Comparison of two dike design strategies

Bachelor Thesis Project

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Preface

I have written this report as part of the Bachelor's thesis for my study Civil Engineering at the University of Twente. In this research, I have created a model based on the study by Hoekstra and De Kok (2008). After the model results were sufficiently close to the results of their study, I used the model to further investigate the concept of an alternative dike design approach to get understanding on the operation and efficiency compared to the existing 'probabilistic' approach.

I would like to thank everyone that helped me in the process of my bachelor's thesis project. In special I would like to thank Ferdinand Diermanse from Deltares and Jaap Kwadijk from the University of Twente for helping me to set up the research plan and for the supervision during my bachelor's thesis. I also, would like to thank Deltares for being able to execute my bachelor's thesis at the company. The virus COVID-19 has asked for quite some improvisation during the process of my project and I am grateful to everyone that helped me in succeeding to improvise in this challenge.

Nino Zuiderwijk

Enschede, 18-06-2020

Summary

This research builds on a study by Hoekstra and De Kok (2008), referred to as H&K. They investigated an ancient dike design philosophy called, the ‘self-learning’ dike (‘SLD’). This strategy might be a safer and more cost effective way of calculating required dike heights, compared to the ‘current’ probabilistic strategy (‘PD’). Results of the study by H&K indicate that the ‘SLD’ is significantly safer and cheaper over a period of 100 simulation years in river Rhine conditions with and without gradual climate change.

In our study, the two dike design strategies are compared in more detail than is done in the study by H&K. Not only are the dike design strategies compared in more circumstances, also the model is analysed to find possible model improvements. The goal is to understand why the results in the study by H&K are as they are. Also, testing these dike design strategies needs to provide insight in which circumstances the ‘SLD’ is a better option than the ‘PD’ design strategy.

The Netherlands yearly invests hundreds of millions of euros in water safety (Van Nieuwenhuizen-Wijbenga, 2018) and for that reason, a ‘new’ design strategy is investigated, called the ‘self-learning’ dike (‘SLD’) design strategy. This approach is easier to implement as the required dike height is nothing more than the highest observed water level plus a certain safety margin, s.

In our study the model and results from the study by H&K are reproduced. The created model is consequently used to further analyse the concept of the ‘SLD’ design approach and compare it with the ‘PD’ design approach. A sensitivity analysis on parameters representing various circumstances, is performed to see how both dike design approach operate under different conditions.

The results of this study show that the relative efficiency of both strategies is sensitive to the conditions for which they were tested. In the study by H&K, the ‘SLD’ scores significantly better due to the fact that the ‘PD’ often requires small dike adaptations, which are combined, more expensive than a single large adaptation of the ‘SLD’. The sensitivity analysis in this study shows that the ‘SLD’ does perform significantly better in the majority of conditions in a gradual climate change scenario compared to the ‘PD’. In the scenario without including climate change, the ‘PD’ performs better, only if adaptation criteria are included that do not allow small adaptations.

The adaptation criteria are chosen such that an equilibrium is found between flood damage cost and dike heightening cost. Such adaptation criteria make the model better represent reality. In the simulations of the study by H&K, all dike adaptations are allowed, while in reality, small adaptations are too costly for the increase in safety level (Personal communication with Diermanse & Kwadijk, 2020). The implementation of adaptation criteria as mentioned here, is beneficial to the efficiency of the ‘PD’ and makes it competing with the ‘SLD’. Simulations over a period of 300 years instead of 100 years, are in favour of the ‘SLD’.

Table of Contents

Table of Figures

Table of Tables

Glossary

H&K	Reference to study by Hoekstra A.Y. and De Kok J. (2008)
'PD'	Dike design strategy called 'Probabilistic' dike
'SLD'	Dike design strategy called 'Self-learning' dike

1. Introduction

The introduction discusses the content of the research and why this research is performed. It is discussed what knowledge is available beforehand and how this study is executed.

1.1. Setting the scene

In the Netherlands, many rivers can be found, of which the rivers Rhine and Meuse are two important ones. Without flood protection, a large part of the Netherlands would be flooded regularly (Parmet, Buishand, Brandsma, & Mülders, 1999). In 1953 a major flood happened in the country and over 1,800 people lost their life (Rijkswaterstaat, 2020). To prevent such disasters from happening and deliver enough safety, the country invests millions of euros every year (Van Nieuwenhuizen-Wijbenga, 2018). For this reason, the design of flood protection is an extensive process that considers the safety level and it's cost, which makes it worth to investigate alternative approaches.

A common used method for the design of flood protection is the probabilistic approach (referred to as 'PD'). This approach calculates the required dike height based on the design discharge corresponding to an exceedance frequency using Stationary Extreme Value Theory. The design discharge is determined using extrapolation of historic Annual Peak discharges. The method contains uncertainties mainly caused by two factors. The first of these two factors is the limited amount of measuring records. The design discharge is derived for a period which is much longer than the observed data. The second source to cause uncertainty is a changing climate. Climate change causes the system to be non-stationary, whereas 'traditional' extreme value theory relies on the assumption of stationarity. More information on stationarity can be found in Chapter 2.1. To be able to implement Extreme Value Theory, the trend in annual maxima (due to climate change) should be removed or a shift should be made towards a non-stationary system (Diermanse F. L. M. et al, 2010).

An alternative dike design strategy is introduced by Hoekstra and De Kok (2008) (referred to as H&K). They investigated the so called 'self-learning' dike design strategy (referred to as 'SLD'). Compared to the 'PD', the 'SLD' strategy is more straightforward and it relies on nothing more than just a simple rule: the height of the dike is the highest observed water level plus a safety margin, s (Hoekstra & De Kok, 2008). The dike is heightened when a newly observed water level exceeds the current dike height excluding safety margin, s.

The 'SLD' design strategy was investigated by H&K to see whether this approach operates better than the 'PD' for a dike ring along the river Rhine. Results of their study indicate that the 'SLD' performs better than the 'PD' in river Rhine conditions over a simulation period of 100 years in current climate change, but especially in changing climate conditions. The results showed that the 'SLD' is more efficient in terms of cost and safety (Hoekstra & De Kok, 2008).

1.2. Objective

The results of H&K indicate that the current approach for dike design is not the most efficient. Besides, more evidence is showing that changes in climate or environment are increasing (Milly, et al., 2008). On top of that, for the rivers Rhine and Meuse a positive trend can be found in annual maxima over the period of 1911-2003. Therefore, if the results of Hoekstra and De Kok (2008) are correct and the 'SLD' design strategy is significantly better in current and climate change conditions, a paradigm shift might be activated in the field of dike design.

H&K investigated both dike design approaches for river Rhine conditions over a period of 100 years in 3 scenario's, an extensive analysis on this study can be found in Chapter 3.2. Deltares, an institute of applied research, asked to further analyse the concept of the 'SLD' to get a better understanding on how this design strategy operates and why it performs significantly better compared to the 'PD' design approach in the study by H&K. Also, Deltares asked to investigate both dike design approaches in other river circumstances, such as river Meuse, to get broader knowledge on this 'newly' researched dike design approaches, the 'SLD'.

The dike design strategies are tested based on the same criteria as H&K: number of adaptations, number of dike overtopping and the total cost, all over a 100-year simulation period. By use of this research and the results, the aim is to provide better inside into both dike design approaches and to provide a more detailed comparison between them to gain more knowledge on whether the 'SLD' design approach is indeed significantly better or only in certain situations.

1.2.1. Research questions

The main goal is to compare the approaches in both stationary and non-stationary systems on at least the following criteria: total cost (maintenance and flood damage), the expected frequency of flooding and the expected frequency of replacement/adjustment of the protection infrastructure. The two main research questions for this research are:

1. *Why does the concept of the 'self-learning' dike design strategy perform better in the circumstances according to the study by Hoekstra and De Kok* (2008)*?*

2. *In which conditions does the 'self-learning' dike design approach perform better than the probabilistic dike design approach (Chapter 3)?*

Research question 2 will be answered using small sub-questions that provide clearer boundaries which helps to keep track of the goal. The sub-questions are:

- How do both dike design strategies perform in circumstances corresponding to the river Meuse at Borgharen?
- Which river and simulation conditions affect the efficiency of the dike design approaches and how?

1.3. Methodology

The two main research questions are addressed using both qualitative as well as quantitative research. In the process of answering both research questions, studies from the past and communication with both supervisors and others determine for which conditions both dike design approaches are analysed and compared.

Research question 1 is answered by reproducing the study by H&K. MATLAB R2017a is used to construct a model which performs equal simulations and generates similar results as the calculation tool used by H&K. The report on their study is examined intensively to collect all required information for reproducing the model. Through this method of answering research question 1, all input used by H&K can be extensively discussed and analysed. Reproduction is an essential step to create knowledge on the operation of the model and remarkable decisions made by H&K can be reconsidered. Output of the model is used to verify the model with the results of the study by H&K. The model is optimized until the results show sufficiently small deviation with the verification study (H&K).

Research question 2 is addressed by analysing the operation of the model created for research question 1 and in the form of discussion with Diermanse F.L.M. and Kwadijk J.C.J. about relevant parameters and validation of the model. This qualitative research will set boundaries for which parameters and possible model improvements are analysed on their influence to the results. For this analysis, MATLAB R2017a is used. Figure 1 shows the steps that are taken in the process to answer research question 1 and 2.

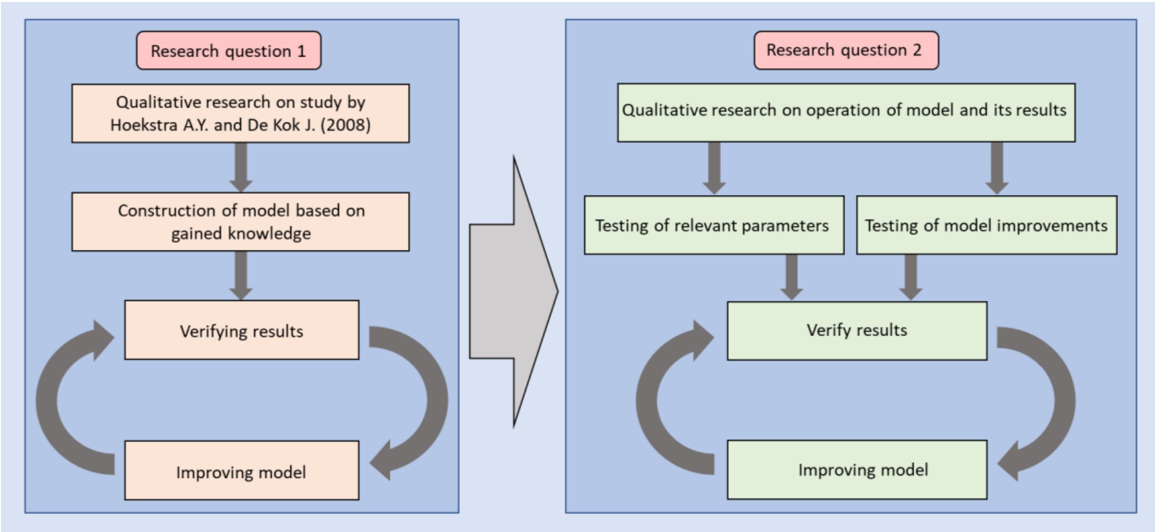

Figure 1 – simplified representation of method steps

2. Description of ‘PD’ and ‘SLD’ dike design strategies

2.1. ‘PD’ design strategy

Standard Extreme Value Theory (EVT) is an approach which can be used to design dikes and is used widely for the design of water management and flood protection structures, such as dikes. The Dutch regulations on dike design estimated a design water level with different return periods for different dike sections (Kind J. , 2012). These values are calculated based on probability distributions for peak discharges using data series from the past. Every 12 years the design water levels are re-evaluated to deal with uncertainties regarding peak discharge variability and respond to climate change.

The approach is based on the assumption of Stationarity. Stationarity means that natural systems are expected to fluctuate within the boundaries of an unchanging envelope of variability (Milly, et al., 2008). The assumptions of a stationary system is acceptable as long as the changes in climate and river basin are neglectable. However, more evidence is showing that the changes in climate or environment are becoming more substantial and more consistent (Milly, et al., 2008).

Due to the probabilistic approach of the design strategy, changing environment will only lead to different design levels long after the change happened (Hoekstra & De Kok, 2008). A trend in new years is neutralized by historic data which do not show a trend. Therefore, this method needs time to detect a trend in discharge statistics. The actions in this strategy are taken to reduce flood risks and are not anticipatory, but following (Hoekstra & De Kok, 2008).

The conceptual model representation of the Probabilistic dike design strategy which relies on Standard Extreme Value Theory is given in Figure 2. The additional dike height is equal to the difference between the initial (current) design water level and the new design water level (with additional 5 years of Maximum Annual discharge data). The design water level is the water level corresponding to the design discharge, determined with the stage discharge relationship. The safety board is 0.5 meters.

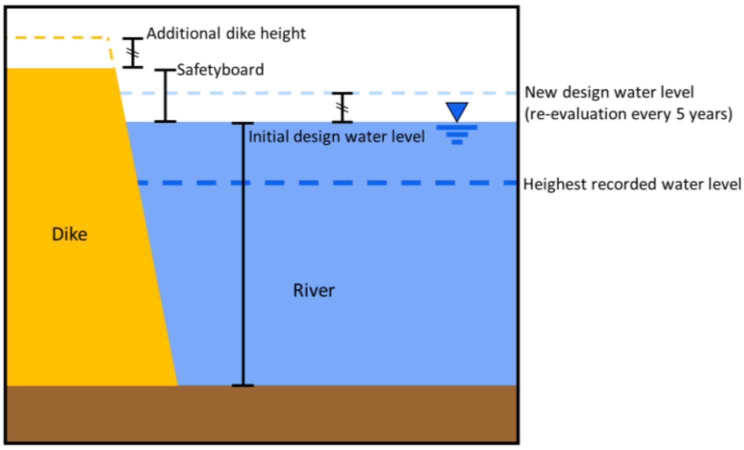

Figure 2 – Conceptual model ‘PD’ design strategy

2.2. ‘SLD’ design strategy (Hoekstra & De Kok, 2008)

The concept of a ‘SLD’ is an ancient design philosophy which suggests determining the required dike height based on the highest measured water level, with an additional safety board. This means the dike design is based on nothing more than just a simple rule, no statistics or other form of probability is applied. The self-learning part of the strategy refers to the immediate adjustments of the dike height after a new extreme flood level is observed. In principle, the dike designed according to this strategy, will grow steadily with changing extreme water levels, if a positive trend can be found. The safety margin will prevent, to some extend, that new record water levels causes a flood.

Figure 3 shows the conceptual model of the ‘SLD’ design strategy. The additional dike height is equal to the difference between the initial (current) highest water level measured and (when higher water level occurs) the new highest water level measured. The safety board of the ‘SLD’ has not been analysed intensively yet and therefore differs in this study, depending on the initial ‘PD’ height. The safety board is determined by subtracting the highest measured water level from the ‘PD’ height.

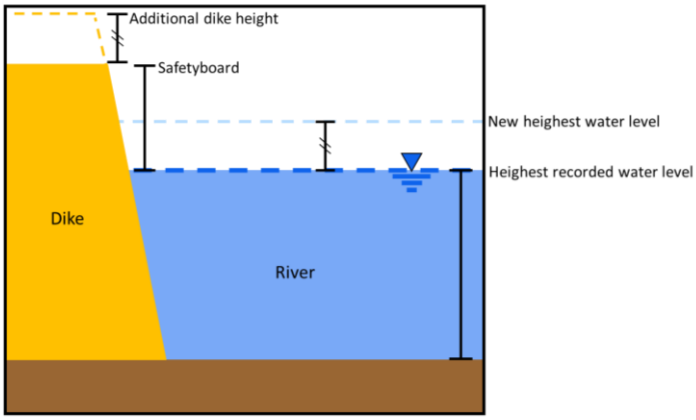

Figure 3 – Conceptual model ‘SLD’ design strategy

3. Tools and Data used

In this chapter, the calculation tool is established. The model is build using the study by H&K. Firstly it is discussed how the two dike design strategies are compared in our study. Next the model is discussed and finally a brief setup of our study is given.

3.1. Comparison of 'SLD' and 'PD' strategies

Deltares asked to extend the knowledge of the previous study by H&K. For this reason, the calculation tool in this study needs to perform similar simulations for the same dike design strategies as in the study by H&K, namely 'SLD' and 'PD'. These two dike design strategies were assessed based on: the amount of overtopping, amount of adaptations and a cost estimation. These criteria are calculated for both dike design strategies over a period of 100 years for which a time series of annual maximum discharges are created based on Gumbel distribution, derived from a set of historic annual maxima (1901-2000) at the gauge station Lobith of the river Rhine.

H&K only considered overtopping as a failure mechanism, with dike heightening (adaptation) as a measure. Reason for this is that the level of spatial detail of the analysis and to which extend all failure mechanisms are included, are very influential to the outcome of a full risk analysis, which is still subjected to considerable uncertainty (Hoekstra A. Y., 2005).

3.2. Repeating H&K analysis

The two strategies, 'SLD' and 'PD' are tested for three discharge scenarios

1. Current peak discharge statistics
2. Current peak discharge statistics including uncertainty
3. Gradual climate change trend which slowly increase peak discharges

Uncertainty in extreme value distribution functions is often displayed as 95% confidence intervals, see Figure 4.

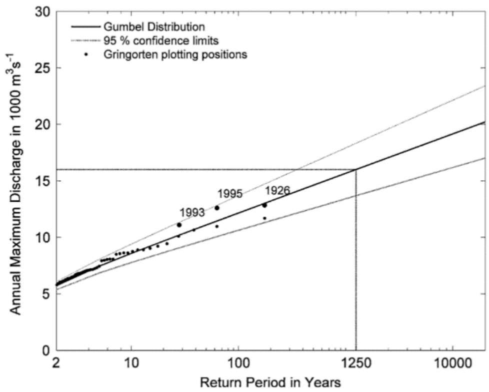

Figure 4 – Gumbel Extreme value distribution (Parmet, et al., 2001)

3.2.1. Statistical analysis

The two dike design strategies are compared based on an extension of the historic discharge data for the gauge station at Lobith of the river Rhine. 100 Years of historical annual maxima is available over the period 1901-2000 (Personal communication with Diermanse & Kwadijk, 2020). This time series is homogenised so as to represent the river conditions of the year 1999 (Diermanse, 2020). This homogenised annual peak discharge data has been used to establish parameters for the Gumbel Extreme Value distribution, which represents the cumulative probability distribution (Bury, 1999; Shaw, 2002):

$$F(Q) = Pr\,ob(q_{max} \le Q) = e^{-e^{-b(Q-a)}}$$ *Equation 1*

The return period is given below:

$$T(Q) = {}^{1}/_{P(Q)} = {}^{1}/_{1 - F(Q)}$$ *Equation 2*

In this equation, Q represents peak discharge [m^3s^{-1-}] and *a* and *b* are parameters which are given below:

$$a = \mu - {}^{y}/_{b} \quad (y = 0.5772)$$
$$b = {}^{\pi}/_{(\sigma * \sqrt{6})}$$ *Equation 3*

μ Is sample mean and σ^2 is sample variance. The stationary Gumbel distribution can be used and is justified in the past (Stedinger, Vogel, & Foufoula-Georgiou, 1993). In the first two scenarios, no trend is assumed in the extension of discharge time series. For the third scenario, a trend in discharge statistics due to gradual climate change is assumed in which, a and b are made time dependent to create a probability distribution which is non - stationary (Khaliq, Ouarda, Ondo, Gachon, & Bobée, 2006). A and b are time dependent by means of common multiplier which ensures that the design discharge is reached in the year 2100.

For the initial year, simulation values for a and b were determined after rescaling the data to ensure that the design discharge for a return period of 1,250 years is 16,000 m^3s^{-1}, 5170 m^3s^{-1} and 6.584*10^{-4} $m^{-3}s^{-1}$ respectively. This value is the design discharge which was established in 2001 (MTPWWM, 2005). The variance of a and b are approximated by:

$$Var(a) \approx {}^{1.16781}/_{Nb^2} \qquad Var(b) \approx {}^{1.10001b^2}/_{N}$$ *Equation 4*

Where N is the number of peak discharges on which the function is fitted.

3.2.2. Simulation

After the probability distribution is established, 100 years of artificial annual maximum discharges can be simulated. This is done by using the inverse of Equation 2. To obtain reliable results, the simulation is repeated 10^5 times. This ensures that the average μ and σ (of the established 100 year time series) over all simulations, approach the Gumbel parameters a and b with less than 1.5%, which were for the initial year, respectively, 5,170 m^3s^{-1} and 6.584*10^{-4} $m^{-3}s^{-1}$. The Q-h relation, used to translated these discharges to water levels, is based on Schielen R.M.J. (2007) and Van den Brink G.M. et al (2007).

The two strategies are evaluated based on the three aforementioned scenarios. How discharges for these scenarios are generated is described below:

<u>Scenario 1:</u>

In the first scenario 100 years of discharges are randomly sampled from the Gumbel distribution using the rescaled a and b values.

<u>Scenario 2:</u>

This scenario includes the uncertainty in the peak discharge statistics. This means, it accounts for the inherent uncertainty of the Gumbel parameters a and b. The difference with scenario 1 is that the parameters a and b are sampled from a normal distribution using the mean and variance of Equation 4. For each year of each simulation, a and b a determined separately.

Scenario 3:

In the last scenario, a and b are made time-dependent such that the design discharge in the year 2100 is equal to 18,000m^3s^{-1} (MTPWWM, 2005). Starting in the first year, the parameters are changed linearly every year to simulate the effect of climate change.

3.2.3. Implementation of 'SLD' and 'PD' design strategies

In this study, both dikes are re-evaluated every 5 years by adding annual peak discharges of the last 5 years to the time series. For both dikes it is checked if overtopping happened in these 5 years. Both dike design approaches are evaluated whether dike heightening is required. For the 'PD' design strategy, the design water level plus safety board of 0.5 m determines the required dike height. The design height of the 'SLD' design strategy is equal to the height of the highest water level measured, plus a safety margin, s. The water level is determined using the stage-discharge relation of the river Rhine at the gauge station Lobith, see Figure 5. H&K only provided a figure to show the stage-discharge relation. For this reason, the stage-discharge relation for the river Rhine, is derived from H&K, figure 3.

In this study, the initial dike height of both dikes is assumed identical, which can be used to determine the safety margin, s. The design water level corresponding to a 1,250 year return period is NAP+17.93 m. After including the safety board of 0.5 m, the initial dike height of the 'PD' is set at NAP+18.43 m (thus the same height is chosen for the 'SLD'). The highest discharge observed at Lobith is 12,849 m^3s^{-1} (Parmet, et al., 2001) which corresponds to NAP+16.59 meters according to the Q-h relation, see Figure 5. The safety margin of the 'SLD' is the initial 'PD' height, minus the highest observed water level, NAP+18.43 – NAP+16.59 = 1.84 m.

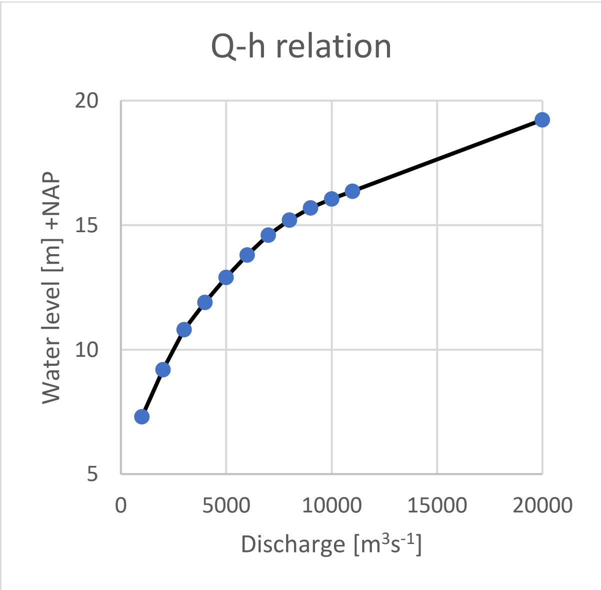

Figure 5 – Q-h relation river Rhine deduced from Hoekstra A.Y. and De Kok J. (2008)

3.2.4. Cost calculation

H&K included a cost calculation over the simulation period, which calculates both the investment cost and the flood damage cost of both dike design approaches. This cost calculation is based on Eijgenraam C.J.J. (2005 & 2006). The paper of H&K does not include detailed information about the implementation of the cost calculation. Therefore, a new cost calculation is established in this study, based on Eijgenraam C.J.J. (2005 & 2006).

For the investment cost of a dike adaptation, equation 5 is used:

$$I(u) = (c + b * u) * e^{\lambda * u} \qquad \textit{Equation 5}$$

In which I is the investment cost for adaptation with height u in cm.
c = 0.5 million euro per km dike (fixed investment cost (Eijgenraam, 2005))
λ = 0.0063 (Eijgenraam, 2005)

The flood damage of an overtopping is calculated using the equation 6:

$$Flood\ damage = (V(mat) * \mu + N * V(imm)) * e^{\gamma * t} e^{(H_t - H_0) * \zeta} \qquad \textit{Equation 6}$$

V(mat) = 5453 million (material cost (Eijgenraam, 2005))
$\mu = 1.115$ (Increase in building cost (Eijgenraam, 2005))
N = 193300 (inhabitants of dike ring 48 (Eijgenraam, 2005))
V(imm) = 5000 euro (0.005 million, (Eijgenraam, 2005))
$\gamma = 1.02$ (Annual economic growth (Eijgenraam, 2005))
t = current year
H_t = dike height in year t
H_0 = initial dike height
ζ = 0.0031 (Eijgenraam, 2005)

3.2.5. Verification

Verification of the calculation tool is done by comparing the results of the calculation tool with the results of the study by H&K. The results of the study by H&K can be found in Table 1. Our reproduction results can be found in Table 2.

Table 1 – Results of study by H&K in river Rhine conditions for the 'PD' and the 'SLD'

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Number of dike overtopping	0.024	0.021	0.034	0.030	0.047	0.037
Number of adaptations	2.13	1.00	2.40	1.09	3.62	1.40
Extra height per adaptation [m]	0.07	0.48	0.07	0.51	0.07	0.51
Fixed costs of dike heightening in 10^6€	22.66	8.15	24.59	8.97	27.05	9.80
Variable cost of dike heightening in 10^6€	4.63	17.46	5.35	21.30	5.66	21.91
Flood damage over 100 years in 10^6€	74.83	69.46	107.19	98.78	121.87	106.11
Total cost in 10^6€	102.11	95.07	137.13	129.06	154.58	137.82

Table 2 – Results of our study in river Rhine conditions for the 'PD' and the 'SLD'

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Number of dike overtopping	0.024	0.022	0.033	0.029	0.046	0.038
Number of adaptations	2.13	1.00	2.40	1.06	3.63	1.38
Extra hight per adaptation [m]	0.07	0.49	0.07	0.52	0.07	0.53
Fixed costs of dike heightening in 10^6€	60.89	28.26	68.62	30.43	102.94	39.23
Variable cost of dike heightening in 10^6€	20.36	126.66	24.40	156.28	35.41	200.75
Flood damage over 100 years in 10^6€	564.35	483.61	826.68	671.11	1,306.90	1,036.80
Total cost in 10^6€	645.59	638.53	919.69	857.82	1,445.20	1,276.80

The results for the adaptation and overtopping criteria of our study adequately agree with the results given in the by H&K. The biggest differences can be found in Scenario 3 for the 'SLD' strategy, though the differences do not exceed 4%. These differences are related to the implementation of model components, which left room for interpretation.

One of these difference could be found in the Q-h relation, since this relation is drawn from a figure from the study by H&K. Retrieving this relation from a figure, might include errors. Also, in this study, a set of historic annual maxima of 100 years (1901-2000) is used, while H&K used 98 years of historic annual maxima (1901-1998). At last, the costs given Table 2, do not comply with the study by H&K, see Table 1. This is most likely due to the difference in the cost function.

3.3. Brief setup of experiment

After reproduction of the results from H&K, it was decided what is relevant to further analyse in this study. A brief setup about what and why is analysed, is discussed here. Chapter 4 contains an extensive explanation of the research performed in this study. The following three topics were selected and carried out:

1. The results H&K indicated that the 'SLD' is significantly more efficient than the 'PD', which would mean that it is beneficial to shift to the 'SLD' strategy. H&K discussed what possible influence certain river characteristics could have on the efficiency of both dike design strategies. We therefore decided to validate some of the hypotheses stated by H&K in their discussion session by means of a sensitivity analysis. The following parameters were selected for the sensitivity analysis: Steepness of the stage-discharge relation; the safety standards; the assumed safety margin and the mean and standard deviation of historic annual maximum river discharges.

2. In the study of H&K, the 'PD' strategy often results in a number of relatively small dike adaptations, which combined are more expensive than one large adaptation of the 'SLD'. The 'PD' requires on average a dike adaptation of 0.07 m, which is not cost efficient (because of high fixed costs, see $I(u) = (c + b * u) * e^{\lambda * u}$
Equation 5 and is therefore not realistic. The operation of the 'PD' design strategy can most likely be substantially improved by adding requirements to a dike adaptation. The strategy can be improved by not allowing for minor adaptations. This is investigated by implementing (i) a threshold value which only allows for dike heightening when the design discharge significantly increases and (ii) a minimum extra dike height threshold once the dike needs to be reinforced.

3. H&K carried out simulations for a period of 100 years. We investigated the influence of this choice on the results, by repeating the analyses for a period of 300 simulation years.

4. Comparing 'SLD' and 'PD' strategies for various river conditions

In this chapter, a broader study is performed to provide insight in how the 'SLD' strategy operates compared to the 'PD' strategy under different circumstances. First of all, the model is used to compare the derived efficiency of both dike strategies in the circumstances corresponding to the river Meuse at Borgharen. Following, the model is used to see how relevant river characteristics affect the efficiency of both dikes, both individually and relative to each other. Thirdly, it is tested whether the strategies can be improved using different criteria for dike heightening. The analyses are finished by investigating the influence of the length of the simulation period on the derived efficiency of the strategies. The model was initially applied for an simulation period of 100 years, in this study we also test the dike design strategies for a period of 300 years.

4.1. Application to the River Meuse

The model is originally designed to simulate river conditions corresponding to the river Rhine at Lobith. This means, the Stage-discharge relation, historic discharge data of the river Rhine and more, are used as input to calculate the results. By adjusting these values to the characteristics of the river Meuse, it can be seen how these two dike design strategies hold in other river circumstances. Extension of the time series for the river Meuse is based on historic discharge data over the period 1911 – 2003. The Q-h relation of the river Meuse is rather different to that of the river Rhine, see Figure 6.

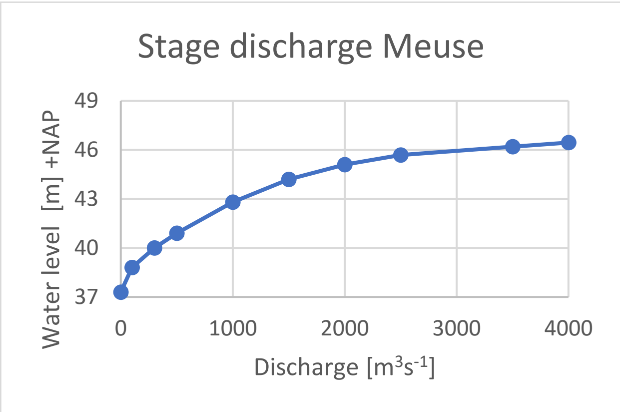

Figure 6 – Q-h relation river Meuse based on Barneveld H. et al (1998), Paap B. et al (2012) and Van Vuuren W. (1999)

To simulate the efficiency of both dike strategies in river Meuse conditions, parameters in the model need to be adjusted. The historic times series is adjusted such that the 1,250-year design discharge corresponds to 3,800m^3s^{-1}, which was established in the year 2001 (Parmet, et al., 2001).

A study on the effects of climate change for the river Meuse by Reuber J. et al (2006), predicts design discharges in the year 2050. For this study it is chosen to use the average case (Reuber J. et al (2006), table 4), a discharge of 4200 m^3s^{-1} in the year 2050 for the river Meuse (Reuber, Schielen, & Barneveld, 2006). This value is extrapolated for 50 more years to represent the design discharge at the end of 100 year period in the model in the conditions of gradual climate change. This leads to a situation in which the first year contains a design discharge of 3800 m^3s^{-1} (Parmet, et al., 2001) which is gradually increased to 4600 m^3s^{-1} over a period of 100 years. The change is introduced in the first year and follows a linear increase.

Dike ring 41 is chosen to estimate cost of both dike design approaches in river Meuse conditions at Borgharen. Reason for this is that dike rings close to Borgharen are not investigated by Eijgenraam E.J.J. (2005) and Dike ring 41 is located along the river Meuse. This dike ring has a length of 37 km long. Values for λ and b in Equation 5 are respectively 0.0033 and 0.02 million euro per km dike. The fixed cost are 1.27 million euro per km dike. ζ, N and V(mat) in Equation 6 are respectively 0.0027, 251200 inhabitants and 7854 million euro for the Meuse river condition.

4.2. Sensitivity analysis

Various river conditions can exist which can affect the efficiency of both dike design approaches. The steepness of the Q-h relation, Safety standards, safety margin and discharge data characteristics are some of many variables which possibly affect the efficiency and are tested in this study. The objective of the sensitivity analysis is to gain insight in which river condition parameters are relevant to the outcome. River conditions to be tested, are small deviations around river Rhine conditions. The sensitivity analysis is executed for scenario 3, which includes gradual climate change. To get insight in the influence of gradual climate change on the dike efficiency, the same simulations are executed for scenario 1. The results of scenario 1 are less extensively analysed and are used to compare the efficiency of both dikes relative to each other, with and without gradual climate change.

4.2.1. Stage discharge relation

The relation between the discharge and the water level is called, stage-discharge relation (Q-h relation). A steep Q-h relation leads to a larger elevation in water levels with an equal increase in discharge. Stage-discharge relations are traditionally established empirically, based on a set of discharge measurements and corresponding water level (Schmidt & Yen, 2001). In this study, the only goal is to gain insight into the influence of the steepness of the Q-h relationship. It is chosen to adjust the linear part of the Q-h relation, which is for discharges of 11,000 m^3s^{-1} and above. In Figure 7, the stage discharge relation with coefficient can be found. Coeff is the multiplier which determines the steepness of the stage-discharge relation, ranging from 0.16 to 0.40 [$m/10^3\ m^3s^{-1}$].

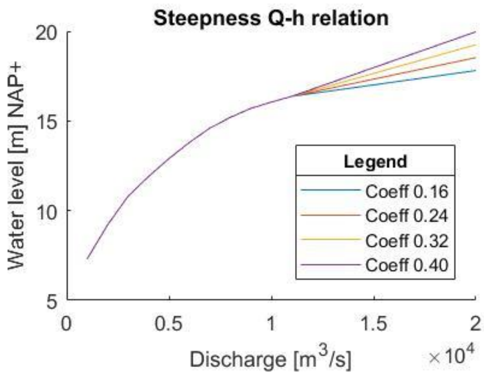

Figure 7 – stage-discharge relations

4.2.2. Safety standards

Safety standards influence the design discharge for the 'PD' and with that, also the safety margin on the 'SLD'. Safety standards, varying from 625 years to 10,000 are tested and correspond to exceedance frequencies of 1/625, 1/1250, 1/2500, 1/5000 and 1/10000.

4.2.3. Safety margin

The safety margin is a parameter which provides additional safety in the design of the dike. The parameter which is referred to is the safety margin as used in the design of the 'PD'. The safety margin of the 'SLD' is derived from the difference between the initial 'PD' height and the highest measured water level, which therefore is dependent on the safety margin of the 'PD'. This is chosen to ensure that both dike design strategies start with the same initial dike height (Hoekstra & De Kok, 2008). For the 'PD' it is chosen to test safety margins ranging from 0 to 2 meters. These values correspond to 1.37 m and 3.37 m safety margin for the 'SLD' design strategy.

4.2.4. Maximum annual peak discharge characteristics

In this study the probability distribution of the annual maximum peak discharge time series is the statistic that is used to establish synthetic time series of discharges. The features of this distribution differ for every river and influence the effectiveness of both the 'PD' and 'SLD' design approach. Of this maximum annual peak discharge data, the mean and standard deviation are adjusted to get hold of the rate of involvement of both river discharge characteristics. To adjust the mean value, the mean is multiplied with a multiplier, x. Then the difference between the original mean and the calculated mean is established and added to all discharge data.

Equation 7, shows the implementation of a standard deviation multiplier. This equation adjusts the standard deviation by multiplying the difference between the discharge in year t (Q_t) and the mean discharge of the complete time series (μ_q), with a multiplier, x. Q_{tq} is the discharge in year t after the adjustment.

$$Q_{tq} = (Q_t - \mu_q) * x + \mu_q \qquad \textit{Equation 7}$$

4.3. Minimum adaptation height

A feature of dike design approach is the magnitude of the minimum adaptation height. Small adaptation will not increase the safety level significantly so that during the re-evaluation 5 years later it may turn out that a new adaptation is required. This results in more adaptations. A minimum dike adaptation height can be set to decrease the number of dike adaptation. Therefore, an analysis is done to get hold of optimal values (in terms of total cost) in scenario 1 and 3 for minimum dike adaptation height and threshold to execute an adaptation.

4.4. Simulation period

In the study by H&K they chose to simulate a period of 100 years. It is not explained why 100 years of simulation time is chosen. The number of simulation years could affect the efficiency of both dike design approaches. This parameter is tested for 300 years instead of 100, to verify whether it affects the relative efficiency of both dike design strategies.

5. Results

5.1. Meuse river

Table 3 shows the results for river conditions corresponding to the river Meuse at Borgharen. The results are in favour of the 'SLD' strategy. Scenario 3 contains gradual climate change, which amplifies the benefits of the 'SLD' approach. Compared to the river Rhine circumstances, gradual climate change in the river Meuse simulation is much larger. The design discharge of the river Meuse is expected to increased with 21.05% (3,800 m^3s^{-1} to 4,600 m^3s^{-1}), while for the river Rhine this expectation is only 12.5% (16,000 m^3s^{-1} to 18,000 m^3s^{-1}). This increases the effect on the number of adaptations and dike overtopping over a period of 100 simulation years. The cost are more efficient for the 'SLD' in all scenarios.

Table 3 – dike performance in river Meuse conditions of 'PD' and 'SLD' dike strategy (B) in 3 scenario's for 100 year simulation period

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Number of dike overtopping	0.0046	0.0044	0.0074	0.0070	0.0151	0.0130
Number of adaptations	2.12	0.84	2.39	0.91	4.88	1.45
Extra hight per adaptation [m]	0.03	0.19	0.03	0.20	0.03	0.21
Fixed costs of dike heightening in 10^6€	99.29	39.36	111.46	42.70	230.11	67.96
Variable cost of dike heightening in 10^6€	5.23	15.90	6.14	18.44	12.92	31.41
Flood damage over 100 years in 10^6€	152.86	133.82	236.50	208.96	671.43	537.60
Total cost in 10^6€	257.37	189.08	354.10	270.10	914.46	636.97

The 'PD' scores similar results for scenario 1 and 2 in number of adaptations compared to river Rhine conditions (Table 2), while the 'SLD' scores significantly lower.

Table 4 shows the highest discharge, mean and standard deviation for the Rhine and Meuse rivers. The difference between the largest peak discharge and mean annual peak discharge, divided by the standard deviation is a measure for variability. The larger this value, the smaller the chance of exceedance of the crest height in future. Since the 'SLD 'is fully based on this characteristic, the number of adaptations is smaller in river Meuse conditions, compared to river Rhine conditions.

Table 4 – Historic data statistical characteristics of rivers Rhine and Meuse

	Rhine	*Meuse*
Mean [m^3s^{-1}]	6,634.5	1,519.9
Standard deviation [m^3s^{-1}]	2,098.4	534.4
Highest discharge [m^3s^{-1}]	12,731	3,175
Number of standard def. from mean to highest discharge	**2.91**	**3.10**

The overtopping frequency of both the 'PD' and the 'SLD' design strategies are much lower in river Meuse conditions, see Table 3, compared to river Rhine conditions, see Table 2. Reason for this is related to the safety board and q-h relation of the river Meuse. H&K added a safety board of 0.5 m to the design water level to determine the crest height. This value can be represented as a percentage of the design discharge, which is influenced by the Q-h relation. In case of the river Rhine, 0.5 m corresponds to 1,574.8 m^3s^{-1} and is 9.8% of the design discharge (16,000 m^3s^{-1}). This value is influenced by both the design discharge and the Q-h relation. A safety board of 0.5 m corresponds to an additional 1000 m^3s^{-1} discharge for the river Meuse, which is 26.32% of the design discharge (3,800 m^3s^{-1}). This is 2.67 times higher than the safety level in river Rhine simulation, which therefore results in a smaller probability for dike overtopping to happen.

5.2. Sensitivity analysis

This section is providing more insight in how river characteristics influence the efficiency of both dike strategies. Five parameters are discussed with regard to their influence in the efficiency of both dike design approaches. The first parameter is the safety standard and is extensively discussed including figures. For all other parameters, the analysis is done in a similar way and figures can be found in Appendix B. The figures contain information on the frequency of adaptation and overtopping, average dike adaptation height and cost. This sensitivity analysis is extensively executed for scenario 3. This scenario includes gradual climate change with an increase in the design discharge of 12.5% over 100 years. This sensitivity analysis is executed for scenario 1 as well, but in this analysis, only a comparison between both dike heightening strategies is carried out, which will elaborate on the difference in efficiency of both dike strategies in a scenario with and without gradual climate change.

5.2.1. Parameter analysis

5.2.1.1. Safety standards

Figure 8, 9, 10, 11, 12, 13, 14 and 15 show all relevant output for different values of the safety standard. The safety standard decreases from left to right on the horizontal axis. It shows that the safety standard has effect on the adaptation frequency of the 'PD' strategy. This is due to gradual climate change, that is included in this scenario. In the calculation of the design discharge, changing Gumbel parameters have a bigger influence with higher safety standards. Therefore, the requirement for adaptation is reached more often with bigger safety standards for the 'PD' design strategy. Stricter safety standards result in a lower frequency of overtopping. This applies to both dike design strategies, since the initial height of the 'SLD' depends on the initial 'PD' height. The overall cost decreases with a decrease in safety standard for both dike design approaches. The 'PD' design strategy benefits most of more strict safety standards.

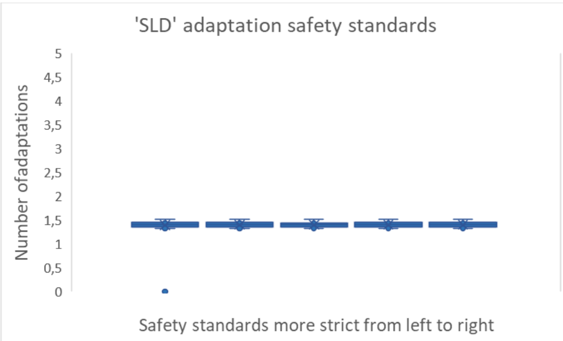

Figure 8 – Effect of safety standards on adaptation frequency 'SLD'

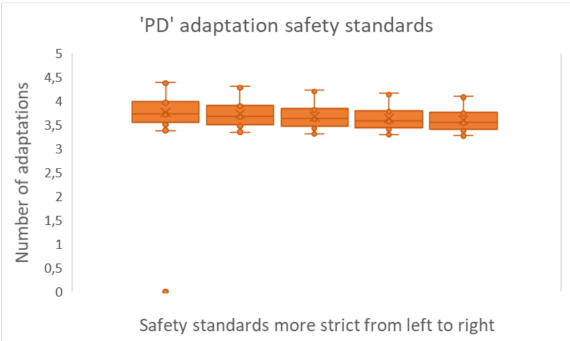

Figure 9 – Effect of safety standards on adaptation frequency 'PD'

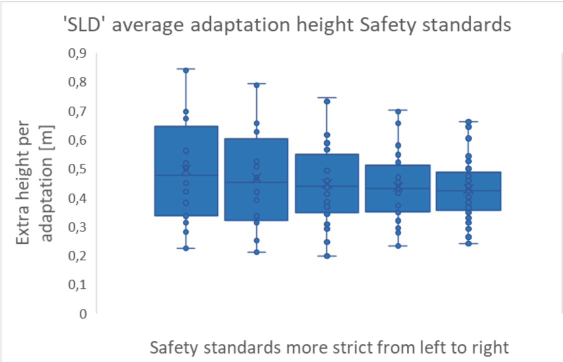

Figure 10 - Effect of safety standards on average adaptation height 'SLD'

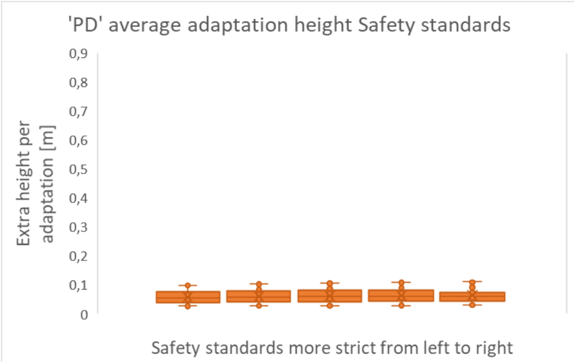

Figure 11 -Effect of safety standards on average adaptation height 'PD'

Figure 12 - Effect of safety standards on overtopping frequency of 'SLD'

Figure 13 - Effect of safety standards on overtopping frequency of 'PD'

Figure 14 - Effect of safety standards on total cost of 'SLD'

Figure 15 - Effect of safety standards on total cost of 'PD'

5.2.1.2. Safety margin

The overtopping frequency of both dike design strategies is reduced with higher safety margin on the 'PD'. A higher safety margin on the 'PD', also increases the safety margin on the 'SLD', since the analysis is set up in such a way to make a fair comparison between the two strategies. Higher safety margins result in higher safety levels and therefore a reduction in number of overtopping. Also the flood damages decrease with larger safety margins especially in case of the 'PD'.

5.2.1.3. Changes in the Standard deviation of annual maximum discharges

In the next sensitivity analysis we looked at (hypothetical) rivers with the same mean annual maximum discharge, but different standard deviations. For rivers with higher standard deviation in the annual maximum discharge compared to the river Rhine, the relative impact of a gradual increase in the mean due to climate change is less severe, see figure in Chapter B.2. The reason for this, is that the increase in standard deviation itself has a substantial impact, which reduces the impact of the increase in mean on the number of adaptations. For higher standard deviations, the total costs of the 'PD' strategy are closer to the total costs related to the 'SLD' strategy. Results show less spread in total costs, for the 'PD' strategy with higher standard deviations, while the spread and mean of the 'SLD' show an increase. Higher standard deviation has more impact on the increase of total cost of the 'SLD' strategy, compared to the total cost of the 'PD' strategy.

5.2.1.4. Q-h relation coefficient

The steepness of the stage discharge relation influences the number of overtopping in 100 years of simulation. Both dike design approaches are affected similarly. With increasing steepness of the stage discharge relation, the overtopping frequency increases, since extreme discharges will result in higher water levels as a result of the increased steepness of the stage-discharge relation. Consequently, both dike design approaches show an increase in cost with an increase in steepness of the stage-discharge relation.

5.2.1.5. *Changes in the mean of annual maximum discharges*

The number of adaptations of both dike design approaches increases with a higher mean value of the discharge statistics. This is due to gradual climate change. A higher mean value results in a steeper increase of annual maximum discharge, which is caused due to gradual climate change. Therefore, the requirement of adaptation is reached more often. For the same reason does the number of overtopping events shows a small increase as well. The costs remain somewhat equal for both dike design approaches, but show a small increase with higher mean value in discharge statistics.

5.2.2. Impacts of combined variation in parameters

The influence of a parameter on the results can be affected by variation in another parameter and can differ between the two dike design strategies. The most important interactions are discussed in this sub-chapter. It is chosen to analyse the results of overtopping frequency and adaptation frequency, since the cost is not a good representation of correlation. Reason for this is, that the cost calculation is highly influenced by the overtopping frequency and much less by the adaptation frequency. We want to understand the impact of combined variation in parameters on both the overtopping frequency as well as adaptation frequency. In Appendix C, figures can be found which show the relation of the frequency of overtopping and the frequency of adaptation, as a result of varying all pairs of parameters.

Whether a parameter affects the impact of parameters on the results is characterized by an unusual trend in the contour lines, where the contour line do not line up somewhat parallel. Appendix C contains figures which show such an 'unusual' trend, which is due to uncertainty in the simulations and the difference in value for the criteria is close to 0. Figure 16 is an example of a contour plot, where contour lines connect combinations resulting in the same 'overtopping frequency'.

The figures in Appendix C.1, indicate that there is no interaction between any pair of parameters. This can be explained by the fact that adaptation frequency relies on the discharge statistics. Interaction can be found if two parameters influence each others influence on the discharge statistics. Only the two parameters, 'changes in mean of annual maximum discharges' and 'changes in standard deviation of annual maximum discharges' affect the discharge statistics. Though, the implementation of variation in these two parameters is done, such that they do not affect each other.

Safety margin is a parameter which clearly interacts to the influence of the q-h relation on the overtopping frequency, see Figure 16 and also to the influence of the standard deviation, see Figure 93 andFigure 94. Figure 16 contains various contour lines that are not parallel to each other. Reason for this interaction is explained using the example of the interaction between the parameters 'stage-discharge relation' and 'safety margin'.

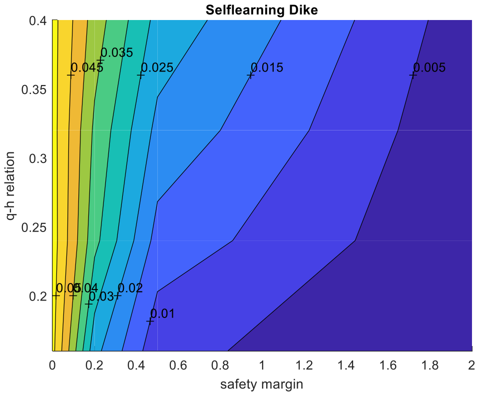

Figure 16 – Contour graph 'q-h relation' Vs. 'safety margin' overtopping frequency

The safety margin is an additional safety added to the design water level, which together forms the crest height. This has influence on the overtopping frequency. The safety margin ranges between 0 and 2 meters in this simulation. The q-h relation influences the height corresponding to a discharge. The extra safety level which is provided by the safety margin is therefore affected by the steepness of the q-h relation coefficient.

The same argumentation can be used to explain why the 'changes in standard deviation of annual maximum discharges' interacts with the 'safety margin' as well, since it affects the discharge statistics. The spread of the discharge statistics is increased with higher standard deviation, resulting in a bigger spread in water levels. An equal safety margin therefore results in a smaller safety level and larger overtopping frequency in a situation with larger standard deviation in annual maximum discharges.

5.2.3. Comparison of 'SLD' and 'PD'

This comparison is based on the 720 simulation results out of the sensitivity analysis. The 'SLD' scores better on number of adaptations and number of overtopping over a period of 100 simulation years. The summarizing statistics of the adaptation and overtopping frequency of both dike design strategies can be found in Table 5. The 'SLD' scores better for both criteria in all simulations. The difference in adaptation frequency between both dike design strategies, ranges between 1.94 and 2.87 in favour of the 'SLD'. It is remarkable that this difference is not reflected in the total cost difference between both strategies, the explanation for this is given below Table 5. In 220 out of 720 simulations, the total costs of the 'PD' design strategy are lower compared to the 'SLD'.

Table 5 – Scenario 3 results of simulation over 720 parameter sets for the 'PD' and the 'SLD'

	'PD'		'SLD'		Difference	
	Min	Max	Min	Max	Min	Max
Adaptation frequency	3.26	4.39	1.31	1.52	1.94	2.87
Overtopping frequency	0	0.2635	0	0.2099	0	0.0687
Total cost	113.35	7630.21	94.10	5875.37	-386.18	2363.61
Number of lowest cost	220		500			

This remarkable notion is explained as follows: besides the adaptation frequency and overtopping frequency, the average adaptation height influences the total costs as well. This value is responsible for the 'PD' to have lower cost in the 220 simulations out of 720. The results for each parameter can be found in Appendix B. It can be seen that all parameters except for 'safety margin' influence the average adaptation height. Stricter safety standards decrease the spread in average adaptation height. Larger standard deviation of the discharge statistics results in higher average adaptation height, which is similar for the steepness of the stage discharge relation. Larger mean discharges result in larger spread, but smaller average dike adaptation heights.

Table 6 – Percentage of simulations in which the costs are in favour of 'PD' scenario 3

Safety standards	**1/625**	**1/1,250**	**1/2,500**	**1/5,000**	**1/10,000**
Percentage prob. dike	19.4%	22.2%	26.4%	34.0%	50.7%
Safety margin [m]	**0**	**0.2**	**0.5**	**2**	
Percentage prob. dike	6.1%	10.0%	24.4%	81.7%	
Q-h relation coeff	**0.16**	**0.24**	**0.32**	**0.40**	
Percentage prob. dike	13.3%	25%	34.4%	49.4%	
Standard dev. mult.	**0.8**	**1.0**	**1.2**		
Percentage prob. dike	22.1%	29.6%	39.2%		
Mean multiplier	**0.8**	**1.0**	**1.2**		
Percentage prob. dike	37.5%	30%	24.2%		

Table 6 shows the percentage in which the 'PD' has lower total costs compared to the 'SLD'. This percentage is based on the 720 simulations of Chapter 5.2.1. For each set of parameters, the percentage in which the 'PD' scores has lower total costs, is shown. The table shows that a higher safety margin, a stricter safety standards, a steeper q-h relation, a higher standard deviation of annual maximum discharges and lower mean of the annual maximum discharges are favourable for the 'PD' design strategy.

Table 7 – Scenario 1 results of simulation over 720 parameter sets for the 'PD' and the 'SLD'

	'PD'		'SLD'		Difference	
	Min	Max	Min	Max	Min	Max
Adaptation frequency	2.11	2.17	1.00	1.01	1.11	1.16
Overtopping frequency	0	0.1393	0	0.1227	0	0.0258
Total cost	68.80	3411.50	65.33	3164.23	-326.03	813.06
Number of lowest cost	400				320	

Table 8 - Percentage of simulations of sensitivity analysis in which the cost are in favour of 'PD' scenario 1

Safety standards	**1/625**	**1/1,250**	**1/2,500**	**1/5,000**	**1/10,000**
Percentage prob. dike	25.7%	36.8%	57.6%	74.3%	83.3%
Safety margin [m]	**0**	**0.2**	**0.5**	**2**	
Percentage prob. dike	31.1%	39.4%	57.2%	94.4%	
Q-h relation coeff	**0.16**	**0.24**	**0.32**	**0.40**	
Percentage prob. dike	26.1%	51.1%	66.7%	78.3%	
Standard dev. mult.	**0.8**	**1.0**	**1.2**		
Percentage prob. dike	44.6%	55.8%	66.3%		
Mean multiplier	**0.8**	**1.0**	**1.2**		
Percentage prob. dike	57.1%	54.6%	55%		

A similar analysis is carried out for scenario 1, i.e. the "current" discharge conditions, without induced climate change. Results of scenario 1, see Table 7 and Table 8, show that the 'PD' has lower total costs in more simulations compared to scenario 3, 400 out of 720. For scenario 3, this number was 220 simulations. This indicates that the 'SLD' strategy is particularly favourable in conditions where climate change causes an increase in extreme river discharges.

5.3. Dike heightening requirements

5.3.1. Minimum adaptation height

Minimum adaptation height is the implementation of a minimum height if dike heightening is required. It is assumed that minimum adaptation height and adaptation threshold are related. The adaptation threshold is investigated in chapter 5.3.2.

To find the optimal value for the minimum adaptation height, the average costs of a set threshold values are put against the minimum adaptation height. Figure 17, gives a clear overview of cost against the minimum adaptation height. As can be seen, for both dike design approaches, the optimal value of the minimum height of dike adaptation, is about 0.7 meter.

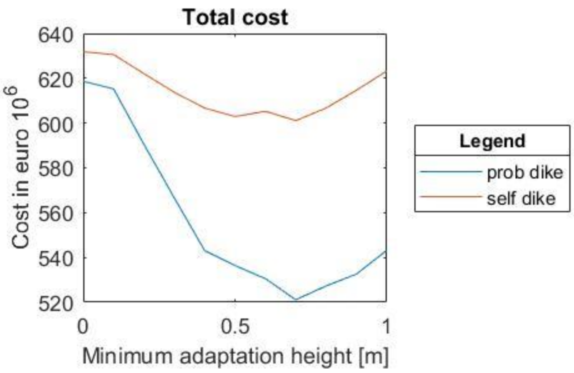

Figure 17 – Total costs of analysis on minimum adaptation height

A minimum value exists due to the ratio between investment cost and flood damage cost. As can be seen in Figure 18 and Figure 19, the frequency of adaptation and frequency of overtopping flattens out with an increasing minimum adaptation height. The frequency of adaptation can be clarified, since a high minimum adaptation increases the height of the first adaptations, which reduces the probability that subsequent adaptations are required in the simulation period. Overtopping will never reach 0, since overtopping can happen before an adaptation is executed. Increasing the adaptation height does not solve this problem and therefore the lines converge to a horizontal asymptote with increasing minimum adaptation height. This horizontal asymptote, Figure 18, is equal to the probability that the maximum discharge out of a 100 samples from the Gumbel distribution exceeds the design discharge plus a safety margin.

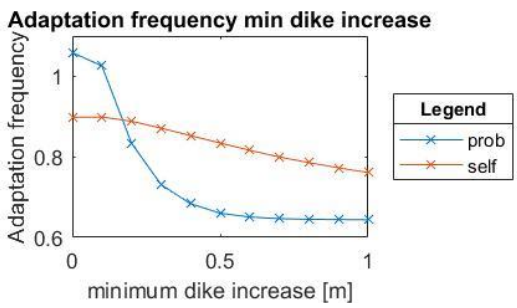

Figure 18 – Adaptation frequency of analysis on minimum adaptation height

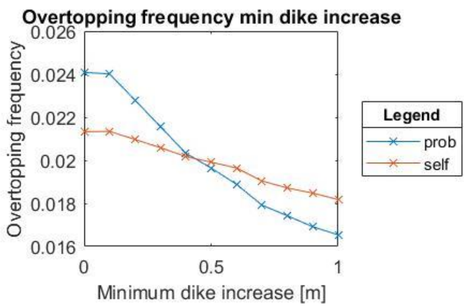

Figure 19 - Overtopping frequency of analysis on minimum adaptation height

Decreasing adaptation and overtopping frequency result in a decline in total cost. Investment cost remains somewhat equal, since the adaptation height is increased with the new adaptation criteria, while adaptation frequency decreases. Flood damage cost decreases, due to lower overtopping frequency. Above 0.7 meter of minimum adaptation height, adaptation and overtopping frequency do not change significantly, while the average adaptation height keeps increasing. Therefore, investment costs keep increasing, which results in an increase in total cost after 0.7 meter of minimum adaptation height, see Figure 17.

5.3.2. Adaptation threshold

Adaptation threshold is a value which needs to be exceeded to heighten a dike, values below this threshold value are not neglected. In Chapter 5.3.1 a minimum adaptation height is found of 0.7m and is taken into account in the analysis of threshold value for the adaptation criterion. As can be seen in Figure 20, the total costs for the 'PD' increase with larger adaptation threshold. For the 'SLD' the total costs remains somewhat equal. This can be related to the frequency of adaptation and overtopping, see Figure 21 and Figure 22.

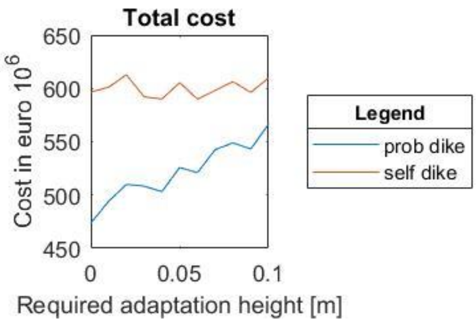

Figure 20 – Total cost of analysis on required adaptation height

The ratio between investment cost and flood damage cost is the main reason for the shape of the total cost curve. The flood damage cost is much higher than the investment cost and therefore, if the overtopping frequency increases, the total costs increase as well. This means for different circumstances (areas with different economic values), these findings will be different.

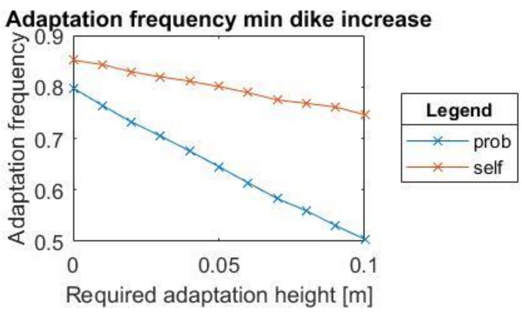

Figure 21 – Adaptation frequency of analysis on threshold value

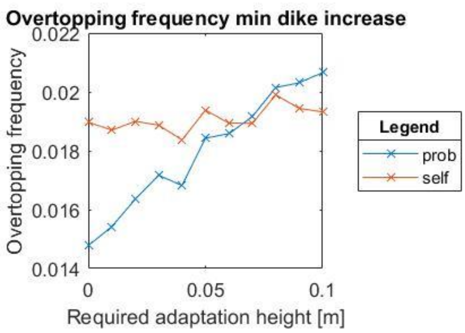

Figure 22 – Overtopping frequency of analysis on threshold value

Figure 20 shows that the total costs for the 'PD' are lowest when the threshold value is 0. This is remarkable, since it suggests that every adaptation is worth the investment. A dike is designed in part to be cost efficient. In the starting year of the simulation, the initial dike height is calculated such that flood cost and investment cost are equal estimates. This is the most optimal cost efficient design and therefore is closer to reality. To create a more realistic situation, the simulation therefore should immediately start with an adaptation. This decreases the flood cost and increases the investment cost. Simulations with different adaptation heights in the initial year show that 0.4 m is a value which results in an equilibrium between flood damage cost and investment cost, see Figure 23.

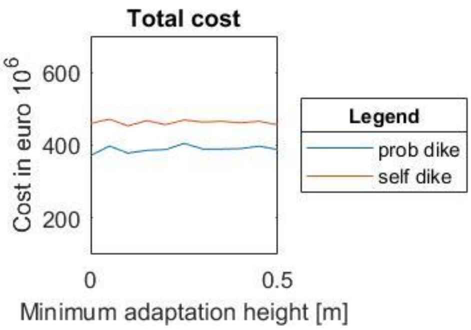

Figure 23 – Total cost of analysis on threshold value, including adaptation initial simulation year

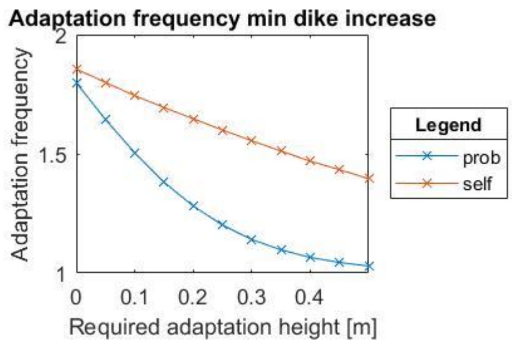

Figure 24 – Adaptation frequency of analysis on threshold value, including adaptation initial simulation year

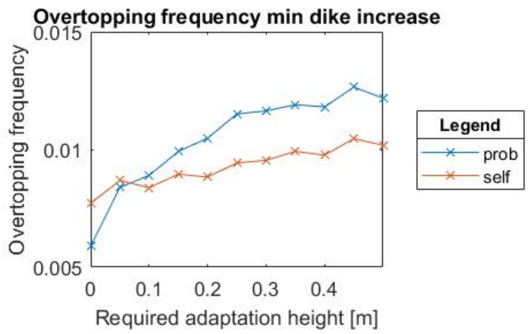

Figure 25 - Overtopping frequency of analysis on threshold value, including adaptation initial simulation year

An initial adaptation height decreases the overall frequency of overtopping, which reduces the flood damage cost and with that the total costs. Though an optimum value for adaptation threshold height can not be found.

In practice, it is not beneficial to execute small dike adaptations. For this reason, the minimum dike adaptation heigth will reduce the adaptation frequency and total costs. Dike adaptation brings besides cost also nuisance with it. These factors make the execution of a dike adaptation well considered. The model used in this study contains various uncertainties in the determination whether or not the dike adaptation criterion is exceeded. For this reason, it is determined that required dike adaptations below 30 cm are ignored. These calculated adaptations are not notably convincing, which therefore are to be neglected in the results.

A similar analysis is done for scenario 3. This analysis results in an optimum of 1 m minimum adaptation height. An optimal threshold value cannot be found for scenario 3, with the same explanation as for scenario 1 (see Chapter 5.3.2), the threshold value is chosen equally, 0.3 m. Based on the assumption that the current dike height is such that an equilibrium can be found in flood damage cost and investment cost, the initial dike height is heightened with 0.4 m as well.

The results of adaptation requirements based on scenario 1 and 3, using an adaptation threshold of 0.3 m, an adaptation in the initial year of 0.4 m and a minimum adaptation of respectively 0.7m and 1m, can be found in Table 9 and Table 10.

Table 9 – results with additional adaptation requirements based on scenario 1 and 3 for the 'PD' and the 'SLD'

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Adaptation requirements Scenario 1						
Number of dike overtopping	0.0119	0.0099	0.0170	0.0139	0.0246	0.0182
Number of adaptations	1.14	1.55	1.19	1.61	1.34	1.80
Average hight per adaptation [m]	0.70	0.91	0.70	0.94	0.70	0.94
Adaptation requirements Scenario 3						
Number of dike overtopping	0.121	0.0098	0.0165	0.0133	0.0236	0.0171
Number of adaptations	1.14	1.54	1.18	1.59	1.34	1.77
Average hight per adaptation [m]	1.00	1.11	1.00	1.14	1.00	1.13

Table 10 – Cost results with additional adaptation requirements based on scenario 1 and 3 for the 'PD' and the 'SLD'

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Adap. Req. Scenario 1						
Fixed costs of dike heightening in 10^6€	32.53	44.23	33.70	45.75	38.25	51.39
Variable cost of dike heightening in 10^6€	86.70	206.18	92.56	231.53	114.84	281.36
Flood damage over 100 years in 10^6€	275.37	217.52	398.25	302.25	665.49	481.63
Total cost in 10^6€	394.60	467.93	524.51	579.53	818.58	814.38
Adap. Req. Scenario 3						
Fixed costs of dike heightening in 10^6€	32.53	43.91	33.79	45.47	38.23	50.43
Variable cost of dike heightening in 10^6€	100.50	237.63	110.60	265.36	146.98	324.42
Flood damage over 100 years in 10^6€	266.49	205.89	407.43	312.29	652.96	462.06
Total cost in 10^6€	399.52	487.43	551.82	623.12	838.17	836.91

As can be seen, required adaptation criteria established according to scenario 1 has lower cost than the required adaptation criteria for scenario 3 in all three scenarios. These results show that it is more cost efficient to design the dike adaptation criteria for scenario 1, whether climate change will or will not happen. This is, because the current dike height (assumed this height includes the 0.4 m initial dike adaptation), is designed for scenario 1.

5.4. Simulation period

Table 11 shows an extension of the simulation period to 300 years. The results, when compared to Table 2, show a small increase in adaptation and overtopping frequency for both dike design approach in scenario 1 and 2. Scenario 3 reveals a huge benefit of the 'SLD' compared to the 'PD'. The 'PD' needs 5 times more adaptations in 300 year simulation compared to 100 year simulation, while the 'SLD' remains below 3 times the frequency of adaptations for 100 year simulation.

Table 11 – results for the 'PD' and the 'SLD' after 300 simulation years

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Number of dike overtopping	0.0645	0.0413	0.0748	0.0468	0.2955	0.1362
Number of adaptations	3.24	1.96	3.43	1.99	18.90	3.47
Extra hight per adaptation [m]	0.06	0.49	0.06	0.50	0.05	0.58

Table 12 – results for the 'PD' and the 'SLD' after 300 simulation years and including the adaptation criteria

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Adaptation requirements Scenario 1						
Number of dike overtopping	0.0335	0.0190	0.0378	0.0218	0.1507	0.0677
Number of adaptations	1.19	2.09	1.22	2.12	2.36	3.06
Average hight per adaptation [m]	0.70	0.91	0.70	0.92	0.70	0.99
Adaptation requirements Scenario 3						
Number of dike overtopping	0.0319	0.0176	0.0369	0.0198	0.1218	0.0593
Number of adaptations	1.20	2.02	1.22	2.04	2.05	2.84
Average hight per adaptation [m]	1.00	1.12	1.00	1.12	1.00	1.17

Table 13 – Cost results for the 'PD' and the 'SLD' after 300 simulation years and including the adaptation criteria

	Scenario 1		Scenario 2		Scenario 3	
	'PD'	'SLD'	'PD'	'SLD'	'PD'	'SLD'
Adap. Req. Scenario 1						
Fixed costs of dike heightening in 10^6€	34.11	59.55	34.84	60.38	67.31	87.20
Variable cost of dike heightening in 10^6€	351.07	340.18	98.07	356.31	257.51	713.96
Flood damage over 100 years in 10^6€	15,345.97	6,554.82	17,877.19	7,461.30	155,794.71	63,154.46
Total cost in 10^6€	15,474.53	6,954.55	18,010.10	7,877.99	156,119.53	63,955.62
Adap. Req. Scenario 3						
Fixed costs of dike heightening in 10^6€	34.13	57.52	34.83	58.14	58.41	80.83
Variable cost of dike heightening in 10^6€	114.04	391.12	119.91	403.87	317.76	755.32
Flood damage over 100 years in 10^6€	15,550.29	5,498.63	17,799.16	6,484.21	120,565.47	55,283.05
Total cost in 10^6€	15,698.46	5,947.27	17,953.90	6,946.22	120,941.64	56,119.20

Table 12 and Table 13 show the results of 300 years of simulation including the adaptation criteria discussed in Chapter 5.3. The 'SLD' scores significantly better in total cost. This can be explained with the ratio between the investment costs and the flood damage cost. Flood damage increases over time due to economic growth, see Chapter 3.2.4. The investment cost is not dependent on time. An extension of simulation years to 300 affects which adaptation requirements score better.

With longer simulation period, the 'PD' scores worse in scenario 1 with adaptation requirement designed for scenario 3. For scenario 2, the 'PD' scores somewhat equal, while in 3 the 'PD' scores significantly better. The 'SLD' benefits in all scenarios of adaptation requirements designed for scenario 3, compared to the adaptation requirements design for scenario 1.

6. Discussion

H&K showed that the 'SLD' design strategy is more efficient than the 'PD' design strategy in river Rhine conditions over a simulation period of 100 years. The benefits of the 'SLD' design strategy are, according to H&K, the biggest in climate change conditions. Our study showed that the efficiency of the 'PD' and 'SLD' design strategies is sensitive to different simulation circumstances.

Simulations in river Meuse conditions showed the benefit of the 'SLD' even more than in river Rhine conditions. The number of overtopping as well as the total costs are lower for the 'SLD' in all three scenario's. Reason for this is that the safety level, provided by a safety margin of 0.5 m against overtopping, is much bigger in river Meuse conditions, than in river Rhine conditions, which is due to the difference in stage-discharge steepness. The advantage of the 'SLD' is most notable in scenario 3, which includes an increase in peak discharges as a result of climate change. Effects of climate change are expected to have bigger effect on the maximum annual discharges of river Meuse than for the river Rhine (MTPWWM, 2005 & Reuber J. et al, 2006). Because of this, the benefits of the 'SLD' are even bigger in simulations of river Meuse circumstances, including climate change. This information indicates that in regions, where peak discharges in rivers are expected to rise to a large extend due to climate change, it is beneficial to implement the 'SLD' design strategy over the 'PD' design strategy.

The sensitivity analyses showed that the 'SLD' has lower total costs in significantly more simulations than the 'PD', when including climate change. Though, if climate change is not included in the simulations, the 'PD' has lower total costs in slightly more simulations. A smaller safety standard, smaller steepness of the q-h relation, lower safety margin, smaller standard deviation in maximum annual discharges and larger mean of maximum annual discharges make the 'SLD' design strategy benefit in the relative efficiency between both dike strategies.

The performance of the 'PD' design strategy relative to the 'SLD' can be improved by adding requirements to the consideration of dike heightening, which are not used by H&K. A minimum height for dike adaptation of 0.7 m, a threshold value to heighten a dike of 0.3 m and an adaptation of the dikes in the initial simulation year of 0.4 m, creates a more realistic situation where the ratio between investment cost and flood damage cost is closer to an equilibrium. The threshold value to heighten a dike, is not calculated using our calculation tool, but is established on common knowledge and discussion with the supervisors, which leaves room for improvement.

Expanding the simulation period of 100 years to 300 years increases the benefits of the 'SLD' design strategy. Climate change causes a serious increase in number of adaptations of the 'PD' design strategy, but only a small increase in number of adaptations of the 'SLD' design strategy. Including the aforementioned dike heightening requirements increases the benefits of the 'PD' and make the number of adaptations in the simulation period smaller for the 'PD'. Though, the land which is protected by the 'PD' will have to deal with more floods, compared to the 'SLD' design strategy. To provide this safety with the 'SLD' design strategy, the safety margin, s (which is added to the highest observed water level) needs to be calculated, which in this study is done using the 'PD' design strategy. This means that, even though the 'SLD' is an easier method to implement, the safety level provided by this dike design strategy, is still based on the probabilistic approach of the 'PD'.

The calculation tool which is used to gather all results in this study, contains uncertainty and is based on the study by H&K. Some input, which are included by H&K cannot be reproduced, since information is missing. The most important uncertainty is found in the reproduction of the cost calculation, which leaves a lot of room for interpretation. The results of the costs calculation in this study, do not comply with the results of H&K, but are checked and respected (Kind J. M., 2020). The number of adaptations and number of overtopping are verified with the study of H&K which showed that, input containing uncertainty does not have a large influence on these two criteria.

7. Conclusion

The efficiency of both dike design strategies in this study relative to each other, is sensitive to changes in the operation of dike heightening or input into the calculation tool. In the conclusions that are made, it should be considered that the calculation tool and operation of both dike design strategies leaves room for improvement.

The ‘SLD’ design strategy scores better in the study by H&K. Reason for this, is the fact that the ‘PD’ design strategy requires relative small adaptations over a simulation period of 100 years in river Rhine conditions, which combined are more expensive than one larger adaptation of the ‘SLD’.

Simulations on river Meuse circumstances at Borgharen and the sensitivity analyses show that the benefits of the ‘SLD’ design strategy are the biggest in climate change conditions. Smaller safety standard and safety margin are beneficial to the ‘SLD’ design strategy. A river with a steeper stage-discharge relation is favourable to the ‘PD’ design strategy. Peak discharges with larger mean and smaller standard deviation compared to river Rhine conditions, make the ‘SLD’ design strategy perform relatively “better”, based on total costs.

It is questionable whether or not the ‘SLD’ is significantly “better” than the ‘PD’ design strategy, since, for example, the implementation of adaptation requirements (such as minimum height of a dike adaptation) does influence the results, such that the relative difference in efficiency of both dike designs strategies is not significant any more. This addition could be one of possibly many model improvements, which make the calculation tool approach reality more closely. This makes the results of the study by H&K and the results of our study doubtful, since the implementation of only one addition, namely the requirements for dike heightening, highly affects the relative efficiency of the dike design strategies.

8. Recommendation

To further investigate the topic discussed in this report, some recommendation can be given to improve the reliability and extend the knowledge of this research. This chapter discusses some of the possible recommendations.

One of the most important improvements to this research is about the cost calculation. The cost calculation of the total cost, investment cost and flood damage are important parameters which give information about the efficiency of a dike. In this research, the cost calculation is not analysed and investigated intensively, even though the total costs determine which dike design strategy is more efficient. To make stronger statements about which dike is better, more intense research is required on the cost calculation. Also, the total cost is constructed out of the investment costs and the flood damage costs. Over time, the flood damage increases due to annual economic growth. The investment cost on the other hand, is not time dependent. Therefore, the flood damage cost is the major contributor to the total cost in long term simulations. This is not realistic since investment cost should grow as well. Therefore, further investigation on the cost calculation should be done to increase the reliability of the conclusions.

Adaptation criteria, such as the minimum adaptation height, are investigated and implemented to approach reality more closely. In this research, adaptation criteria are investigated based on the investment cost and the increase in safety level. To expand the reliability of the adaptation criteria, other nuisance should be taken into account as well. Further research on, for example, negative externalities, such as noise nuisance, should be investigated intensively. Adaptation criteria are a model improvement which is investigated in this study, in which we did not focus especially on improving the model. A validation study on the calculation tool, might be valuable to gain more realistic and reliable results.

In the establishment of safety margin, s, for the 'SLD' design strategy, as mentioned in Chapter 6, the probabilistic strategies are essential. Investigating a different method to establish the safety margin, s, might increase the performance of the 'SLD', but this should be investigated in future research.

In our study, we shortly discussed the relation of the performance of the 'SLD' design strategy with the number of standard deviations from the mean to the maximum peak discharge of the time series. In this study, this relation is not further analysed, but might be a valuable addition to understanding the way the 'SLD' operates. Also the influence of the magnitude of climate change might be an important addition. This study showed that larger increase in design discharge (river Meuse conditions) over a simulation period of 100 years is in favour to the 'SLD' design strategy, but to what extend is not analysed.

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A. Mathematical model

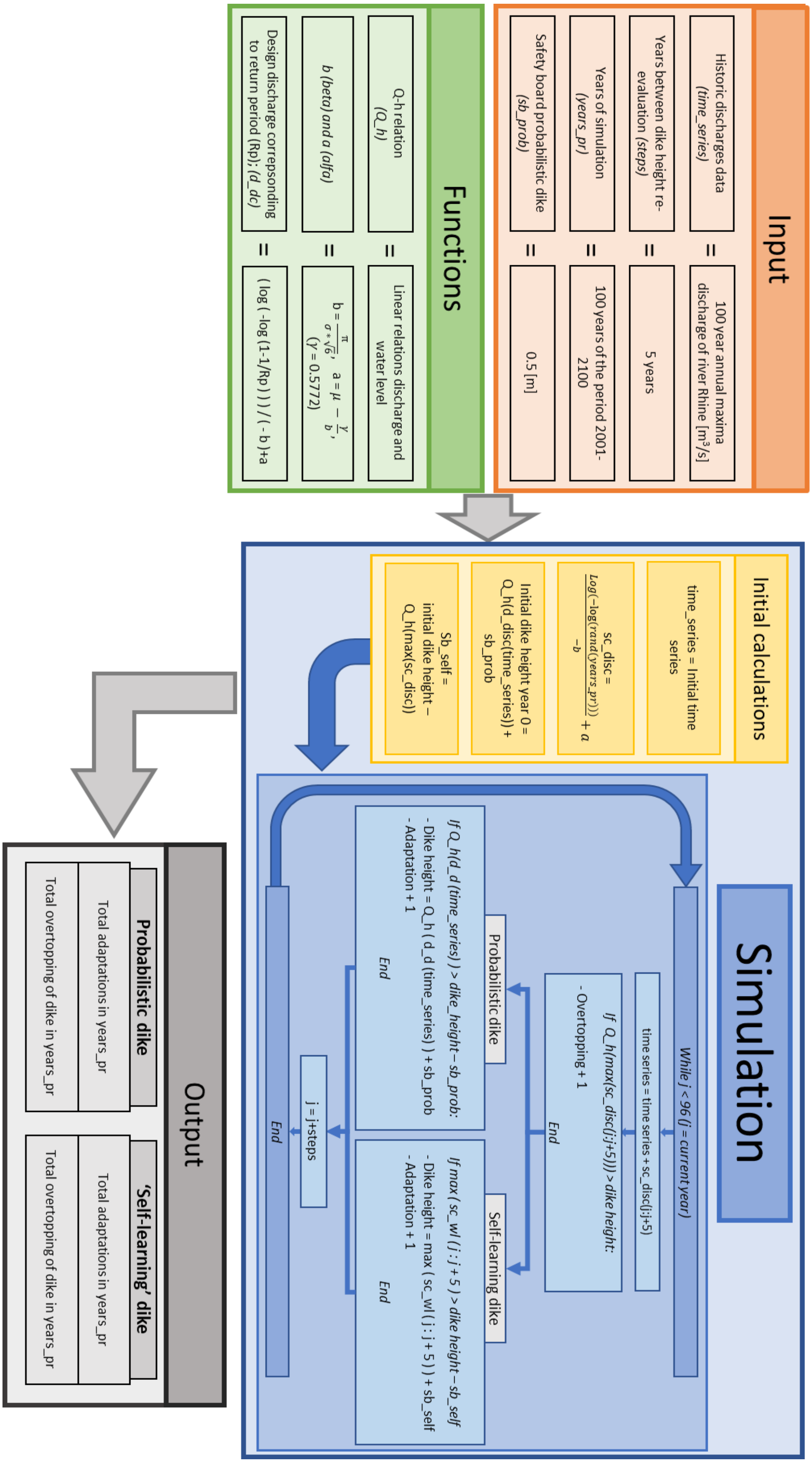

Figure 26 - Simplification of model including adaptation and overtopping frequency

B. Sensitivity analysis

In this chapter, figures can be found which show the results of the sensitivity analysis per parameter. Results are given for 'adaptation frequency', 'average adaptation height', 'overtopping frequency' and the total cost. The results are visualized in boxplots, to show the mean and spread. The value of the parameter cannot be read from the figures, but are increased from left to right. All parameter sets can be found in Chapter 4.2.

B.1 Safety margin

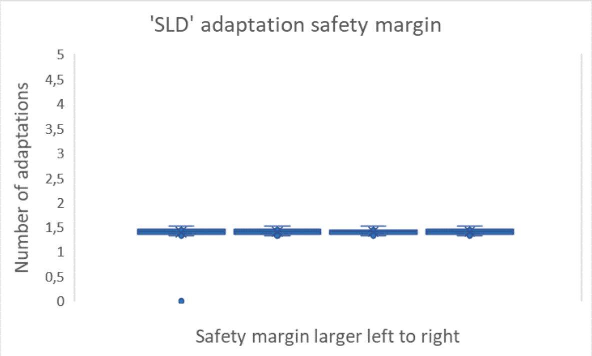

Figure 27 – Adaptation frequency for sensitivity safety margin 'SLD'

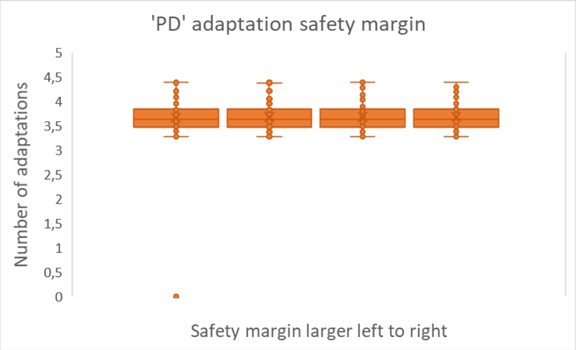

Figure 28 – Adaptation frequency for sensitivity safety margin 'PD'

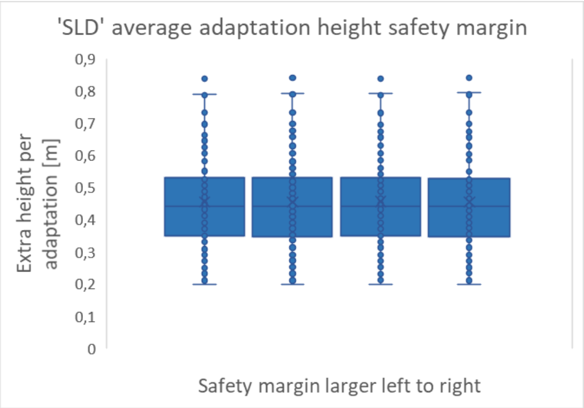

Figure 29 – Average adaptation height for sensitivity safety margin 'SLD'

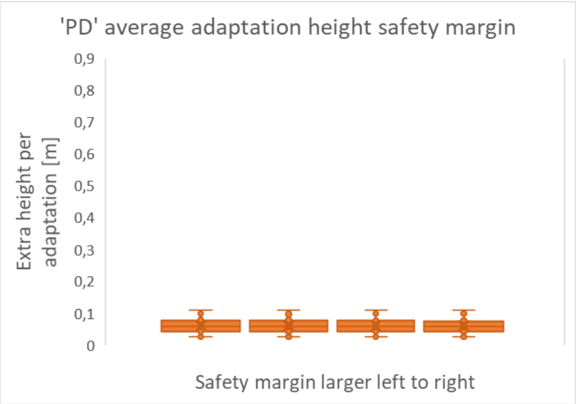

Figure 30 – Average adaptation height for sensitivity safety margin 'PD'

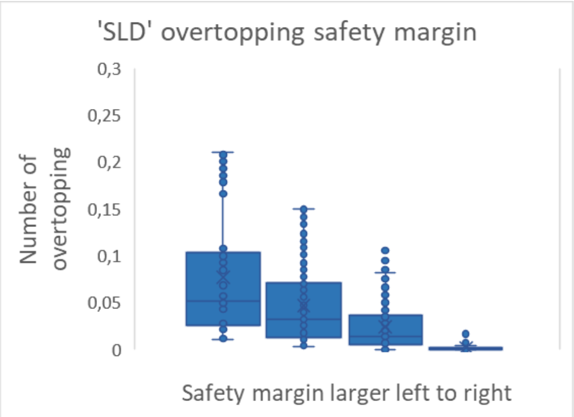

Figure 31 – Overtopping frequency for sensitivity safety margin 'SLD'

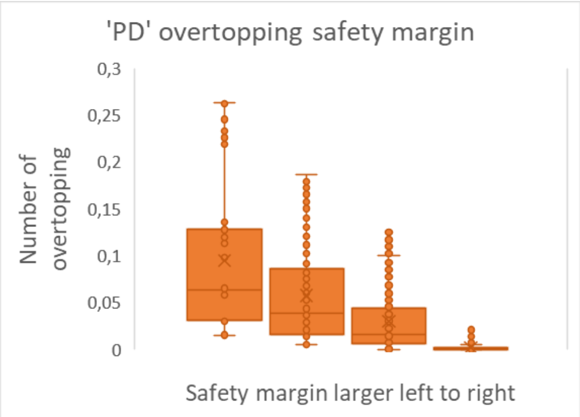

Figure 32 – Overtopping frequency for sensitivity safety margin 'PD'

Figure 33 – Total cost for sensitivity safety margin 'SLD'

Figure 34 – Total cost for sensitivity safety margin 'PD'

B.2. Standard deviation multiplier

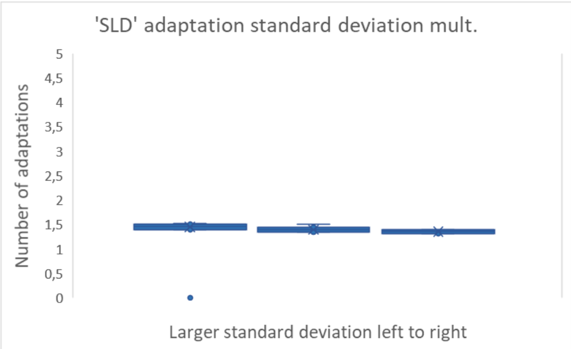

Figure 35 – Adaptation frequency for sensitivity standard deviation 'SLD'

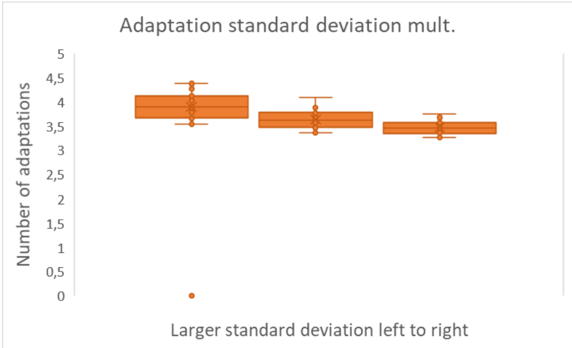

Figure 36 – Adaptation frequency for sensitivity standard deviation 'PD'

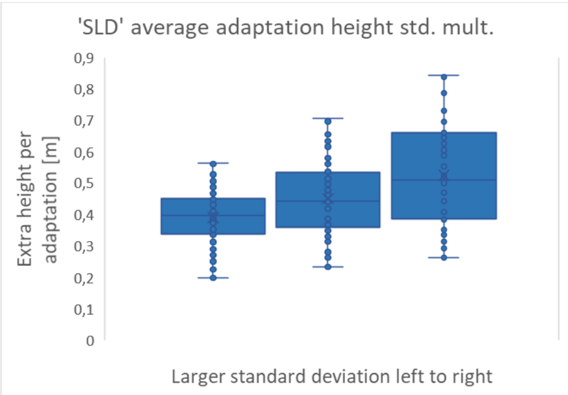

Figure 37 – Average adaptation height for sensitivity standard deviation 'SLD'

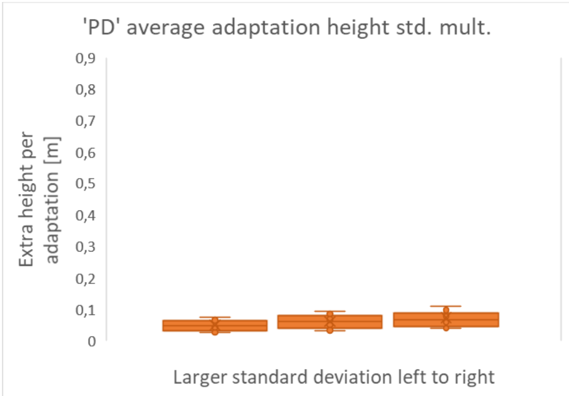

Figure 38 – Average adaptation height for sensitivity standard deviation 'PD'

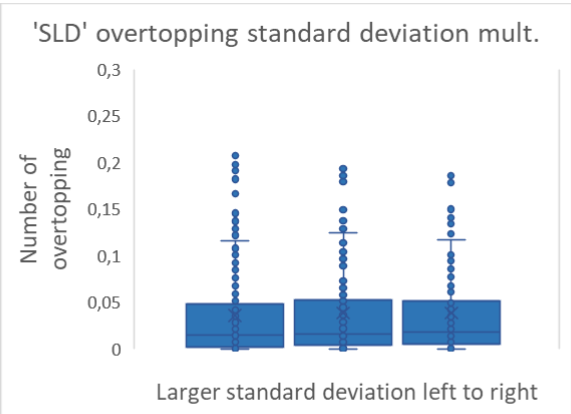

Figure 39 – Overtopping frequency for sensitivity standard deviation 'SLD'

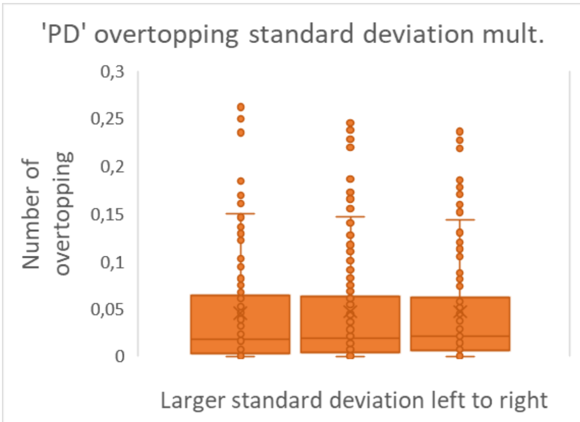

Figure 40 – Overtopping frequency for sensitivity standard deviation 'PD'

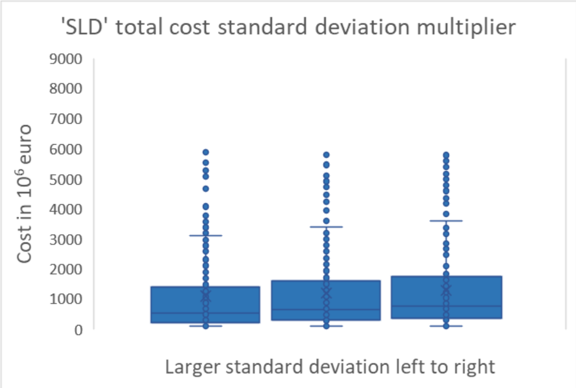

Figure 41 – Total cost for sensitivity standard deviation 'SLD'

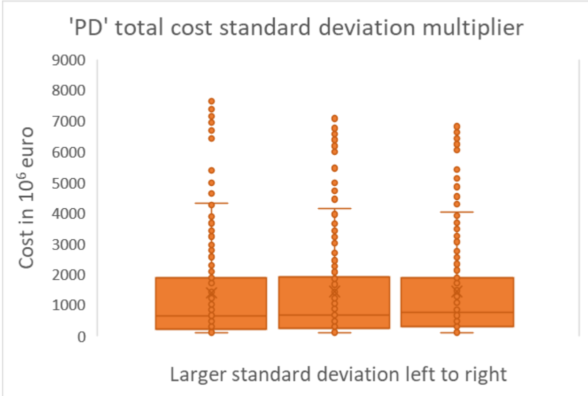

Figure 42 – Total cost for sensitivity standard deviation 'PD'

B.3. Stage discharge relation

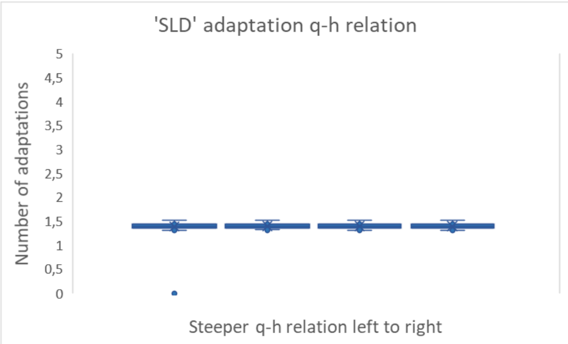

Figure 43 – Adaptation frequency for sensitivity q-h relation 'SLD'

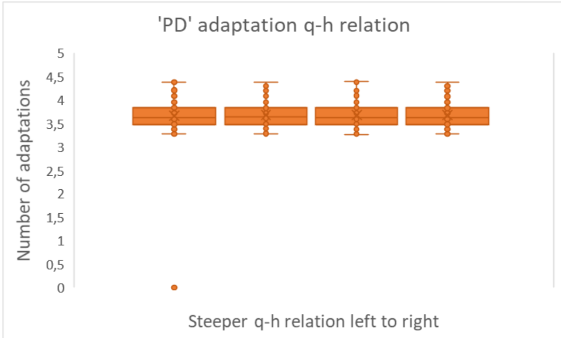

Figure 44 – Average adaptation height for sensitivity q-h relation 'PD'

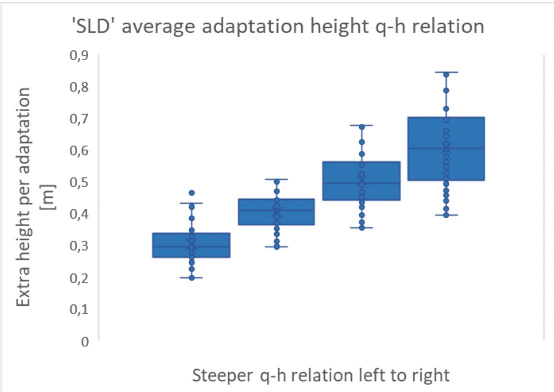

Figure 45 – Average adaptation height for sensitivity q-h relation 'SLD'

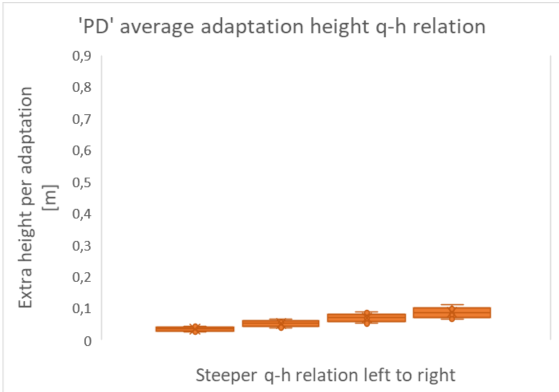

Figure 46 – Average adaptation height for sensitivity q-h relation 'PD'

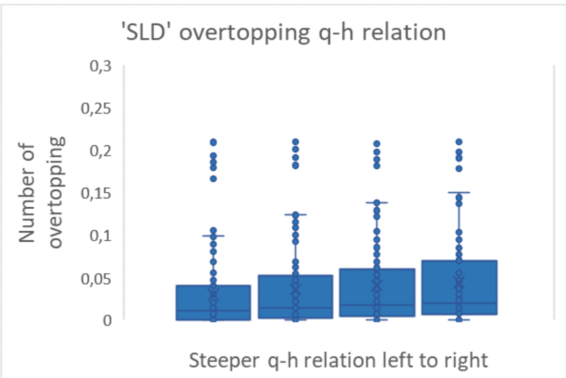

Figure 47 – Overtopping frequency for sensitivity q-h relation 'SLD'

Figure 48– Overtopping frequency for sensitivity q-h relation 'PD'

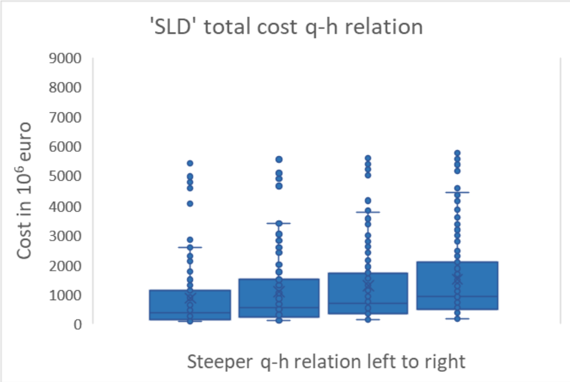

Figure 49 – Total cost for sensitivity q-h relation 'SLD'

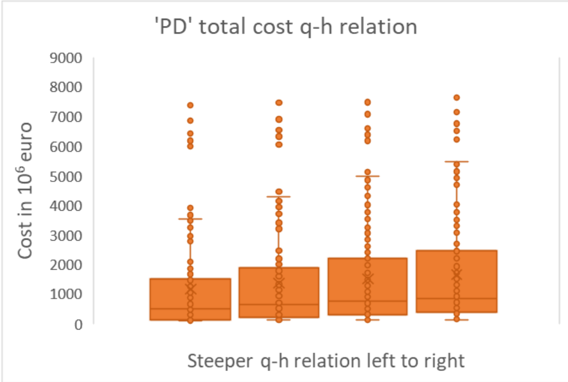

Figure 50 – Total cost for sensitivity q-h relation 'PD'

B.4. Mean multiplier

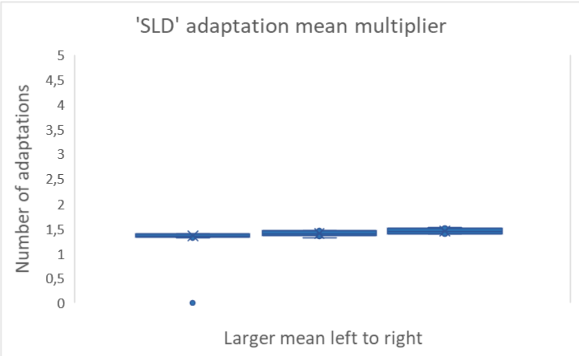

Figure 51 – Adaptation frequency for sensitivity mean 'SLD'

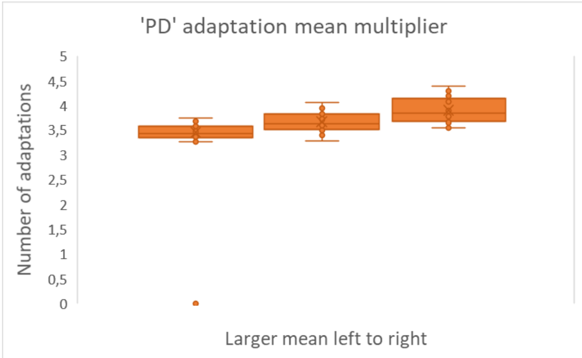

Figure 52 – Average adaptation height for sensitivity mean 'PD'

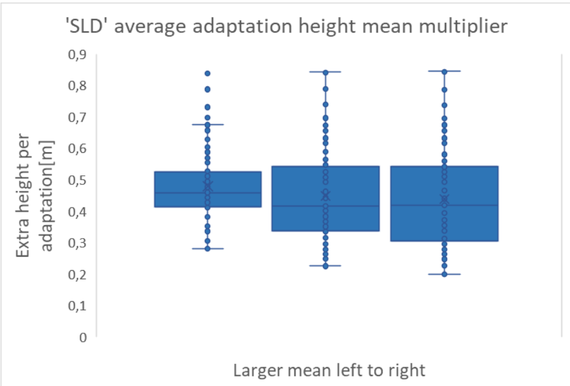

Figure 53 – Average adaptation height for sensitivity mean 'SLD'

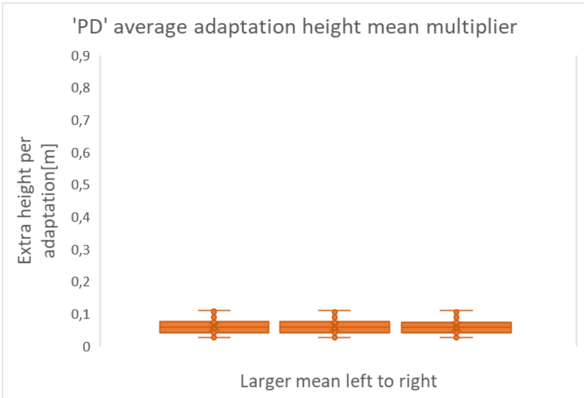

Figure 54 – Average adaptation height for sensitivity mean 'PD'

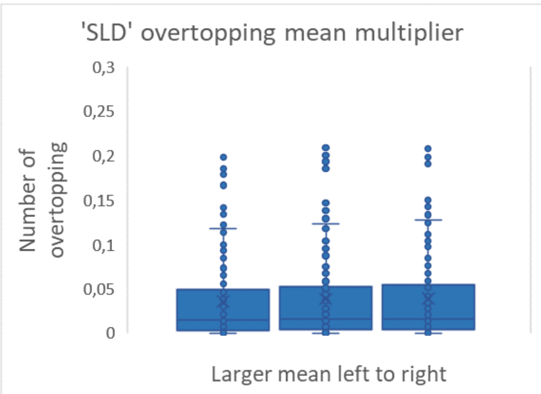

Figure 55 – Overtopping frequency for sensitivity mean 'SLD'

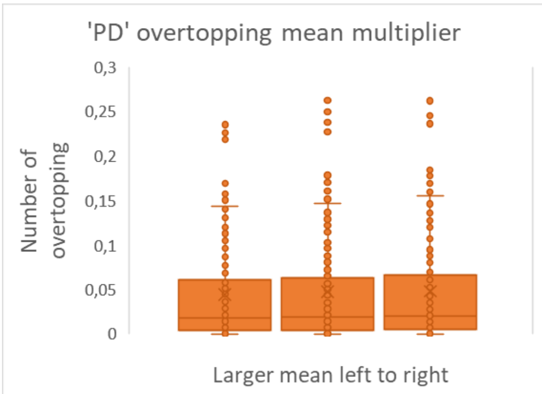

Figure 56 – Overtopping frequency for sensitivity mean 'PD'

Figure 57 –Total cost for sensitivity mean 'SLD'

Figure 58 –Total cost for sensitivity mean 'PD'

C. Covariance of condition parameters

This chapter contains figures of the correlation analysis for all pairs of parameters. On the left side, figures for the ‘PD’ can be found, on the right hand side, the ‘SLD’. Some of the figures in the adaptation frequency, show odd shapes. The variation of the adaptation frequency in these odd shapes is close to 0 and therefore can be ignored. The first sub-chapter contains the results for the adaptation frequency, followed by the overtopping frequency.

C.1. Adaptation frequency

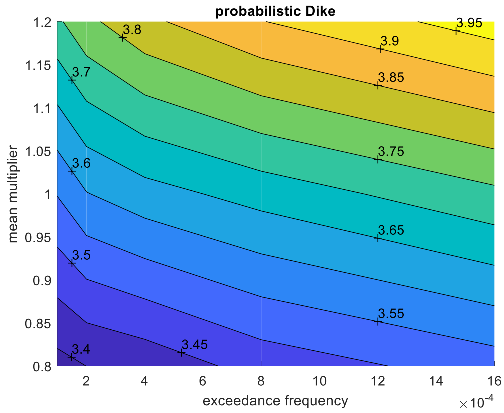

Figure 59 – Mean vs. Exc. Freq. Adaptation frequency ‘PD’

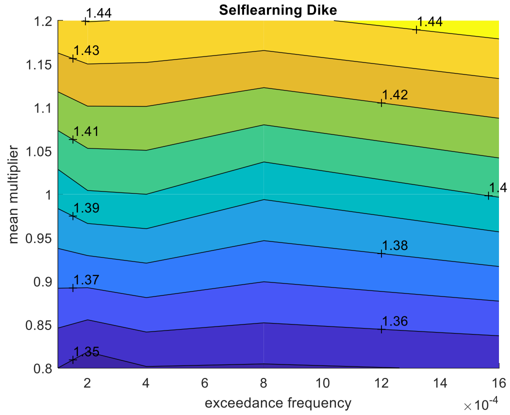

Figure 60 - Mean vs. Exc. Freq. Adaptation frequency ‘SLD’

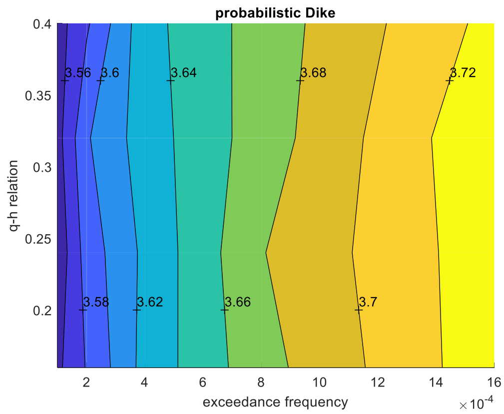

Figure 61 – q-h rel. vs Exc. Freq. Adaptation frequency ‘PD’

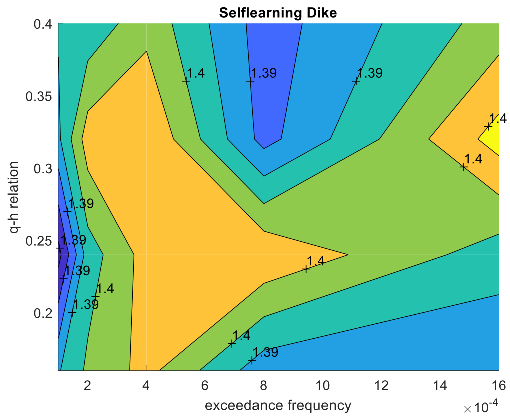

Figure 62 - q-h rel. vs Exc. Freq. Adaptation frequency ‘SLD’

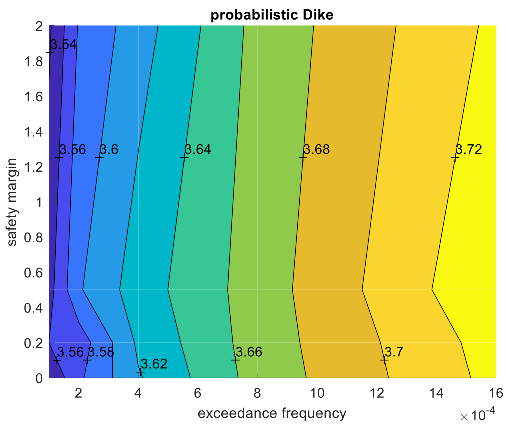

Figure 63 – Saf. Mar. Vs. Exc. Freq. Adaptation frequency ‘PD’

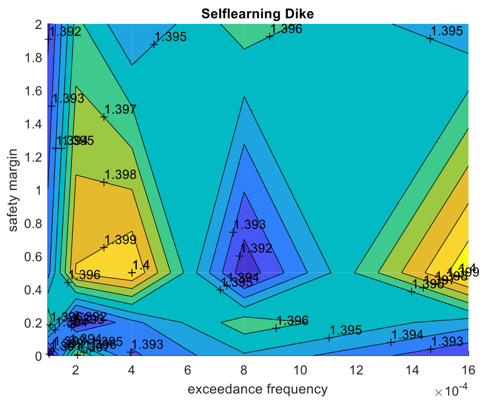

Figure 64 - Saf. Mar. Vs. Exc. Freq. Adaptation frequency ‘SLD’

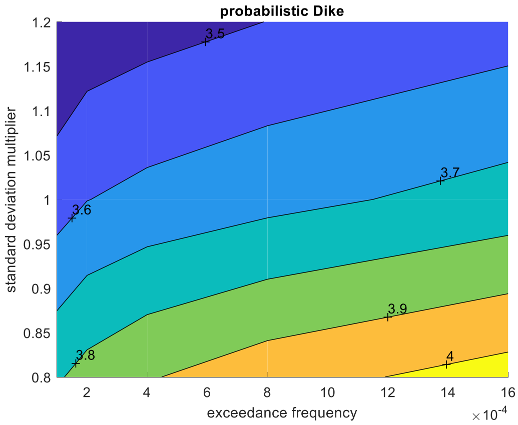

Figure 65 – Stan. Dev. Vs. Exc. Freq. Adaptation frequency ‘PD’

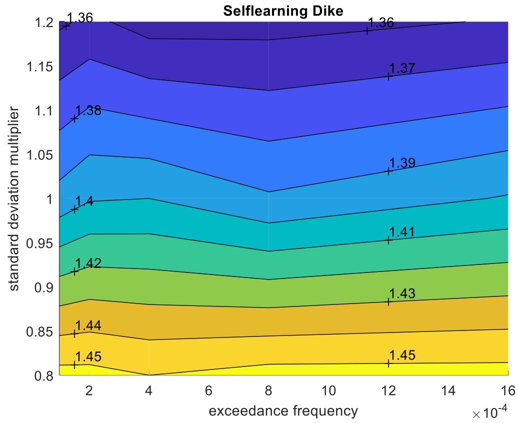

Figure 66 - St. Dev. Vs. Exc. Freq. Adaptation frequency ‘SLD’

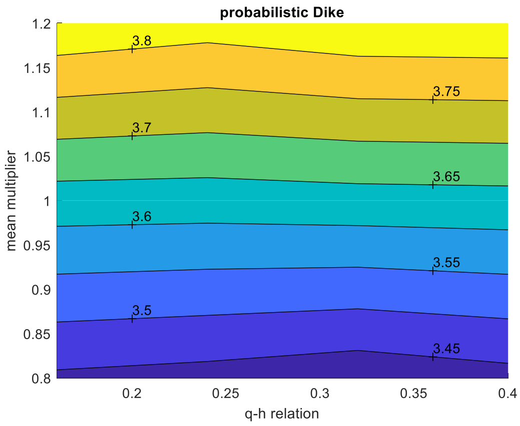

Figure 67 – Mean Vs. Q-h rel. Adaptation frequency ‘PD’

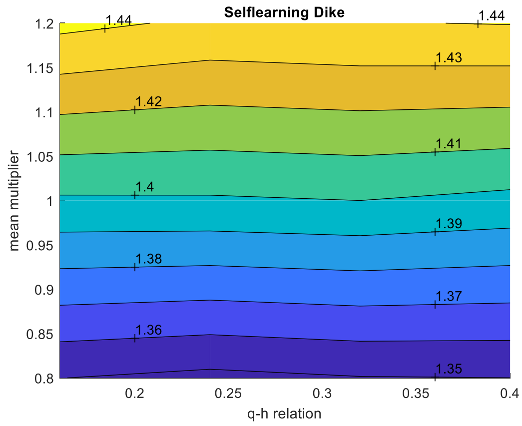

Figure 68 - Mean Vs. Q-h rel. Adaptation frequency ‘SLD’

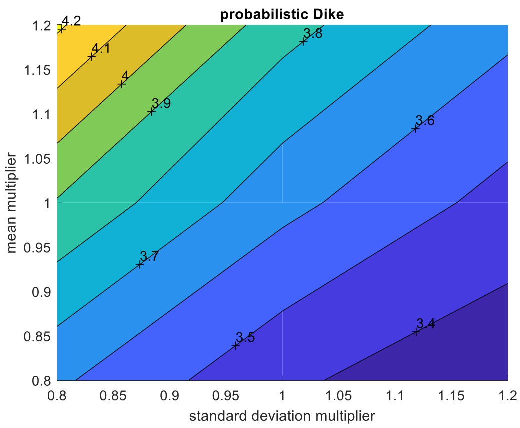

Figure 69 – Mean Vs. St. Dev. Adaptation frequency ‘PD’

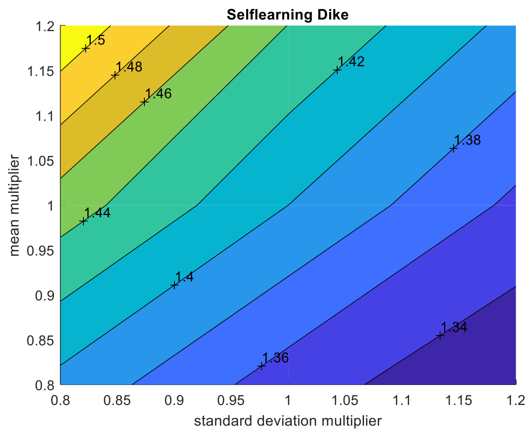

Figure 70 - Mean Vs. St. Dev. Adaptation frequency ‘SLD’

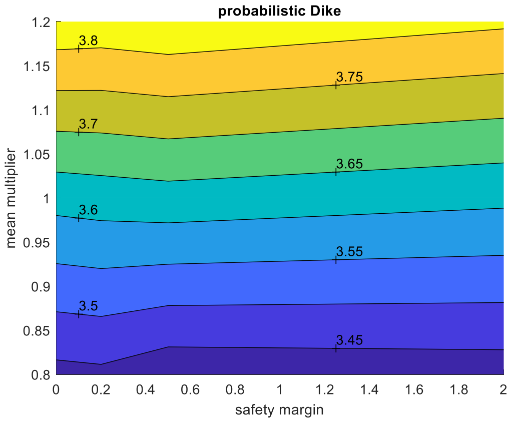

Figure 71 – Mean Vs. Saf. Mar. Adaptation frequency 'PD'

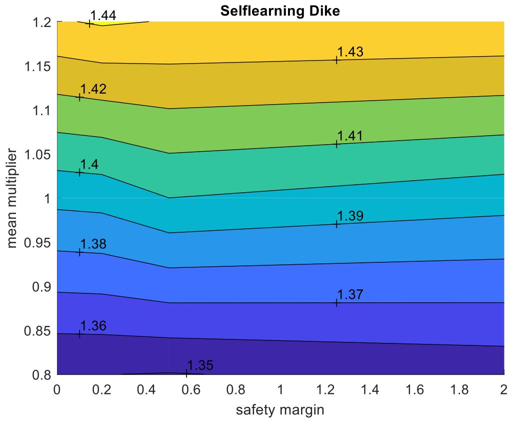

Figure 72 – Mean Vs. Saf. Mar. Adaptation frequency 'SLD'

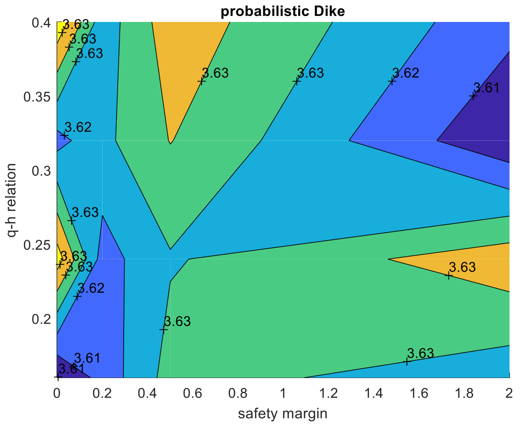

Figure 73 – q-h Rel. Vs. Saf. Mar. Adaptation frequency 'PD'

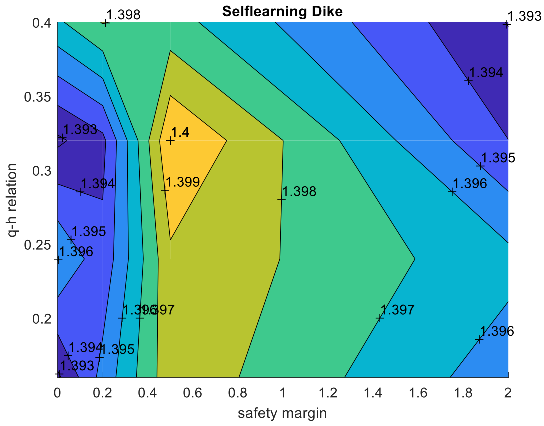

Figure 74 – q-h Rel. Vs. Saf. Mar. Adaptation frequency 'SLD'

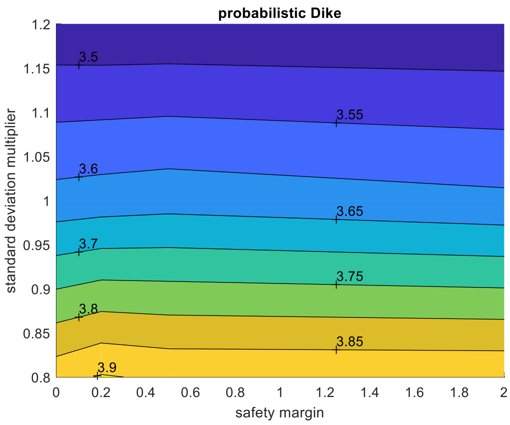

Figure 75 – St. Dev. Vs. Saf. Mar. Adaptation frequency 'PD'

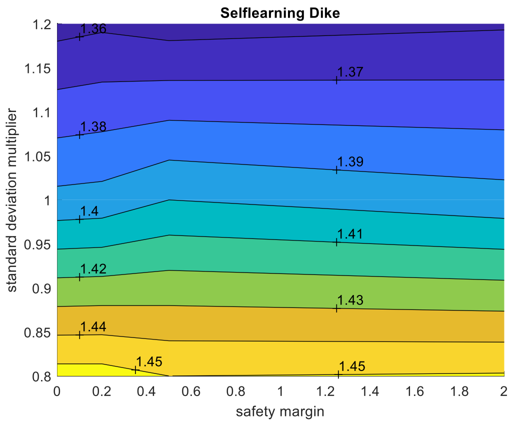

Figure 76 – St. Dev. Vs. Saf. Mar. Adaptation frequency 'SLD'

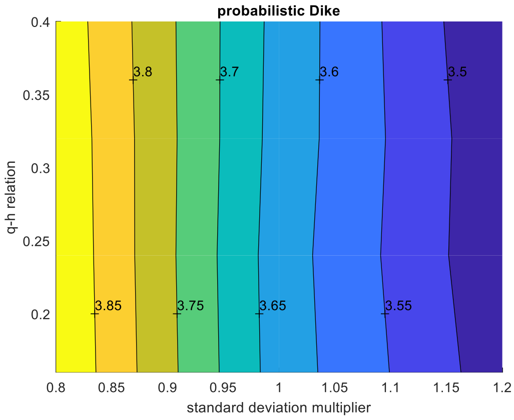

Figure 77 – q-h Rel. Vs. St. Dev. Adaptation frequency 'PD'

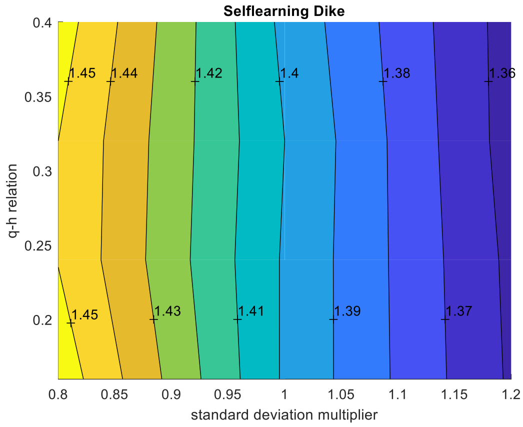

Figure 78 - q-h Rel. Vs. St. Dev. Adaptation frequency 'SLD'

C.2. Overtopping frequency

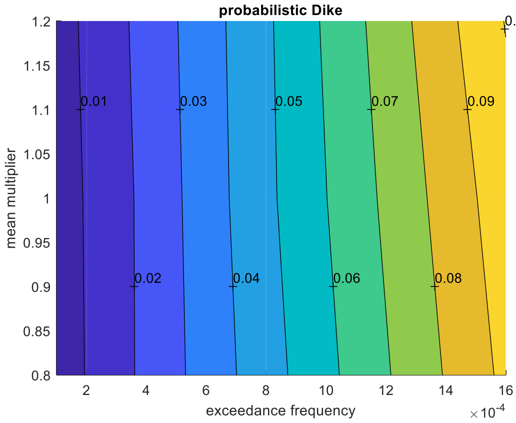

Figure 79 – Mean Vs. Exc. Freq. Overtopping frequency 'PD'

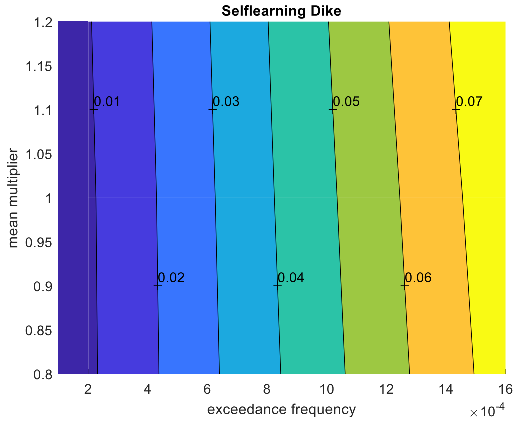

Figure 80 – Mean Vs. Exc. Freq. Overtopping frequency 'SLD'

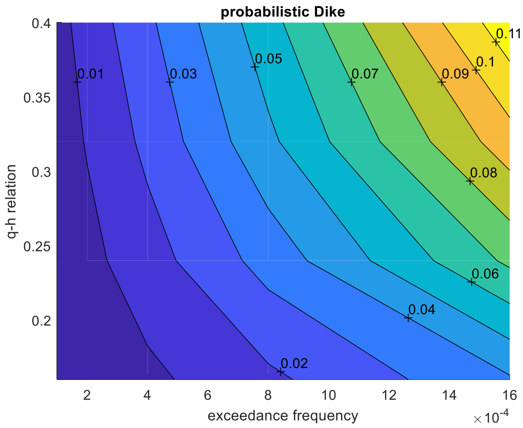

Figure 81 – q-h Rel. Vs. Exc. Freq. Overtopping frequency 'PD'

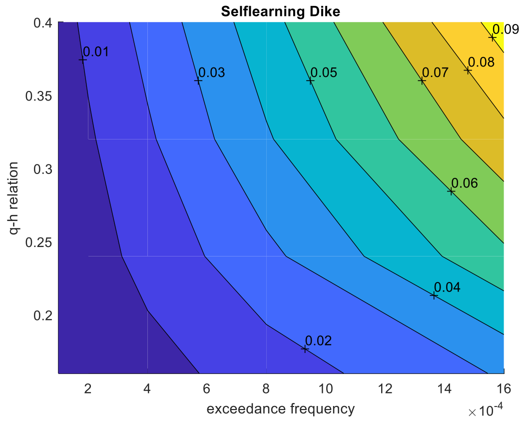

Figure 82 - q-h Rel. Vs. Exc. Freq. Overtopping frequency 'SLD'

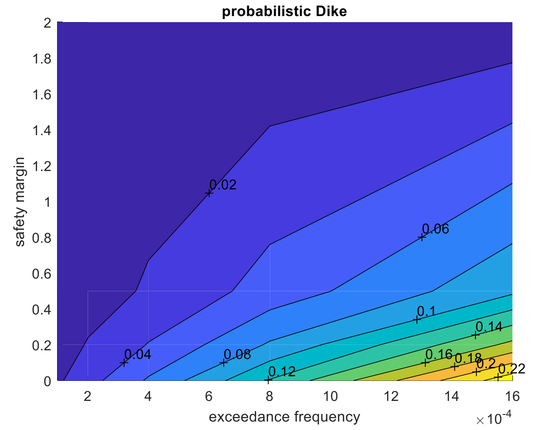

Figure 83 – Saf. Mar. Vs. Exc. Freq. Overtopping frequency 'PD'

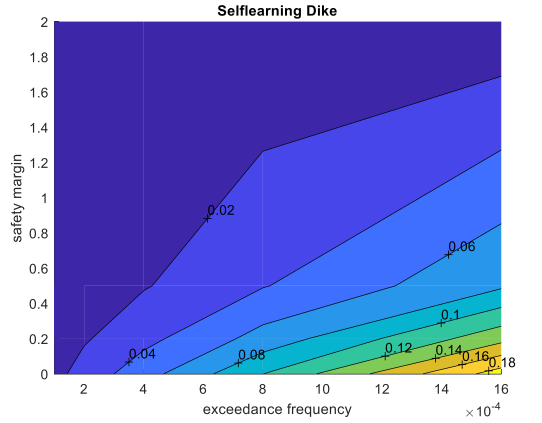

Figure 84 – Saf. Mar. Vs. Exc. Freq. Overtopping frequency 'SLD'

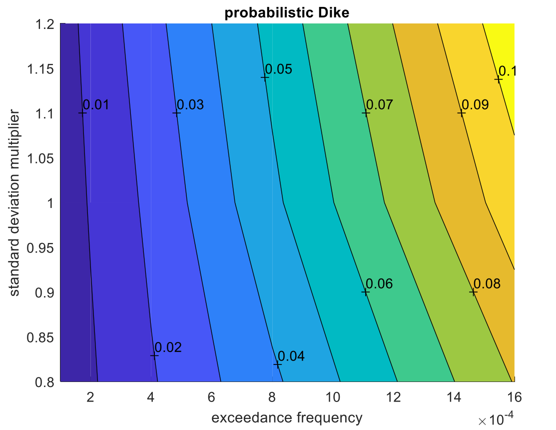

Figure 85 - St. Dev. Vs. Exc. Freq. Overtopping frequency 'PD'

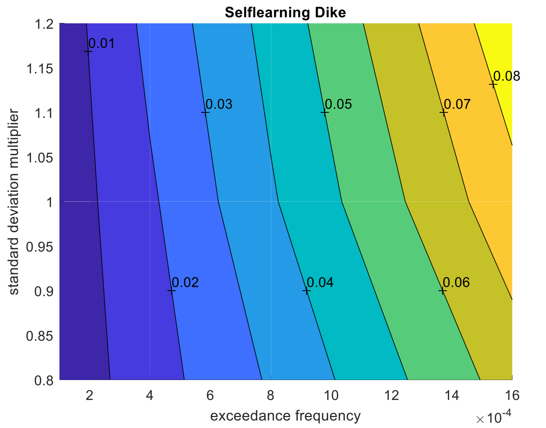

Figure 86 – St. Dev. Vs. Exc. Freq. Overtopping frequency 'SLD'

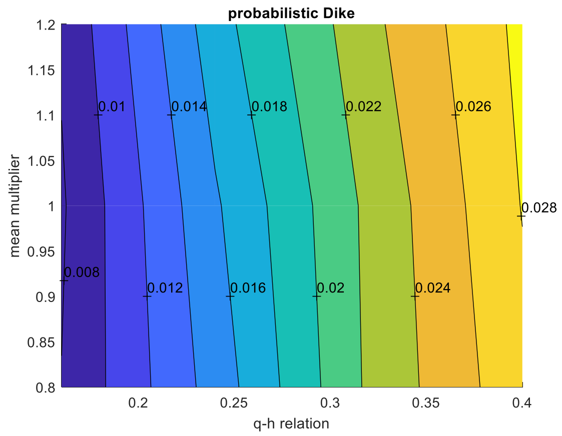

Figure 87 – Mean Vs. q-h Rel. Overtopping frequency 'PD'

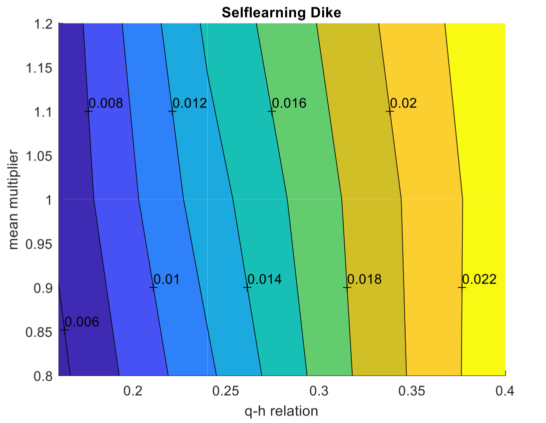

Figure 88 - Mean Vs. q-h Rel. Overtopping frequency 'SLD'

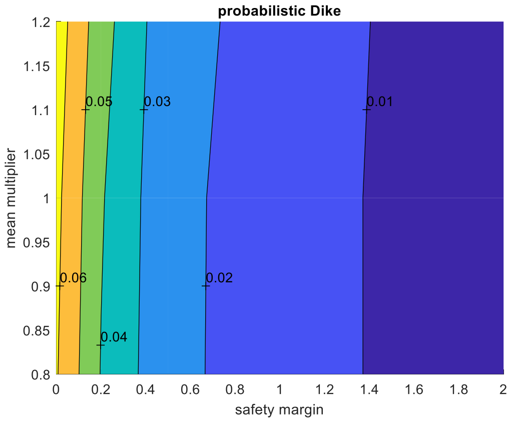

Figure 89 – Mean Vs. Saf. Mar. overtopping frequency 'PD'

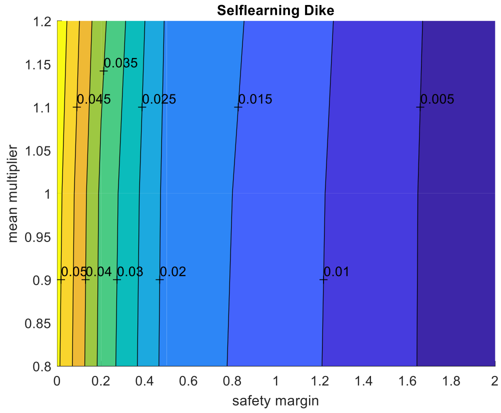

Figure 90 - Mean Vs. Saf. Mar. overtopping frequency 'SLD'

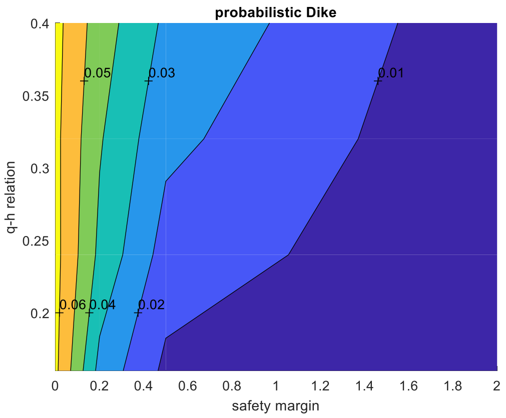

Figure 91 – q-h Rel. Vs. Saf. Mar. Overtopping frequency 'PD'

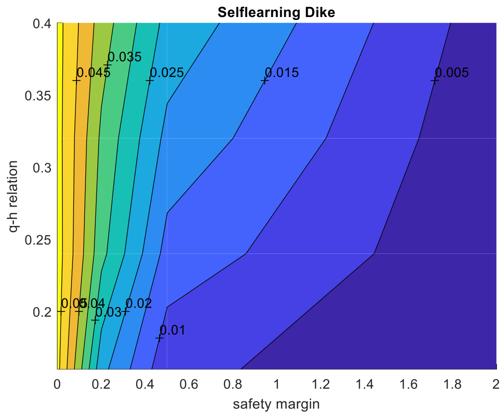

Figure 92 - q-h Rel. Vs. Saf. Mar. Overtopping frequency 'SLD'

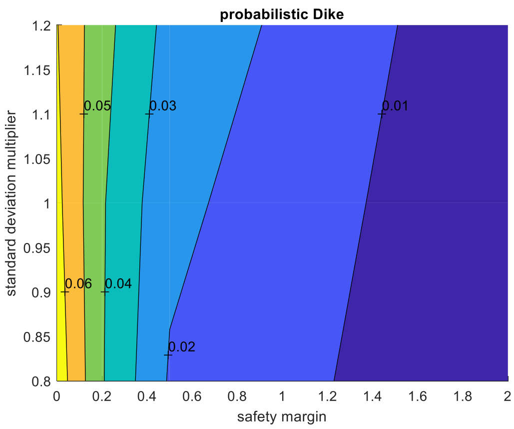

Figure 93 – St. Dev. Vs. Saf. Mar. Overtopping frequency 'PD'

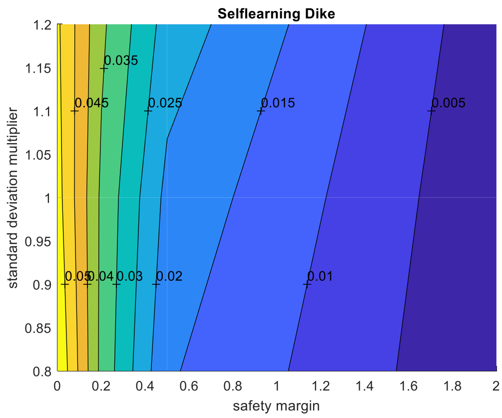

Figure 94 - St. Dev. Vs. Saf. Mar. Overtopping frequency 'SLD'

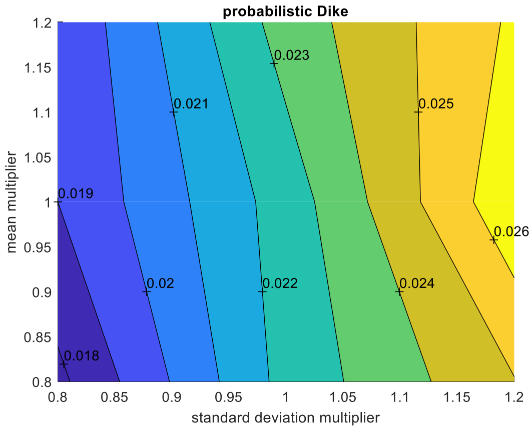

Figure 95 – Mean Vs. St. Dev. Overtopping frequency 'PD'

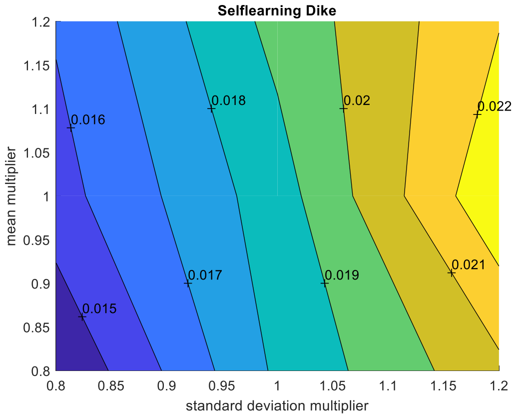

Figure 96 - Mean Vs. St. Dev. Overtopping frequency 'SLD'

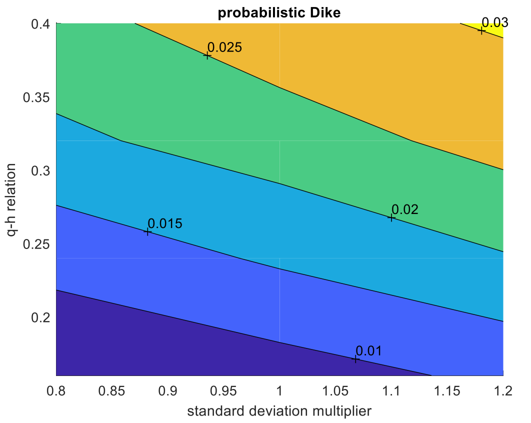

Figure 97 – q-h Rel. Vs. St. Dev. Overtopping frequency 'PD'

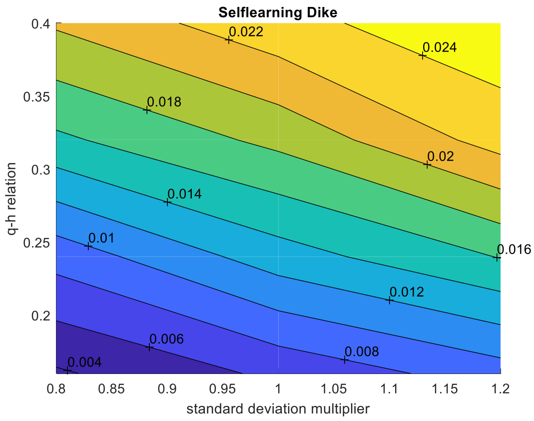

Figure 98 - q-h Rel. Vs. St. Dev. Overtopping frequency 'SLD'

D. Q-h relations

The q-h relations of the Rhine and Meuse rivers are established based on points in the graph. These points can be found in the tables below. Table 14 shows the points of the q-h relation of the river Rhine. Table 15 shows the points corresponding to the q-h relation of the river Meuse.

Table 14 – Q-h relation river Rhine

Discharge [m^3s^{-1}]	1000	2000	3000	4000	5000	6000	7000	8000	9000	10000	11000	20000
Waterlevel [m] NAP+	7,3	9,2	10,8	11,9	12,9	13,8	14,6	15,2	15,7	16,1	16,4	19,2

Table 15 – Q-h relation river Meuse

Discharge [m^3s^{-1}]	0	100	300	500	1000	1500	2000	2500	3500	4000
Waterlevel [m] NAP+	37,3	38,8	40	40,9	42,8	44,2	45,1	45,7	46,2	46,2