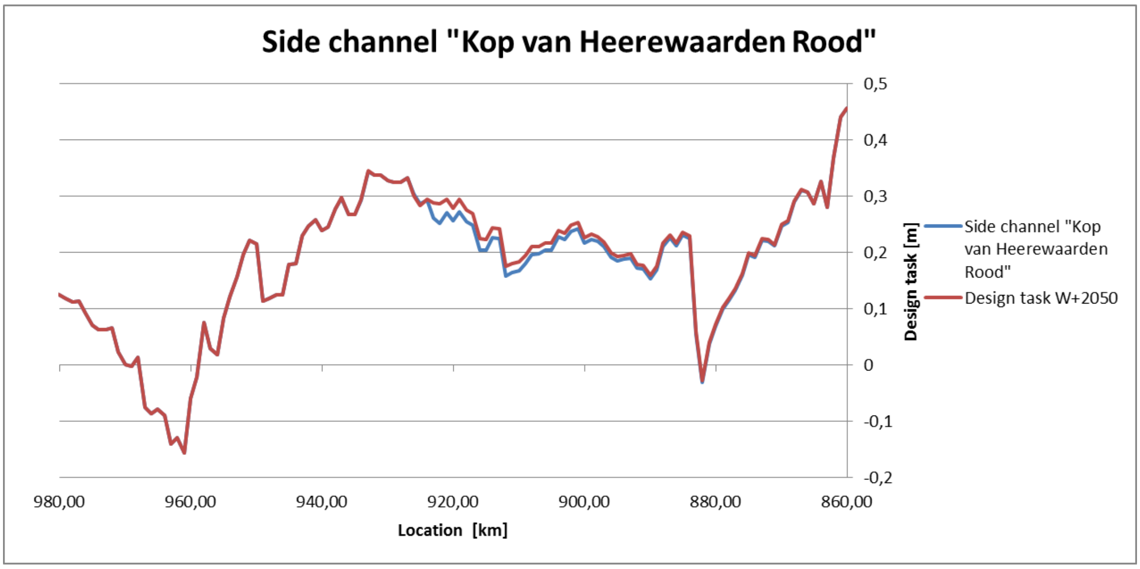


Figure D-3: Kop van Heerewaarden, reduction of design task

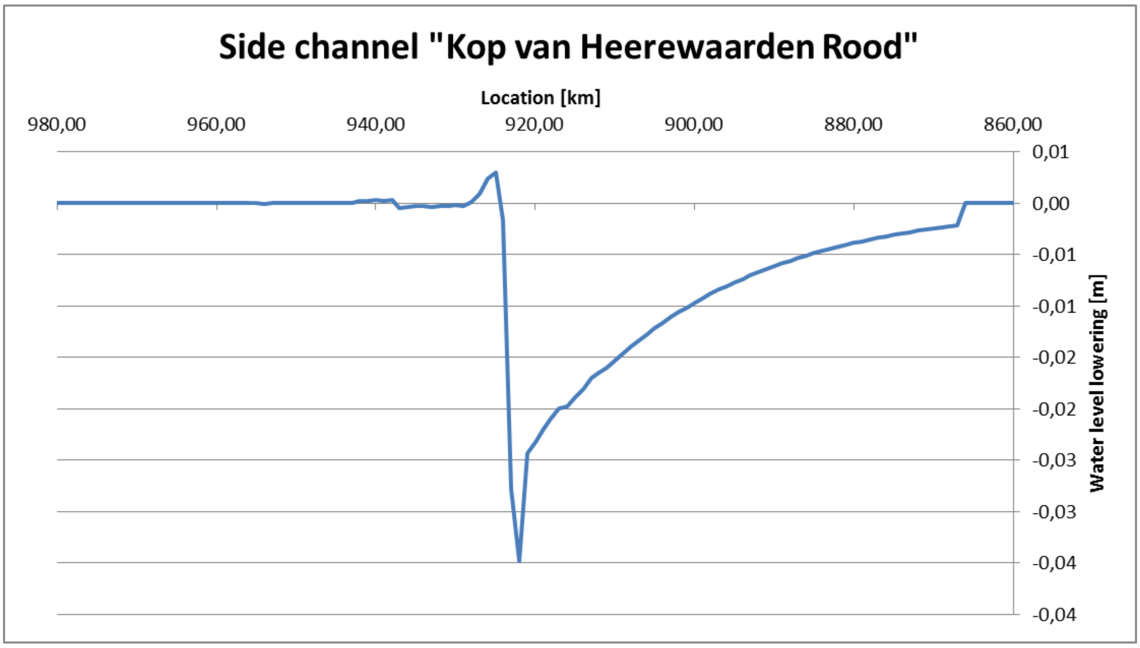


Figure D-4: Kop van Heerewaarden, water level effect

Side channel "Kerkenwaard-Tuil"

This is also a WaalWeelde project. It contains a side channel which is created in the floodplains.

Table D-3: Project details "Kerkenwaard-Tuil"

Floodplain measure	Side channel "Kerkenwaard-Tuil"
Project size	3 km
Project location	Km 934-937
Project reach	68 km
Maximum MHW reduction	0.08 m
Investment costs	24.8 M€

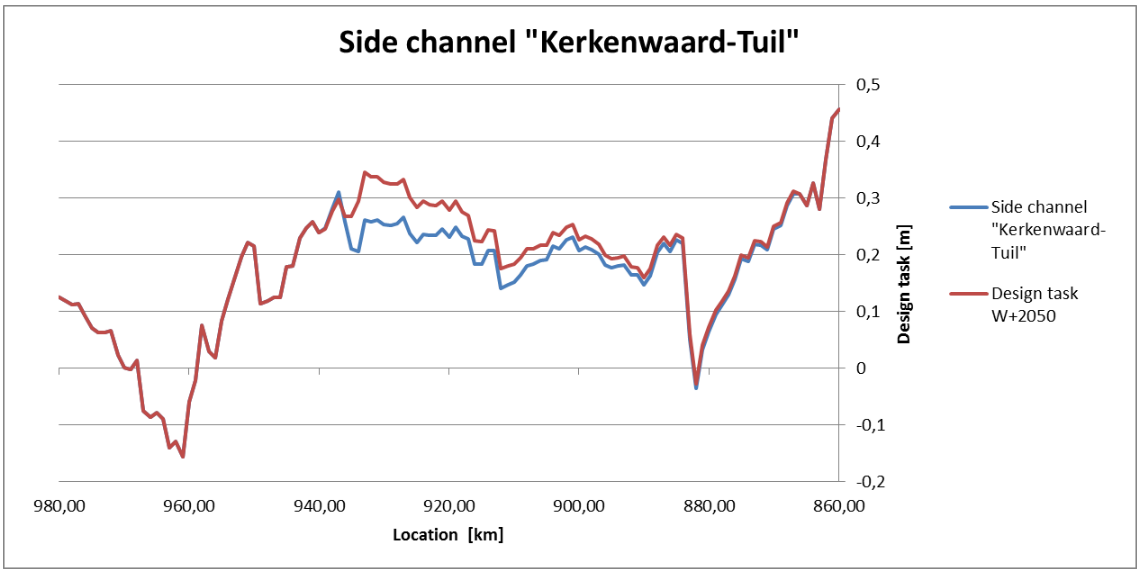


Figure D-5: Kerkenwaard-Tuil, reduction of design task

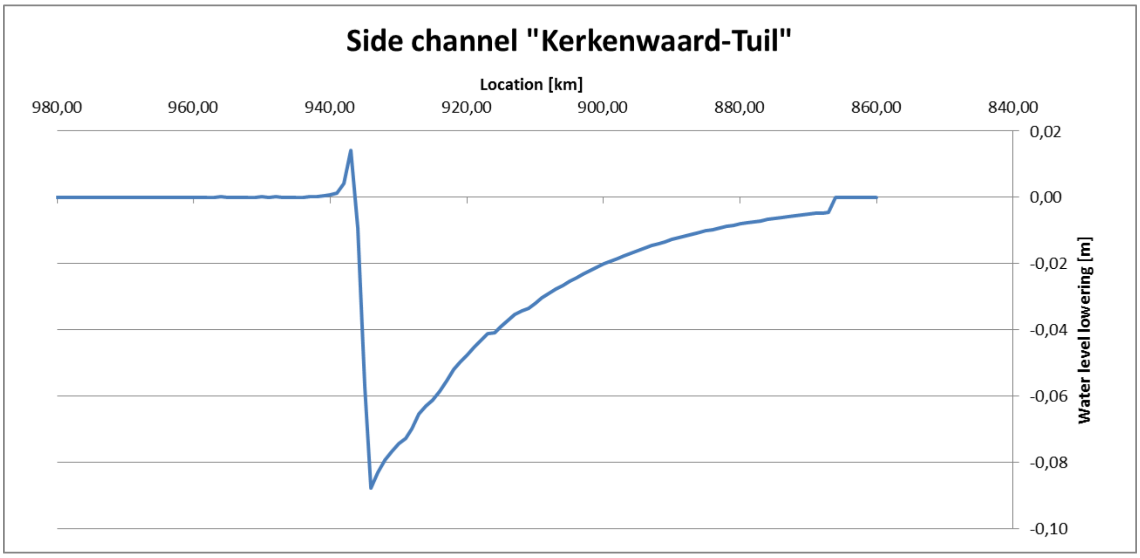


Figure D-6: Kerkenwaard-Tuil, water level effect

D.1.2 Excavation

Excavation of the floodplains directly increases the flood conveyance capacity of the floodplains, causing a lower water level at the upstream side of the measure. This measure will be effective from the moment the floodplains start flowing along. The water level lowering effect will be relatively more for moderate discharges since the increase of the perimeter is then relatively larger than for the MHW discharge.

Excavation "Willemspolder natuur"

This is a pretty large excavation. The project is 5 km long and the effect reaches over 50 km. The measure is schematized by program office RfR, commissioned by DPR. The measure is calculated with the hydraulic model of DPR. The measure again shows an increase in the water level at the downstream side of the measure. Also downstream from the measure some water level reduction is seen. This might be caused by the hydraulic roughness but this is not sure.

Table D-4: Project details "Willemspolder natuur"

Floodplain measure	Excavation "Willemspolder natuur"
Project size	5 km
Project location	Km 908-913
Project reach	>50 km
Maximum MHW reduction	0.04 m
Investment costs	117.3 M€

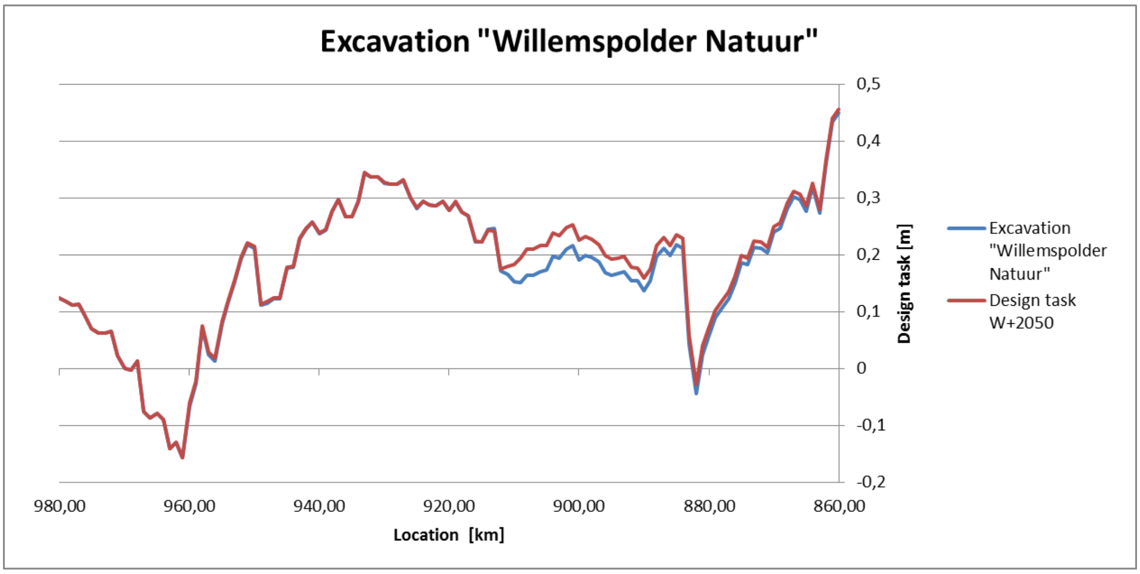


Figure D-7: Willemspolder natuur, reduction of design task

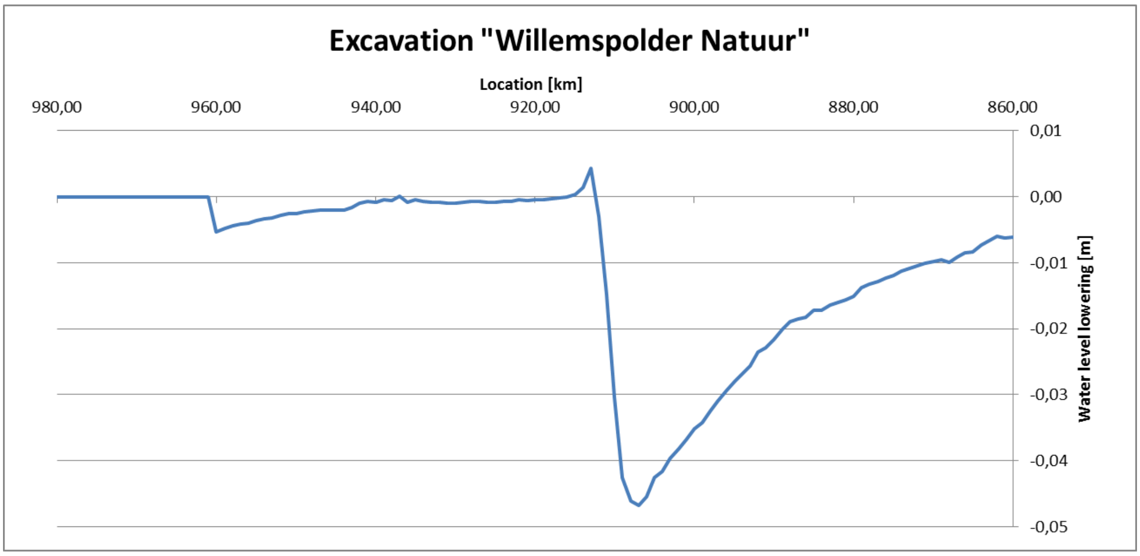


Figure D-8: Willemspolder natuur, water level effect

Floodplain project "Stiftsche uiterwaarden"

In the floodplains a gully is excavated, increasing the flood conveyance capacity of the floodplains. This gully may be connected to the river but that is not taken into account in this calculation. Also summer dikes and other smaller dikes are removed in order to improve the flow through the floodplains. The elaboration is done for WaalWeelde, and thus the calculations are done with a recent hydraulic model.

Table D-5: Project details "Stiftsche uiterwaarden"

Floodplain measure	Floodplain project "Stiftsche uiterwaarden"
Project size	2 km
Project location	Km 919-920
Project reach	56 km
Maximum MHW reduction	0.09 m
Investment costs	122.1 M€

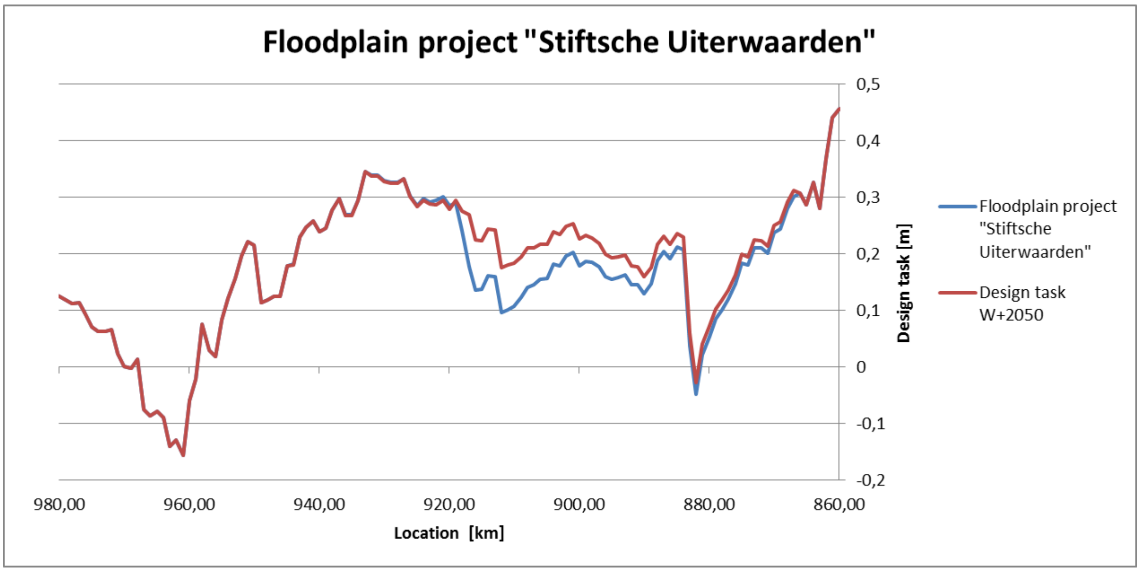


Figure D-9: Stiftsche uiterwaarden, reduction of design task

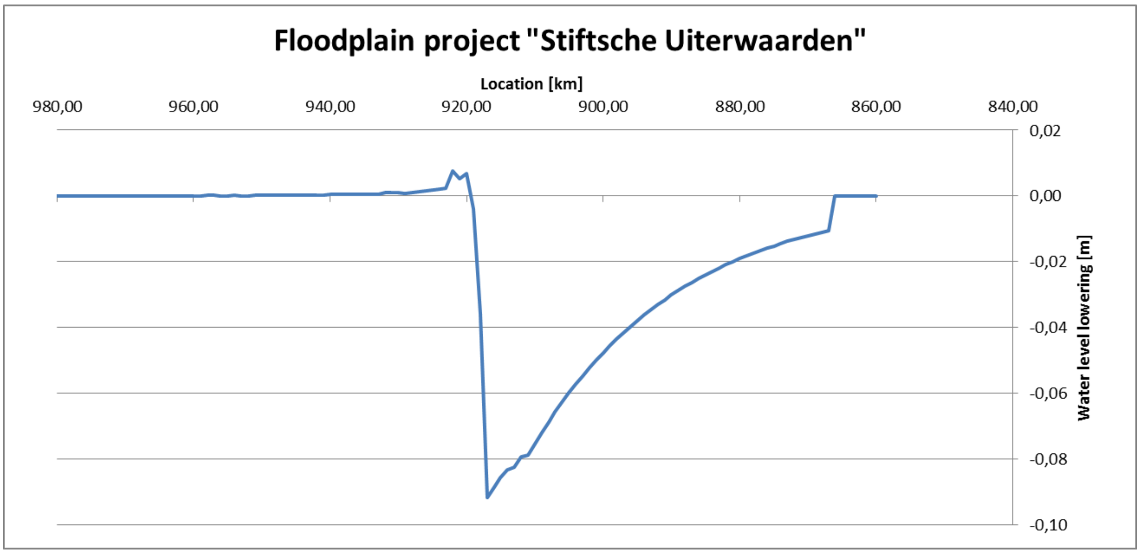


Figure D-10: Stiftsche uiterwaarden, water level effect

Floodplain Project "Kop van Heerewaarden groen"

This project is elaborated for the WaalWeelde programme. In the floodplains gullies are excavated, while maintaining a high water free area.

Table D-6: Project details "Kop van Heerewaarden Groen"

Floodplain measure	Floodplain project "Kop van Heerewaarden groen"
Project size	2 km
Project location	Km 922-924
Project reach	59 km
Maximum MHW reduction	0.06 m
Investment costs	45.8 M€

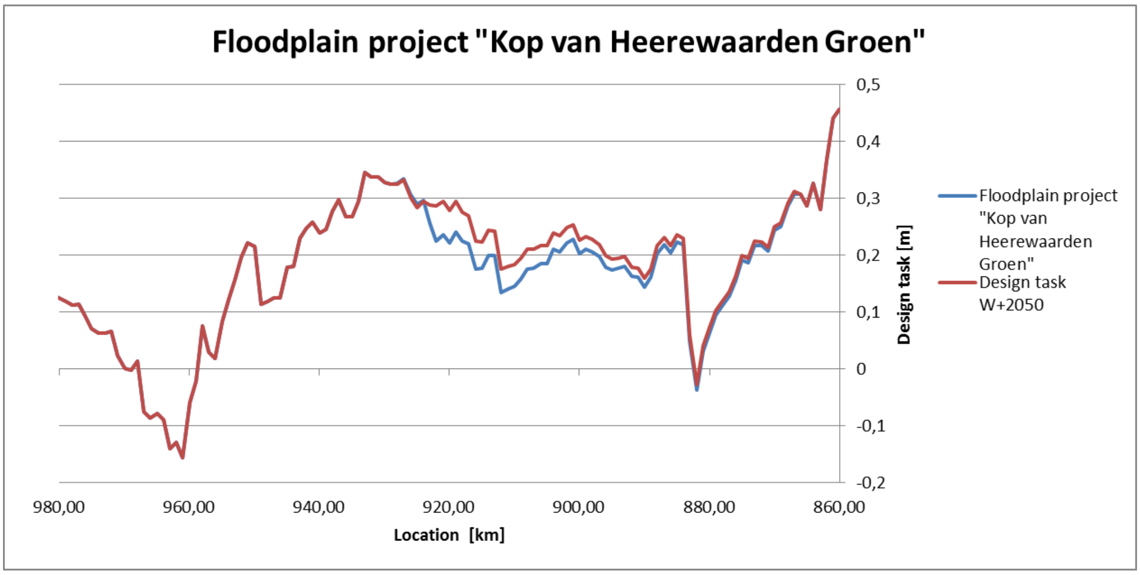


Figure D-11: Kop van Heerewaarden, reduction of design task

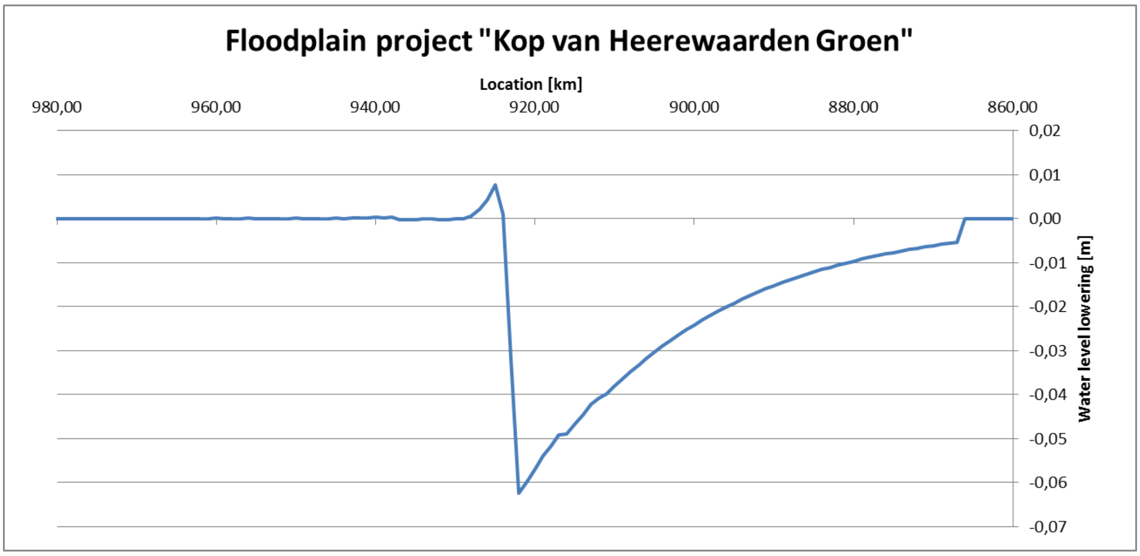


Figure D-12: Kop van Heerewaarden, Water level effect

D.1.3 Dike repositioning

Dike repositioning means that the winter dikes are being placed backward, so that the floodplains become wider. This increases the capacity of the floodplain and it will lead to lower water levels starting from the discharge where the floodplains start flowing. The effect will be approximately equal for all discharges. Dike repositioning will in all cases be combined with dike improvement, since the new dike will be constructed stronger than the old dike.

Dike repositionings Varik Heesselt

This contains three separate dike repositioning measures, being dike repositioning at Varik (PKB), dike repositioning at Heesselt (PKB) and another dike repositioning at Heesselt (WaalWeelde). Water level effects of PKB projects are obtained from the old blokkendoos PKB. These measures have been calculated some years ago with another reference situation and another flow model. These numbers are thus less reliable.

Table D-7: Project details "Varik-Heesselt"

Floodplain measure	Dike repositionings Varik-Heesselt
Project size	3 km
Project location	Km 922-925
Project reach	59 km
Maximum MHW reduction	0.08 m
Investment costs	54.5 M€

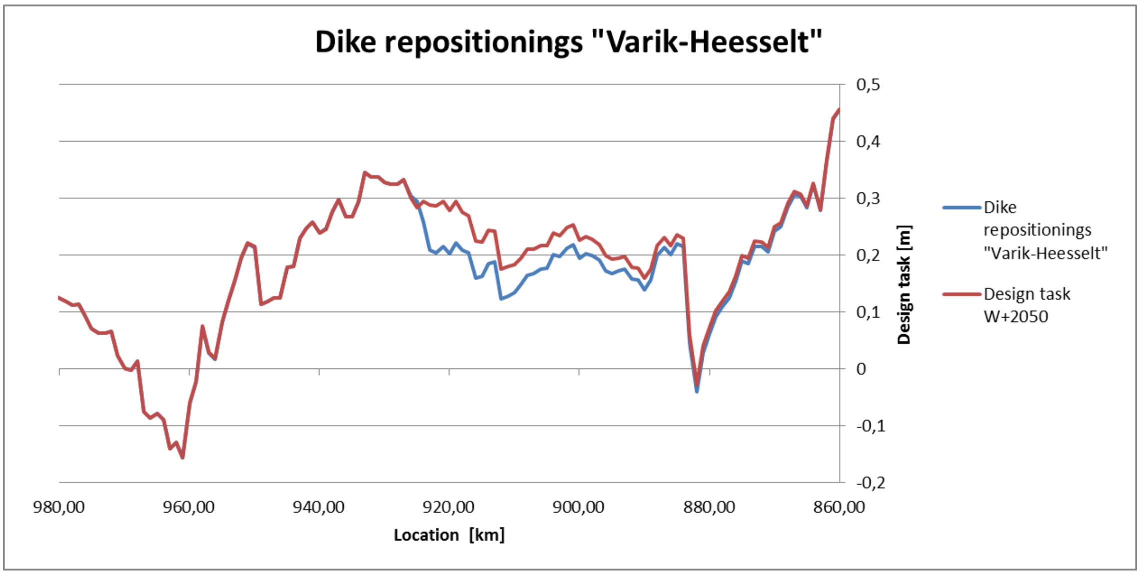


Figure D-13: Varik-Heesselt, reduction of design task

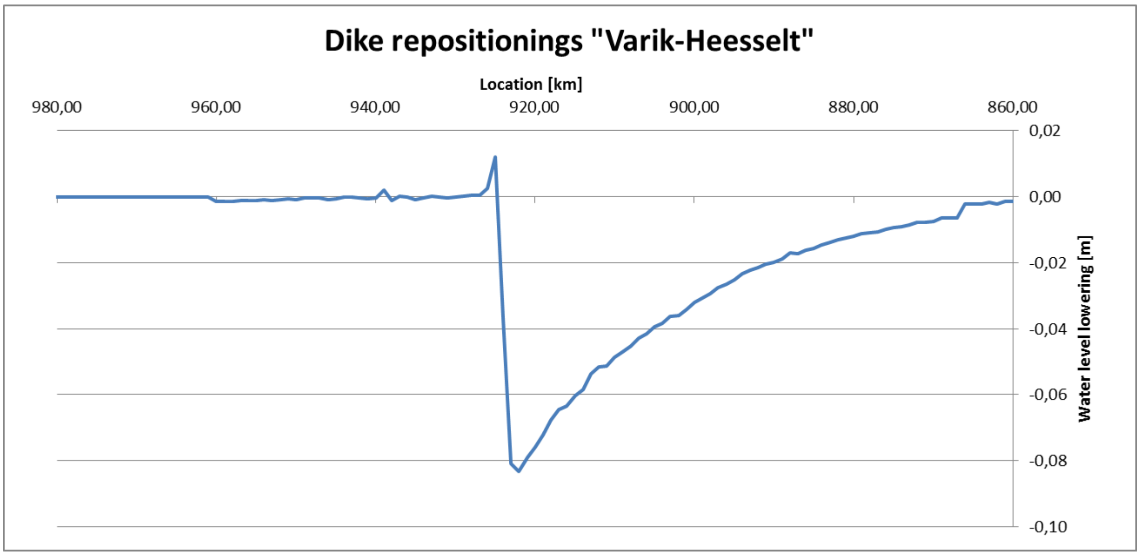


Figure D-14: Varik-Heesselt, water level effect

Dike repositioning "Hurwenensche Uiterwaarden"

This is part of a larger project, containing also a side channel and floodplain excavation. It is part of the WaalWeelde project.

Table D-8: Project details "Hurwenensche uiterwaarden"

Floodplain measure	Dike repositioning "Hurwenensche uiterwaarden"
Project size	1 km
Project location	Km 932-933
Project reach	54 km
Maximum MHW reduction	0.04 m
Investment costs	22.9 M€

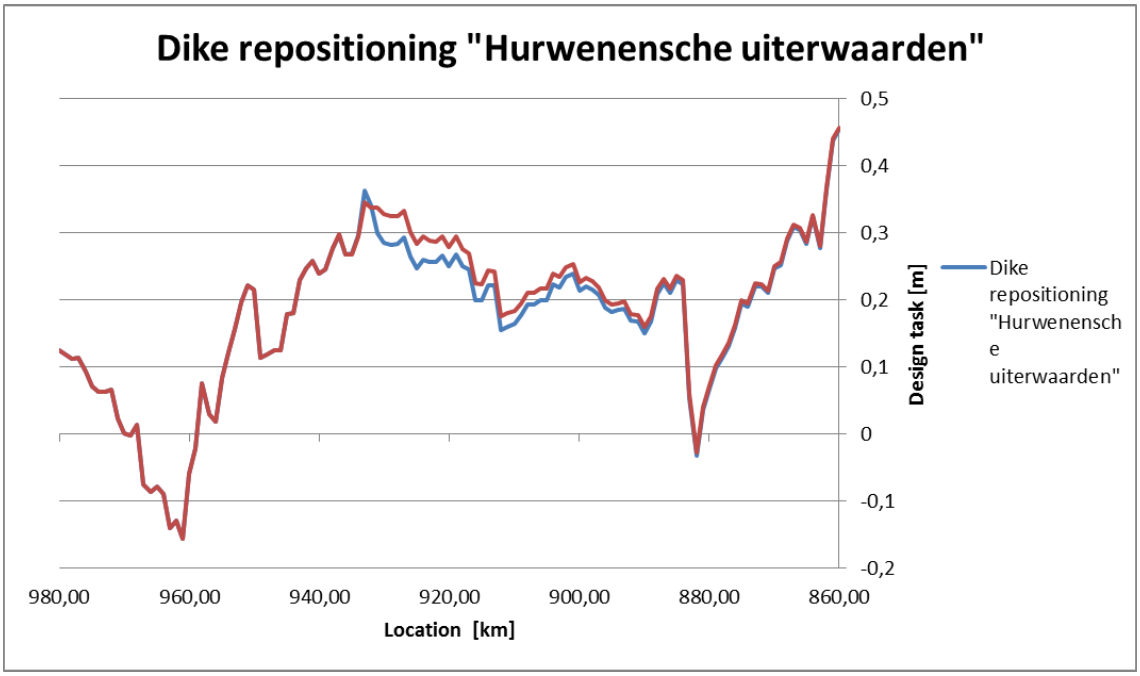


Figure D-15: Hurwenensche uiterwaarden, reduction of design task

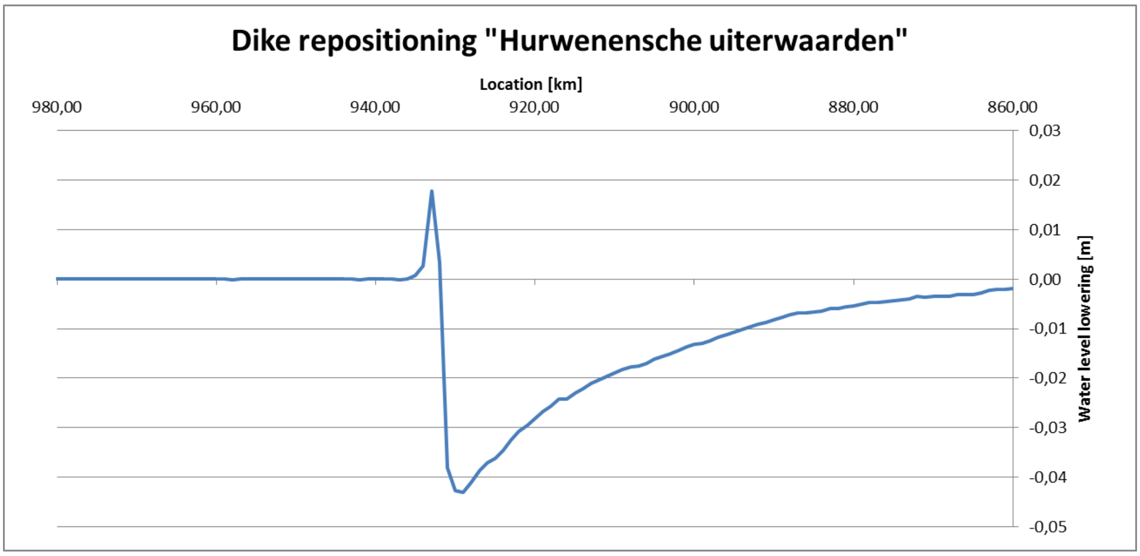


Figure D-16: Hurwenensche uiterwaarden, water level effect

Dike repositioning "Werkendam Noord"
This dike repositioning is part of the Delta Programme Rivers.

Table D-9: Project details "Werkendam Noord"

Floodplain measure	Dike replacements "Hurwenensche uiterwaarden"
Project size	3 km
Project location	Km 957-960
Project reach	47 km
Maximum MHW reduction	0.10 m
Investment costs	176.7 M€

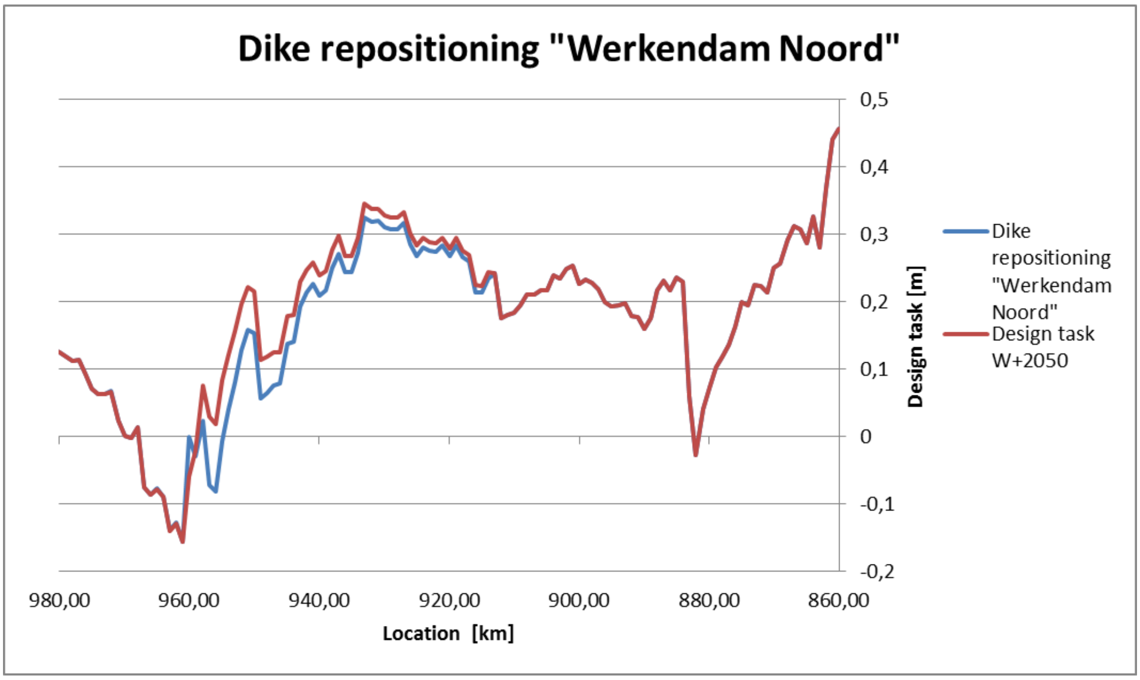


Figure D-17: Werkendam Noord, reduction of design task

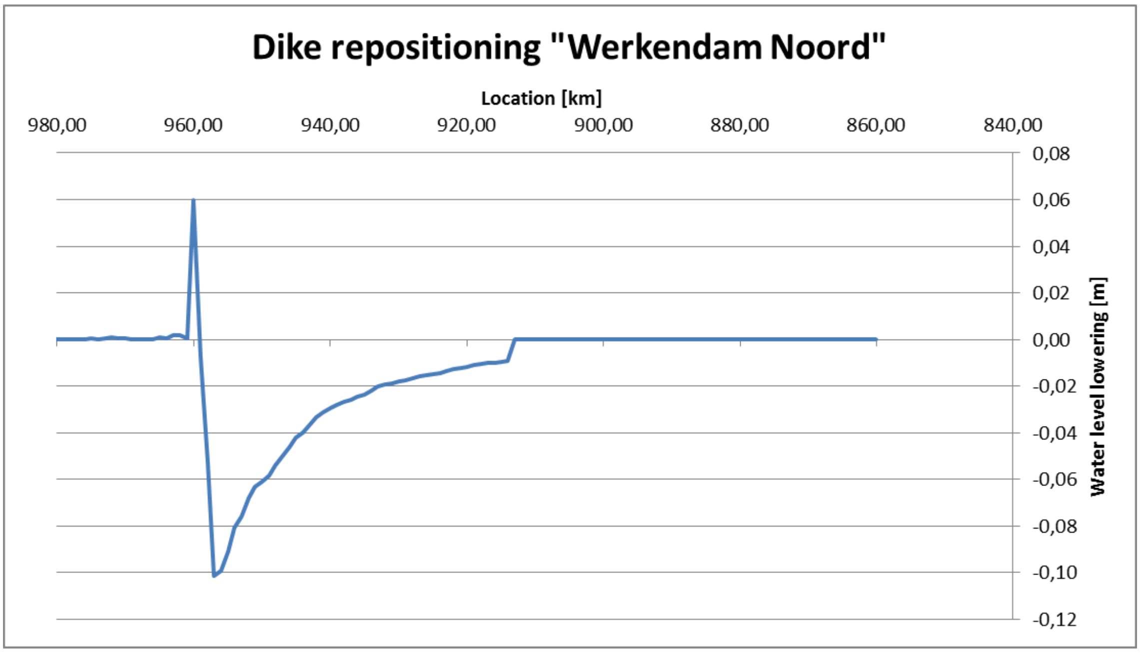


Figure D-18: Werkendam Noord, water level effect

D.1.4 Assessment of relations in floodplain measures

Increase of the floodplain capacity can be a very effective measure to bring down the water level for high discharges. Depending on the size of the measure the effect may vary from several cm up to more than 20 cm. Larger projects like dike repositioning will have an effect of around 8-10 cm. Excavations will have less effect, this is around 6 cm or up to 9 cm for a very large project. Side channels can be very effective, up to 23 cm for a very large project. But also small project can have an effect of around 10 cm easily.

For all projects assessed above the water level effect graph looks somewhat the same. The water level slightly increases at the most downstream end of the location. This is due to the sudden decrease in flood conveyance capacity of the river. This rise can vary from 1 cm up to 6 cm. The water level reducing effect increases approximately linear with the length of the measure. The backwater curve then shows an exponential development starting from the most upstream location of the measure. The reach of a measure is always in the order of 50 km, a little less or more depending on whether the river has bifurcations in it. Off course a more effective measure will have a longer reach in case no bifurcations are present.

To see whether there is any relation between the size of the project, the reach of the project, the maximum water level reduction and the water level rise on the downstream side of the measure, these values will be plotted in a number of graphs. Also the relation between investment and effectiveness will be assessed, but this will be done in the last chapter of this analysis. Even though there is only a small number of measures to be assessed, it may be possible to find some relations between dimension and effect.

Project length vs water level effect
First of all the length of the measure in relation to the water level effect is shown in Figure D-19. The figure makes a distinction between the three types of floodplain measures. This is done because different types of measures can have different effects, even though the length of the project is equal.

The figure shows that there is no evident relation between the length of a project and its water level lowering effect. For excavation measures the longest project is even the least effective. The results of these figures are of course flattered, because the width and depth of a measure are not taken into account. A short measure can be much wider and/or deeper than a long measure and therefore be much more effective than the longer ones. This is probably the case with the excavation measures. Only for the side channels it seems that a longer measure will also be more effective. Dike repositioning shows that two measures of equal length can have different effect.

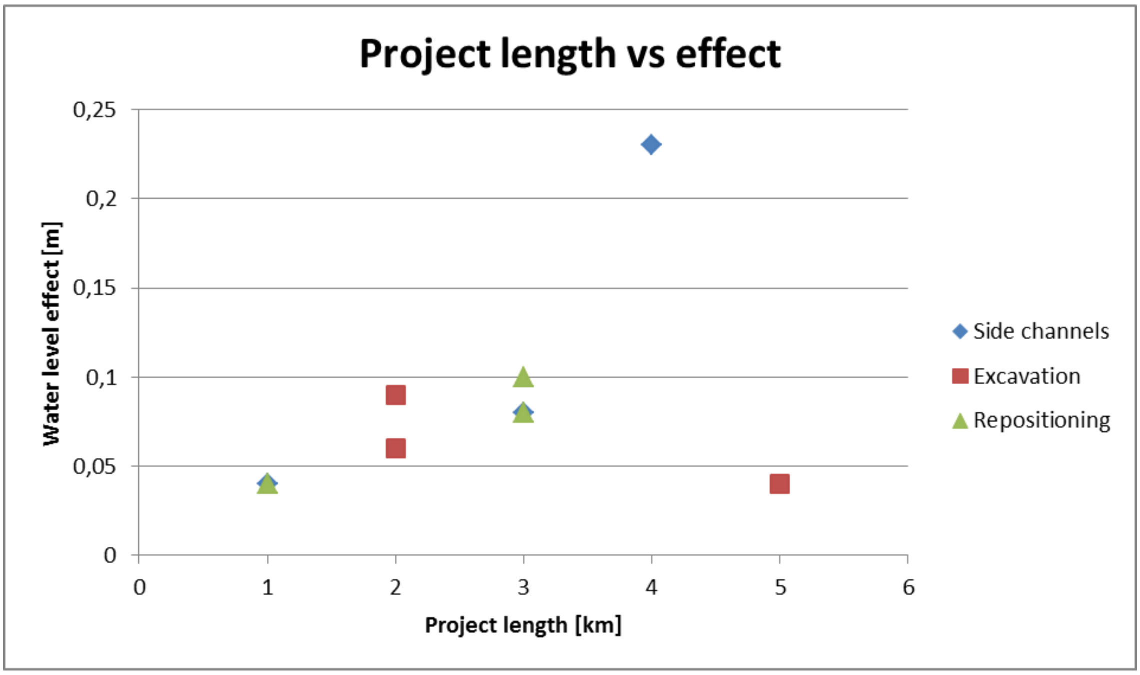


Figure D-19: Relation project length and water level effect

Project length vs reach
Figure D-20 shows the relation between the length of the project and the reach of the project. Also here it seems that there is no apparent relation between the dimension and the effect of the measure. This can partly be explained by upstream bifurcations, which reduce the distance over which the measure is effective. Another reason is again the lack of other dimensions of the measure.

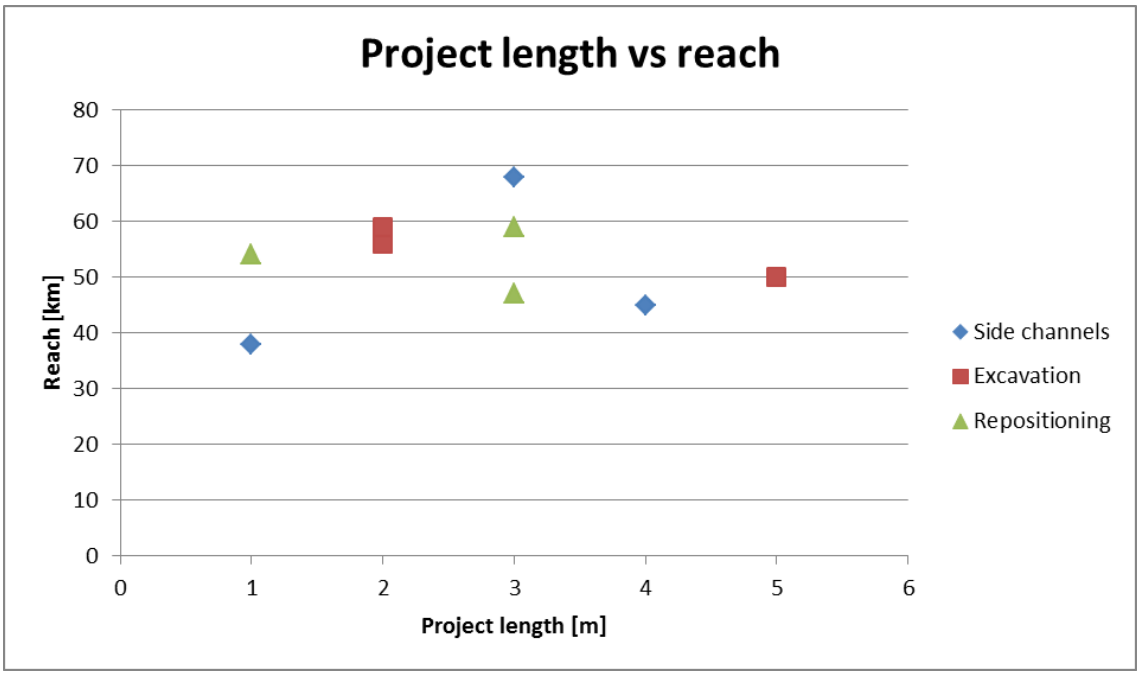


Figure D-20: Relation Project length vs reach

Other relations

Figure D-21 and Figure D-22 show the relation between water level effect and downstream rise and reach respectively. These figures show that there are no apparent relations between these properties whatsoever. This is actually the case for all relations between properties. The only properties that seem to have some sort of relation are the project length and the water level effect of side channels, as has been treated above.

The main reason that there are no apparent relations is the lack of information about the dimensions of the measures. The only available dimension is the length of the project, and this is even a round-off value. If more information would be available, for example the width and depth of a measure, the amount of ground movement or the capacity (in m^3) of a measure, it would be possible to discover some relation between the dimensions and the effectiveness. Also it would be a good idea to assess more measures than has been done now. The relation between cost and effect will be assessed below.

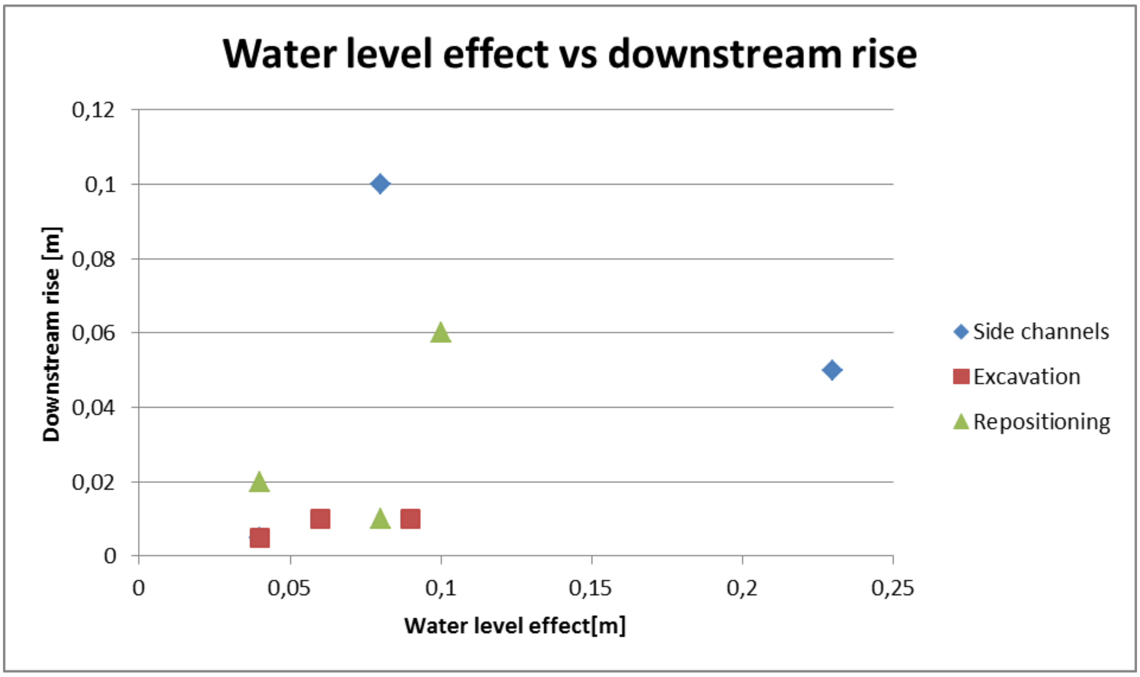


Figure D-21: Relation water level effect and downstream rise

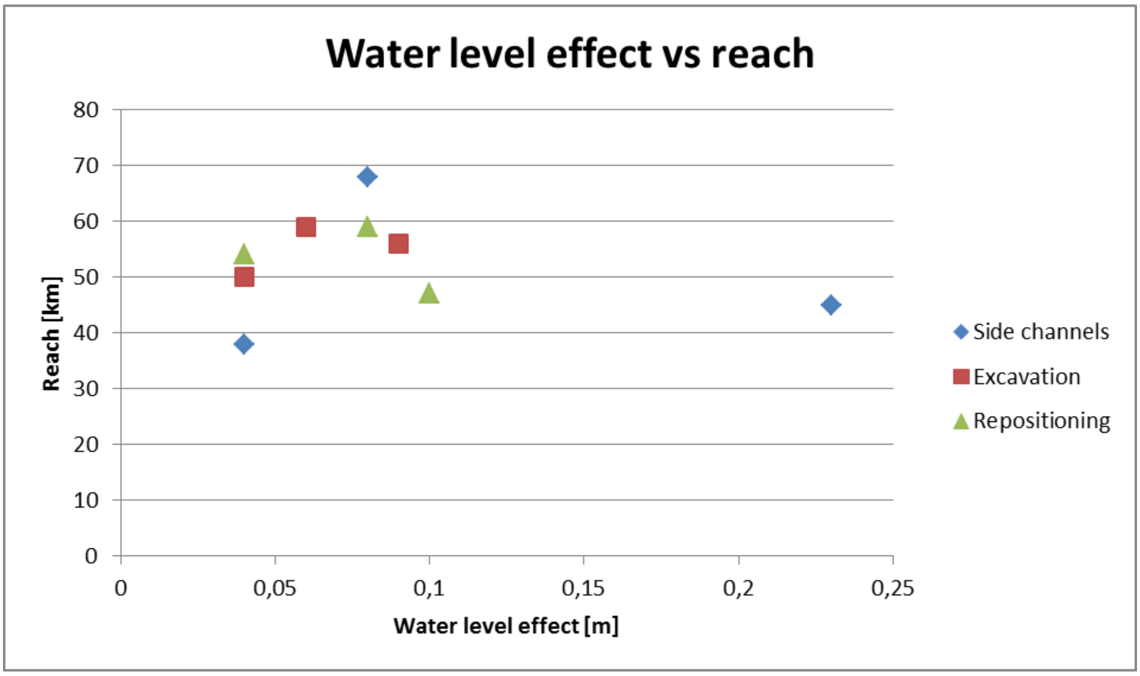


Figure D-22: Relation water level effect and reach

D.1.5 Effect on the Q-h relation

The way in which a measure affects the water level given a certain discharge depends on the type of floodplain measure. Dike repositioning will have effect from the moment the floodplains start flowing along, and its effect will increase for higher discharges. It is assumed here that this increase is linear, since the widening of the floodplain will be more effective for higher water levels. Excavation will be effective from the moment the floodplains start flowing along. The relative effect will be maximal at lower water levels and this will decline for higher discharges. For higher discharges the water level effect will reach a maximum value, since when the extra space is ‘filled’ the water level effect no longer increases. The same holds for a side channel, only the effect is larger and it starts at a higher discharge. The effect of these measures on the Q-h relation can be seen in Figure D-23

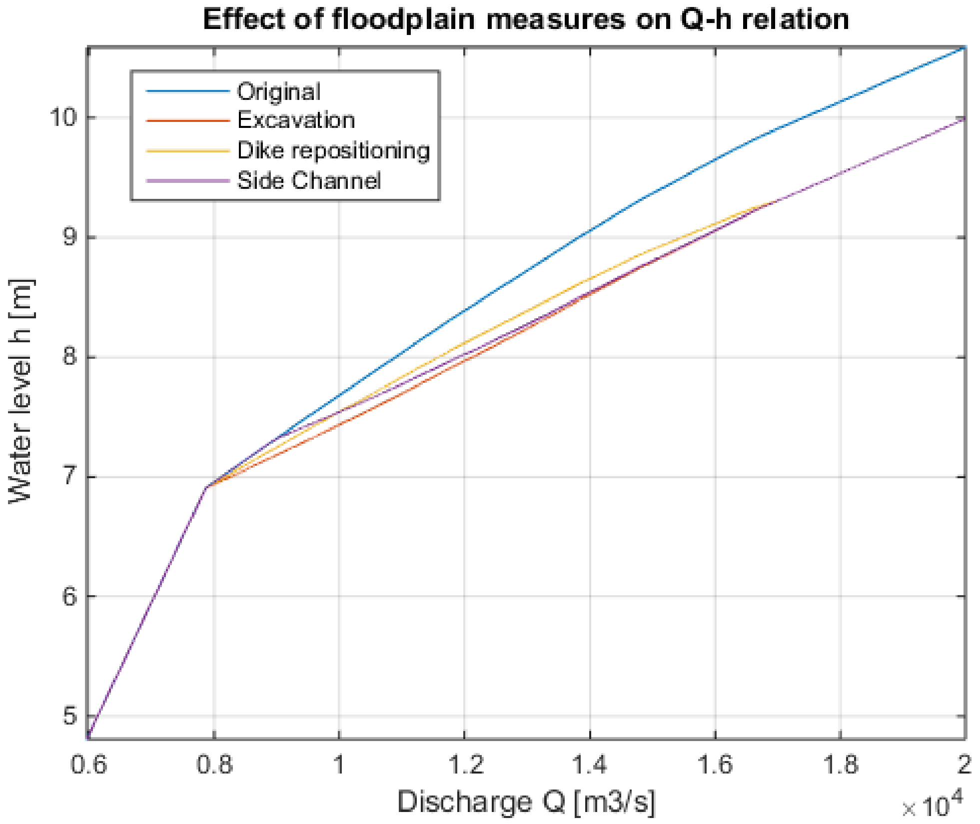


Figure D-23: Effect of dike repositioning on Q-h relation

To calculate the Q-h relation for a certain location along the river stretch the effect on the Q-h relation should be described in relative water level lowering. The water level reduction at MHW can be read from the water level effect graphs for a measure. With this maximum water level reduction and the relative reduction for lower discharges, the Q-h relation can be specified for all locations along the river stretch. These new Q-h relations will be used to assess the total effectiveness of a measure on the probability of failure of a dike reach.

D.1.6 Conclusions floodplain measures

Based on this analysis it is hard to draw conclusions about the relation between project size and effectiveness. A lot of information is missing in order to draw conclusions about the relation between the dimensions and the effectiveness. The effectiveness of the measures can vary widely due to the actual dimensions of the measure, the location of the measure and other factors. This makes every measure unique and this emphasizes the need to assess every measure individually. To make calculations with different floodplain measures some assumptions will have to be made, based on the effects described above.

For all the types of floodplain measures an 'average' variant will be put together on order to be able to make some calculations on the effectiveness of these measures for all discharge levels. All measures will have a water line based on one of the real measures. The length will probably be a little longer than 50 km, since the line will not be kinked by bifurcations. The length of the 'imaginary' dike reach is only 16 km, but that should not be a problem. In reality there will also be measures which extend beyond the limits of the dike reach they serve.

The side channel will have a length of 3 km, and the maximum water level reduction is 20 cm. This is quite a big effect, but practice shows that side channels are very effective measures. The downstream water level rise will be 5 cm. The water line is based on the line from "Sleeuwijk".

The excavation measure will have a length of 2 km, the maximum water level reduction is 6 cm. The water level rise on the downstream side will be 1 cm.

The dike repositioning will have an effect of 8 cm, with a length of 3 km. The downstream water level rise will be 2 cm.

For all measures it counts that spatial integration is not taken into account. It is thus assumed that the floodplains and the surroundings offer enough space to construct the measures in such a way that the proposed effect is reached. Also no bifurcations are taken into account so the water level effect graph will be smooth. The water lines for the three types of measures are presented in Figure D-24.

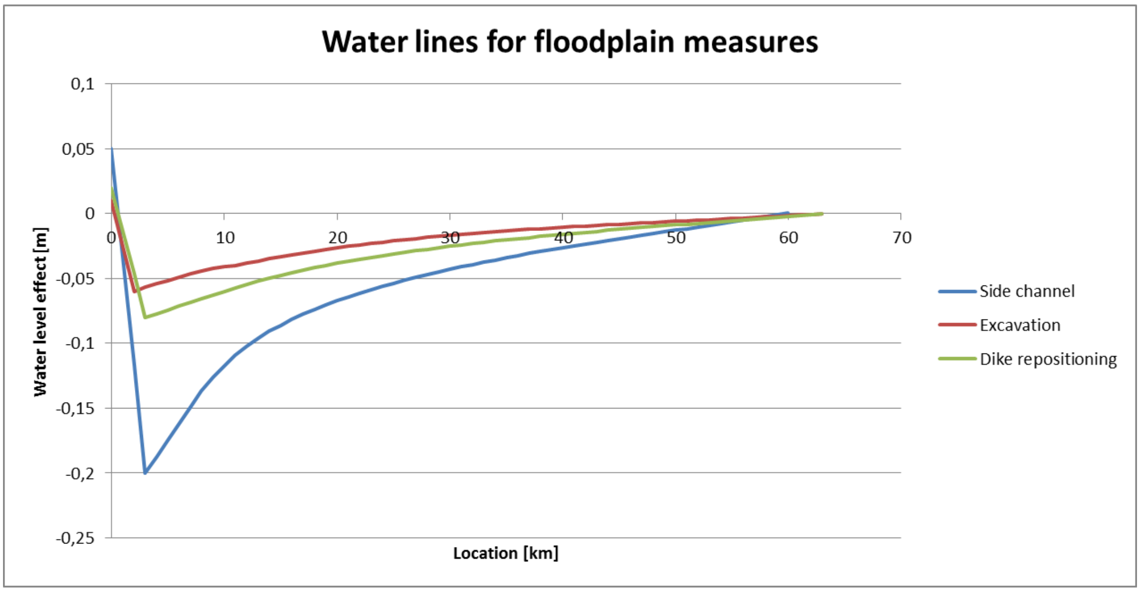


Figure D-24: Water lines for floodplain measures

D.2 Lowering of groynes

Groyne lowering is not taken into account in the Blokkendoos, but since it could be of interest for flood defences where piping is a big problem, the effect of this measure will still be assessed. For the effect of this measure on the Q-h relation some assumptions need to be made. First of all it is assumed that all of the dike sections that are regarded contain groynes. All the groynes will be flooded at a discharge of 3000 m^3/s. From this discharge (and the corresponding water level) the effectiveness will increase linearly with the discharge. The water level lowering effect is maximal at Q = 5000 m^3/s. The reduction of the water level is then 15 cm. This is a reasonable estimate for the Waal since this has been calculated for the same river stretch by HKV (DHV, 2009). The water level lowering will linearly decrease for higher discharges until the water level effect is 10 cm for the maximum discharge of 17,000 m^3/s (Huthoff, et al., 2011). This effect on the Q-h relation is valid for all reaches.

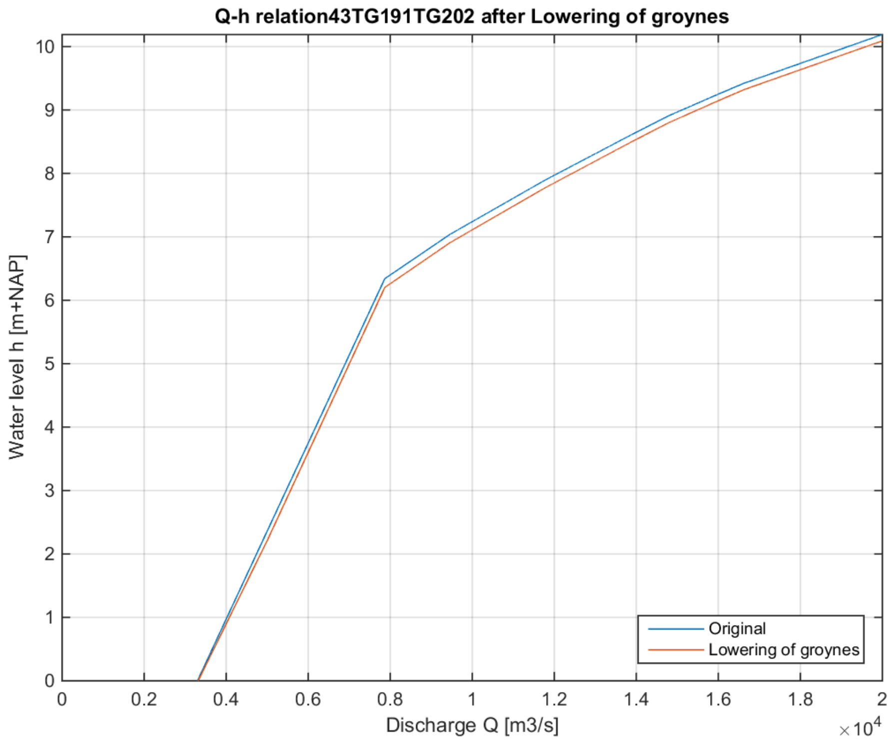


Figure D-25: Effect of groyne lowering on Q-h relation

D.3 Summer bed projects

Measures taken in the summer bed will most certainly contain dredging activities. Dredging is supposed to be more effective in the lower water levels. Before the floodplains are flowing the water level reduction will be approximately equal to the thickness of the dredged layer. For higher water levels the increase of the flood conveyance capacity will be small in comparison to the high flood conveyance capacity of the winter bed. Therefore the water level reduction will be less. The disadvantage of dredging is that the measure will most certainly be only temporary, since sedimentation in the river will cause the dredged area to be filled up again eventually.

Summer bed deepening "Bovenmerwede and Nieuwe Merwede"

This project dredges over a stretch of 17 km. This is a project of the PKB, so the numbers are a bit different from the actual effect. What is interesting is that this measure has effect on both the upstream and downstream side of the measure. The measure is located on the red indicated area in Figure D-27. It can be seen that the downstream effect is smaller than the upstream effect, but it is still present, and the water level does not rise at the downstream end. The effect is maximal at the upstream end of the measure, and since the measure is so effective its reach is very long.

Table D-10: Project details dredging summer bed

Summer bed measure	Summer bed deepening “Bovenmerwede and Nieuwe Merwede”
Project size	17 km
Project location	Km 954-971
Project reach	127 km
Maximum MHW reduction	0.37 m
Investment costs	97.6 M€

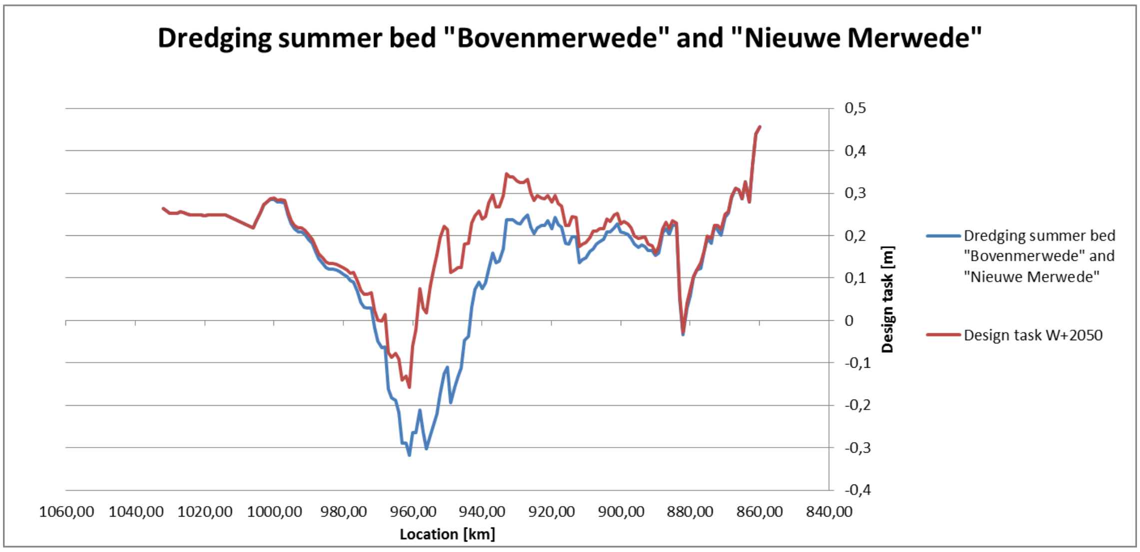


Figure D-26: Dredging summer bed, reduction of design task

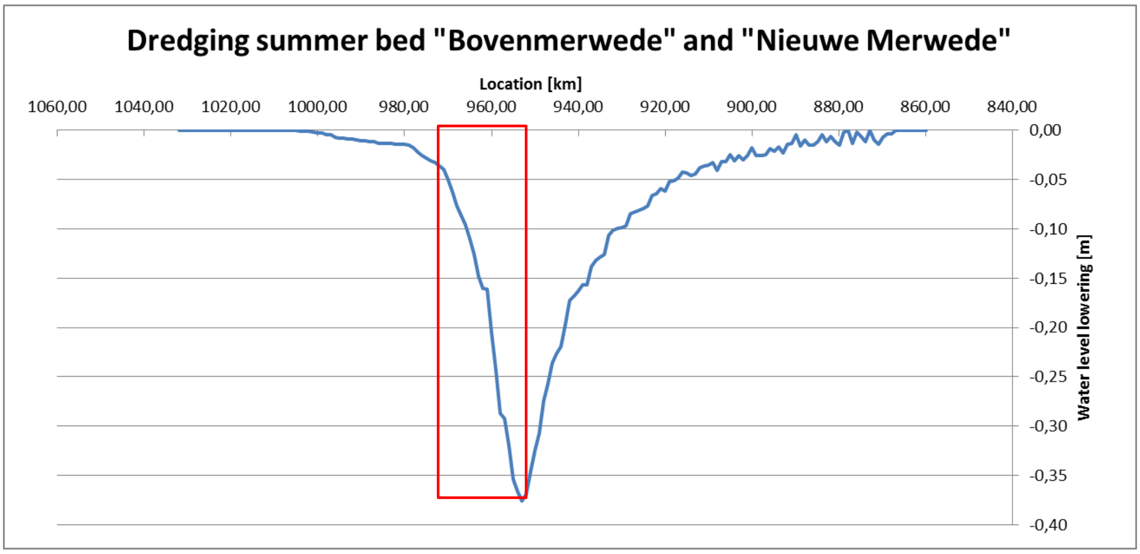


Figure D-27: Dredging summer bed, water level effect

Widening summer bed “Beneden Merwede”

This project widens the summer bed over a stretch of 6 km. This project is a part of the DPR. Its effect is different from the deepening of the summer bed. Although very small, this measure also shows a water level rise at the downstream end. The project is much less effective than the previously discussed dredging measure.

Table D-11: Project details widening summer bed

Summer bed measure	Summer bed widening "Beneden Merwede"
Project size	6 km
Project location	Km 963-969
Project reach	23 km
Maximum MHW reduction	0.03 m
Investment costs	31.7 M€

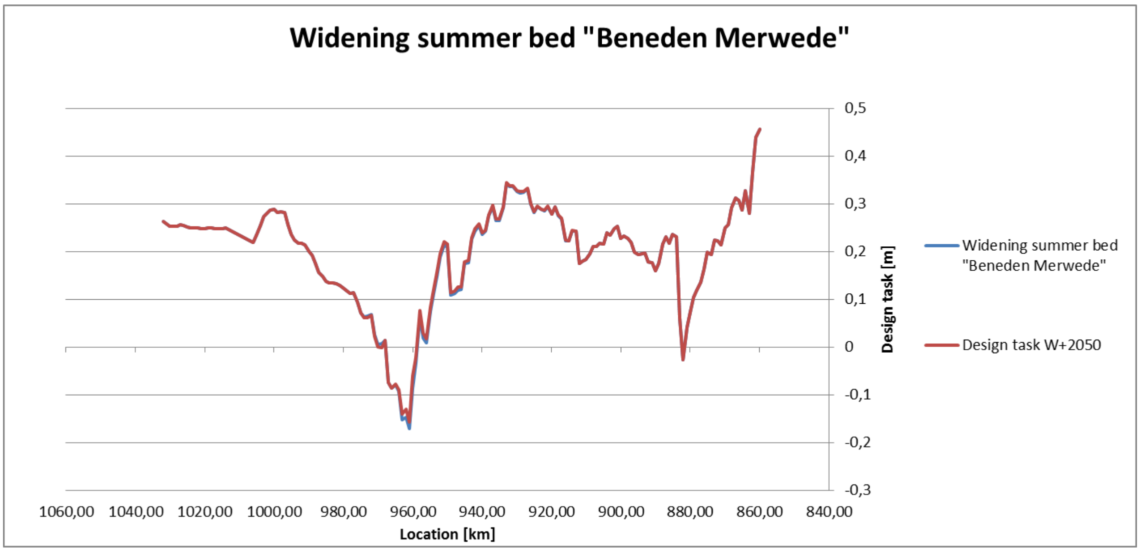


Figure D-28: Widening summer bed, reduction of design task

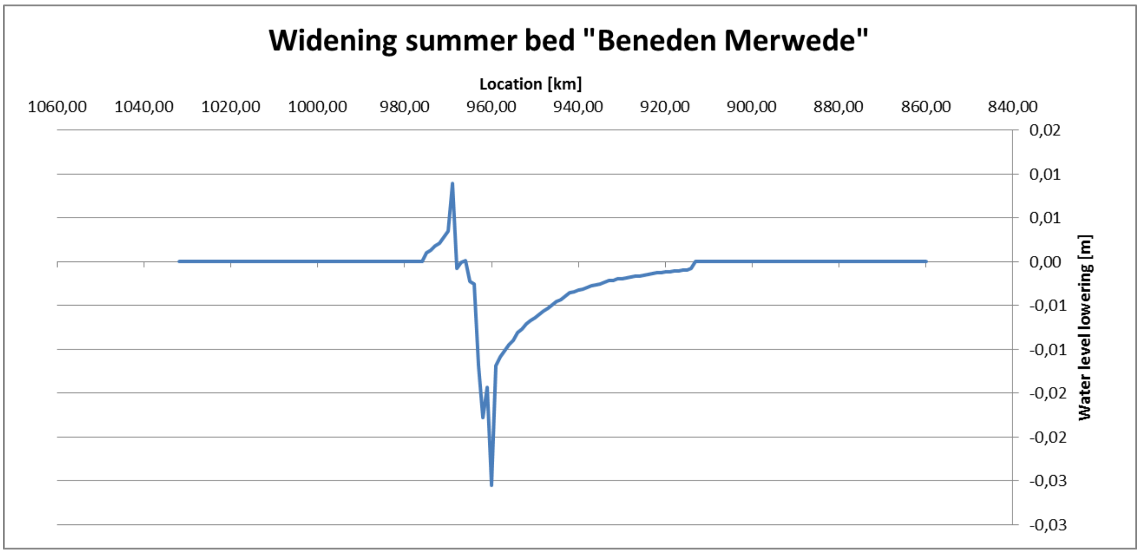


Figure D-29: Widening of summer bed, water level effect

D.3.1 Conclusion summer bed measures

Widening of the summer bed has a different effect on the water level than lowering the summer bed. Dredging of the summer bed has an advantage that it also has a positive downstream effect. Dredging of the summer bed is also more effective on this river stretch. Since water level reduction will be less for the higher discharges, the dredged layer will be even thicker than the maximal water level lowering of 37 cm.

If a dredging project is used for calculation, the water level reduction will be equal to the thickness of the dredged layer until the point where the floodplains start flowing along. For the considered dike reach this is at the point where the Q-h relation has a kink. This is at 7867 m^3/s. From there the effect will linearly decline until it is half of the initial reduction at a discharge of 17,000 m^3/s. From there the effect will remain constant. This is an assumed Q-h relation change, based on ‘engineering judgement’.

D.4 Dike improvement

Dike improvement is also not incorporated in the blokkendoos. Nevertheless this is still a type of measure which is very interesting, and it’s effectiveness for different discharge levels needs to be assessed. The effectiveness of dike raising will be assessed by raising the dike with 10, 20, 30 and 40 cm. This will be done by simply shifting the fragility curves of the failure mechanism overflow/overtopping to the right. The effectiveness that is calculated for this measure can be verified by making design calculations in PC-Ring.

Failure mechanisms piping and macro stability are a little harder to assess since they are not solely water level related. For the assessment of piping use is made from a rule of thumb by Bligh. This rule says that for one metre higher water level a piping berm of 18 m is needed (Technische Adviescommissie voor de Waterkeringen, 1999). This value is conservative as it counts for sand. The fragility curve will be shifted to the higher water levels with 25, 50, 75 and 100 cm in order to pretend the construction of a berm of 4.5, 9, 13.5 and 18 m.

For cost-efficiency a good comparison can be made when the same amount of ground movement is done for dike improvement as would be in a spatial measure. In this way the effectiveness of the measure can be assessed in terms of money and ground movement.

D.5 Application

To calculate the Q-h relation for a certain location along the river stretch the effect on the Q-h relation should be described in relative water level lowering (Figure D-23 and Figure D-25). The water level reduction at MHW can be read from the water line graphs for a specific measure. With this maximum water level reduction and the relative reduction for lower discharges, the Q-h relation can be specified for all locations along the river stretch. These new Q-h relations will be used to assess the total effectiveness of a measure on the probability of failure of a dike reach.

When calculations are made on the model dike section, the upstream end of the measure lies just outside of reach. Otherwise too less of the reach will be available to assess the effectiveness.

D.6 Cost analysis

To see which type of measure is the most cost-efficient, also an analysis is made on the effectiveness and the cost. This is shown in Figure D-30. This figure shows that for side channels and for dike repositioning it holds that a higher investment leads to a higher effect. For excavations this is not the case. Although it should be borne in mind that spatial measures also improve the quality of the landscape. So a higher investment may not lead to a larger water level reduction in all cases, but it does give the environment extra value this is not taken into account in this analysis. A more thorough cost-benefit analysis will be made later.

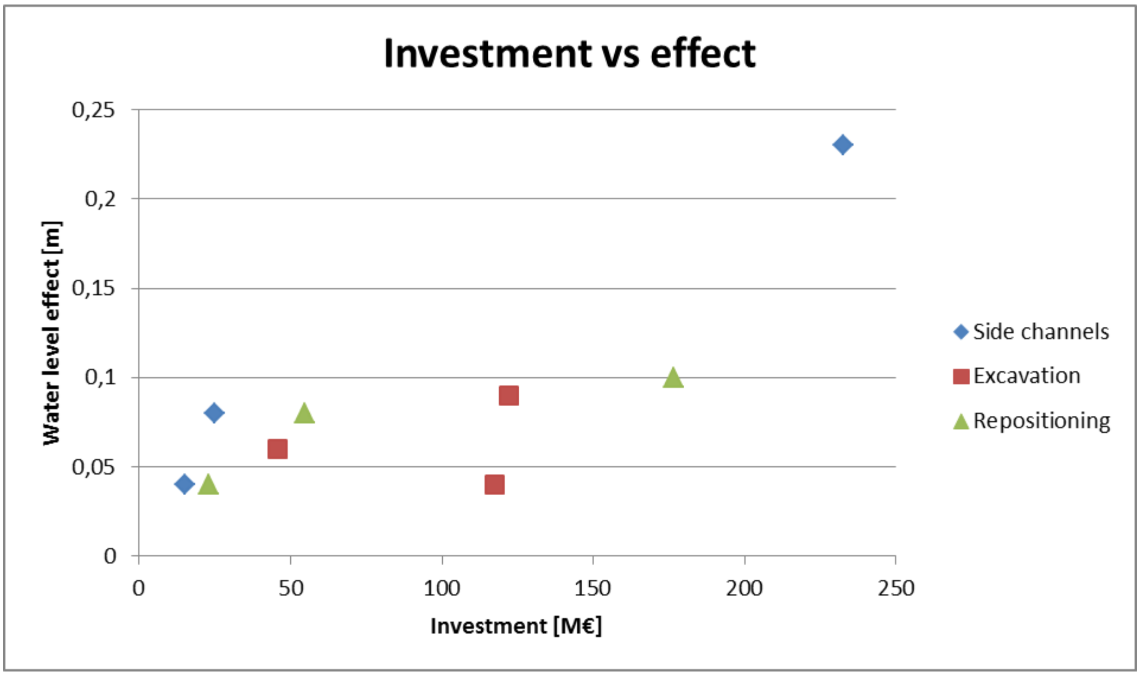


Figure D-30: Relation investment and water level effect

E. Matlab scripts

E.1 Probability of failure per section

This script was used to calculate the probability of failure per dike section for a certain failure mechanism.

```
% Jelle van Zuijlen, maart 2015
% Afstudeerwerk
% Berekent faalkans per Dijkvak en per faalmechanisme
% Berekent voor een traject de faalkans per dijkvak

% faalmechanismen:
% 1 overtopping
% 2 macro stabiliteit
% 3 piping
% 4 stabiliteit binnentalud

clear;close;clc;
dbstop if error

%% Specificaties
%Vak = '43TG191TG202';          % vaknaam zonder punten
Faalmechanisme = '1';           % kies 1 of 2 of 3 of 4
Werklijn = 'HR2006';            % kies HR2006 of W+2050
Locatie = 'bank';               % kies bank of axis
Methode = 'interp';             % kies 2eorde of 3eorde of interp
h = 0.00:0.05:14.00;            % range waterstanden
Measure = 'dik';                % kies non/sid/exc/rep/gro/dre/dik
effect = 0.5;                   % grootte dijkverbetering

% Debietpunten Q-h relatie:
Q = [5965   7867    9459    11763   13881   14794   16000   16619
20000];

Maatregel = 0;
if Measure == 'sid';
    Maatregel = 'Side channel';
elseif Measure == 'exc';
    Maatregel = 'Excavation';
elseif Measure == 'rep';
    Maatregel = 'Repositioning of dikes';
elseif Measure == 'gro';
    Maatregel = 'Lowering of groynes';
elseif Measure == 'dik';
    Maatregel = 'Dike improvement';
elseif Measure == 'non';
    Maatregel = 'No measure';
end
```

```
mech = 0;
if Faalmechanisme == '1';
    mech = 'Overflow/overtopping';
elseif Faalmechanisme == '2';
    mech = 'Macro stability';
elseif Faalmechanisme == '3';
    mech = 'Piping';
elseif Faalmechanisme == '4';
    mech = 'Damage and erosion outer slope';
end

%% Data laden
% Laad lijst met vaknamen
load('Vakken');

faalkans0=1;
pfring0=1;
error0=1;

for v = 1;%:length(Vakken);
% Try en catch: als xlsread een error geeft (omdat de combinatie
% vak-faalmechanisme niet bestaat) geeft de hele file als uitkomst
dat
% Pf(v)=0 en Error(v)=0

try
    Vak = Vakken{v,1};

% Faalkans PC-Ring
Pfring(v) =
xlsread(['C:\Users\Zuijlen.HKV.000\Dropbox\Afstuderen\2.
Synthese\Excel\',Vak,'-',Faalmechanisme,'.xlsx'],1,'D2');
pfring0=Pfring(v);

% Inladen waarden fragility curve
fragility =
xlsread(['C:\Users\Zuijlen.HKV.000\Dropbox\Afstuderen\2.
Synthese\Excel\',Vak,'-',Faalmechanisme,'.xlsx'],2,'A11:D291');

%% Plotten fragility curve
% Fragility curve, werklijn en Q-h relatie worden in figuur 1
geplot
% Plot fragility curve
waterst = fragility(:,1);
faalk = fragility(:,4);
fail = zeros(length(faalk),1);

figure(1);
subplot(2,2,1);
plot(waterst,faalk)
title('Fragility curve')
```

```
xlim([0 14])
% ylim([0 1])
xlabel('Water level h [m+NAP]')
ylabel('Probability of failure')
grid on

if Measure == 'dik';
    fail = zeros(length(faalk),1);
    fail(50:length(faalk),1)=max(faalk);
    for l = 1+(effect/0.05):length(faalk);
        fail(l) = faalk(l-(effect/0.05));
        if l == 281
            break
        end
    end
    subplot(2,2,1);
    hold on
    plot(waterst,fail);
    legend('Original',Maatregel,'location','southeast');
    hold off
    faalk = fail;
end

%% Plotten waterstanden
% Inladen van de waterstanden voor de oever
hoever = xlsread(['C:\Users\Zuijlen.HKV.000\Dropbox\Afstuderen\2.
Synthese\Excel\',Vak,'-',Faalmechanisme,'.xlsx'],1,'B20:J20');
% hoever = hbank+effect;
% Inladen van de waterstanden voor de as
has = xlsread(['C:\Users\Zuijlen.HKV.000\Dropbox\Afstuderen\2.
Synthese\Excel\',Vak,'-',Faalmechanisme,'.xlsx'],1,'B24:J24');

% Fit maken van de waterstanden
% Kiest eerst de locatie uit 'Specificaties'
% Kiest methode uit 'Specificaties
% Plotten originele Q-h punten en juiste fit in 1 grafiek

% Keuze locatie: oever
if Locatie == 'bank';
    subplot(2,2,2);
    plot(Q,hoever,'o')

    if Methode == '3eorde';
        [p,~,mu] = polyfit(hoever,Q,3);
        Q2 = polyval(p,h,[],mu);
    end

    if Methode == 'interp';
        Q2 = interp1(hoever,Q,h,'linear','extrap');
    end
```

```
    if Methode == '2eorde';
        p = polyfit(hoever,Q,2);
        Q2 = polyval(p,h);
    end
end

% Keuze locatie: as
if Locatie == 'axis';
    subplot(2,2,2);
    plot(Q,has,'o')

    if Methode == '3eorde';
        [p,~,mu] = polyfit(has,Q,3);
        Q2 = polyval(p,h,[],mu);
    end

    if Methode == 'interp';
        Q2 = interp1(has,Q,h,'linear','extrap');
    end

    if Methode == '2eorde';
        p = polyfit(has,Q,2);
        Q2 = polyval(p,h);
    end
end

hold on
plot(Q2,h);
title('Q-h relation');
xlim([0 30000]);
ylim([0 14]);
xlabel('Discharge Q [m3/s]')
ylabel('Water level h [m+NAP]')

grid on
hold off

if ~or(strcmp(Measure,'non'),strcmp(Measure,'dik'))% if measure is
not 'non', so there is a measure!
    hmea =
xlsread(['C:\Users\Zuijlen.HKV.000\Documents\MATLAB\NewQh\newqh',Va
k,'-',Measure,'.xlsx'],1,'A2:A20002');
    q = [0:1:20000];
    Q3 = interp1(hmea,q,h,'linear','extrap');
    subplot(2,2,2);
    hold on
    plot(Q3,h);
    legend('Original','Fit',Maatregel,'Location','southeast');
    hold off
end
```

```
%% Waterstanden omrekenen naar probability density
%Definieer werklijnen
if Werklijn == 'HR2006';
    a1 = 1620.7;
    b1 = 5893.3;
    a2 = 1517.78;
    b2 = 5964.63;
    a3 = 1316.43;
    b3 = 6612.61;
end

if Werklijn == 'W+2050';
    a1 = 1929.11;
    b1 = 5893.34;
    a2 = 1805.86;
    b2 = 5978.76;
    a3 = 1566.54;
    b3 = 6749.10;
end
% % Plot werklijn
% % Dit moet na terugkeertijd omdat de T berekend wordt

X = [0.1 1 2 10 25 100 1250 10000];

w0=1;
for i=1:length(X)
    W(i)=a1.*log(X(i))+b1;
    if X(i)>2;
        W(i)=a2.*log(X(i))+b2;
    end
    if W(i)>25;
        W(i)=a2.*log(X(i))+b2;
    end
    w0=W(i);
end

subplot(2,2,4);
semilogx(X,W);
title(Werklijn);
xlim([0.10 10000]);
ylim([0 20000]);
xlabel('Return period [year]');
ylabel('Discharge at Lobith [m3/s]');
grid on

% Bereken de terugkeertijd per waterstand
t0 = 1;
for i=1:length(Q2);
        T(i)=exp(((Q2(i)-b1)./a1));
    if T(i)>2;
```

```
        T(i)=exp(((Q2(i)-b2)./a2));
    end
    if T(i)>25;
        T(i)=exp(((Q2(i)-b3)./a3));
    end
    t0=T(i);
end

%% Afmaken figuur 1
% Set title and title size
ha = axes('Position',[0 0 1 1],'Xlim',[0 1],'Ylim',[0
1],'Box','off','Visible','off','Units','normalized','clipping','off
');
tit = text(0.5, 1,['\bf ',Vak,'-',mech,'
',Maatregel,''],'HorizontalAlignment','center','VerticalAlignment',
'top');
s = tit.FontSize;
tit.FontSize = 12;

% Set figuresize and print
hFig = figure(1);
set(hFig, 'Position', [100 100 700 475])
set(gcf, 'Visible', 'off');
print(['Knoppen ',Vak,'-',Faalmechanisme,'',Measure,''],'-dpng');

% F-waarde bepalen per waterstand
f0=1;
for j=1:length(T);
    F(j)=1-(1-exp(-1/T(j)));
    f0=F(j);
end

% Probability density is de afgeleide van F
d0=1;
for k=2:(length(F)-2);
    D(k)=(((3.*F(k+1))-(4.*F(k))+(1.*F(k-1)))/0.1);
    d0=D(k);
    D(1)=0;
    D(281)=0;
end

%% Calculate probability of failure per water level
% Creeer figuur met fragility curve, probability density en
probability of
% failure
% Bereken faalkans per waterstand
pf0=1;
for m=1:length(D);
    if m > length(faalk);
       faalk(m) = max(faalk);
    end
```

```
    Pf(m)=D(m).*faalk(m);
    pf0=Pf(m);
end

% Plot fragility curve in de bovenste grafiek
figure(2);
subplot(3,1,1);
plot(h,faalk);
title(['',Vak,'-',mech,' ',Maatregel,'']);
xlim([0 14]);
ylim([0 1]);
ylabel('Probability of failure');
grid on

% Plot probability density
subplot(3,1,2);
plot(h,D);
xlim([0 14]);
ylim([0 1]);
ylabel('Probability density')
grid on

%% Totale faalkans en tabel wegschrijven
% Faalkans is gelijk aan de oppervlakte onder de onderste grafiek
in figuur
% 2. Deel de grafiek in staafjes van 0.05 m breed en bereken de som
van
% alle staafjes voor de totale faalkans.
z0=1;
for n=1:length(Pf);
    Z(n) = 0.05.*Pf(n);
    z0 = Z(n);
end

Faalkans(v) = sum(Z);
faalkans0=Faalkans(v);

if Faalkans(v) > Pfring(v);
    Error(v) = Faalkans(v)./Pfring(v);
else Error(v) = Pfring(v)./Faalkans(v);
end
error0=Error(v); % Error geldt alleen voor HR2006. Voor ander
klimaat scenario geldt andere
% pfring.

% Plot probability of failure
subplot(3,1,3);
plot(h,Pf);
xlim([0 14]);
xlabel('Water level h [m+NAP]')
ylabel('Probability of failure')
```

```
% text(0,1,['',Vak,'-',Faalmechanisme,'']);
grid on

an = annotation(figure(2),'textbox',...
    [0.15 0.27 0.25 0.04],...
    'String',{['Pf = ',num2str(Faalkans(v)),'']},...
    'FitBoxToText','on');

%% Als er een maatregel is gaat ie het hier opnieuw doorrekenen
if ~or(strcmp(Measure,'non'),strcmp(Measure,'dik'))% if measure is
not 'non', so there is a measure!
        % Bereken de terugkeertijd per waterstand

delete(an);

    t_mea0 = 1;
    for i=1:length(Q3);
            T_mea(i)=exp(((Q3(i)-b1)./a1));
        if T_mea(i)>2;
            T_mea(i)=exp(((Q3(i)-b2)./a2));
        end
        if T_mea(i)>25;
            T_mea(i)=exp(((Q3(i)-b3)./a3));
        end
        t_mea0=T_mea(i);
    end

        % F-waarde bepalen per waterstand
    f_mea0=1;
    for j=1:length(T_mea);
        F_mea(j)=1-(1-exp(-1/T_mea(j)));
        f_mea0=F_mea(j);
    end

    % Probability density is de afgeleide van F
    d_mea0=1;
    for k=2:(length(F_mea)-2);
        D_mea(k)=(((3.*F_mea(k+1))-(4.*F_mea(k))+(1.*F_mea(k-
1)))/0.1);
        d_mea0=D_mea(k);
        D_mea(1)=0;
        D_mea(281)=0;
    end

    % Bereken faalkans per waterstand
    pf_mea0=1;
    for m=1:length(D_mea);
        if m > length(faalk);
           faalk(m) = max(faalk);
        end
        Pf_mea(m)=D_mea(m).*faalk(m);
```

```
        pf_mea0=Pf_mea(m);
    end


        % Plot probability density
    subplot(3,1,2);
    hold on
    plot(h,D_mea);
    hold off
    legend('Original',Maatregel,'location','southwest');

        z_mea0=1;
    for n=1:length(Pf_mea);
        Z_mea(n) = 0.05.*Pf_mea(n);
        z_mea0 = Z_mea(n);
    end

    Faalkans_mea(v) = sum(Z_mea);
    faalkans_mea0=Faalkans_mea(v);


    % Plot probability of failure
    subplot(3,1,3);
    hold on
    plot(h,Pf_mea);
    hold off
    legend('Original',Maatregel,'location','southwest');

an = annotation(figure(2),'textbox',...
    [0.15 0.27 0.25 0.04],...
    'String',{['Pf = ',num2str(Faalkans(v)),''];['Pf ',Maatregel,'
= ',num2str(Faalkans_mea(v)),'']},...
    'FitBoxToText','on');

end



% Set size specifications for figure 2
hFig = figure(2);
set(hFig, 'Position', [100 100 450 550]);
set(gcf, 'Visible', 'off');
print(['Probability of failure ',Vak,'-
',Faalmechanisme,'',Measure,''],'-dpng');
delete(an);
% Tabel wegschrijven met waterstand h, debiet Q, terugkeertijd T,
% onderschrijdinskans F en kansdichtheid waterstand D, faalkans
faalk en
% gecombineerde faalkans Pf

Samenvatting = table(h',Q2',T',F',D',faalk,Pf');
```

```
Samenvatting.Properties.VariableNames = {'WaterLevel_h'
'Discharge_Q' 'ReturnPeriod' 'F' 'ProbablilityDensity'
'ConditionalPf' 'Pf'};
writetable(Samenvatting,['C:\Users\Zuijlen.HKV.000\Documents\MATLAB
\tabel',Vak,'-',Faalmechanisme,'.xlsx']);

catch
    Pfring(v) = 0;
    Faalkans(v) = 0;
    Error(v) = 0;
    Faalkans_mea(v) = 0;
end
end

% Traject = table(Vakken,Faalkans',Error');
% Traject.Properties.VariableNames = {'Vakken' 'Faalkans' 'Error'};
%
writetable(Traject,['C:\Users\Zuijlen.HKV.000\Documents\MATLAB\traj
ect-',Faalmechanisme,'-',Methode,'-',Measure,'.xlsx']);
```

E.2 Q-h per measure

This scripts makes new Q-h relations for every dike section given a measure.

```
%% Q-h relations for floodplain measure
% Jelle van Zuijlen, April 2015

% There are three types of floodplain measures
%
% Excavation is effective from the moment the floodplains start
flowing
% along.
%
% Dike repositioning is effective from the moment the floodplains
start
% flowing along, the effecivenes rises linearly up to maximum
effect for
% the highest discharge
%
% Side channel is effective from high discharges. It should be
specified
% from which discharge this is happening.

clear all;
close all;
clc;

%% Specifications
Measure = 'rep';             % Choose: sid or exc or rep or gro
```

```
Effect = 0.193290473;          % Water level effect in m for highest
discharge
Floodplains = 7867;            % Discharge [m3/s] from which
floodplains flow
FlowStart = 9000;              % For Side channel: discharge where it
starts flowing
Vak = '43TG191TG202';          % Specifiy section
Faalmechanisme = '1';          % Necessary for loading Q-h relations
Location = 'bank';             % kies bank of axis

Maatregel = 0;
if Measure == 'sid';
    Maatregel = 'Side channel';
elseif Measure == 'exc';
    Maatregel = 'Excavation';
elseif Measure == 'rep';
    Maatregel = 'Repositioning of dikes';
elseif Measure == 'gro';
    Maatregel = 'Lowering of groynes';
end

%% Original Q-h relation
% laden Q-h relaties en locaties, effect per vak etc
load ('Vakken');
load ('xcoor');
load ('effectxcoor');

for v = 1;%:length(Vakken);
    Vak = Vakken{v,1};

    if Measure == 'sid';
        Effect = effectxcoor(v,1);
    elseif Measure == 'exc';
        Effect = effectxcoor(v,2);
    elseif Measure == 'rep';
        Effect = effectxcoor(v,3);
    end

% Inladen van de waterstanden voor de oever
hbank = xlsread(['C:\Users\Zuijlen.HKV.000\Dropbox\Afstuderen\2.
Synthese\Excel\',Vak,'-',Faalmechanisme,'.xlsx'],1,'B20:J20');

% Inladen van de waterstanden voor de as
has = xlsread(['C:\Users\Zuijlen.HKV.000\Dropbox\Afstuderen\2.
Synthese\Excel\',Vak,'-',Faalmechanisme,'.xlsx'],1,'B24:J24');

Q = [5965   7867    9459    11763   13881   14794   16000   16619
20000];
% h = [4.81 6.91    7.49    8.31    9.02    9.31    9.65    9.82
10.59];
```

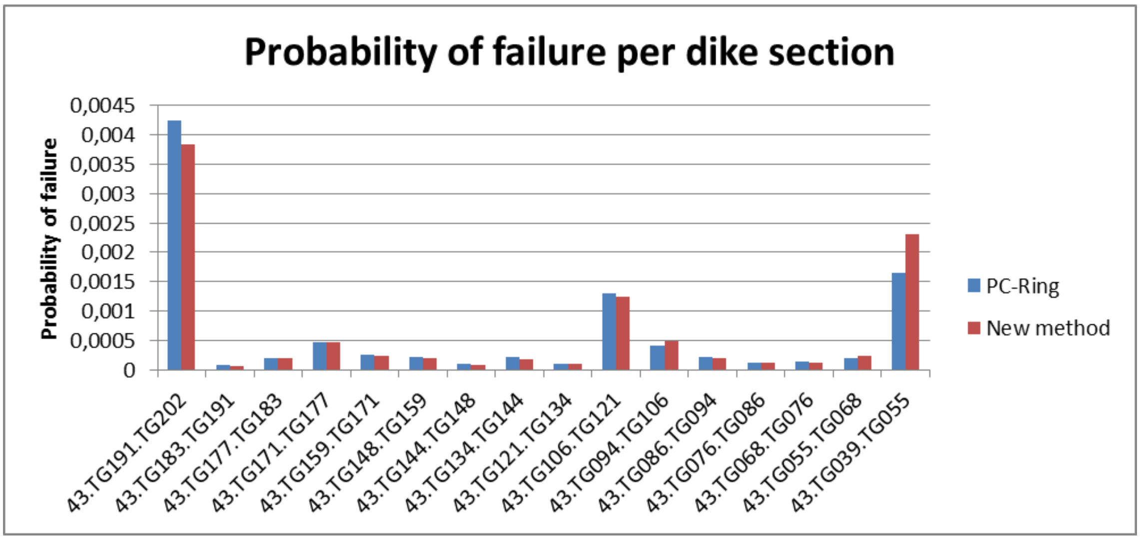
```
if Location == 'bank';
    h = hbank;
elseif Location == 'axis';
    h = has;
end

% plot (Q,h);

%% Interpolation for many Q's
q = [0:1:20000];
p = interp1(Q,h,q,'linear','extrap');

plot(q,p)
if Measure == 'gro';
    xlim([0 max(Q)])
    ylim([0 max(h)])
else
    xlim([min(Q) max(Q)])
    ylim([min(h) max(h)])
end

title(['Q-h relation',Vak,' after ',Maatregel,''])
xlabel('Discharge Q [m3/s]')
ylabel('Water level h [m+NAP]')
grid on

nul = zeros(1,length(q));
reduction0=1;
for i=1:length(q);
    Reduction(i)=nul(i);
    reduction0=Reduction(i);
end

% Measure = excavation
if Measure == 'exc';
    a = (-9133)^2/Effect;
    x = [1:1:9134];

    for i=(Floodplains+1):17001;
        Reduction(i) = (1/a).*-(x(i-Floodplains)-9133).^2+Effect;
        reduction0=Reduction(i);
    end
    for i=17002:length(q);
        Reduction(i)=Effect;
        reduction0=Reduction(i);
    end
    new = p-Reduction;
end

% Measure = dike replacement
if Measure == 'rep';
```

```
    Red = linspace(0,Effect,(17001-(Floodplains)));

      for i = (Floodplains+1):17001;
          Reduction(i) = Red(i-(Floodplains));
          reduction0=Reduction(i);
      end
      for i = 17002:length(q);
          Reduction(i) = Effect;
          reduction0=Reduction(i);
      end
    new = p-Reduction;
end

% Measure = side  channel
if Measure == 'sid'
    n = 17001-(FlowStart);
    a = (-n)^2/Effect;
    x = [1:1:n];

    for i=(FlowStart+1):17001;
        Reduction(i) = (1/a).*-(x(i-FlowStart)-n).^2+Effect;
        reduction0=Reduction(i);
    end
    for i=17002:length(q);
        Reduction(i)=Effect;
        reduction0=Reduction(i);
    end
    new = p-Reduction;
end

% Measure = groyne lowering
if Measure == 'gro';
    Reduction(1,5000:i)=0.10;
    Red = linspace(0,0.15,2001);
    Low = linspace(0.15,0.10,12000);

    for i=3001:5001;
        Reduction(i)=Red(i-3000);
        reduction0=Reduction(i);
    end
    for i=5002:17001;
        Reduction(i)=Low(i-5001);
        reduction0=Reduction(i);
    end
    new = p-Reduction;
end


hold on
plot(q,new)
legend('Original',Maatregel,'Location','Southeast')
```

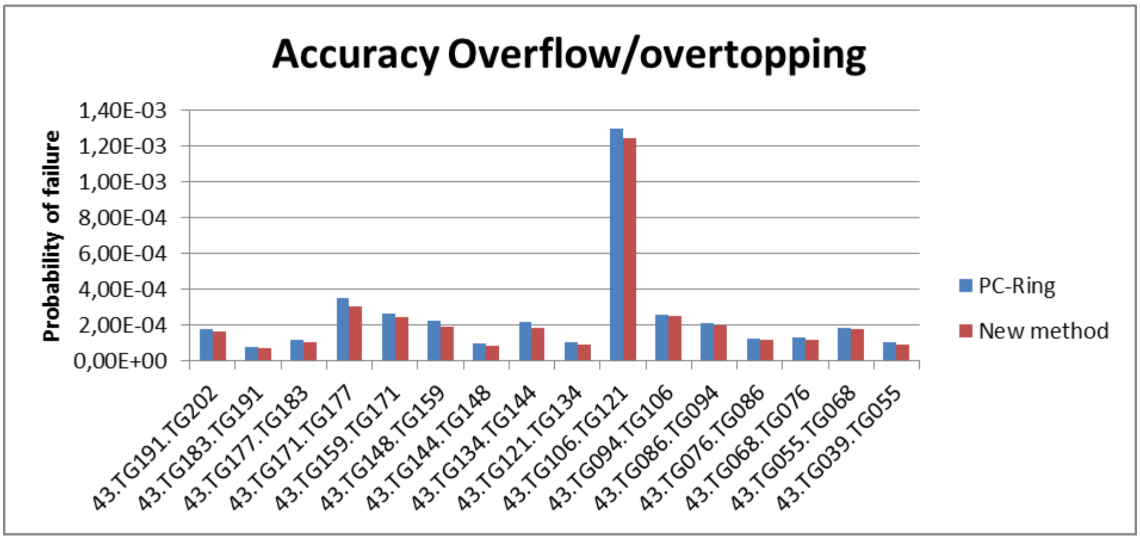
```
hold off

fig = figure(1);
set(gcf, 'Visible', 'off');
saveas(fig,['C:\Users\Zuijlen.HKV.000\Documents\MATLAB\NewQh\newqh'
,Vak,'',Measure,''],'png');

tabel = table(new');
writetable(tabel,['C:\Users\Zuijlen.HKV.000\Documents\MATLAB\NewQh\
newqh',Vak,'-',Measure,'.xlsx']);

lines(:,v) = [new(9460);new(14795);new(20001)];
end
```

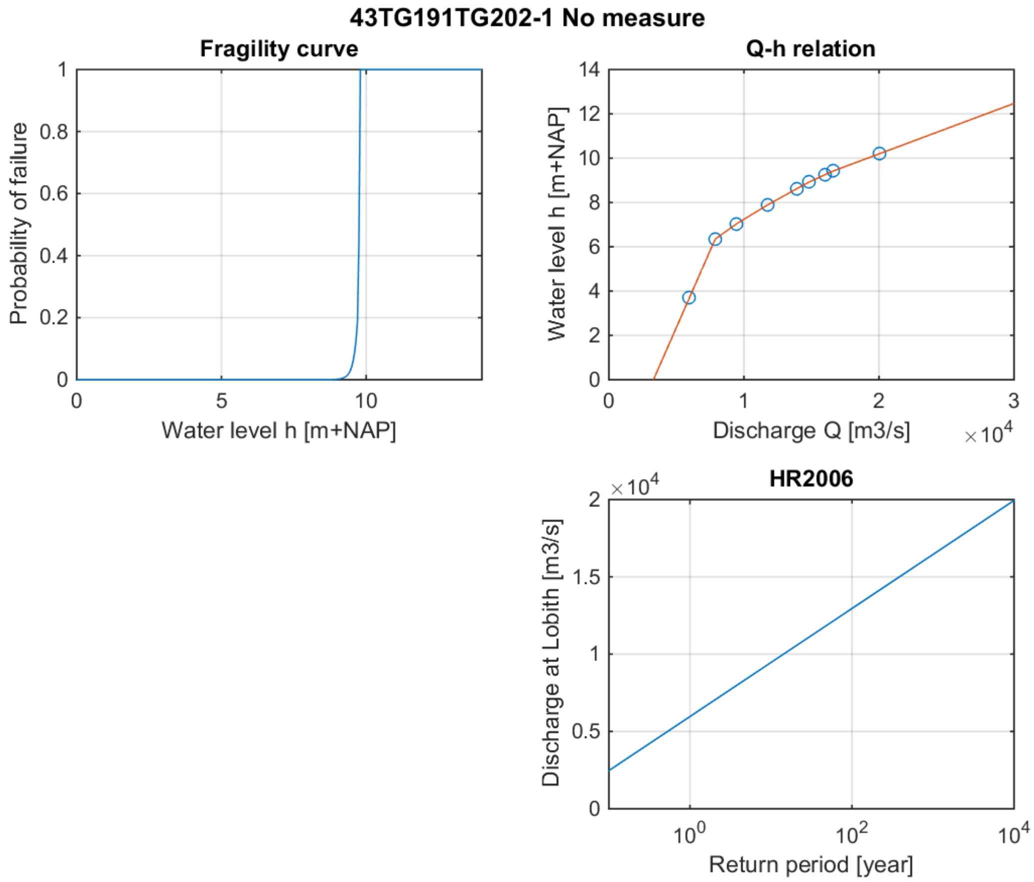
F. Figures

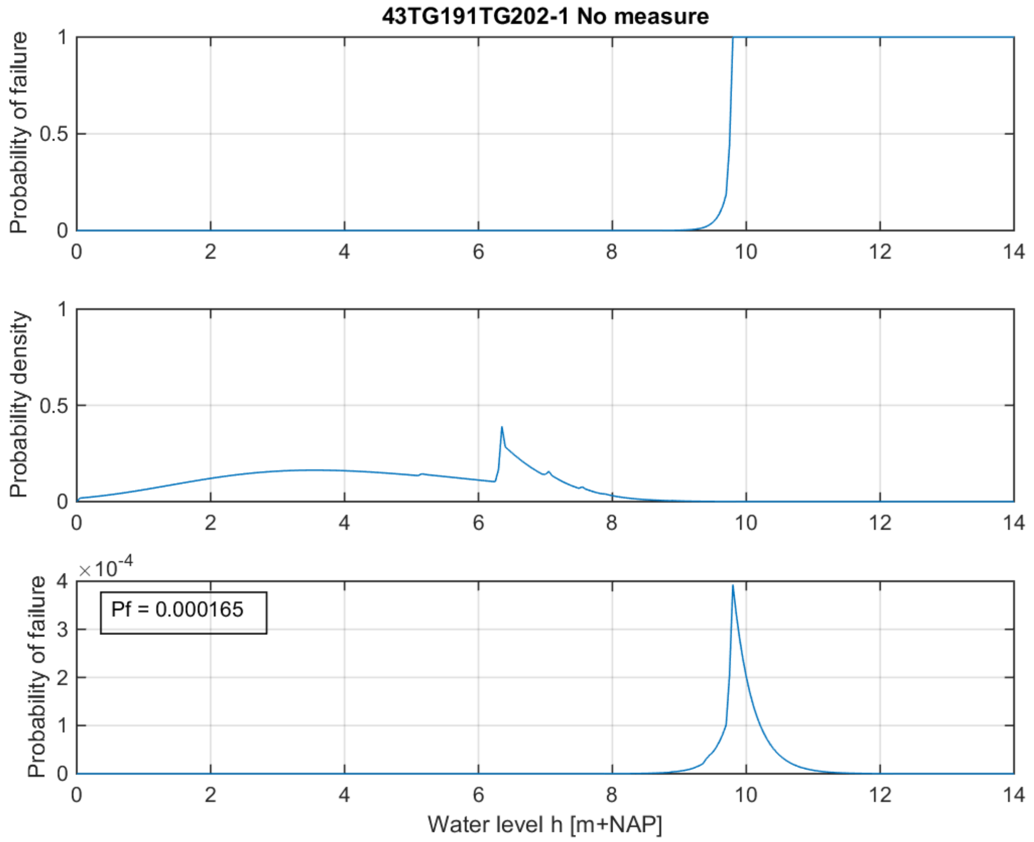
F.1 No measure



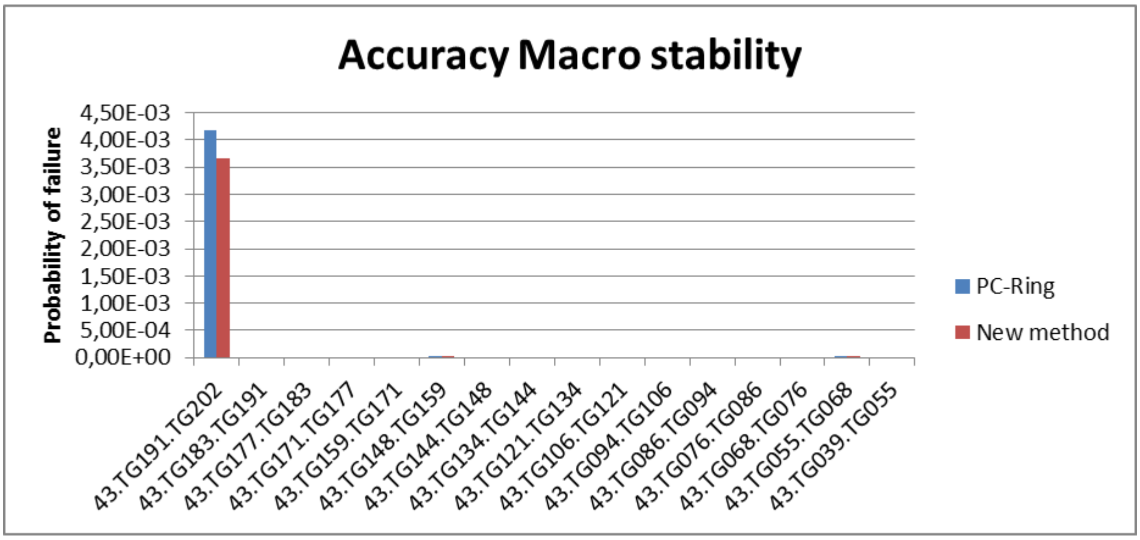
F.1.1 Overflow/overtopping

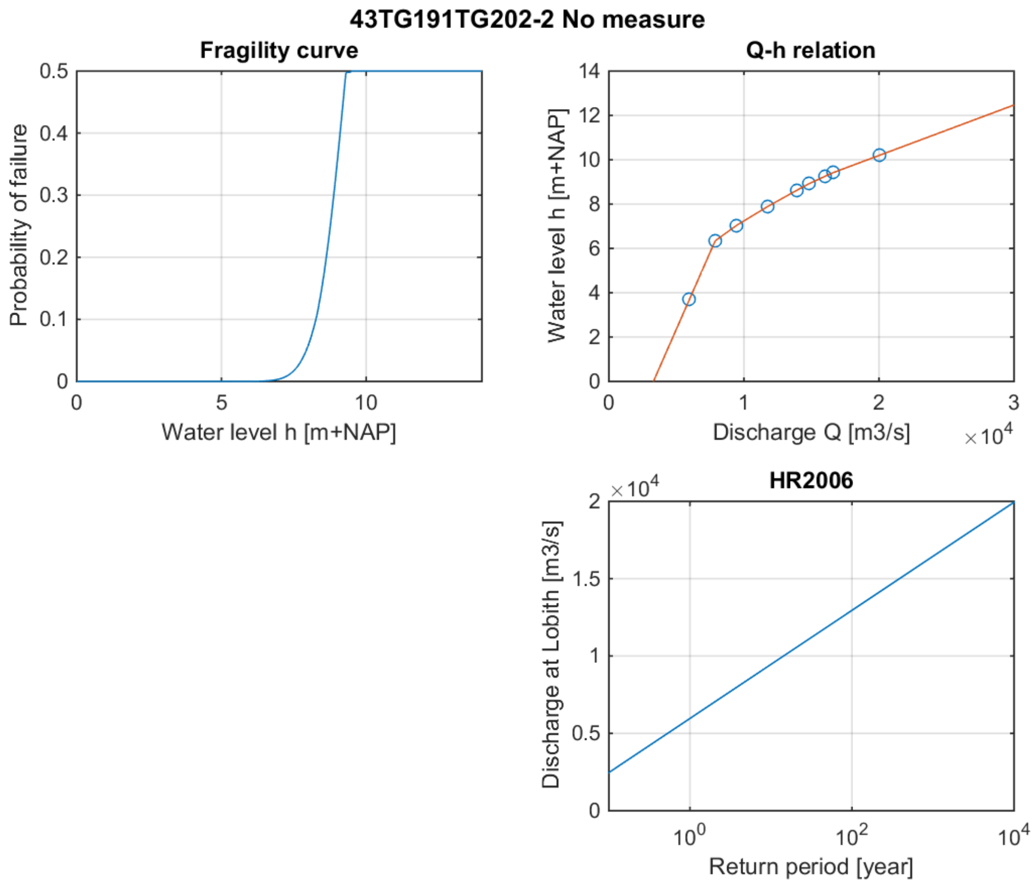


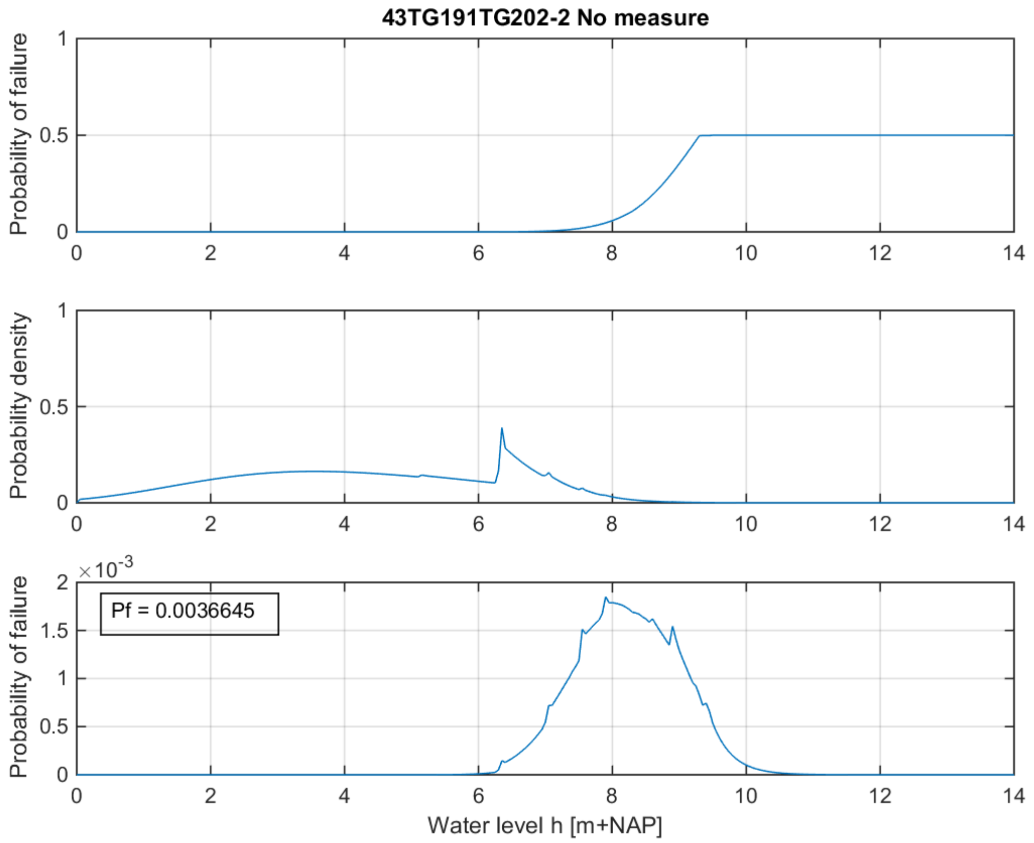

43TG191TG202-1 No measure
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
HR2006
Discharge at Lobith [m3/s]
Return period [year]


43TG191TG202-1 No measure
Probability of failure
Probability density
Pf = 0.000165
Water level h [m+NAP]

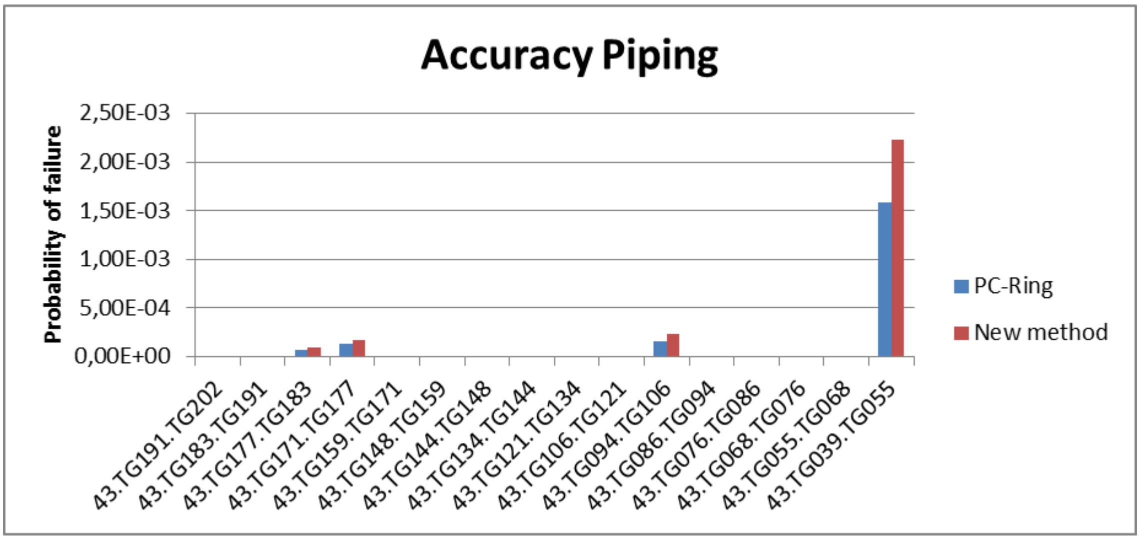
F.1.2 Macro stability

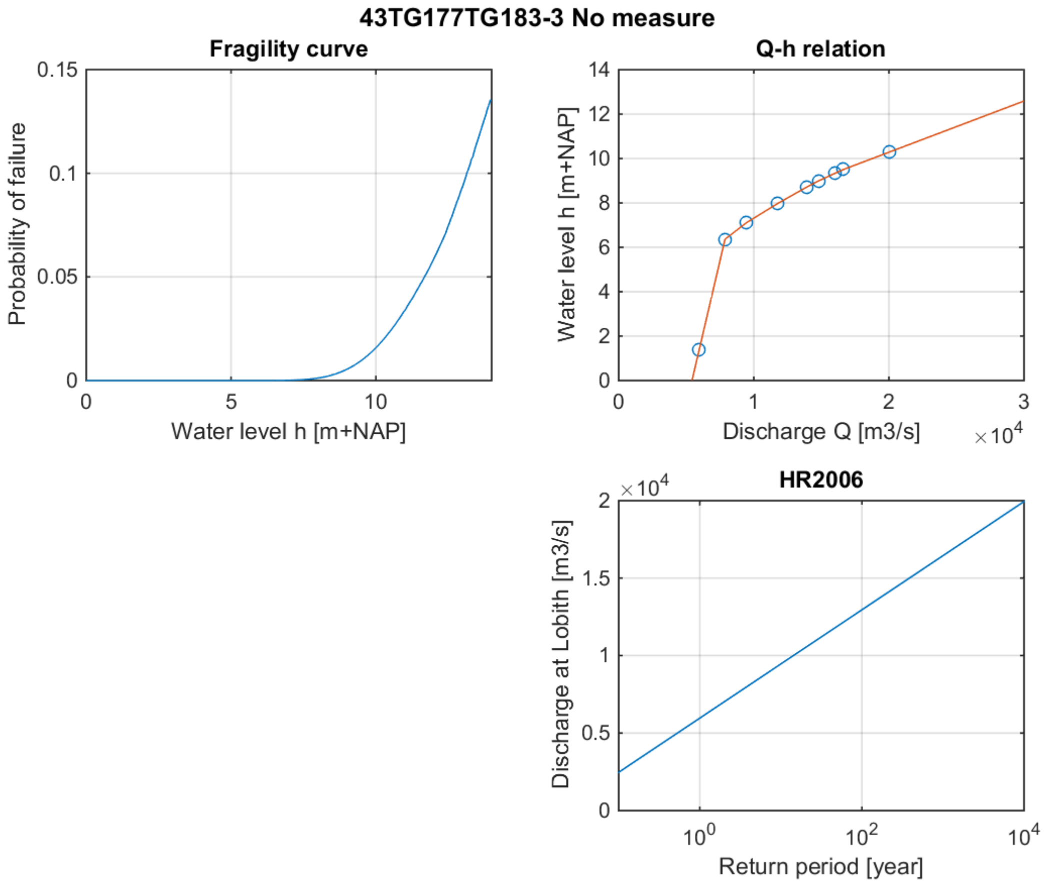


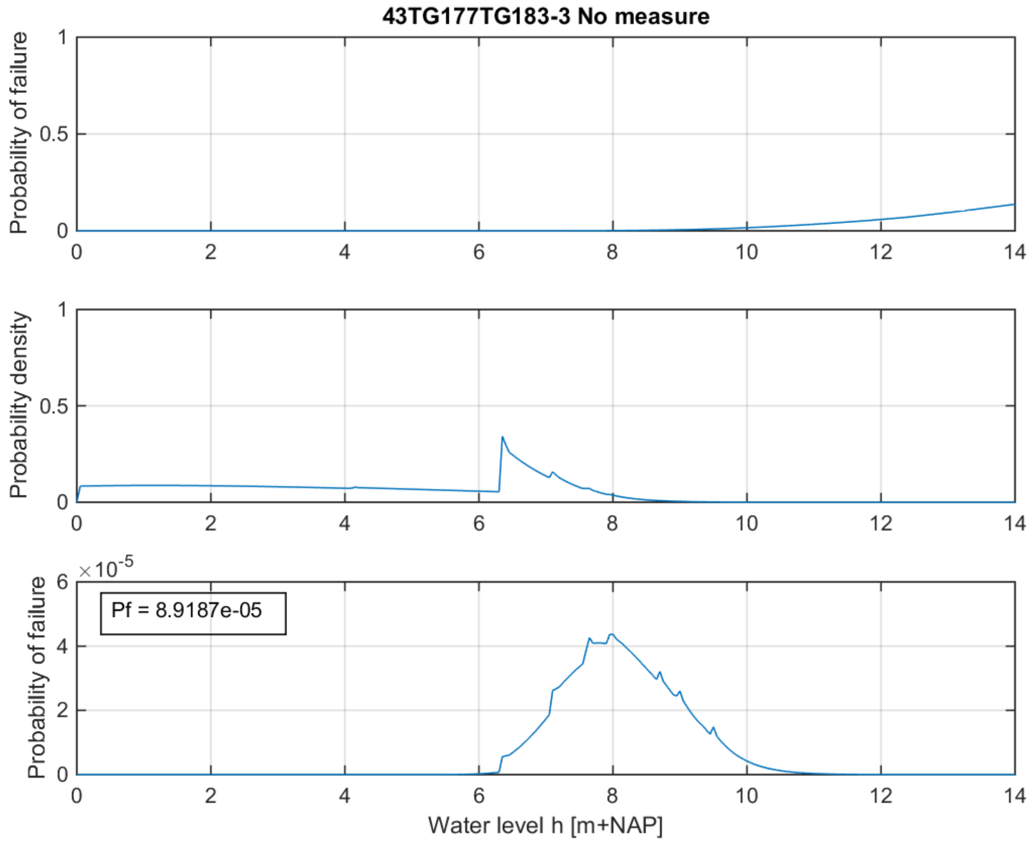



43TG191TG202-2 No measure
Probability of failure
Probability density
Probability of failure
Pf = 0.0036645
×10^-3
Water level h [m+NAP]

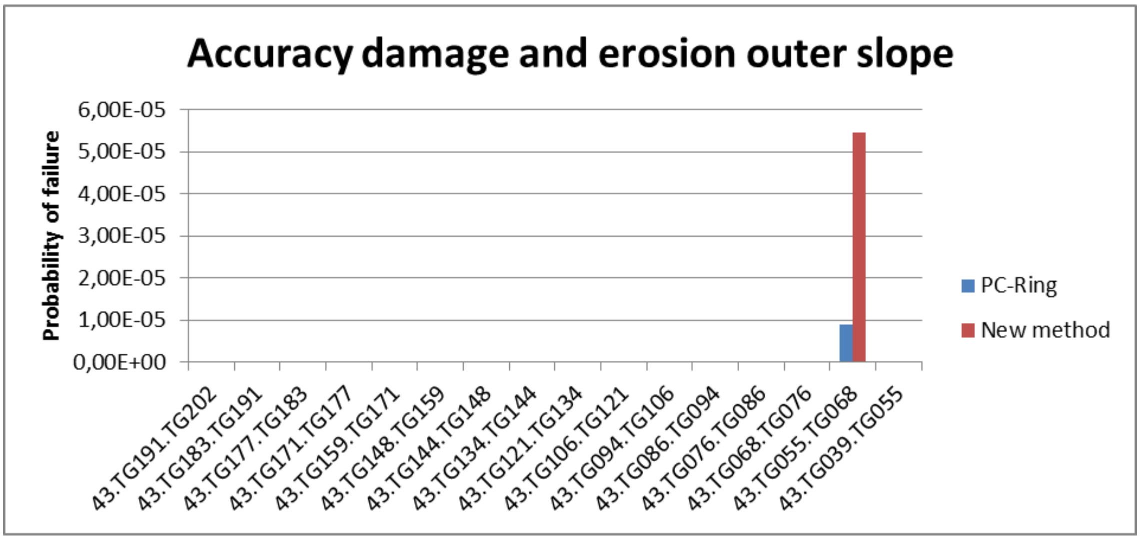
F.1.3 Piping


Accuracy Piping
Probability of failure
PC-Ring
New method
43.TG191.TG202
43.TG183.TG191
43.TG177.TG183
43.TG171.TG177
43.TG159.TG171
43.TG148.TG159
43.TG144.TG148
43.TG134.TG144
43.TG121.TG134
43.TG106.TG121
43.TG094.TG106
43.TG086.TG094
43.TG076.TG086
43.TG068.TG076
43.TG055.TG068
43.TG039.TG055

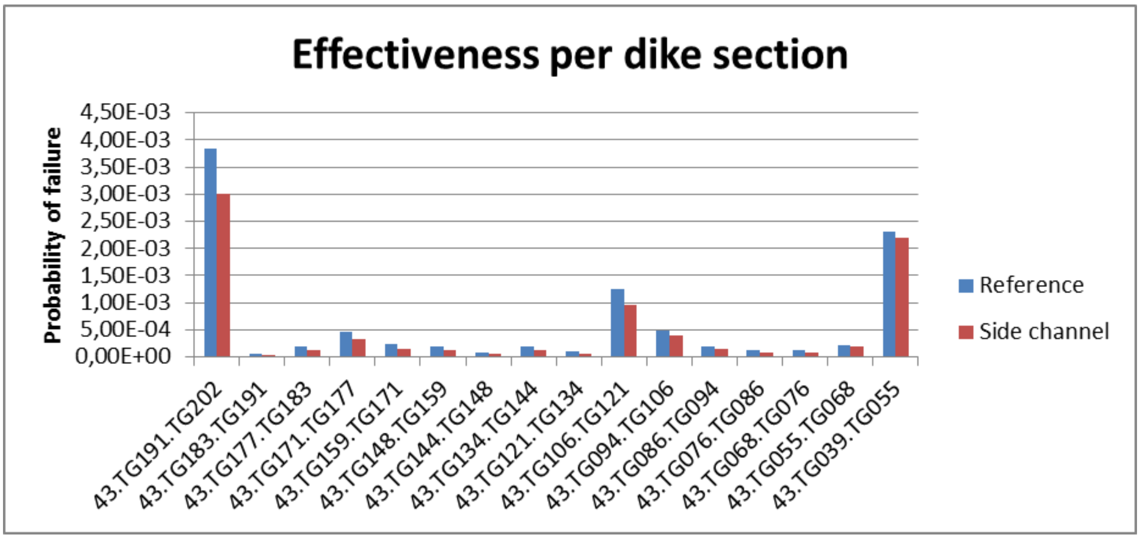

43TG177TG183-3 No measure
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
×10^4
HR2006
×10^4
Discharge at Lobith [m3/s]
Return period [year]


43TG177TG183-3 No measure
Probability of failure
Probability density
×10^-5
Probability of failure
Pf = 8.9187e-05
Water level h [m+NAP]

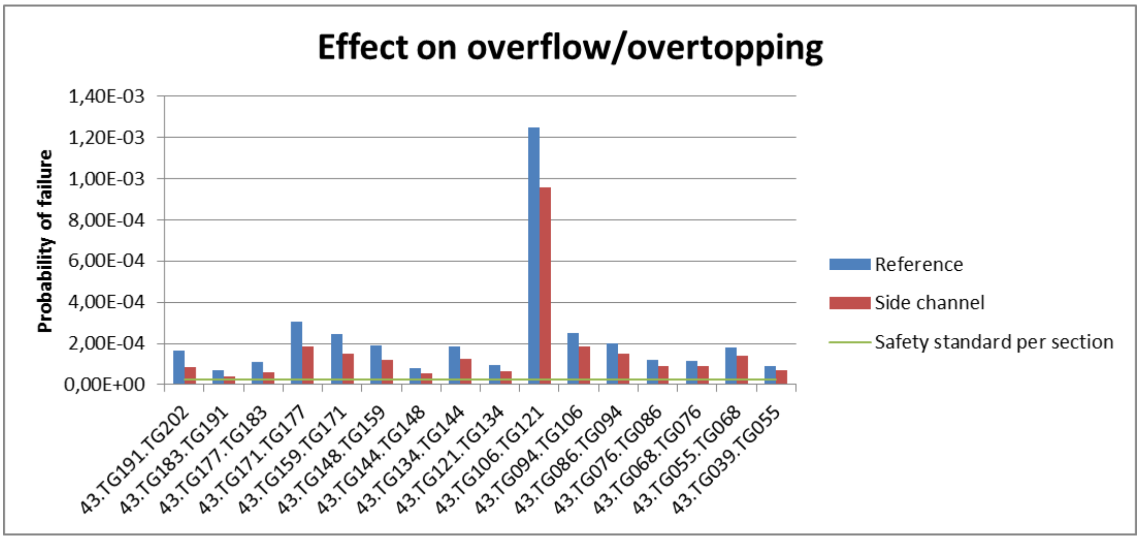
F.1.4 Damage and erosion outer slope

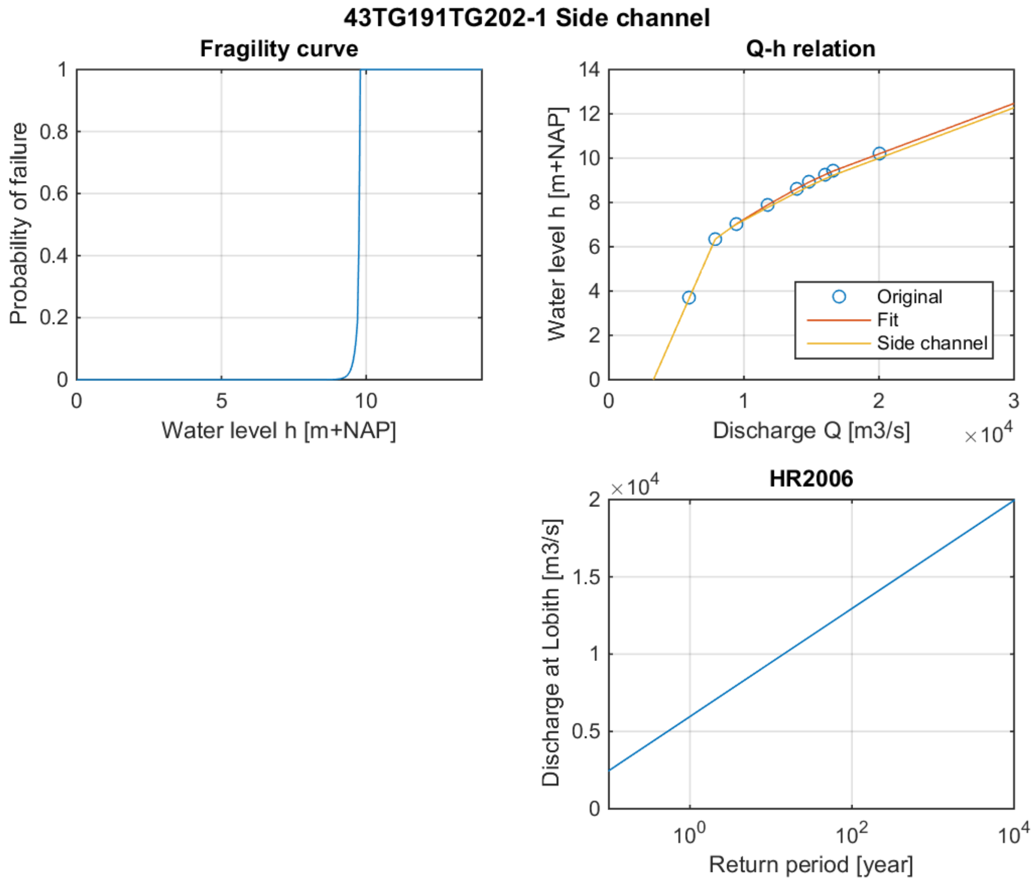


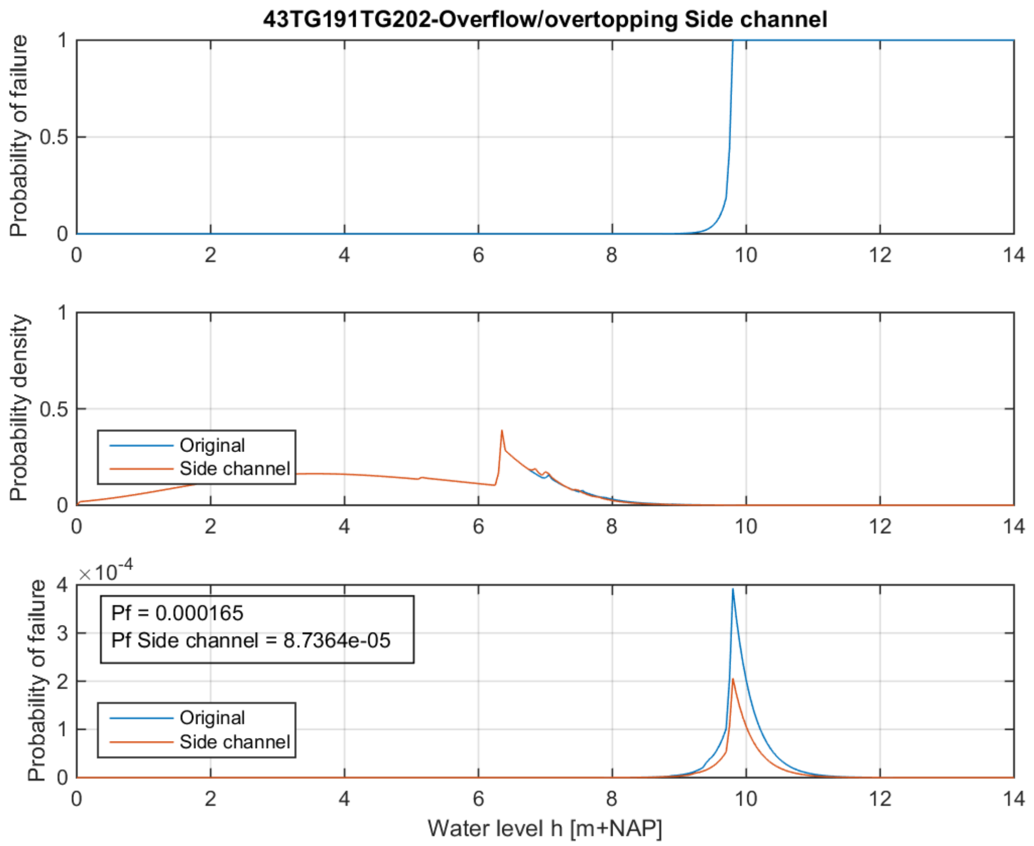
F.2 Side channel



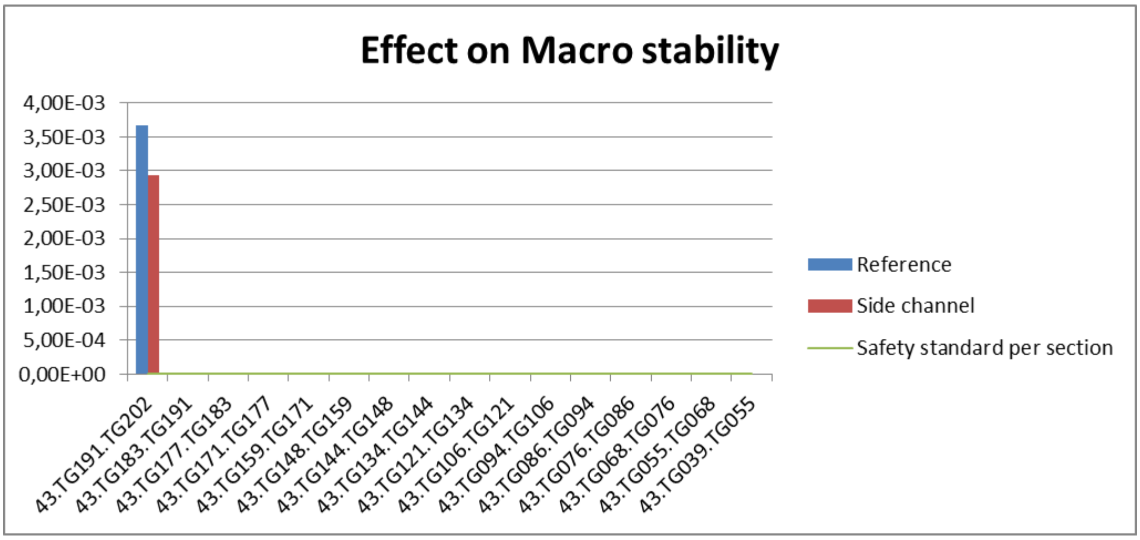
F.2.1 Overflow/overtopping

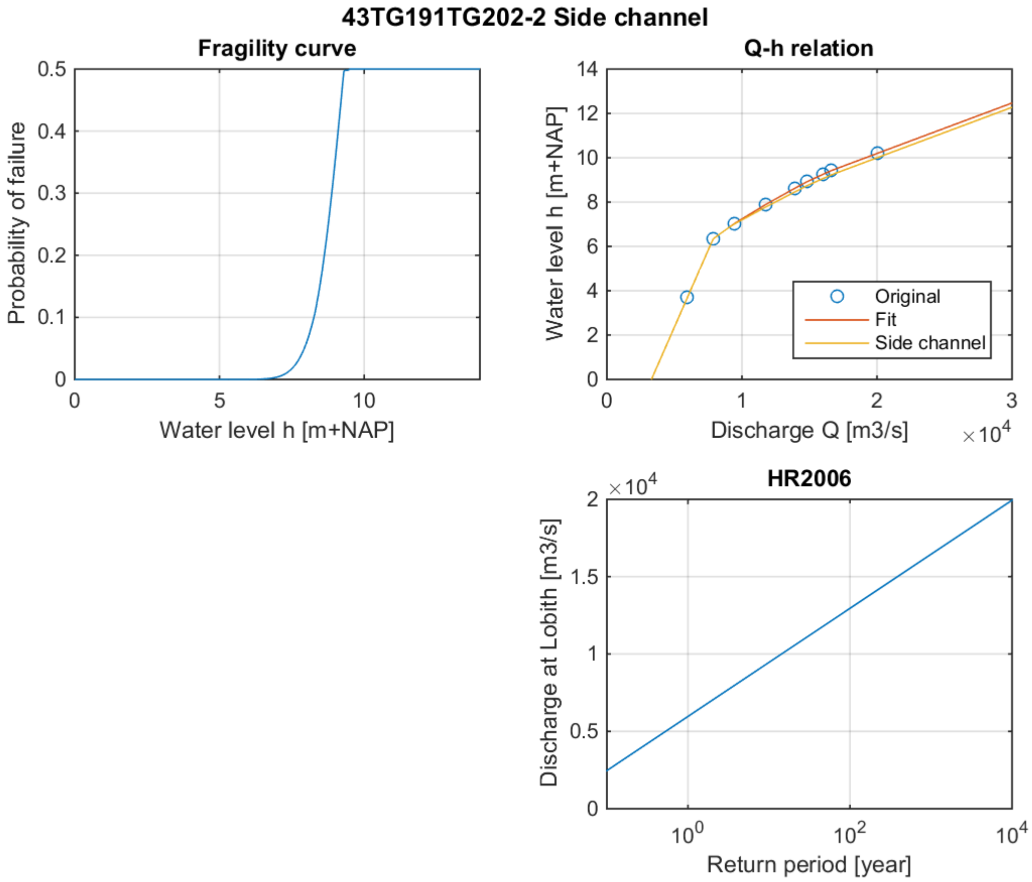


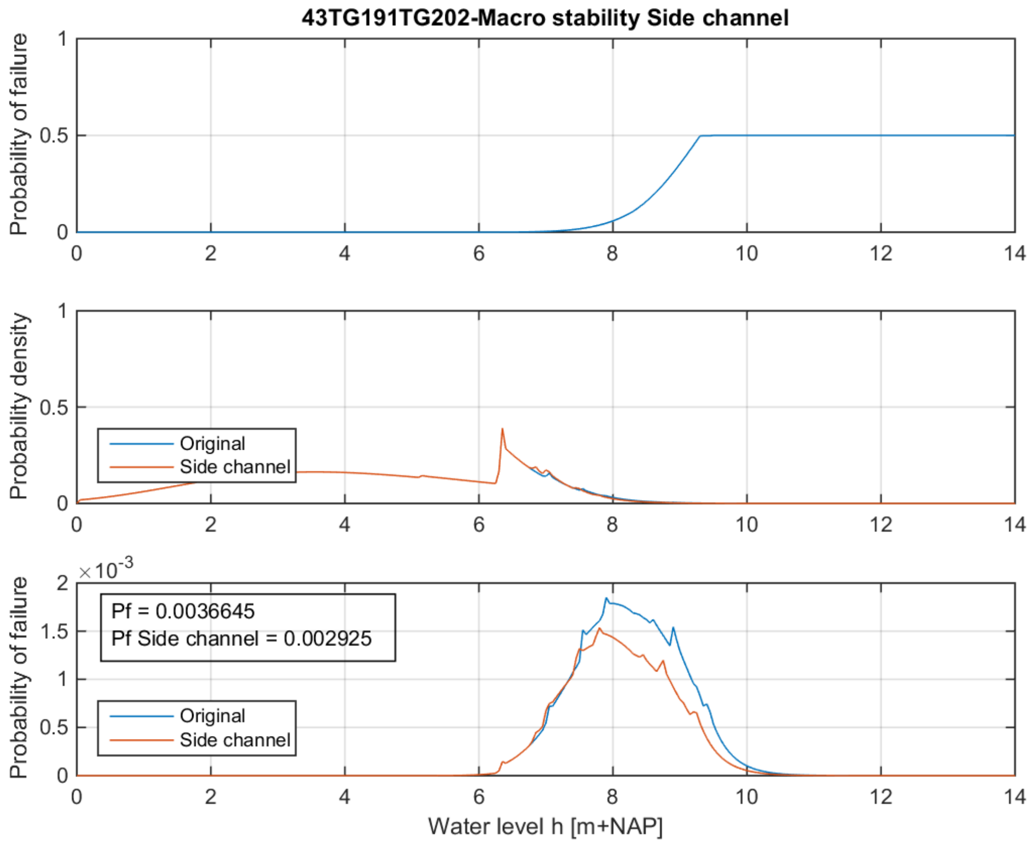

43TG191TG202-1 Side channel
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
×10^4
Original
Fit
Side channel
HR2006
×10^4
Discharge at Lobith [m3/s]
Return period [year]


43TG191TG202-Overflow/overtopping Side channel
Probability of failure
Probability density
Original
Side channel
×10^-4
Probability of failure
Pf = 0.000165
Pf Side channel = 8.7364e-05
Original
Side channel
Water level h [m+NAP]

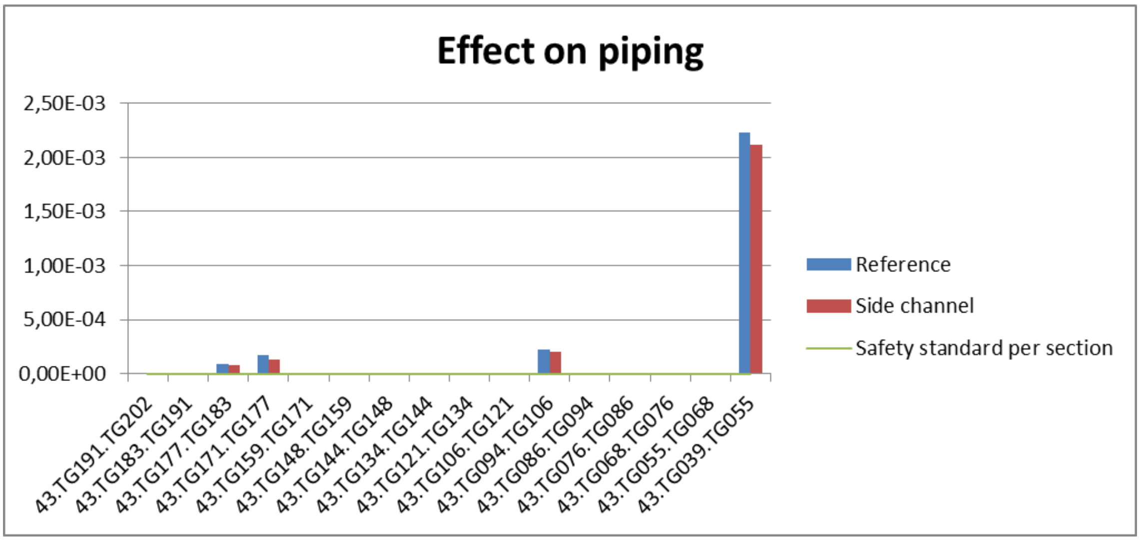
F.2.2 Macro stability

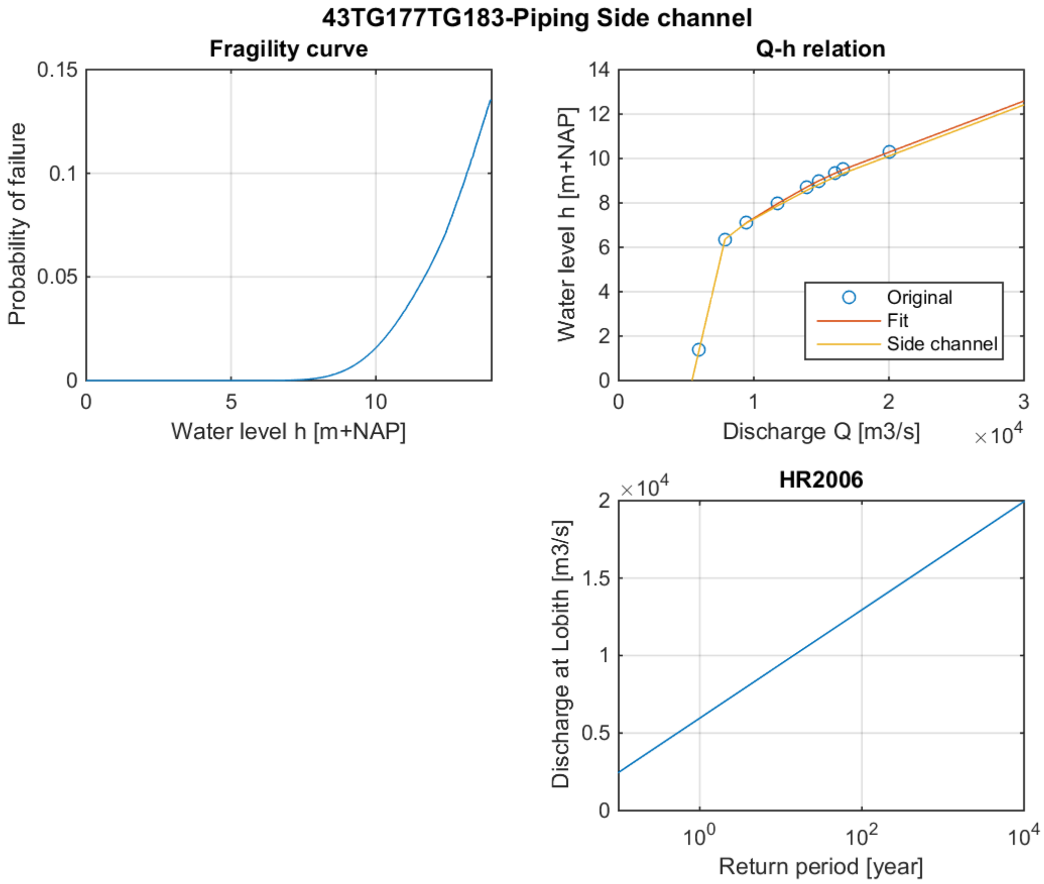


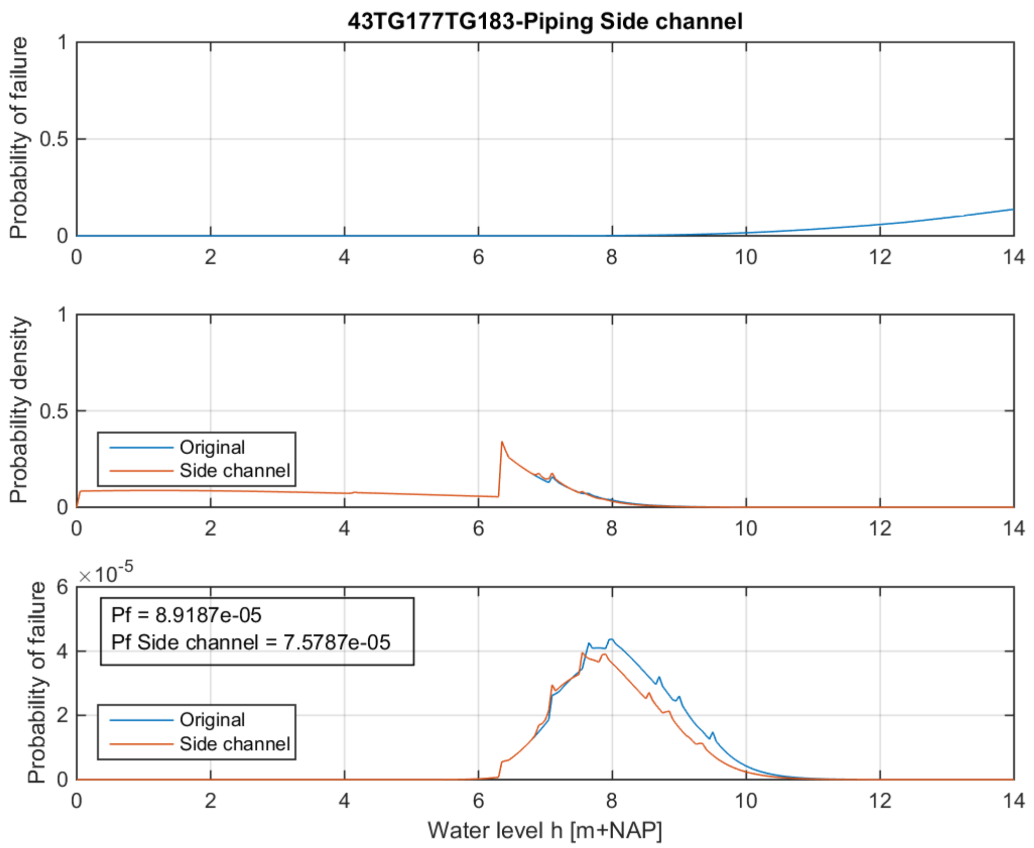



43TG191TG202-Macro stability Side channel
Probability of failure
Probability density
Original
Side channel
×10^-3
Pf = 0.0036645
Pf Side channel = 0.002925
Water level h [m+NAP]

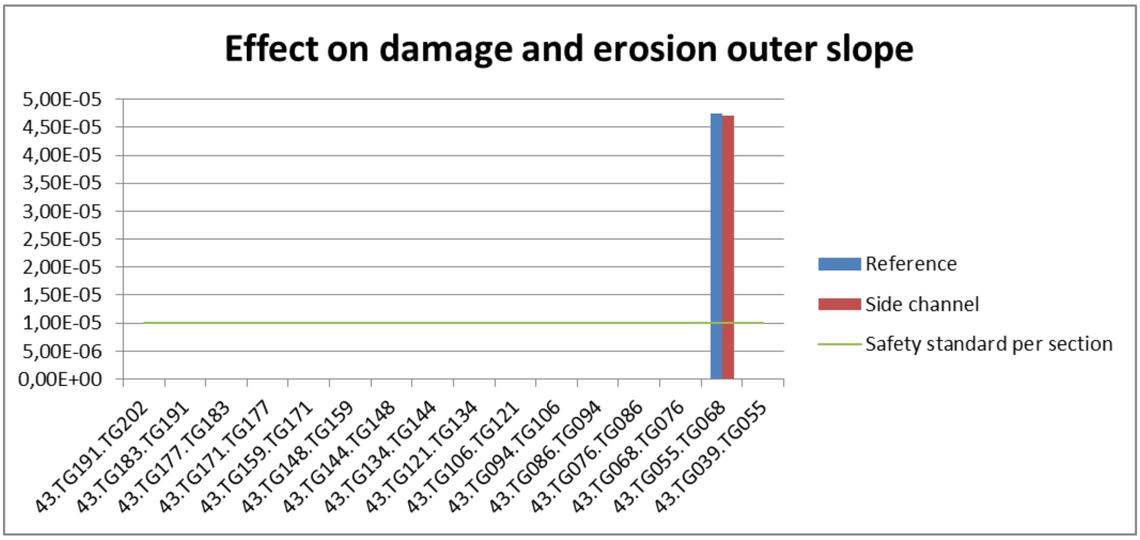
F.2.3 Piping


Effect on piping
2,50E-03
2,00E-03
1,50E-03
1,00E-03
5,00E-04
0,00E+00
43.TG191.TG202
43.TG183.TG191
43.TG177.TG183
43.TG171.TG177
43.TG159.TG171
43.TG148.TG159
43.TG144.TG148
43.TG134.TG144
43.TG121.TG134
43.TG106.TG121
43.TG094.TG106
43.TG086.TG094
43.TG076.TG086
43.TG068.TG076
43.TG055.TG068
43.TG039.TG055
Reference
Side channel
Safety standard per section

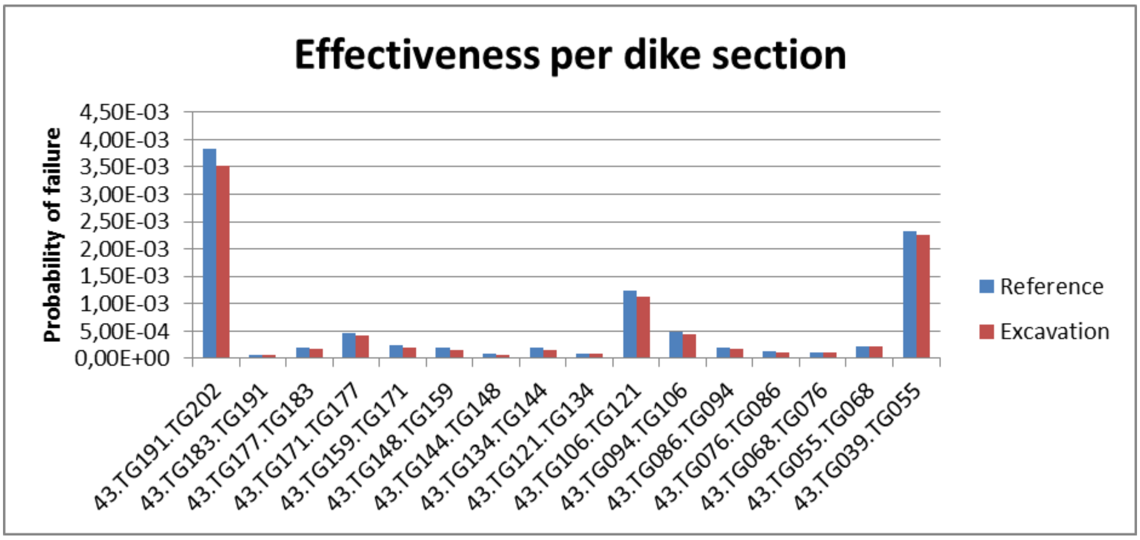

43TG177TG183-Piping Side channel
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
Original
Fit
Side channel
HR2006
Discharge at Lobith [m3/s]
Return period [year]


43TG177TG183-Piping Side channel
Probability of failure
Probability density
Original
Side channel
Probability of failure
Pf = 8.9187e-05
Pf Side channel = 7.5787e-05
Original
Side channel
Water level h [m+NAP]

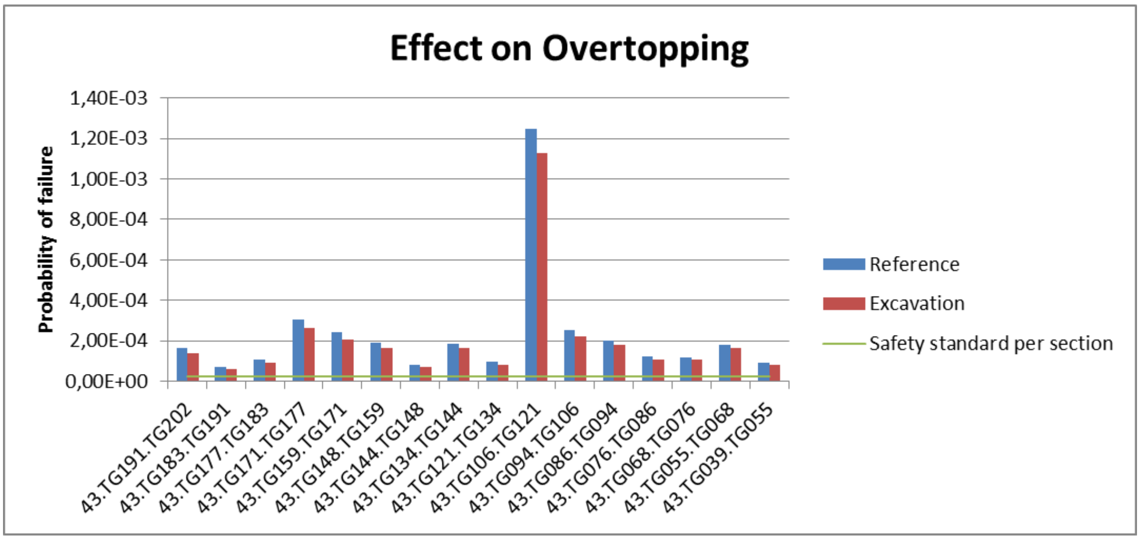
F.2.4 Damage and erosion outer slope

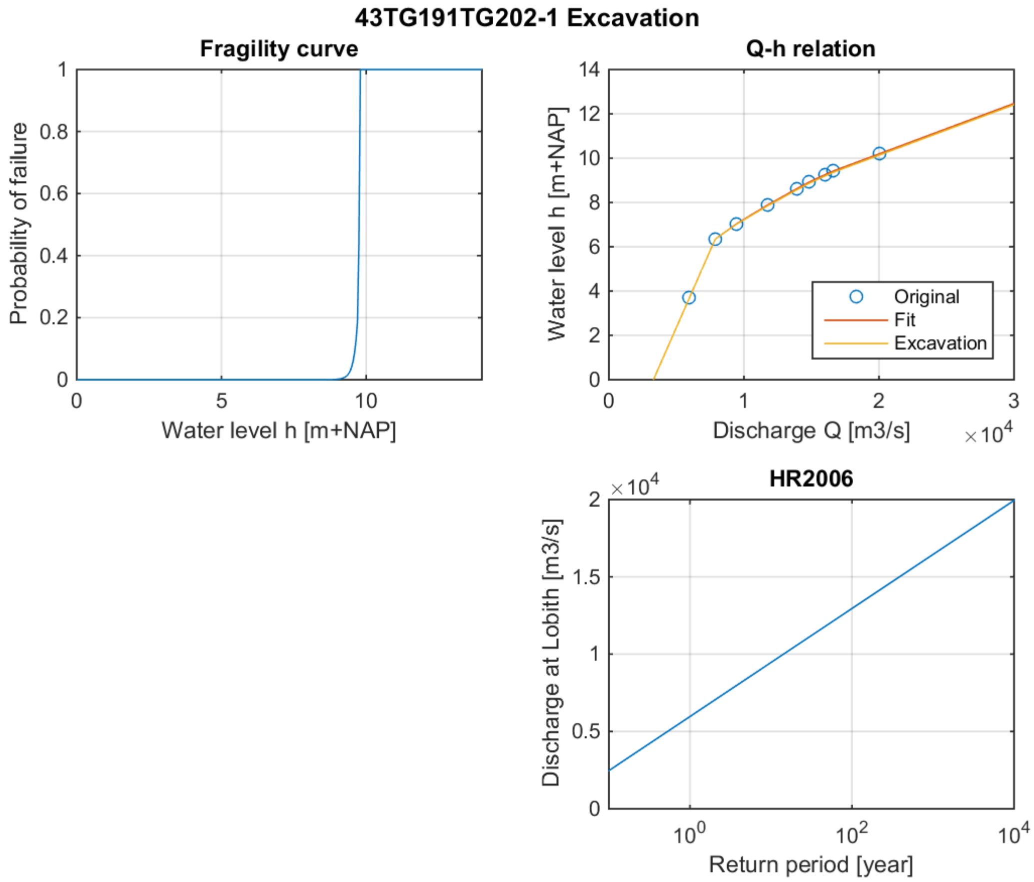


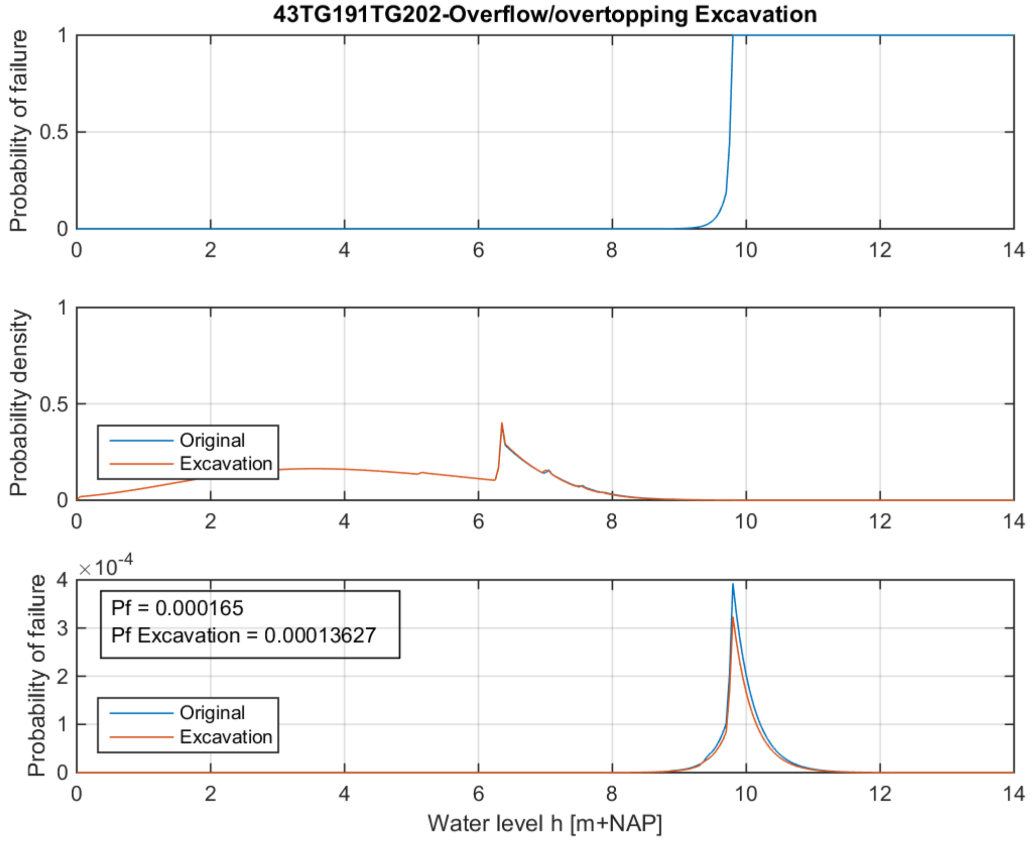
F.3 Excavation



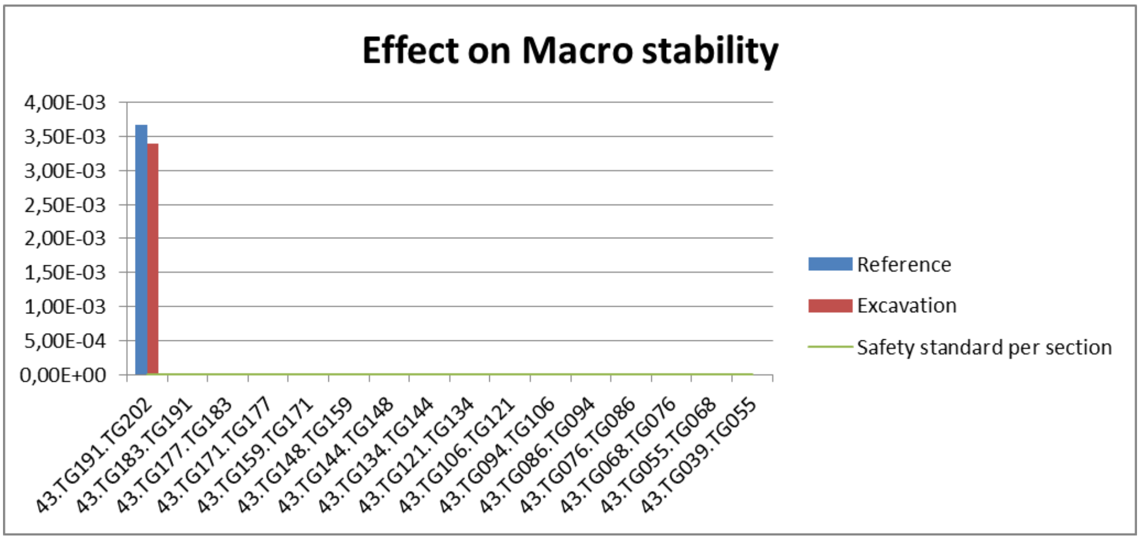
F.3.1 Overflow/overtopping

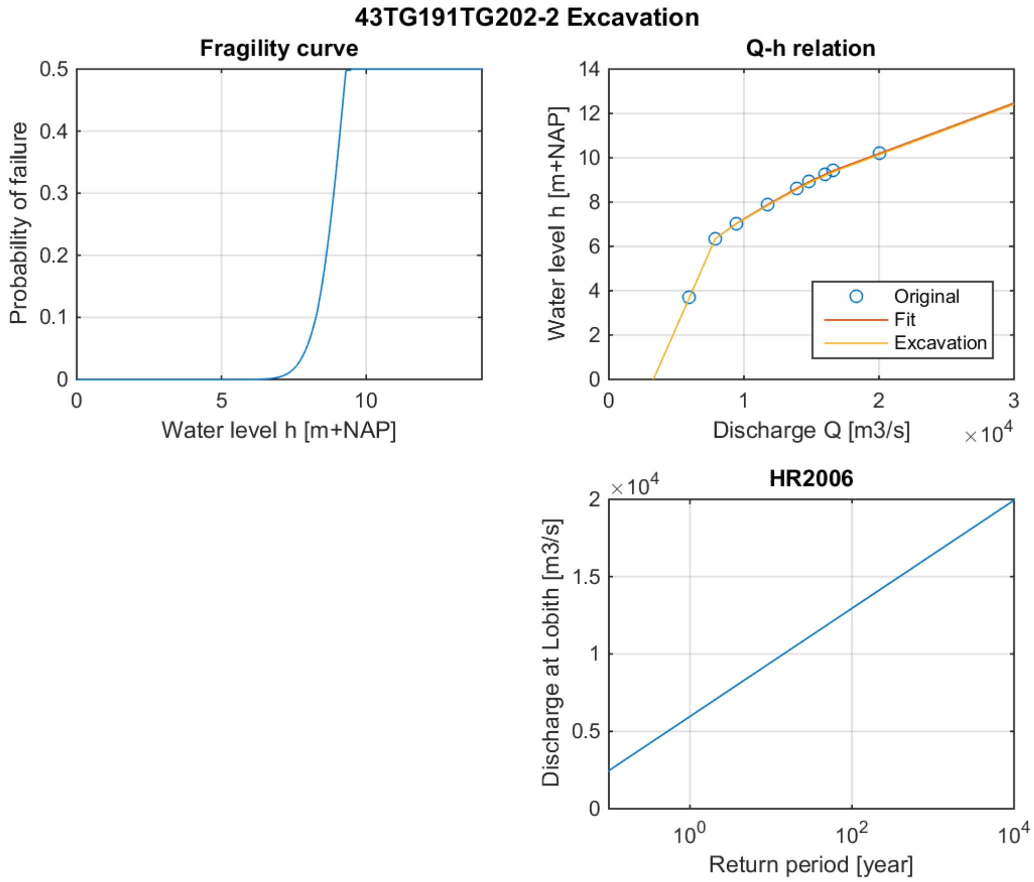


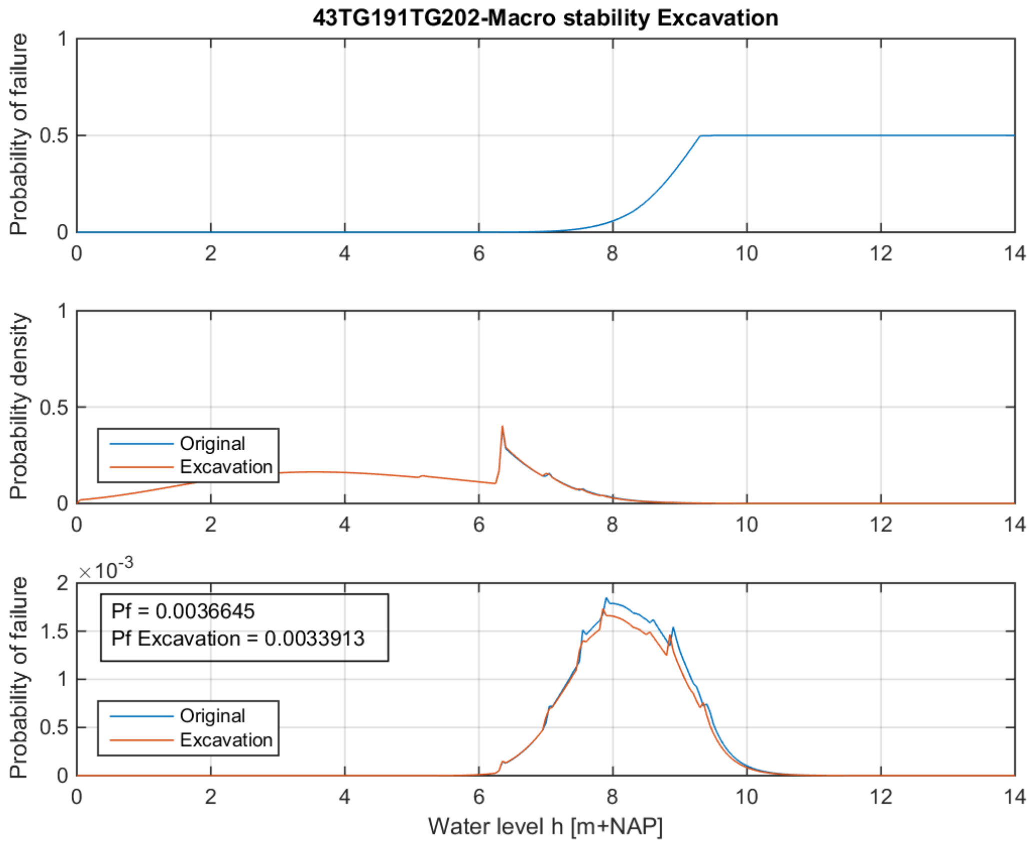

43TG191TG202-1 Excavation
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
×10^4
Original
Fit
Excavation
HR2006
×10^4
Discharge at Lobith [m3/s]
Return period [year]


43TG191TG202-Overflow/overtopping Excavation
Probability of failure
Probability density
Original
Excavation
×10^-4
Probability of failure
Pf = 0.000165
Pf Excavation = 0.00013627
Original
Excavation
Water level h [m+NAP]

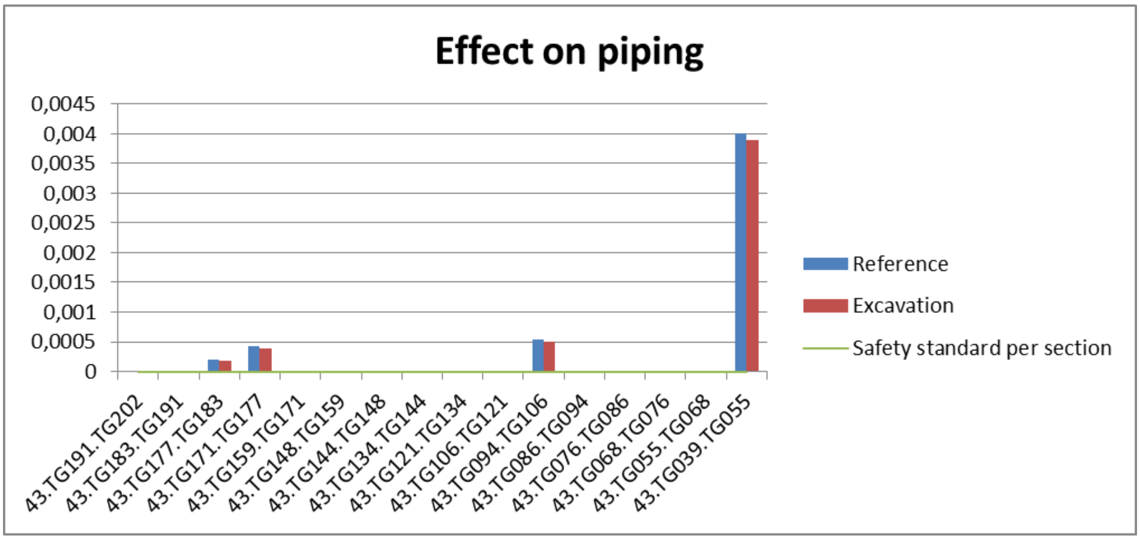
F.3.2 Macro stability

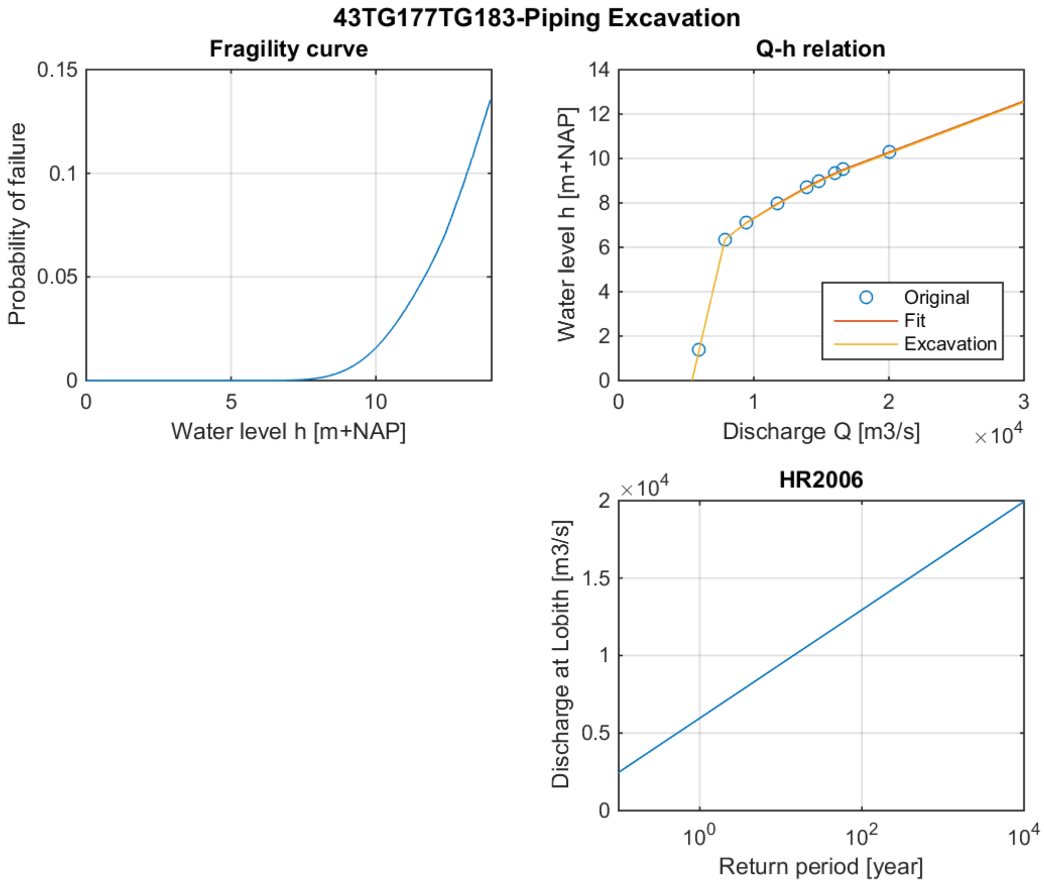


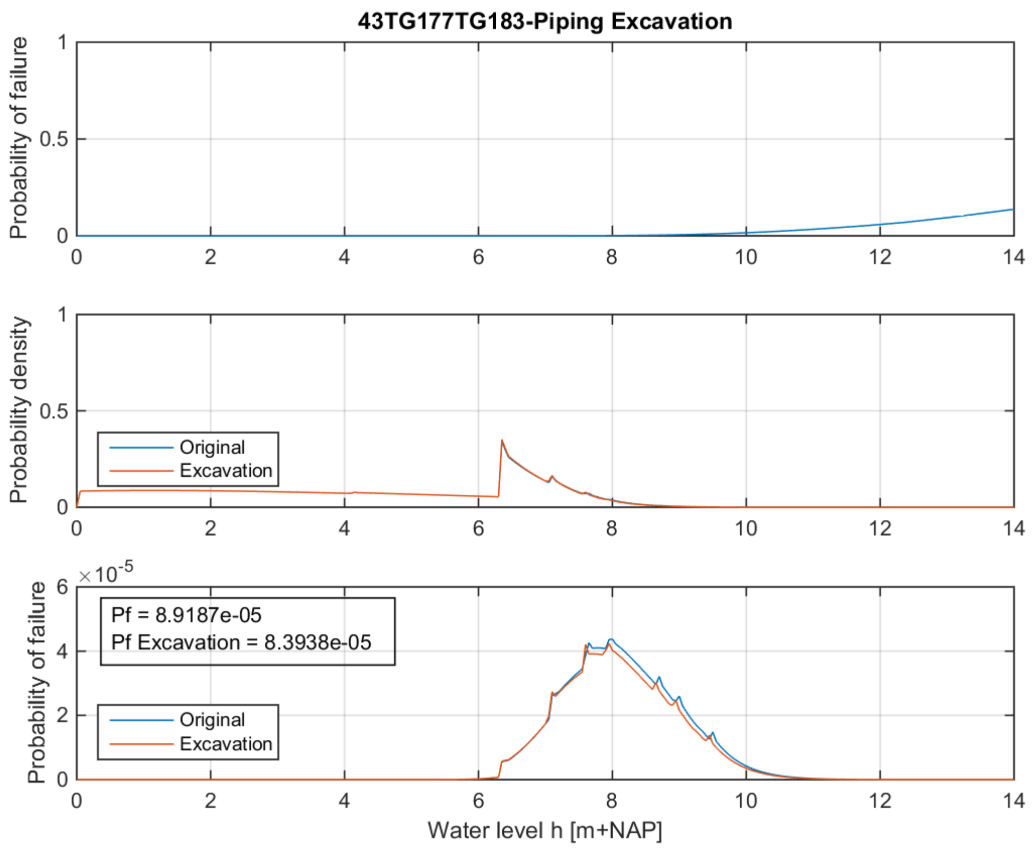


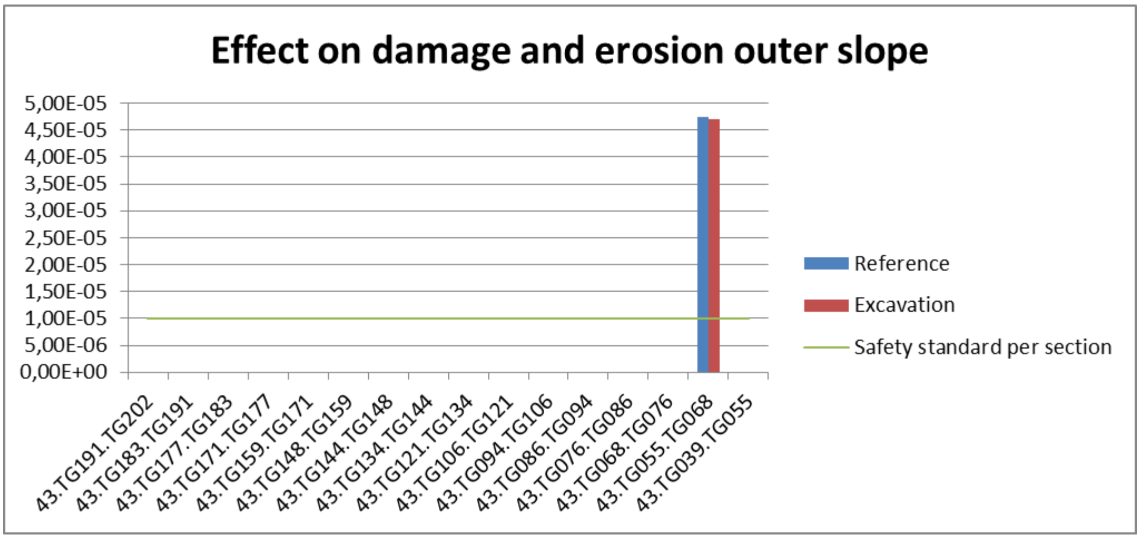

43TG191TG202-Macro stability Excavation
Probability of failure
Probability density
Original
Excavation
Pf = 0.0036645
Pf Excavation = 0.0033913
Original
Excavation
Water level h [m+NAP]

F.3.3 Piping

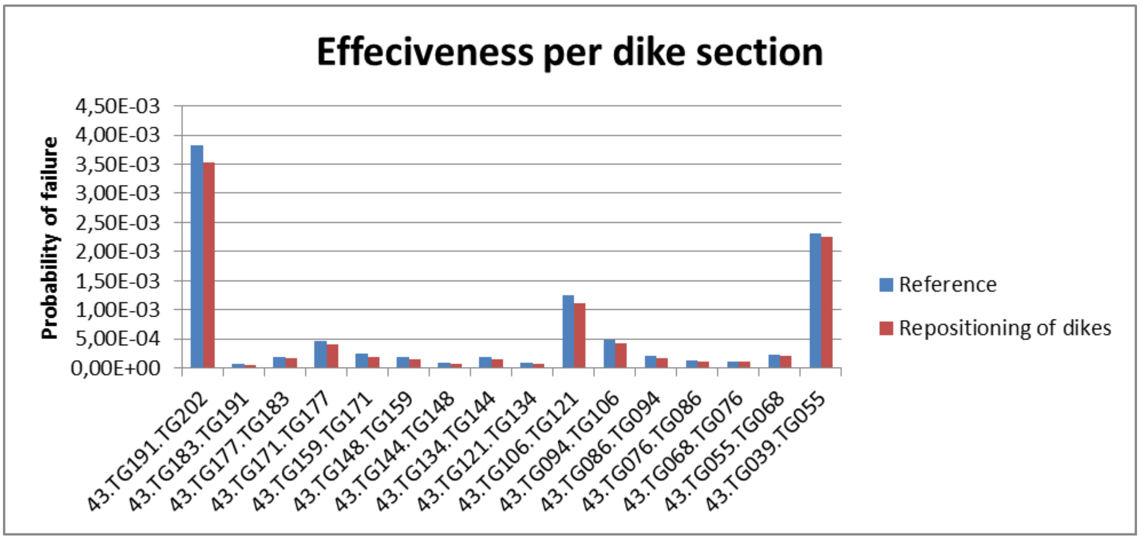

Effect on piping
Reference
Excavation
Safety standard per section
43.TG191.TG202
43.TG183.TG191
43.TG177.TG183
43.TG171.TG177
43.TG159.TG171
43.TG148.TG159
43.TG144.TG148
43.TG134.TG144
43.TG121.TG134
43.TG106.TG121
43.TG094.TG106
43.TG086.TG094
43.TG076.TG086
43.TG068.TG076
43.TG055.TG068
43.TG039.TG055


43TG177TG183-Piping Excavation
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
Original
Fit
Excavation
HR2006
Discharge at Lobith [m3/s]
Return period [year]

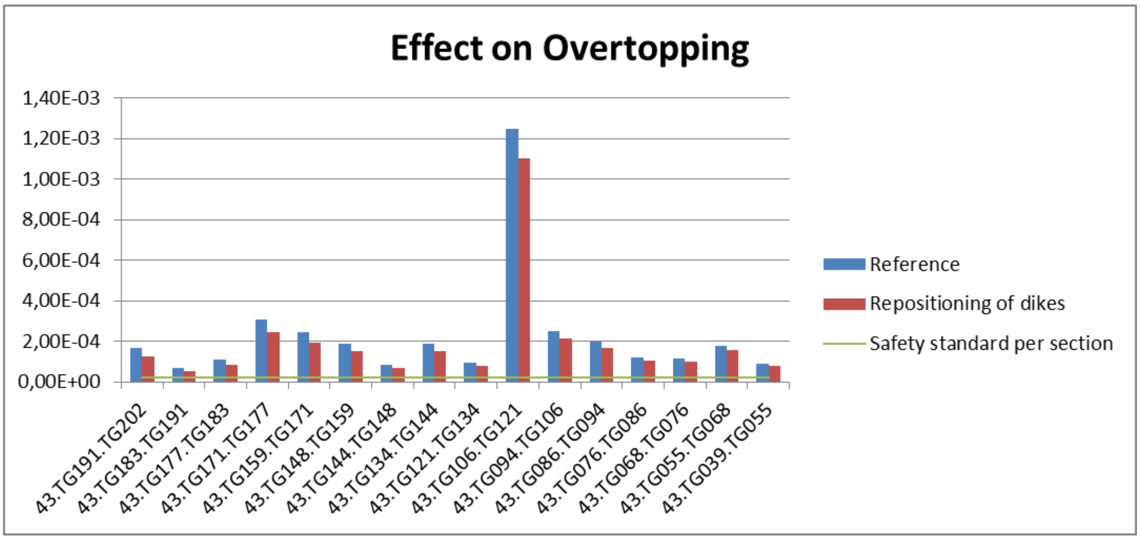

43TG177TG183-Piping Excavation
Probability of failure
Probability density
Original
Excavation
Pf = 8.9187e-05
Pf Excavation = 8.3938e-05
Original
Excavation
Water level h [m+NAP]

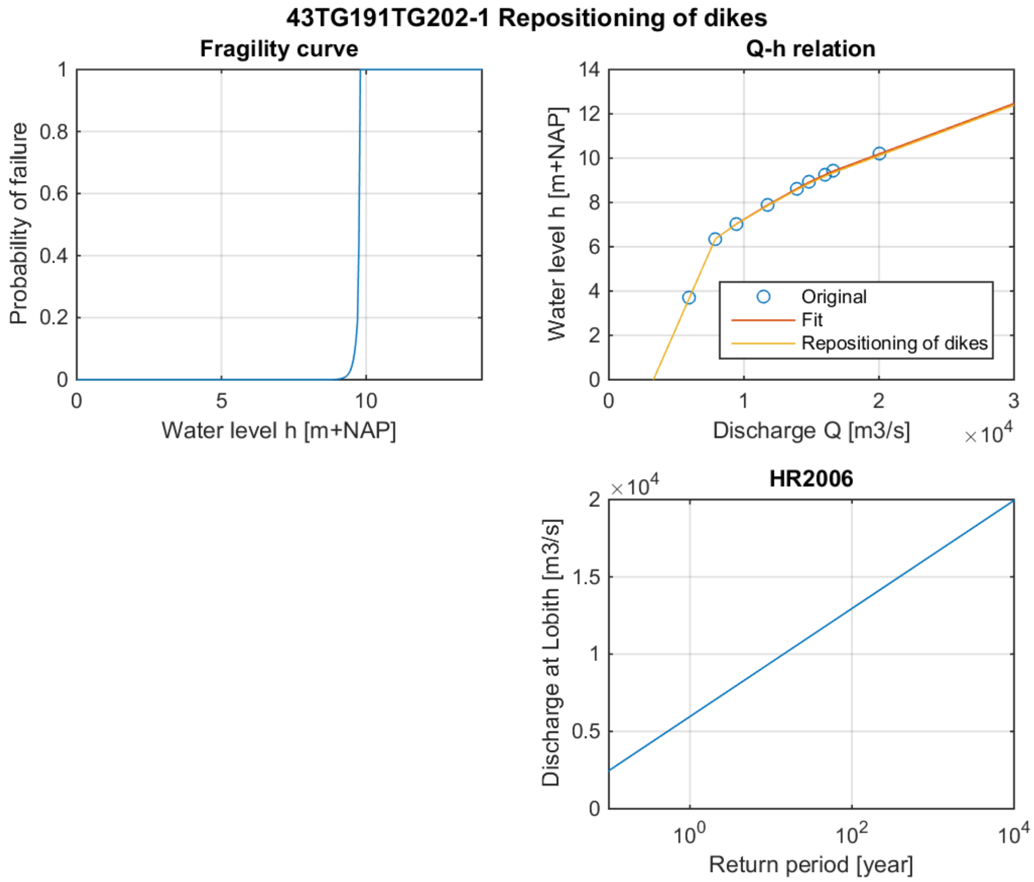


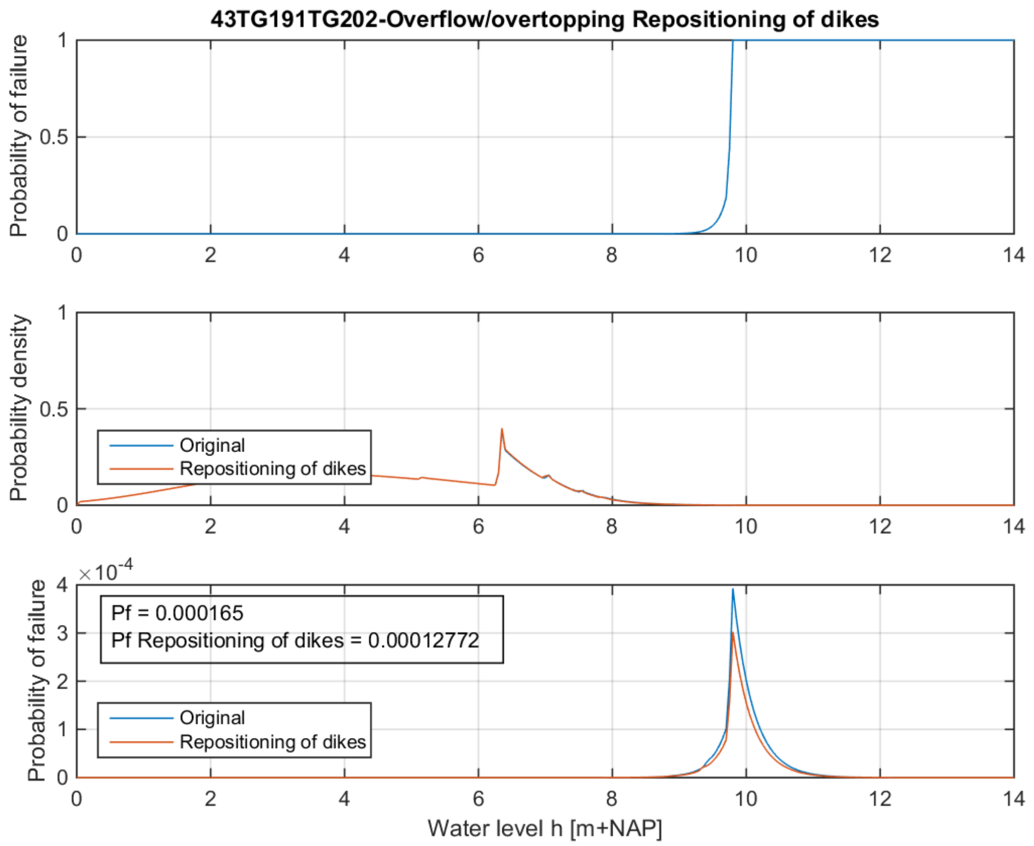
F.4 Repositioning of dikes



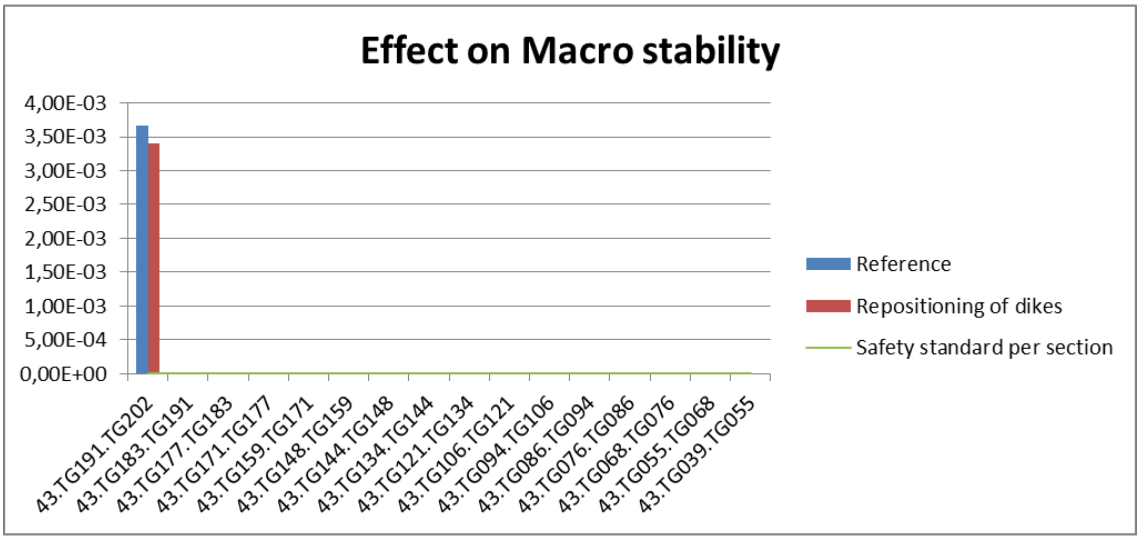
F.4.1 Overflow/overtopping

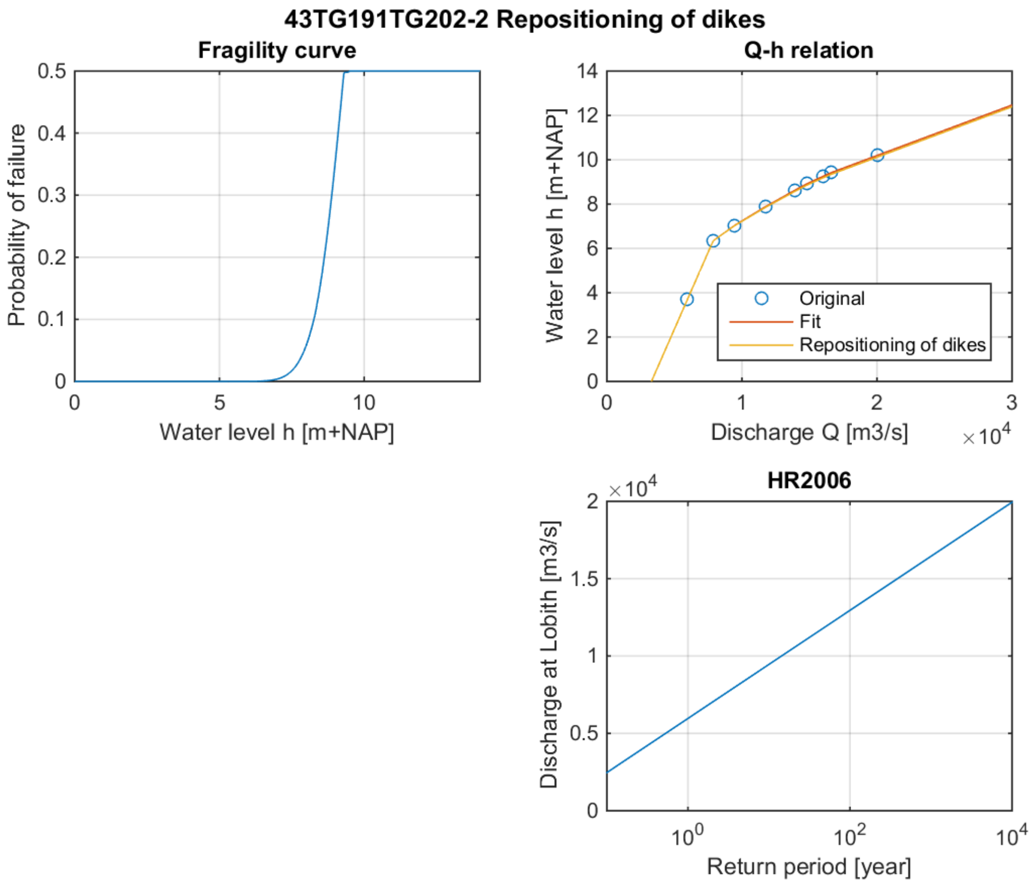


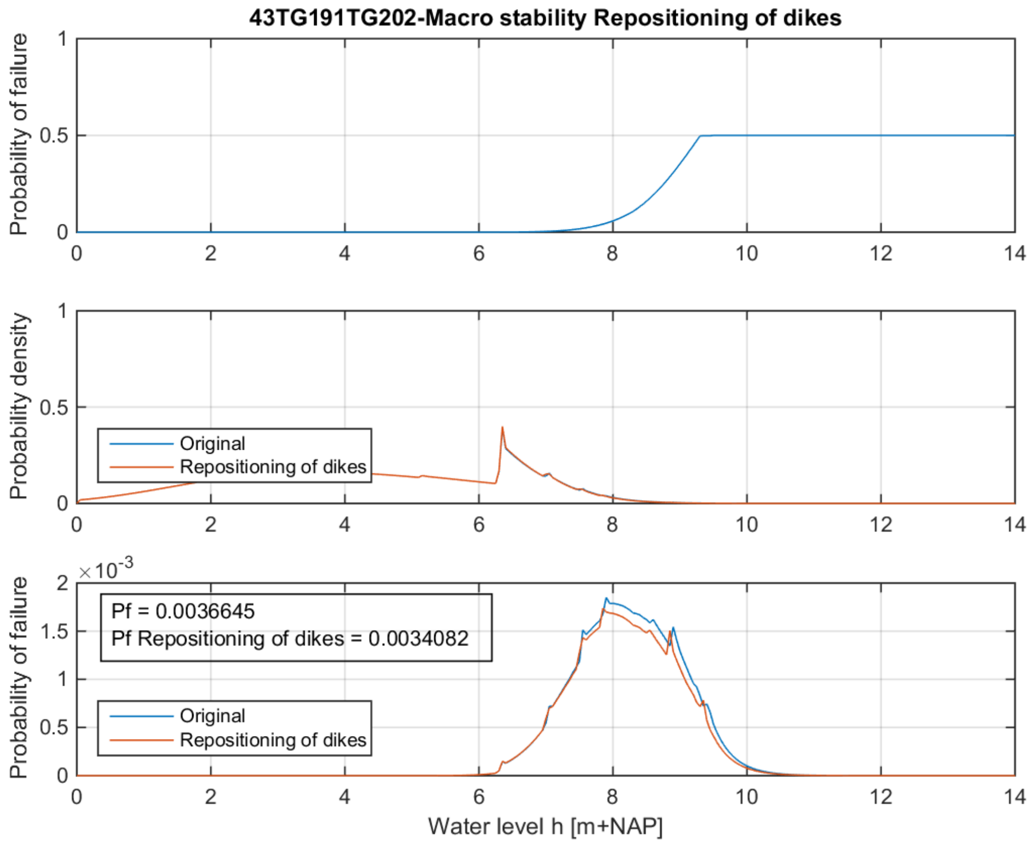

43TG191TG202-1 Repositioning of dikes
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
Original
Fit
Repositioning of dikes
HR2006
Discharge at Lobith [m3/s]
Return period [year]


43TG191TG202-Overflow/overtopping Repositioning of dikes
Probability of failure
Probability density
Original
Repositioning of dikes
Pf = 0.000165
Pf Repositioning of dikes = 0.00012772
Probability of failure
Original
Repositioning of dikes
Water level h [m+NAP]

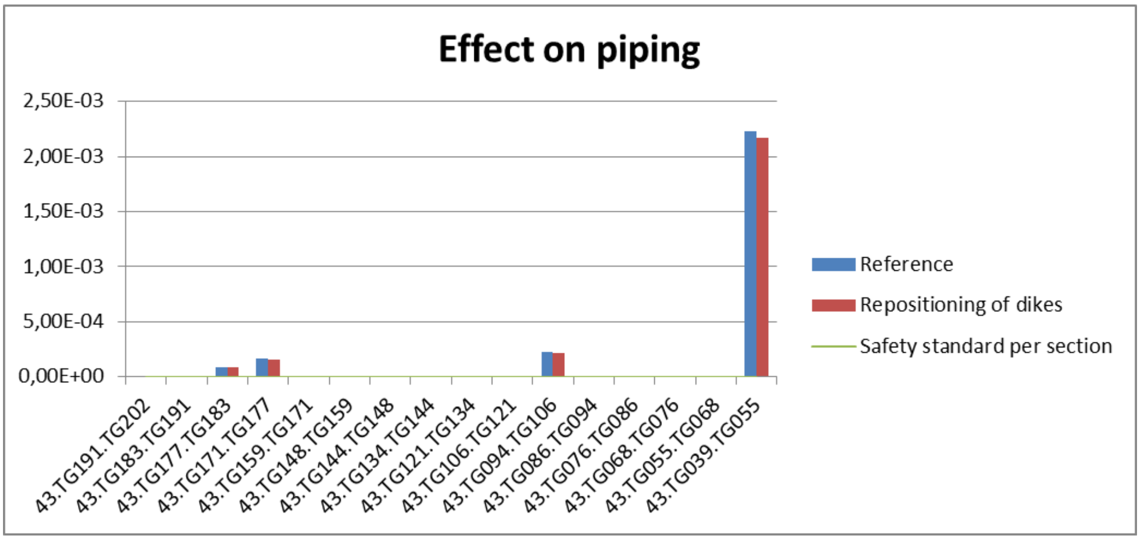
F.4.2 Macro stability

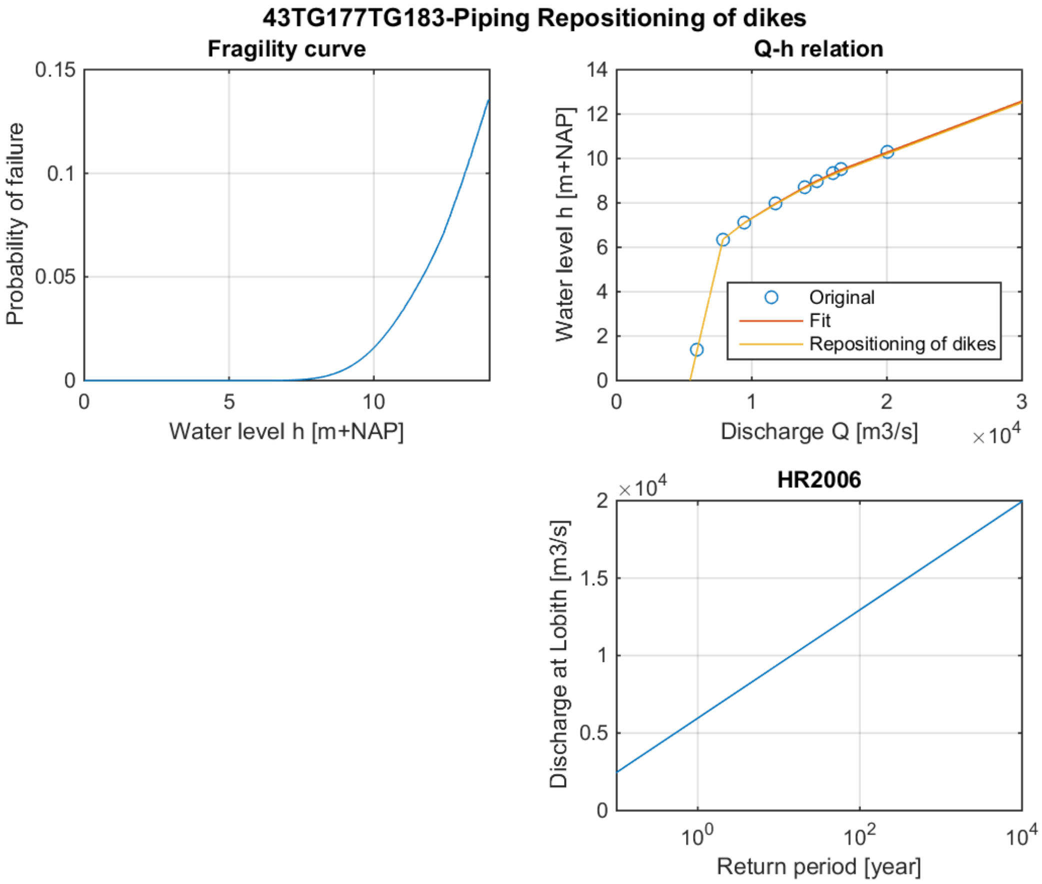


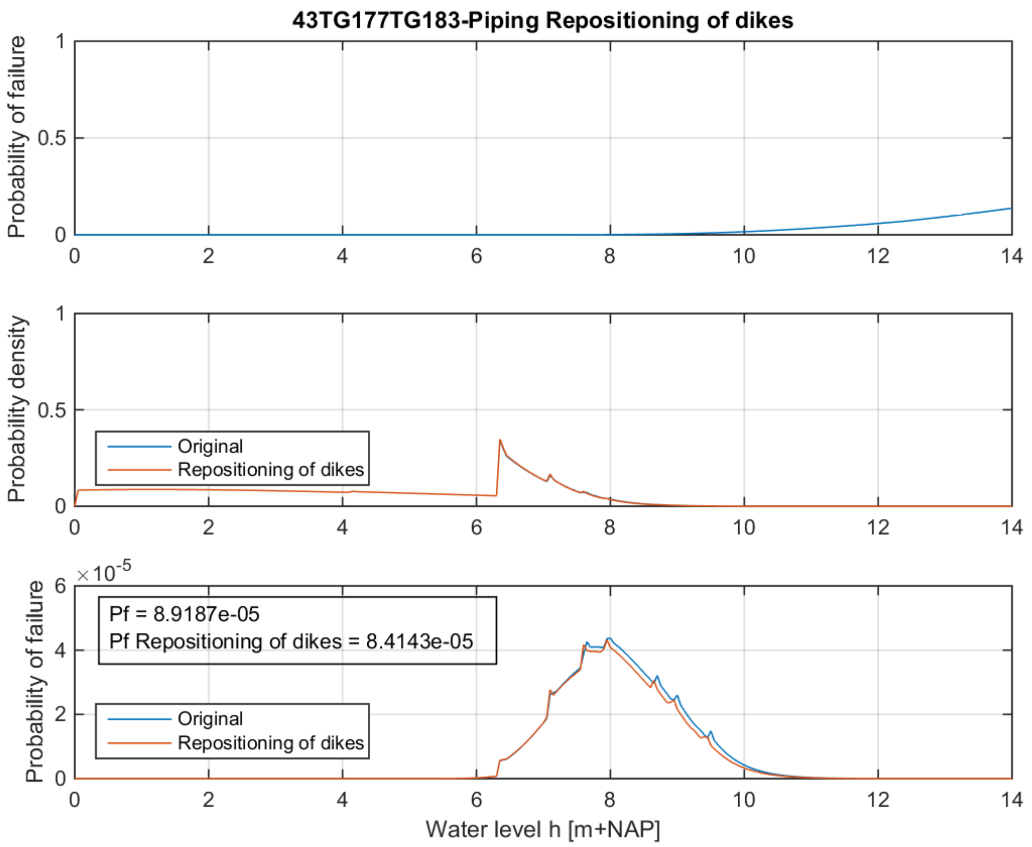



43TG191TG202-Macro stability Repositioning of dikes
Probability of failure
Probability density
Original
Repositioning of dikes
Pf = 0.0036645
Pf Repositioning of dikes = 0.0034082
Water level h [m+NAP]

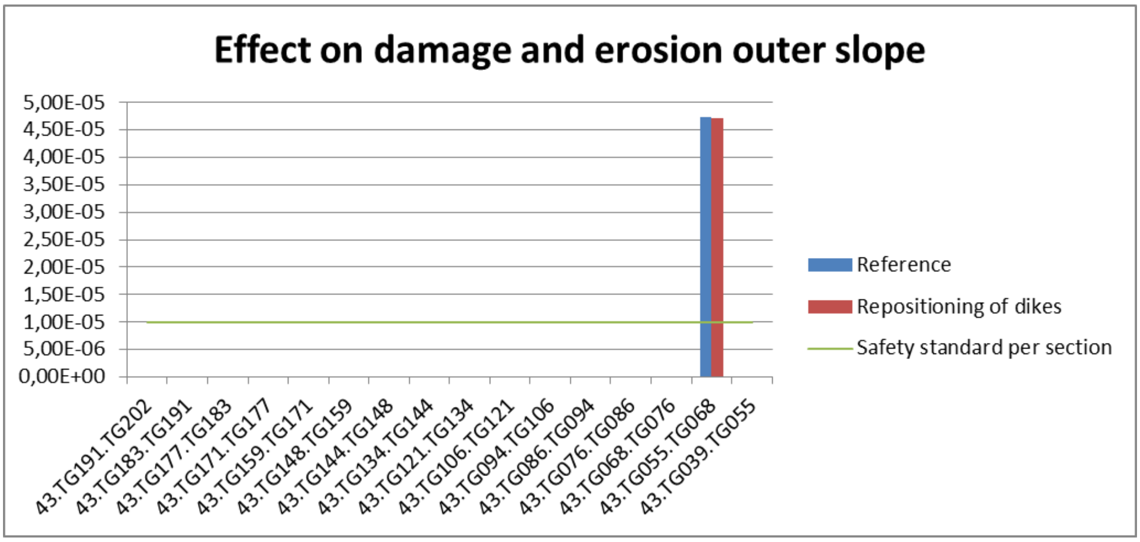
F.4.3 Piping


Effect on piping
Reference
Repositioning of dikes
Safety standard per section
43.TG191.TG202
43.TG183.TG191
43.TG177.TG183
43.TG171.TG177
43.TG159.TG171
43.TG148.TG159
43.TG144.TG148
43.TG134.TG144
43.TG121.TG134
43.TG106.TG121
43.TG094.TG106
43.TG086.TG094
43.TG076.TG086
43.TG068.TG076
43.TG055.TG068
43.TG039.TG055

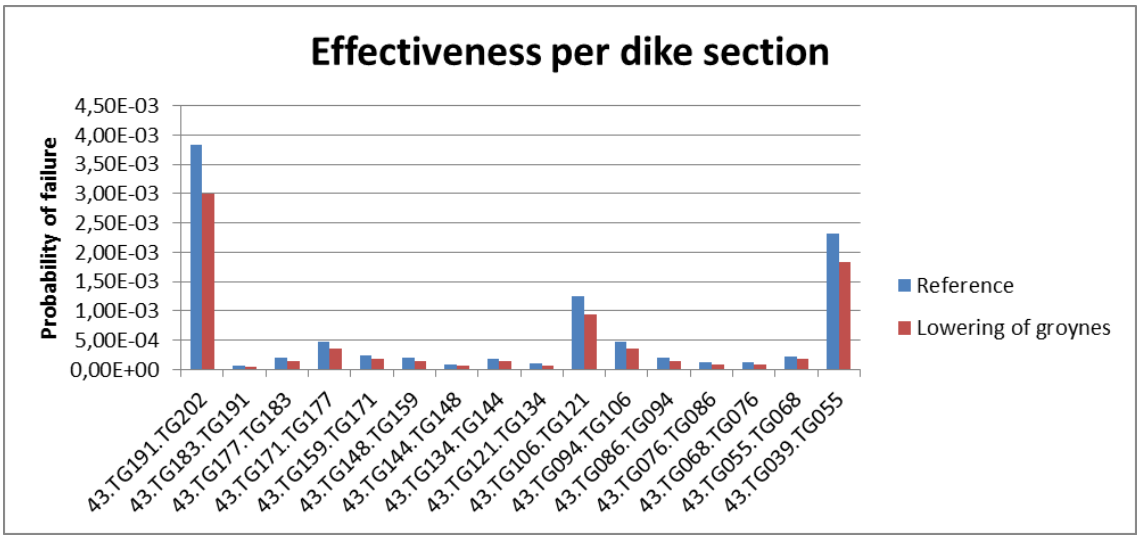

43TG177TG183-Piping Repositioning of dikes
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
Original
Fit
Repositioning of dikes
HR2006
Discharge at Lobith [m3/s]
Return period [year]


43TG177TG183-Piping Repositioning of dikes
Probability of failure
Probability density
Original
Repositioning of dikes
Pf = 8.9187e-05
Pf Repositioning of dikes = 8.4143e-05
Original
Repositioning of dikes
Probability of failure
Water level h [m+NAP]

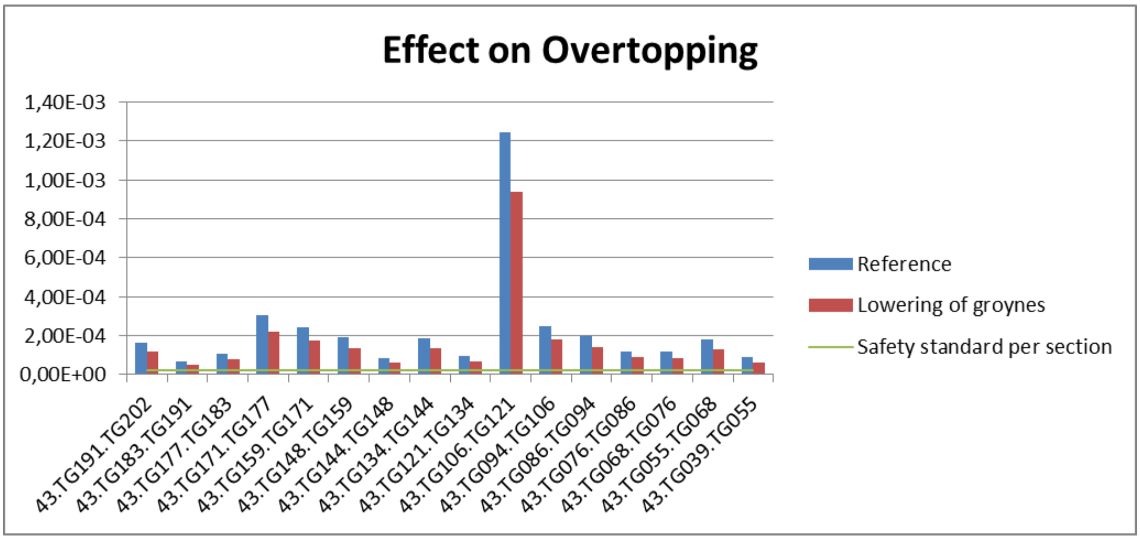
F.4.4 Damage and erosion outer slope

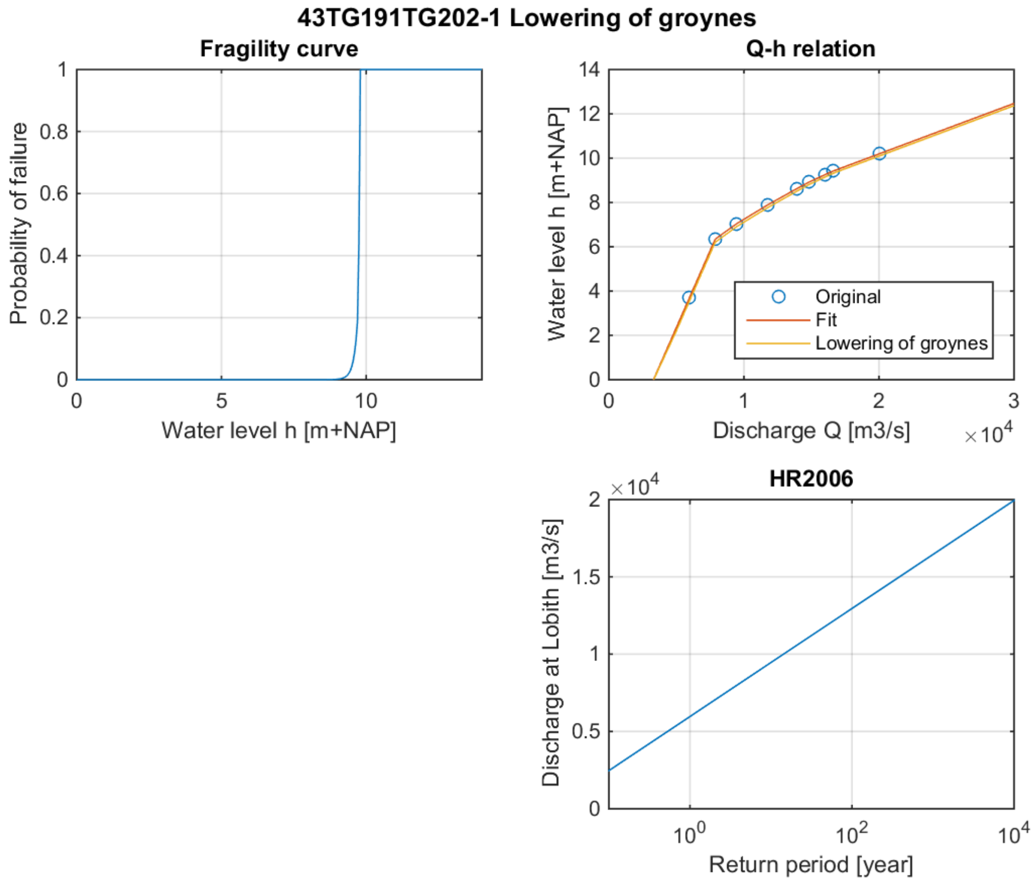


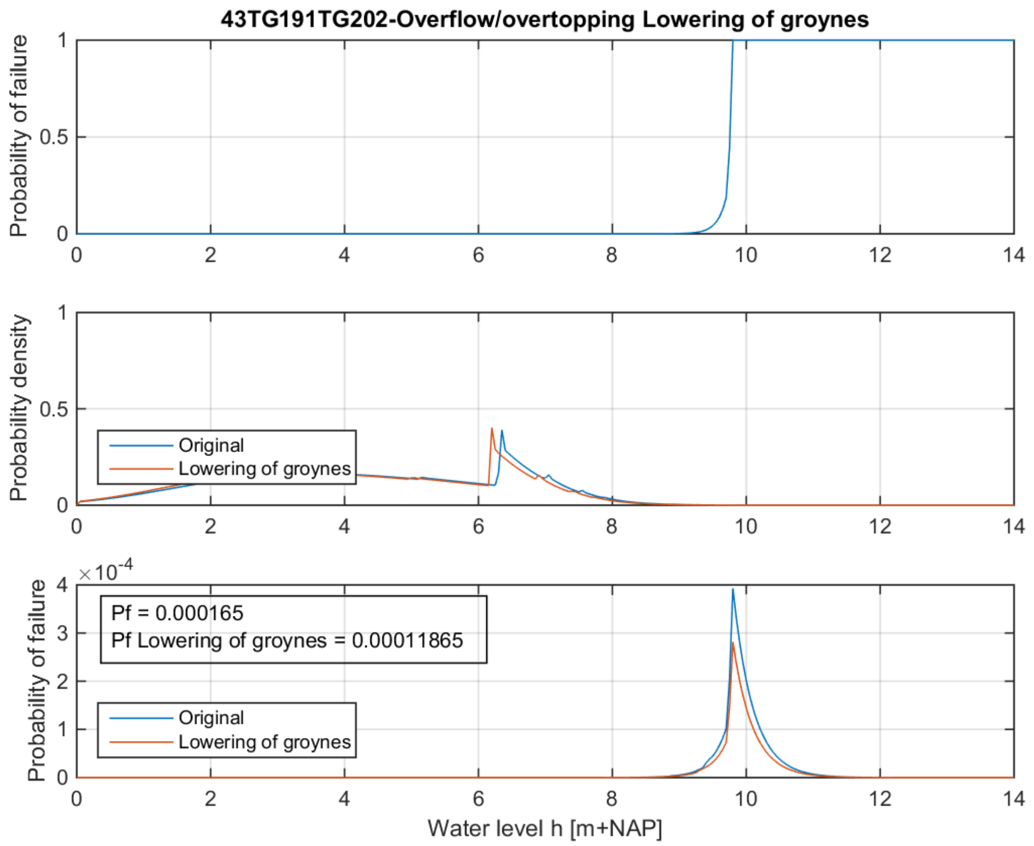
F.5 Lowering of groynes



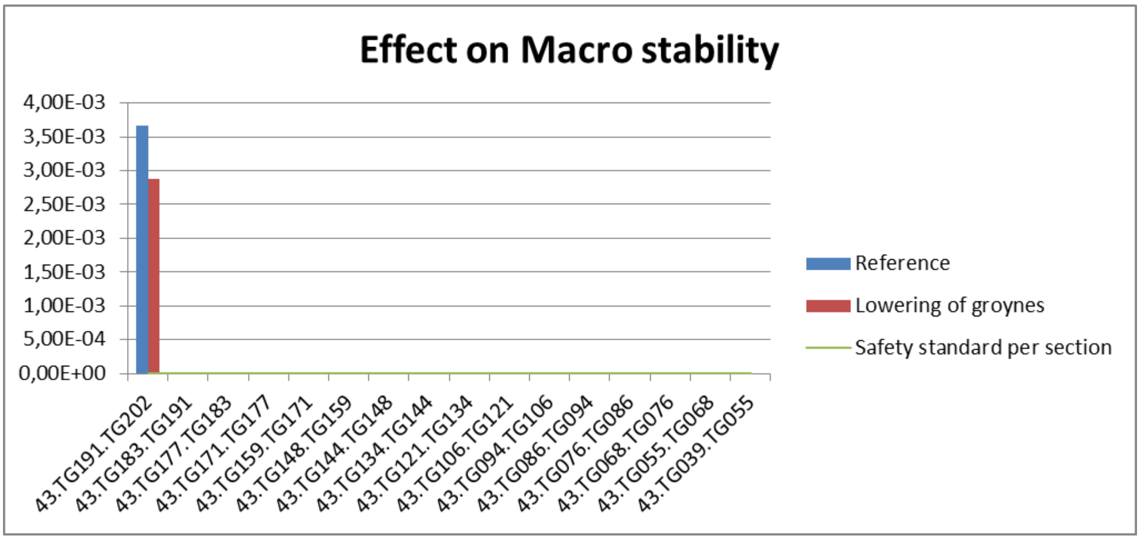
F.5.1 Overflow/overtopping

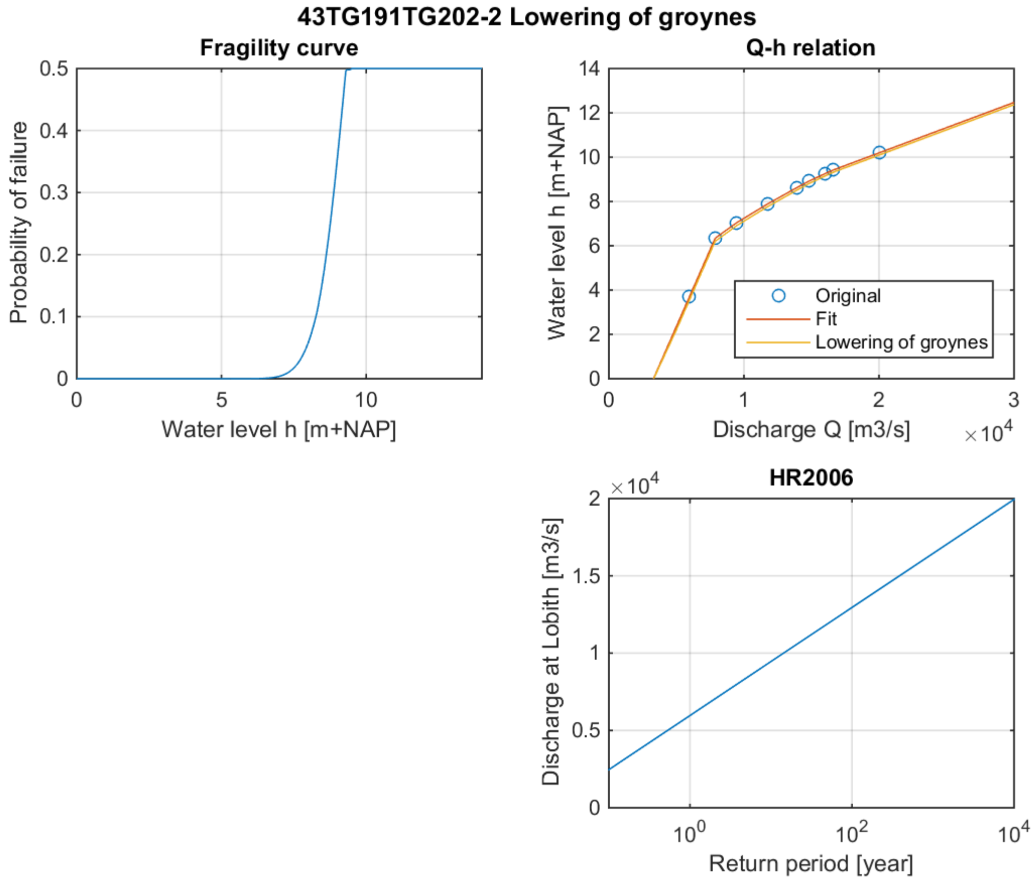


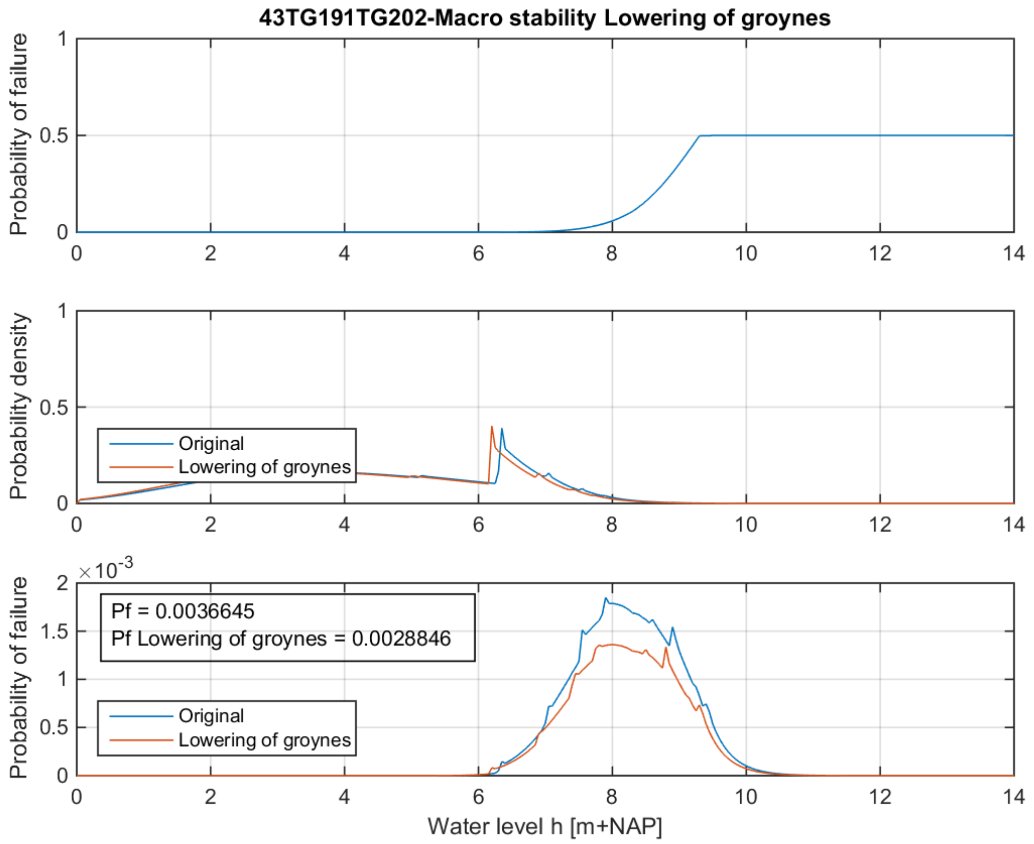

43TG191TG202-1 Lowering of groynes
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
×10^4
Original
Fit
Lowering of groynes
HR2006
×10^4
Discharge at Lobith [m3/s]
Return period [year]


43TG191TG202-Overflow/overtopping Lowering of groynes
Probability of failure
Probability density
Original
Lowering of groynes
×10^-4
Probability of failure
Pf = 0.000165
Pf Lowering of groynes = 0.00011865
Original
Lowering of groynes
Water level h [m+NAP]

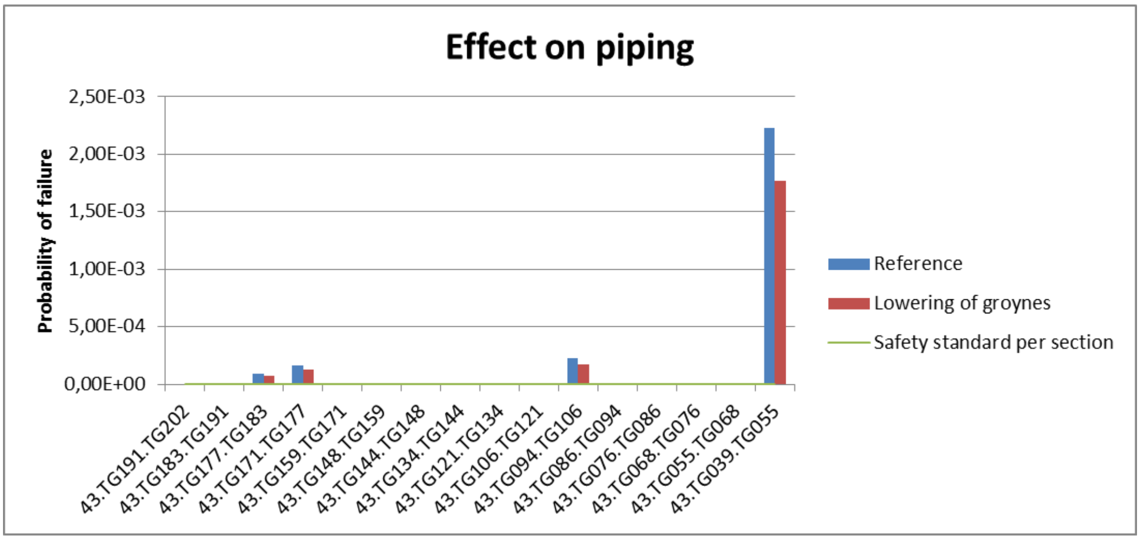
F.5.2 Macro stability

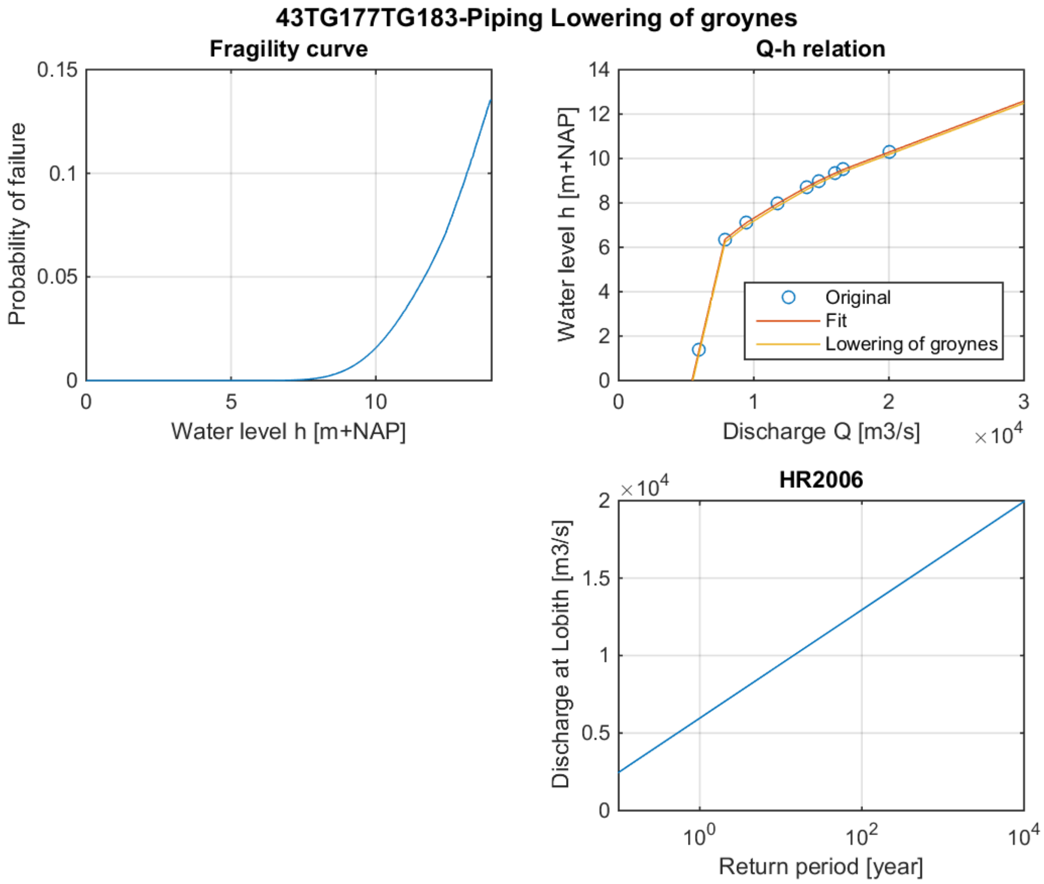


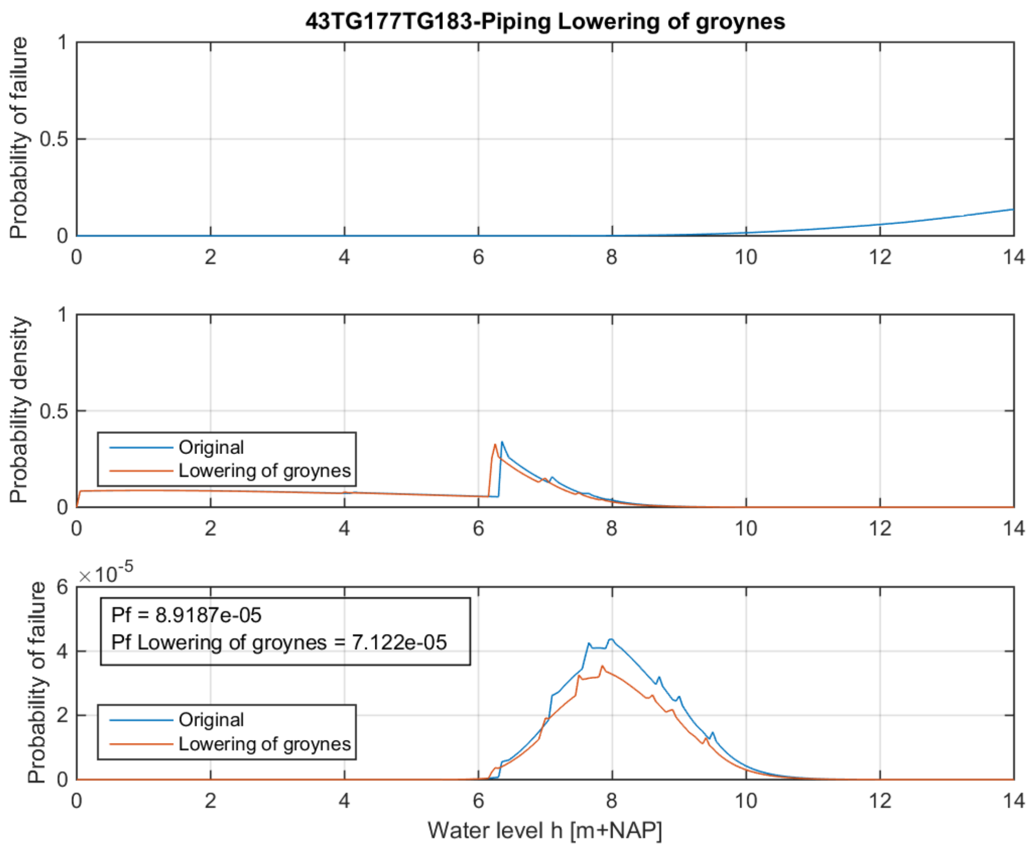



43TG191TG202-Macro stability Lowering of groynes
Probability of failure
Probability density
Original
Lowering of groynes
×10^-3
Pf = 0.0036645
Pf Lowering of groynes = 0.0028846
Water level h [m+NAP]

F.5.3 Piping


Effect on piping
Probability of failure
2,50E-03
2,00E-03
1,50E-03
1,00E-03
5,00E-04
0,00E+00
43.TG191.TG202
43.TG183.TG191
43.TG177.TG183
43.TG171.TG177
43.TG159.TG171
43.TG148.TG159
43.TG144.TG148
43.TG134.TG144
43.TG121.TG134
43.TG106.TG121
43.TG094.TG106
43.TG086.TG094
43.TG076.TG086
43.TG068.TG076
43.TG055.TG068
43.TG039.TG055
Reference
Lowering of groynes
Safety standard per section


43TG177TG183-Piping Lowering of groynes
Fragility curve
Probability of failure
Water level h [m+NAP]
Q-h relation
Water level h [m+NAP]
Discharge Q [m3/s]
Original
Fit
Lowering of groynes
HR2006
Discharge at Lobith [m3/s]
Return period [year]


43TG177TG183-Piping Lowering of groynes
Probability of failure
Probability density
Original
Lowering of groynes
Pf = 8.9187e-05
Pf Lowering of groynes = 7.122e-05
Water level h [m+NAP]

F.5.4 Damage and erosion outer slope