Risk based flood management of sewer systems

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by

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PREFACE

Before you lies the master thesis 'Risk based flood management of sewer systems'. This thesis is part of the Master of Civil Engineering at the TU Delft. It is the final product of the master track Water Management in which I followed the specialization Urban Drainage and Watermanagement.

My interest for water management in general rose during the Bachelor Civil Engineering at the University of Twente. During the initial phase of the master Watermanagement at the TU Delft I was fascinated by the challenging field of urban drainage and water management. For me, the challenges in this field lie mainly in dealing with uncertainties, conflicts of interest and having only limited space to come up with solutions.

I arrived at the topic of this thesis during my internship at Witteveen+Bos. Talking about the possibilities to also carry out a master thesis at this firm has ultimately led to the thesis report that lies in front of you. The thesis topic was interesting for me as it included many different research techniques, among others a literature review, modelling and performing a questionnaire. Besides that, the fact that this thesis tried to fill some knowledge gaps in the applicability of risk based flood management of sewer systems, was very motivating for me.

I would like to thank Erik Dekker, my daily supervisor at Witteveen+Bos, for his help, guidance and support. Furthermore, I would like to thank Johan Post and Francois Clemens for their critical, but constructive feedback and for steering the research in the right direction. I want to thank Wim Luxemburg for his willingness to join the thesis committee. I would like to thank my colleagues at Witteveen+Bos, in particular Bram Stegeman, for the fruitful discussions, pleasant working environment and refreshing lunch walks. Last, but not least, I want to thank my family and friends, in particular my beloved girlfriend Gerdineke, for their motivating and unconditional support. Amongst them, I also want to particular thank my parents for raising me and giving me the possibility to do this study.

I hope you enjoy reading.

Maarten Baan

Rijssen, August 2016

ABSTRACT

Traditional sewer flood management is often norm-based. An alternative approach is risk-based sewer flood management. In risk-based sewer management it is possible to decide upon which measures to take in a more expedient way; it is possible to judge which measure is the best in reaching the goal of reducing risks due to flooding. A risk is defined as the chance that an event takes place multiplied with the effect of such an event. By analysing the type of effects and the frequency at which they occur, it is possible to compare different locations where flooding occurs.

The main goal of this master thesis is to develop a method for using risk-based flood management of sewer systems. This method has to be appropriate for comparing the risks at different flooding locations and compare possible measures to prevent flooding based on their expediency. The norm used for optimizing the method is that it must result in a general and straightforward applicable method giving an unambiguous result. The following research question covers the goal of the master thesis: '*How can risk-based flood management of sewer systems be implemented in a general applicable way?*'

First an existing method for risk-based flood management is analysed. This method is used as a basis for the method developed in this thesis. In order to try to improve the existing method, the weak points of this method are altered in the method that is developed in this thesis.

The method in this thesis determines the risk level using the severity score. The severity score is obtained by multiplying the occurring effects by their accompanying weights. These weights depend on the severity of effects as judged by the municipality. The risk level is then dependent on the severity score and the return period of the situation in question. Besides the risk level, also the annual expected severity score is calculated based on the severity score of a range of return periods, varying from small (0.5 years) to large (100 years). The annual expected severity score is useful to make a distinction between locations with the same risk level.

In order to come to a well-founded choice regarding the design storms to use in the method, the Dutch design storms and composite design storms are compared. The underlying principles of both types of design storms are analysed and compared. Besides that, the difference between using a full precipitation series and the design storms is analysed for a couple of sewer systems. Based on these considerations, the composite design storms are used in the method.

Based on a literature study the relevant effects of sewer flooding are obtained. The effects that are taken into consideration to determine the risk level are: flooding of buildings, risk of casualties, infection risk, traffic disruption and flooding of public space. For each of the effects, the way in which they can be quantified is determined.

In order to be able to judge the applicability of the method, the uncertainty involved in using the 1D/2D-model and allocating the effect category weight are obtained. The model uncertainty is obtained by varying a couple of relevant model parameters and analysing the influence of this variation on the results. From this analysis it follows that the subcatchment area is the most influential parameter.

The uncertainty in allocating the effect category weight is obtained by analysing the results of a questionnaire in which respondents are asked to give their judgment about the severity of the relevant effects. This analysis shows that mainly the judgments about intangible effects, like risk of casualties, infection risk and traffic disruption are very wide spread.

A comparison of the influence of the model uncertainty and the uncertainty involved in allocating the effect category weights shows that the latter has a much larger influence on the risk level and annual expected severity score than the first, although the influence of model uncertainty is not negligible.

The developed method in this thesis is a general applicable method for risk based flood management of sewer systems that gives unambiguous results. The main weak point of the method is the allocation of the effect category weights. In order to improve the method in future, attention has to be given to this aspect. To reduce the relative small influence of model uncertainty, a detailed investigation of the amount and characteristics of subcatchment area surcharging to the sewer system is recommended.

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APPENDICES

1 INTRODUCTION

Properly functioning sewer systems are an important asset of urban areas, as these systems are a vital part of the urban drainage system, which protects the urban area from pluvial flooding (Ten Veldhuis, 2010). Pluvial flooding is caused by rainfall-generated overland flow pounding on the urban surface due to overloading of the urban drainage system. The interest in urban flood risk has grown over the last few decades, as the frequency of flooding and damage caused by urban flood events have increased (Ten Veldhuis, 2010). Besides that, climate change predictions increase concern about urban flood risk in cities around the world (Wilby, 2007).

1.1 Problem definition

In traditional sewer flood management, the standard is norm-based flood management. The general standard is that a sewer system must be capable of handling the rainfall event 'bui08' from the 'Leidraad Riolering', without flooding. This rainfall event has a return period of 2 years. This standard is in contradiction with the European norm EN752:2008, which prescribes different return periods for different regions, like villages and cities, and objects, like tunnels and hospitals. The Netherlands is the only country in the European Union that hasn't implemented these norms. The possibility exists that for certain locations the effects of flooding are very severe for more extreme rainfall events than 'BUI08', for example the flooding of a company with a lot of damage or the flooding of a tunnel, whereas the system is capable of handling 'bui08' without flooding. The problem of the approach using only 'BUI08' as norm, is that for such locations no measures are taken based on the results of 'BUI08'. On the other hand, it is also possible that measures are taken based on 'bui08' although the effects of flooding don't increase in severity for more intense rainfall events; there is only hindrance and no real damage.

Thus, norm-based flood management does not always result in investing the available budget for sewer improvement at locations where it is most urgent. A possible solution to this problem is using risk- instead of norm-based flood management of sewer systems. In risk-based sewer management it is possible to decide upon which measures to take in a more expedient way; it is possible to judge which measure is the best in reaching the goal of reducing risks due to flooding. The effectiveness of the measure can be judged based on the reduction in risk compared to the costs of the measure. Decisions about which measures to take are no longer based on norms, but on a firm risk assessment.

A risk is defined as the chance that an event takes place multiplied with the effect of such an event. Effects are for example reduced health protection, safety, liveability or durability. Also financial damage can be an effect of flooding due to an extreme rainfall event. By analysing the type of effects and the frequency at which they occur, it is possible to compare different locations where flooding occurs. For example, it is possible that a rare event with large effects has the same risk as an event with little effects that occurs often. In risk-based sewer management it is possible to compare measures based on how much the risk is reduced by performing the measures. This comparison can be combined with a comparison of the costs of the measures, resulting in a firm based choice which measure to perform.

1.2 Scope and goal

The main goal of this master thesis is to develop a method for using risk-based flood management of sewer systems. This method has to be appropriate for comparing the risks at different flooding locations and compare possible measures to prevent flooding based on their expediency. The goal of the master thesis is to optimize the present method for risk-based flood management of sewer systems that is developed at the proeftuin Enschede by Rioned (Hartemink & Meijer, 2015). The norm used for optimizing the method is that it must result in a general and straightforward applicable method giving an unambiguous result. In other words, the norm is to find the simplest method, which fulfils the following criteria:

- general applicability;
- only interpretable in one way. Risk assessment may not be dependent on the person using the method;
- required data has to be overall available / accessible / free;
- explainable in an easy way.

1.3 Research question

The following research question covers the goal of the master thesis: '*How can risk-based flood management of sewer systems be implemented in a general applicable way?*'

1.4 Report structure

In chapter 2 the method developed at the proeftuin Enschede by Rioned is described in more detail. Chapter 3 describes the way in which the available method can be improved. In chapter 4 the appropriate set of rainfall events to use for the risk assessment is determined. In chapter 5 an overview is made of effects that can be considered and how to consider these effects. Chapter 6 describes how the current method is optimized based on the results of the previous chapters. In the chapters 7 and 8, an uncertainty analysis is presented, indicating which part of the method contains the highest amount of uncertainty. Chapter 9 contains the discussion, conclusions and recommendations.

2 ANALYSIS OF AVAILABLE METHOD

The method for risk-based flood management of sewer systems developed at the Proeftuin Enschede by RIONED and STOWA (Hartemink & Meijer, 2015) is described and analysed in paragraph 2.1. Missing aspects and weak points of the method discovered during the analysis are described in paragraph 2.2.

2.1 Proeftuin-method

The method for risk-based flood management of sewer systems developed at the Proeftuin Enschede by RIONED and STOWA (Hartemink & Meijer, 2015), in the remainder of the report referred to as the Proeftuin-method, forms the starting point of the methodology that is developed in this research. A Proeftuin is a project that stimulates innovation and makes the results known to a large public. The results however, are no overall accepted techniques or methodologies, but a representation of the experience with experiments. This makes the results very appropriate for further development.

The essence of risk-based management in general is to control risks by making choices based on the consequences caused by a malfunctioning system. To come to a transparent assessment, risk-based management uses matrices. In the Proeftuin-method three matrices are used, namely the so-called effect matrix, risk matrix and causal matrix.

The effect matrix describes the consequences of an event with a certain level of severity for the different categories taken into account. In table 1 the categories and their indicators as used in the Proeftuin-method are presented. Based on the consequences of a certain event, a severity level is given to each category. In the Proeftuin-method six different severity levels are applied, namely very small, small, moderate, substantial, severe and very severe. The category with the highest severity level is normative and used as a representative severity level for the event.

Table 1: Effect matrix Proeftuin-method

	Security and health	Quality of the living environment (accessibility, liveability of public and private space)	Finances	Reputation
Severity level	- Number of casualties - Number of ill or wounded people	- Extension of the area - Importance of roads - Number of roads - Exceptional area's	Amount of damage in euro's	- Number of complaints - Political consequences - Negative publicity

The second matrix used, is the risk matrix. This matrix allocates a risk level to each possible combination of severity level and return period. A low severity level with a small return period can have an equal risk as a higher severity level with a larger return period. Every risk level has a certain score, which is used in a later stadium to determine the expediency of possible measures. The lowest risk level, 'Very low', has a score of 0.01. The increase of the score is a factor 10 per level, giving a score of 1,000 to the highest risk level, 'Extremely high'. The risk matrix applied in the Proeftuin-method is given in table 2.

Table 2: Risk matrix Proeftuin-method

		Frequency of occurrence					
		(almost) impossible	unlikely	Possible	likely	regular	frequent
		< 1/1000 year	> 1/1000 year < 1/100 year	> 1/100 year < 1/10 year	> 1/10 year < 1 year	> 1 year < 1 / month	> 2 / month
Level of severity	Very severe	Moderate risk (1)	High risk (10)	Very high risk (100)	Extremely high risk (1,000)	Extremely high risk (1,000)	Extremely high risk (1,000)
	Severe	Low risk (0.1)	Moderate risk (1)	High risk (10)	Very high risk (100)	Extremely high risk (1,000)	Extremely high risk (1,000)
	Substantial	Very low risk (0.01)	Low risk (0.1)	Moderate risk (1)	High risk (10)	Very high risk (100)	Extremely high risk (1,000)
	Moderate	Very low risk (0.01)	Very low risk (0.01)	Low risk (0.1)	Moderate risk (1)	High risk (10)	Very high risk (100)
	Small	Very low risk (0.01)	Very low risk (0.01)	Very low risk (0.01)	Low risk (0.1)	Moderate risk (1)	High risk (10)
	Very small	Very low risk (0.01)	Very low risk (0.01)	Very low risk (0.01)	Very low risk (0.01)	Low risk (0.1)	Moderate risk (1)

The last matrix used in the Proeftuin-method, is the causal matrix. This causal matrix is in fact a translation of the general effect matrix in terms of flooding. It indicates which consequences a rainfall event has to have for a certain level of severity. A Dutch version of the causal matrix used in the Proeftuin-method is presented in Appendix I.

The Proeftuin-method consists in short of the following steps:

1 use an event with a given return period in a hydraulic model to acquire the effects that occur;
2 use the results of step 1 to fill out the causal matrix and obtain the level of severity for the different categories;
3 use the risk matrix to obtain the risk level for the different effect categories, based on the return period and the level of severity;
4 based on the results of step 3, determine which locations have an unacceptable risk. For these locations measures can be designed and judged on expediency. Measures which are most expedient are then given the highest priority in realisation. Two possible methods are given to judge the expediency of a measure:

    1 the first method only takes into account the reduction of risk from the category with the highest risk:

$$\frac{Initial\ risk\ score\ of\ normative\ category - remaining\ risk\ score\ of\ normative\ category}{costs\ of\ the\ measure/10{,}000} \quad (1)$$

    2 the second method takes into account the reduction of risk for all categories:

$$\frac{(A + B + C + D + E + F) - (A' + B' + C' + D' + E' + F')}{costs\ of\ the\ measure/10{,}000} \quad (2)$$

where:
$A = initial\ security\ and\ health\ risk\ score$ and $A' = remaining\ security\ and\ health\ risk\ score$
$B = initial\ accessibility\ risk\ score$ and $B' = remaining\ accessiblity\ risk\ score$
$C = initial\ liveability\ of\ public\ space\ risk\ score$ and $C' = remaining\ live.\ of\ publ.\ sp.\ risk\ score$
Etc.

The described procedure makes it possible to implement risk based flood management of sewer systems and judge measures on their expediency. It is possible to apply the basics of the Proeftuin-method in a different setting. An example of such an application is the adaption of the risk and effect matrix by Witteveen+Bos. This adaption results in a method which is more appropriate for comparing different locations where flooding occurs, instead of analysing the system as a whole. The adapted method is in the remainder of the report referred to as the W+B-method. The adapted risk and effect matrix are presented in appendix I. These matrices show that the basic principles of the Proeftuin-method are still applied, but that the levels of severity, return periods, risk levels and categories taken into account are changed.

2.2 Missing and weak aspects

The overall structure of the procedure used at the Proeftuin-method has been described in paragraph 2.1. The procedure has some weak points and is at some aspects not elaborated in enough detail to be general applicable. These aspects are described in this paragraph.

Rainfall events

The first aspect that is not elaborated in enough detail is which set of rainfall events to use for the different return periods. As different rainfall events are available to represent the same return period, it has to be investigated which set of events is the most appropriate to use for the risk based flood management. The return period of the rainfall event itself and of the effects caused by the rainfall event do not have to be equal. The use of the right rainfall event is of more importance in risk-based management than compared to norm-based management, as in risk-based management different return periods are used and the effects of an event are important, whereas in norm-based management a single event is used to examine the system. This aspect is elaborated in chapter 4.

Once a rainfall event is determined for every return period, the hydraulic modelling program Infoworks ICM, in the remainder of the report referred to as ICM, is used to determine the effects caused by the event. This gives a next point of attention, as these calculations are subject to uncertainties. The uncertainties are analysed in chapter 7.

Effect matrix

Translating effects in a common metric involves a lot of uncertainty as many assumptions are required (Ten Veldhuis, 2010). The division of the effects in different categories avoids that all effects have to be translated into a common metric, in most cases monetary terms (Hammond et al., 2015). However, in the effect matrix severity levels are assigned to the different categories of effects. To determine the severity levels, the different effect categories need to be equivalent (Leitao et al., 2013). This implies that the categories are in fact still translated in a common metric, be it in an indirect manner. An example is given for the effect matrix used in the W+B method. For the category liveability of private terrain, the effect is severe if more than 10 dwellings are flooded. For the category companies the effect is severe if water enters one or more companies or shops. This would imply that flooding of 10 dwellings is comparable to flooding of 1 shop. The validity of this assumption is questionable as it is very uncertain if flooding of 10 dwellings would result in as much flood damage as the flooding of 1 shop. The effect matrix of the Proeftuin-method uses an effect category finances and an effect category health, see table 1. Dividing these categories in equivalent levels implies that a certain number of casualties is equivalent with a certain amount of damage, thus indirectly translating casualties into monetary terms. These examples show that using severity levels is a weak aspect of the method as different effects are compared in an indirect way. Applying risk based flood management of sewer systems implies that different effects have to be compared; however, it is better to do this in a direct way.

Besides the uncertainties and assumptions involved in allocating the effect categories in equivalent severity levels, a division into severity levels will give problems in classifying cases situated close to the border between two levels. For example, in the Proeftuin-method, it doesn't matter if this border is exceeded by 1 or 80 buildings. If the border is exceeded, the severity level increases always one level. The W+B-method does not have such big differences, as specific locations are analysed instead of the complete system. However, using distinct levels of severity will always result in a border between the categories and difficulties in classifying cases that are close to that border.

As shown in the two preceding paragraphs, the use of severity levels in the effect matrix has some serious shortcomings and missing aspects. Therefore, the possibility of improving the effect matrix or applying alternative methods will be investigated, based on a literature study, data availability and practical applicability. This aspect is elaborated in chapter 3.

To fill out the effect matrix, a large amount of additional information is required, for example when is a manhole cover lifted, how is the number of casualties and ill / wounded people related to flood depth / extent? Thereby it is questionable if all aspects taken into account are relevant and practically applicable and it is not sure if all relevant aspects are considered. The W+B-method shows that it is possible to use a different set of categories. Which categories are applied in the optimized method, is elaborated in chapter 5.

Risk matrix

The risk matrix used in the Proeftuin-method is transparent and in principle appropriate to use in the method developed in this research. However, the classification of the frequency of occurrence in the risk matrix is problematic, as the border between two classes is for example precisely defined at 10 years. This makes it unclear where to put an event with a return period of 10 years. An easy solution to this problem is applied in the W+B-method, by defining categories with a specific return period instead of a range of return periods. In addition to changing the categories of return periods, the adaptations applied to the effect matrix can have implications for the risk analysis. This aspect will be elaborated in chapter 6.

Expediency of measures

To judge the expediency of measures, two suitable methods where developed in the Proeftuin-method. However, it is not described which of the two methods has to be applied. As the goal of this research is to develop an unambiguous method, one particular method to judge the expediency has to be applied. The choice of this method is elaborated in chapter 6.

3 ADJUSTMENT OF METHOD

The Proeftuin-method and the W+B-method based on it are described and analysed in chapter 2. The largest shortcoming was the use of severity levels in the effect matrix, which compares the different effect categories in an indirect way and introduces difficulties in classifying cases close to the border between two severity levels. The advantage of using matrices is their transparency and that they facilitate the decision process of policy makers and their communication with civilians. In paragraph 3.1 the possibility of improving the method is investigated. In paragraph 3.2 an alternative approach to risk assessment is described.

3.1 Adaptation of Proeftuin-method

To improve the Proeftuin-method, the use of severity levels in the effect matrix has to be eliminated. The procedure proposed in this paragraph achieves this, without losing the transparency of the Proeftuin-method. The procedure consists of the following steps:

1. The municipality has to assign weights to the different effect categories based on how severe they categorize an effect. These weights are called effect category weights (ECW).
2. A range of severity scores has to be assigned to each box in the risk matrix.
3. Use ICM to obtain the effects for the given return periods. The return periods taken into account are 0.5, 1, 2, 5, 10, 20, 50 and 100 years.
4. To obtain the severity score for each return period, multiply the effects with the corresponding effect category weights.
5. Use the risk matrix to determine the risk level for each return period, based on the calculated severity scores at step 4.
6. For locations with a risk level that is judged as unacceptable by the municipality, measures can be designed and judged on their expediency by repeating steps 3-5 with the adjusted situation. How the expediency of measures is computed, is described in chapter 6.

To clarify the proposed procedure, a simple, fictive example is described here in short. The municipality has assigned the effect category weights 1, 2 and 3 to the effects flooded houses, shops and main roads respectively, as they judge that the flooding of a shop is twice as severe as the flooding of a house and the flooding of a main road is thrice as severe as the flooding of a house. The risk matrix presented in table 3 is used to determine the risk level. For clarity sake, this matrix uses only 4 return periods.

Table 3: Example of risk matrix

Frequency of occurrence (return period)			
T: 100 y	T: 10 y	T: 2 y	T: 1 y
Moderate (> 15)	High (> 15)	Very high (> 15)	Extremely high (> 15)
Low (11-15)	Moderate (11-15)	High (11-15)	Very high (11-15)
Negligible (6-10)	Low (6-10)	Moderate (6-10)	High (6-10)
Negligible (1-5)	Negligible (1-5)	Low (1-5)	Moderate (1-5)

Using ICM, the effects as given in table 4 are predicted for two locations. In table 4 it can be seen that location II has the highest risk level, namely 'High' for a return period of 2 years, so this location gets the highest priority in designing measures.

Table 4: Example of effects with their accompanying severity score and risk level

Location	Return period [years]	Flooded houses (ECW: 1)	Flooded shops (ECW: 2)	Flooded main roads (ECW: 3)	Severity score (Risk level)
I	1	1	0	1	4 (Moderate)
	2	2	0	2	8(Moderate)
	10	5	1	2	13 (Moderate)
	100	6	1	2	14 (Low)
II	1	0	0	0	0 (None)
	2	6	1	1	11 (High)
	10	7	1	1	12 (Moderate)
	100	7	1	1	12 (Low)

The first advantage of the adapted procedure compared to the Proeftuin-method is that the different effects are made equivalent in a direct way, instead of an indirect way, via the severity levels. Besides that, the problem of classifying cases close to a border of severity levels is eliminated. The only border that is left is the one between the risk levels. Another advantage is that the adaption accommodates that all effects are taken into account in the assessment of the risk level and not only the normative severity level from a particular effect, as was the case in the Proeftuin-method. This gives a more complete image of the risk that occurs at different locations. In the remainder of the report, the adapted method is referred to as the Severity Score-method.

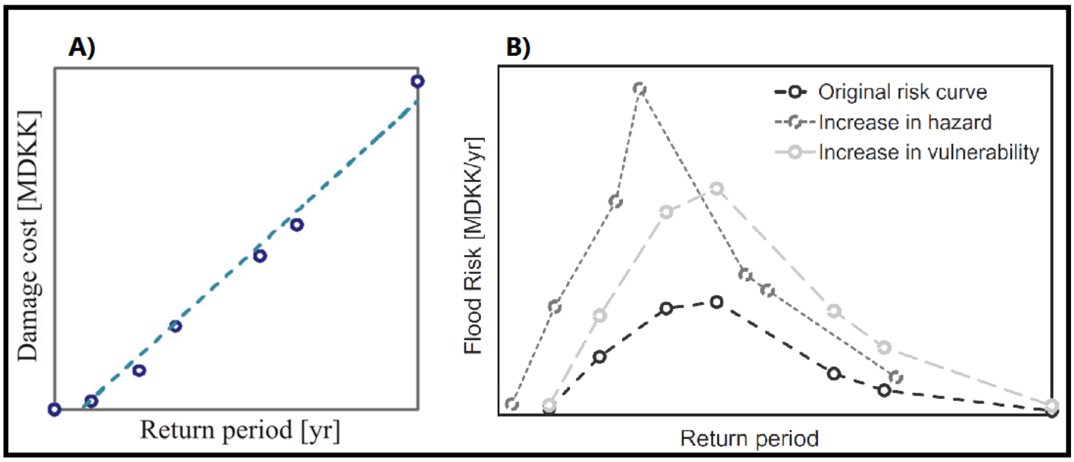
3.2 Expected annual damage method

A fundamental different way to perform a risk assessment is calculating the expected annual damage (EAD), a strong indicator to give insight in the vulnerability of a given area and the expediency of proposed measures (Olsen et al., 2015). The first part of this method is analogous to the Proeftuin- and Severity Score-method. First the expected flood extent for different return periods is obtained by using design storms as input for a 1D/2D-model that calculates the flood extent. These results are used to obtain the vulnerability of an area, which includes for example the number of houses or roads flooded (Zhou et al., 2012).

The next steps in this method differ from the Proeftuin- and Severity Score-methods. The vulnerability for each return period is translated into an amount of damage cost, for example based on the amount of flooded houses, basements, manholes, shops and roads, see figure 1A (Zhou et al., 2012). This implies that all effects are translated in monetary terms.

Dividing the damage cost by the accompanying return period gives the flood risk for each return period as a certain amount of expected damage per year. Plotting and connecting these values for each return period results in a risk density curve (RDC), as shown in figure 1B (Zhou et al., 2012). The RDC shows which return periods have a large contribution to the expected annual damage. In figure 1B also the effect of increased hazard, for example due to climate change and increased vulnerability, for example due to an increase in flooded shops, on the RDC is shown. An increasing hazard will result in a shift towards more frequent return periods and an increase in vulnerability will result in a vertical shift as the damage increases but the occurrence of flood events remains the same. The expected annual damage is obtained by integration of the flood damage over all return periods (Zhou et al., 2012). The result of this method is a certain expected annual damage for each area, which indicates vulnerable areas by a high expected annual damage.

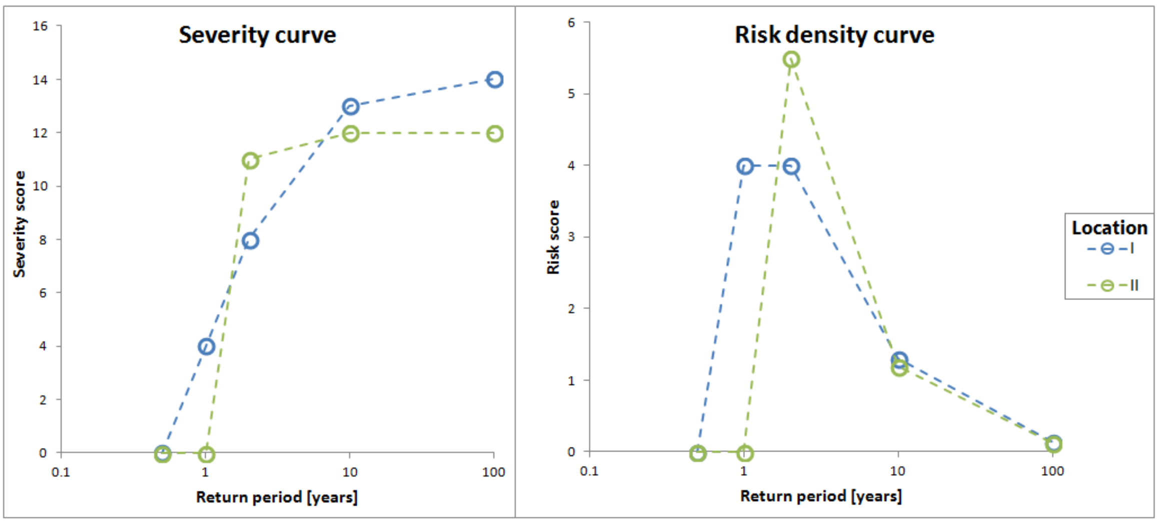
Figure 1: Example of: A: Damage cost per return period; B: Risk Density Curve



As mentioned, the calculation of the expected annual damage as usually applied, requires that all effects are translated into monetary terms. As discussed in paragraph 2.2 this translation introduces a lot of uncertainties and assumptions (Ten Veldhuis, 2010), which makes it questionable if it is useful to do this. However, it is also possible to apply the principles of the method to calculate the expected annual severity score. The higher this value, the larger the risk caused by flooding. The severity score for each return period is obtained in the same way as proposed in paragraph 3.1. The expected annual severity score is then obtained by integration of the severity score over all return periods. Plotting the severity scores for different return periods would yield a graph like figure 1A. After that, the risk score, which is defined as the severity score per year, is obtained by dividing the severity score by the corresponding return period. Plotting the risk scores for different return periods would yield a graph like figure 1B. Such a graph can be used to get insight in which return periods have a large contribution to the expected annual severity score.

To clarify the proposed method, the same simple example as used in paragraph 3.1 is used. The resulting severity curve and risk density curve are shown in figure 2A and B respectively.

Figure 2: Severity curve and risk density curve for example situation from paragraph 3.1



As mentioned, the expected annual severity score is calculated by integration of the severity curve. Olsen et al. (2015) have compared different methods of solving the integral of the damage curve, the counterpart of the severity curve, namely:

- an analytical solution, which uses the assumption that the relation between flood damage and return period is log-linear, which is a common assumption in flood damage assessments (Apel et al., 2004);

- a numeric solution, using the trapezoidal rule, which doesn't require the assumption of a log-linear relationship between flood damage and return period.

If small return periods with negligible effects till very large return periods with a negligible annual damage are included, both methods give comparable and accurate results (Olsen et al., 2015). Former applications of the W+B-method have shown that damage doesn't increase a lot beyond a return period of 10 years. This implies that assuming a log-linear relationship between the damage and the return period doesn't hold in urban pluvial flooding in the Netherlands, which makes the analytical solution unsuitable. Therefore, the numeric solution, using the trapezoidal rule is used to obtain the expected annual severity score, with the following formula:

$$Expected\ annual\ severity\ score = 0.5 * \sum_{i=1}^{n-1} \left(\frac{1}{T_i} - \frac{1}{T_{i+1}}\right) * (SS_i + SS_{i+1}) \quad (3)$$

where:

$$n = number\ of\ return\ periods\ taken\ into\ account$$
$$T_i = i^{th}\ return\ period\ taken\ into\ account$$
$$T_{i+1} = i + 1^{th}\ return\ period\ taken\ into\ account$$
$$SS_i = severity\ score\ of\ i^{th}\ return\ period\ taken\ into\ account$$
$$SS_{i+1} = severity\ score\ of\ i + 1^{th}\ return\ period\ taken\ into\ account$$

For the simple example presented in paragraph 3.1, this results in an expected annual severity score of 10.4 for location I and 8.4 for location II. The conclusion drawn would be that location I has a higher flood risk. This is in contradiction with the conclusion drawn in paragraph 3.1, where it was concluded that location II has the highest flood risk, because it had a risk level 'High' for a return period of 2 years. The contradiction is caused by the fact that the method in paragraph 3.1 only takes into account the normative return period, which coincides with the highest point in the risk density curve, see figure 2B. The method proposed in this paragraph takes into account the contribution of all return periods, which in some cases can lead to different conclusions. This can be seen as an advantage of this method, as the risk assessment is not based on the peak return period but on the contribution of all return periods. Besides that, the expected annual damage can be used to make a distinction of severity in locations with the same risk level. A disadvantage is that the method is more abstract, which makes it less suitable in communication with policy makers and citizens.

3.3 Optimized method

Based on the preceding paragraphs, the base of the optimized method is the Severity Score-method, described in paragraph 3.1. This method has the advantage that the allocation of effects in equivalent severity levels is eliminated, whereas the transparency of the Proeftuin-method is preserved. Besides that, the determination of the risk level is based on all effects and not only on the normative effect.

When the Severity Score-method is performed, calculating the expected annual severity score as described in paragraph 3.2 doesn't require much additional effort. The annual severity score can be used to check if the determination of the risk level is not based on a peaked risk density curve, which could lead to a distorted image of the mutual severity of the risk at different locations. Another useful application of the expected annual severity score is to make a distinction of severity in locations with the same risk level. For these reasons, the expected annual severity score is computed in the optimized method.

The following aspects still need to be elaborated in more detail:

- which design storms are used (chapter 4);
- which effects are taken into account (chapter 5);
- how the effect category weights are determined (chapter 6);
- which range of severity scores is allocated to each box in the risk matrix (chapter 6);
- how the expediency of measures is determined (chapter 6).

4 DESIGN STORMS

In this chapter, a design storm for each return period is acquired based on a literature study and some testing on four sewer systems. Design storms are used instead of full precipitation series, because of the high computational costs that come with simulating a long precipitation series. From a scientific point of view, this is not a valid reason. However, the method has to be practically applicable by engineering companies, which require low computational costs. To justify the use of design storms, their use is compared to using a full precipitation series for four sewer systems. As mentioned in chapter 3, the return periods taken into account are 0.5, 1, 2, 5, 10, 20, 50 and 100 years. Two different types of design storms are analysed, namely the Dutch design storms (Stichting Rioned, 2004) and the composite design storms, among others used in Belgium (Vaes & Berlamont, 1996), Canada (McKelvie, 1982) and Denmark (Zhou et al., 2013). Also the effect of climate change on the design storms is taken into consideration.

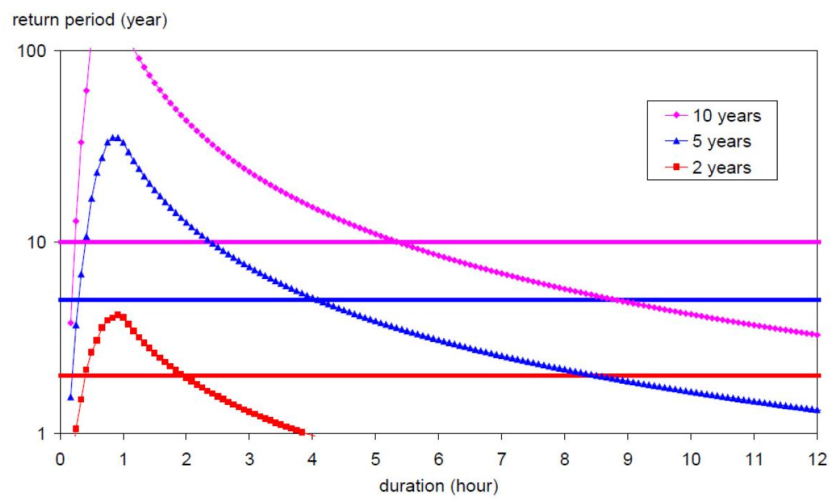
In paragraph 4.1 the underlying principles of the Dutch and composite design storms are compared. In paragraph 4.2 the two types of storms are compared with using a precipitation series. In paragraph 4.3 it is described which series is used to construct the design storms. Paragraph 4.4 deals with the influence of climate change on the design storms.

4.1 Comparison underlying principles

In this paragraph the underlying principles of the Dutch and the composite design storms are compared. These principles are described in Appendix II. Based on this comparison and the analysis done in paragraph 4.2, a conclusion is drawn about which type of storm is most appropriate to use for the flooding analysis.

The main weak point of the Dutch design storms is that the relation between the precipitation volume and the length of the event is incorrect. If the Dutch design storms are compared with the IDF-relations of De Bilt, large differences are found (Vaes, Willems, Berlamont, et al., 2002). Due to the relative low peak, the volume is underestimated for very short events (< 30 min.). Due to the short duration of the storm, the volume is overestimated for moderate storm durations (30-120 min.) and underestimated for long events (> 120 min.). A Flemish composite design storm however has a constant return period for all aggregation levels, because it is constructed based on the IDF-relationships. This is shown schematically in figure 3. In this figure the dotted lines represent the return period for the Dutch design storms based on their maximum precipitation volume for different aggregation levels, when compared to the corresponding volume obtained from the IDF-relations of De Bilt. The solid lines show the constant return period for the Flemish composite storms.

Figure 3: Return period of the Dutch design storms for different aggregation levels (Vaes, Willems, Berlamont, et al., 2002)



Another weak point of the Dutch design storms is that the return period is based on the rank in the ordered series of peak intensities from the historic precipitation series with a length of 25 years. Using this method, an accurate statement can only be made for return periods till approximately 1/20 of the length of the concerning precipitation series (Vaes, Willems, & Berlamont, 1994a, 1994b). Therefore, the peak intensities found for the return periods of 2, 5 and 10 years are quite unreliable. For the Flemish composite design storms, extreme value analysis is used to determine the intensities for return periods, so these intensities are more reliable. However, also the use of extreme value analysis is unreliable for very large return periods, as the results are in that case based on only a small part of the precipitation data.

A last disadvantage of the Dutch design storms regarding the return period of the storms is that the largest return period is 10 years. For the risk analysis however, also more extreme events with a return period till 100 years are required. Besides this, the Dutch design storms are based on a series from 1955-1979, which is already 40 - 60 years ago. Changes in precipitation extremes and characteristics since this time are not taken into account. The Flemish composite designs storms are based on a more recent period, namely 1970-2007 (see paragraph 4.3).

Applying the Dutch design storms to sewer systems with a large capacity, requires to assume some pre-filling, as the situation can occur that part of the system is filled before the extreme event takes place (Stichting Rioned, 2004). The advantage of the composite design storms is that the antecedent and posterior precipitation give a reasonable representation of the variability in precipitation for return periods larger than 1 year (Vaes, 1999). Due to the antecedent precipitation, no pre-filling has to be assumed.

Besides using these design storms, it is possible to use an actual fallen event. Such events connect better to the imagination of policy makers and residents. However, using such a storm has a principle disadvantage. It will be difficult to allocate a return period to the event, as the return period won't be constant for different aggregation levels, so the storm will have a different return period for different locations in the sewer system. Therefore, it is advised to not use an actual fallen event in the risk analysis. It can be used to validate the 1D/2D-model regarding the modelled and actual flood extent if reliable measurements are available.

As argued in the preceding paragraphs and summarized in table 5, the Dutch design storms have some serious shortcomings, whereas the Flemish composite design storms give a good representation of the effects, for different aggregation levels, at larger return periods. Based on this information, the conclusion is drawn that the design storms that are constructed based on the principles of the Flemish composite design storms are more appropriate to use in the flooding analysis. In order to justify this conclusion, the differences in flood volumes calculated by using the design storms or a full precipitation series are analysed for a couple of sewer systems in paragraph 4.2.

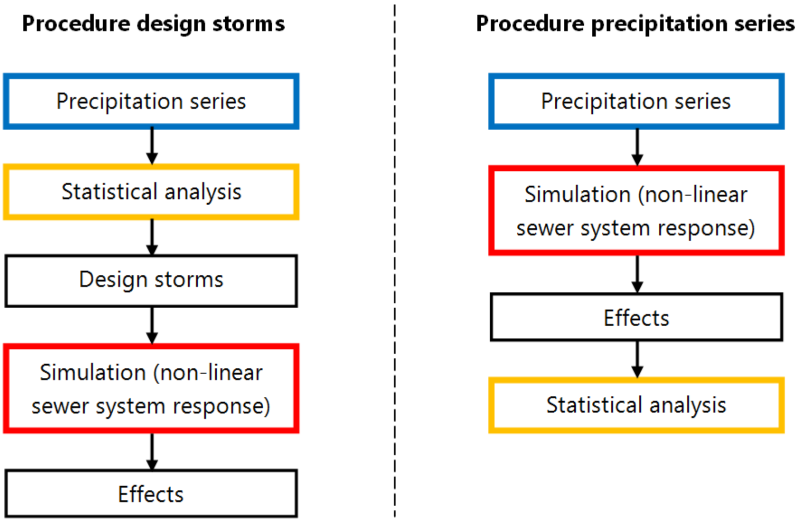
Table 5: Shortcomings Dutch design storms as compared with Flemish designs storms

	Dutch design storms	**Flemish composite design storms**
Return period	Underestimation for very short and long storms. Overestimated for storms with a moderate length. Largest return period is 10 years.	Return period of event is constant for all aggregation levels, due to direct relation with IDF-relations.
Determine peak intensity	Based on the rank in ordered precipitation series. Unreliable for T > 1 year.	Obtained with use of extreme value distribution, more reliable till return periods of 10 year. Less reliable for larger return periods.
Series used	1955 - 1979: Not a recent precipitation series and no full multi-decennial cycle (see paragraph 4.3).	1970 - 2007: Recent precipitation series, 1 full multi-decennial cycle (see paragraph 4.3).
Variability in precipitation	For systems with large storage capacity, pre-filling has to be assumed.	Antecedent and posterior precipitation give reasonable representation of variability in precipitation for larger return periods (T > 1 year)

4.2 Comparison of flood volumes

A weak point of using design storms in general is that they do not account for the non-linearity in sewer systems. This non-linear behaviour is caused by pressured flow and the use of control structures, like internal weirs, pumping cellars, etc. (Vaes & Berlamont, 1996). Due to this non-linear behaviour the return period of the effect of the event is not equal to the return period of the event itself. Besides that, the stochastic elements of rainfall events, i.e. dry periods between events and unique combinations of intensities and durations, will also have an influence on the occurrence of flooding. Using precipitation series instead of a single design storm will incorporate the non-linear behaviour and stochastic elements of rainfall events in a better way, because the statistical analysis is performed on the effects. This implies that the non-linear behaviour and stochastic elements are situated in front of the statistical analysis. When using design storms, the statistical analysis is performed on the precipitation series, so before the non-linear behaviour. This difference is schematized in figure 4.

Figure 4: Procedure design storms versus precipitation series



For larger return periods the influence of the non-linearity in the system vanishes due to the discharging weirs (Vaes et al., 2004). Therefore, it is expected that the return period of the event and the effects of the event will coincide more for larger return periods than for smaller return periods. In this paragraph the results of both procedures will be compared for the two types of design storms.

4.2.1 Procedure of comparison

To compare the calculated flood volume when using a full precipitation series with the calculated flood volume when using the Dutch or composite design storms, the following procedure is followed:

- For four sewer systems, simulate a 1D-sewer model using different precipitation input, namely:
  - full precipitation series of De Bilt from 1955-1979;
  - BUI06 (T = 1 year), BUI08 (T = 2 years), BUI09 (T = 5 years) and BUI10 (T = 10 years);
  - Composite design storms based on the principles of the Flemish composite design storms with a return period of 1, 2, 5 and 10 years. The storms that are used are based on the IDF-relation from the series of De Bilt from 1955-1979 (Vaes, Willems, Berlamont, et al., 2002) and presented in Appendix IV.
- For all manholes, compute the maximum flood volume each time the manhole is flooded during the simulation of the full precipitation series. To do this in a proper way, the events that cause flooding of a manhole have to be independent. A flooding event is said to be independent if there is at least 10 hours without flooding in between. This criterion is based on the reference system with a pump over capacity of 0.7 mm/h and 7 mm storage, implying that a full system is emptied in 10 hours.

- The flood volume belonging to a certain return period is based on the rank in the ordered series of flood volumes. As the precipitation series has a length of 25 years, the position for each return period is:
  - 10 years: the average of the 2nd and 3rd largest flood volume;
  - 5 years: the 5th largest flood volume;
  - 2 years: the average of the 12th and 13th largest flood volume
  - 1 year: the 25th largest flood volume
- For each return period, the Normalized Root Mean Square Error (NRMSE) is calculated for both the Dutch design storms as the composite design storms using the following formula:

$$NRMSE = \frac{\sqrt{\frac{\sum_{i=1}^{n}\left(X_{obs,i} - X_{model,i}\right)^2}{n}}}{\overline{X_{obs}}} \quad (4)$$

  where:

$$n = the\ total\ number\ of\ manholes$$
$$X_{obs,i} = the\ flood\ volume\ obtained\ from\ the\ precipitation\ series\ for\ manhole\ i$$
$$X_{model,i} = the\ flood\ volume\ obtained\ from\ the\ design\ storms\ for\ manhole\ i$$
$$\overline{X_{obs}} = the\ average\ flood\ volume\ of\ flooded\ manholes, obtained\ from\ the\ precipitation\ series$$

- Besides that, the NRMSE is calculated between the Dutch design storms and the composite design storms, with the same formula, only now the parameters represent:

$$n = the\ total\ number\ of\ manholes$$
$$X_{obs,i} = the\ flood\ volume\ obtained\ from\ the\ Dutch\ design\ storm\ for\ manhole\ i$$
$$X_{model,i} = the\ flood\ volume\ obtained\ from\ the\ composite\ design\ storm\ for\ manhole\ i$$
$$\overline{X_{obs}} = the\ average\ flood\ volume\ of\ flooded\ manholes, obtained\ from\ the\ Dutch\ design\ storm$$

This procedure is followed for four sewer systems in the Netherlands, with different characteristics, in order to investigate the general applicability of the design storms. The systems and their characteristics are given in table 6.

Table 6: Characteristics of sewer systems

System	Paved area (ha)	Inhabitants	Ground level (m)	Storage volume (m^3)	Total sewer length (m)	Sewer diameters (m)
De Hoven	12.7 ha	2,200	5.70 - 9.00 m	865 m^3	7,431 m	0.3 - 0.75 m
Loenen	15.8 ha	3,066	17.0 - 29.5 m	2,064 m^3	12,299 m	0.125 - 1.25 m
Haarlem	341.3 ha	51,351	-0.27 - 2.30 m	42,222 m^3	148,580 m	0.1 - 1.5 m
Velp	166.4 ha	28,214	9.0 - 51.9 m	33,880 m^3	85,573 m	0.1 - 1.5 m

4.2.2 Results of comparison

The resulting NRMSE for both types of design storms are presented in table 7 for all systems. At De Hoven and Loenen no flooding occurs for a return period of 1 year. For each return period the storm with the lowest NRMSE is marked green.

Table 7: NRMSE of comparison of precipitation series and design storms

System	Comparison	Return period			
		1 year	2 years	5 years	10 years
De Hoven	Dutch vs. Series	-	0.22	0.25	0.66
	Composite vs. Series	-	0.74	0.35	0.29
	Dutch vs. Composite	-	0.35	0.12	0.31
Loenen	Dutch vs. Series	-	0.17	0.41	1.29
	Composite vs. Series	-	0.17	0.29	0.11
	Dutch vs. Composite	-	0.03	0.22	0.53
Velp	Dutch vs. Series	0.28	0.12	0.29	0.97
	Composite vs. Series	0.38	0.19	0.18	0.11
	Dutch vs. Composite	0.09	0.09	0.18	0.46
Haarlem	Dutch vs. Series	0.50	0.86	1.61	2.18
	Composite vs. Series	0.53	1.00	1.57	1.81
	Dutch vs. Composite	0.10	0.25	0.23	0.31

As mentioned, the non-linear system behaviour decreases for larger return periods (Vaes et al., 2004). Thus it is expected that the NRMSE between the design storm and the precipitation series will decrease if the return period increases. This is the case for the composite design storms in all systems except for Haarlem. The Dutch design storms show at all systems however an increase of the NRMSE when the return period increases to 5 and 10 years. This is a possible indication that the stochastic elements of the rainfall events are modelled in a better way by the composite design storms than by the Dutch design storms. This coincides with the finding of Vaes (1999) that the antecedent and posterior precipitation give a reasonable representation of the variability in precipitation. The lower NRMSE for the Dutch design storms as compared to the composite design storms at small return periods is than possibly caused by the fact that the non-linear system behaviour and incorrect modelling of the stochastic elements of the rainfall events in the series level each other out.

The NRMSE based on the differences between the Dutch and composite design storms are largely in the same order of magnitude as the difference between the NRMSE's based on the differences between the design storms and the precipitation series. This likely indicates that the differences between flooding at the design storms and the precipitation series occur largely at the same manholes for the Dutch and composite design storms, but that the amount of flooding is different.

Based on this analysis it is concluded that the results give an indication that the composite design storms model the stochastic elements of the rainfall events in the series better than the Dutch design storms. This fact combined with the conclusions drawn in paragraph 4.1 forms the base to choose for using the composite design storms in the method.

4.2.3 Shortcomings of using spatially uniform precipitation

For both design storms and a full precipitation series, spatial variability of the precipitation is not taken into account, when using spatially uniform precipitation intensities. When designing a system properly to have no flooding for a return period T, there will not be a flooding at each location more than once in T years (Vaes, 2006). However, for the complete city flooding can occur more than once in T years, but on different locations, due to the large variability in space and time of the rainfall. A composite design storm gives an indication of the composed effects over the complete sewer system for a certain return period. For example, the composite design storm results in flooding at location A and B. However, flooding at location A can in reality occur during another event than flooding at location B.

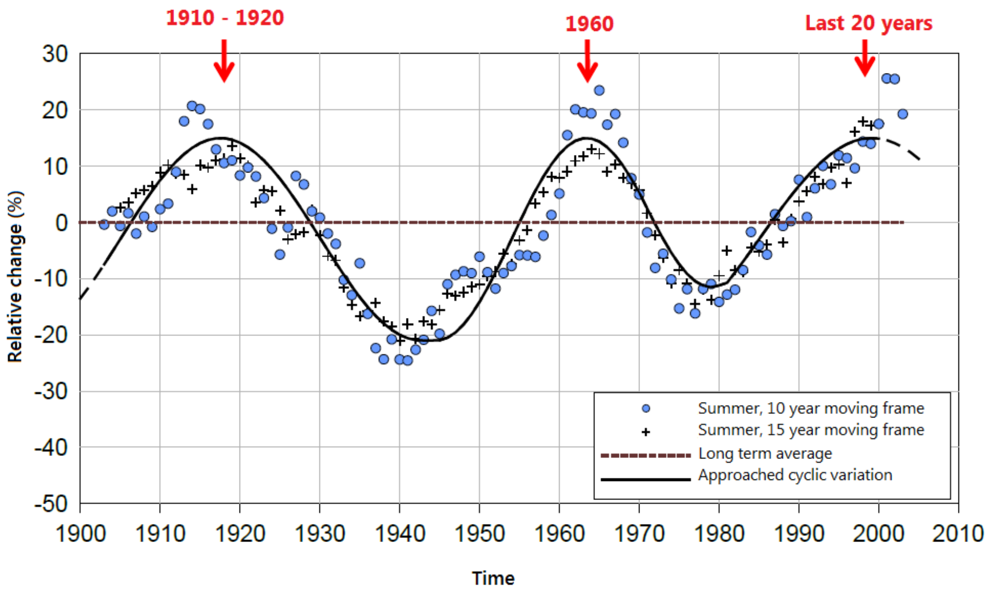
Also the direction in which the storm moves over the system is of importance. If the storm moves in the same direction as the main discharge direction, flooding will be more severe, than when it moves perpendicular or opposite to this direction. Vaes, Willems, and Berlamont (2002) developed a method to take into account the movement direction of the storm. However, this method has high computational costs and requires a lot of statistical analysis of the results. Therefore, this method is not used in the methodology developed in this research, but the inaccuracy caused by not taking the spatial variability into account is accepted.

4.3 Constructing composite design storms

In this paragraph the data used to construct the composite design storms and the resulting storms are presented. To construct design storms based on the principles of the Flemish composite storm, a long, validated precipitation series with a short measuring interval is required. In the Netherlands the longest validated precipitation series is measured at De Bilt, this series has a measuring frequency of once per hour. This is too low to use for hydrodynamic sewer calculations. For such calculations measuring frequencies of at least once per quarter are required. The only validated precipitation data with higher frequency are the maximal 5, 10, 15 and 30-minute precipitation intensity per year for the period 1906 - 1990 (Buishand & Wijngaard, 2008). However, this is not a full series and it can very well be that the second highest intensity from a certain year is much higher than the highest intensity from another year. Therefore, these data are not appropriate as a basis for the design storms. For Ukkel, Belgium, a validated precipitation series with a measuring frequency of once per 10 minutes, measured with the same device, is available for the period 1898 - 2007. This is an ideal precipitation series for the construction of composite design storms if the series is representative for the Netherlands. To investigate if this is the case, Vaes, Willems, Berlamont, et al. (2002) have compared the IDF-relations of Ukkel with those of De Bilt. From this comparison it follows that the IDF-relations are identical. As composite design storms are based on the IDF-relations, this implies that composite design storms constructed using the precipitation series of Ukkel are representative for use in De Bilt. As there are no significant differences in the Netherlands regarding extreme precipitation intensities for short storm durations (Overeem, 2009), the composite design storms are not only representative for use in De Bilt, but for the complete country.

Using the precipitation series of Ukkel, IDF-relations where constructed by Vaes et al. (2004) for the period 1967-1993 and by Willems (2011) for the periods 1970 - 2007. The precipitation series of Ukkel shows multi-decennial oscillations regarding the extreme precipitation intensities. The relative change in extreme precipitation intensities is given in figure 5 for the months June-July-August (summer) (Willems, 2011). This figure clearly shows the cyclic variations. The IDF-curves constructed by Willems (2011) for the period 1970 - 2007 include one complete cycle and will therefore give a better estimate of the extreme precipitation than the ones constructed by (Vaes et al., 2004) for the period 1967 - 1993 as this period is mainly in the bottom of the cyclic variation. Therefore the IDF-relations constructed by Willems (2011) will be used to construct the composite design storms. The procedure of constructing the composite design storms is presented in Appendix III and the resulting storms are presented in Appendix IV.

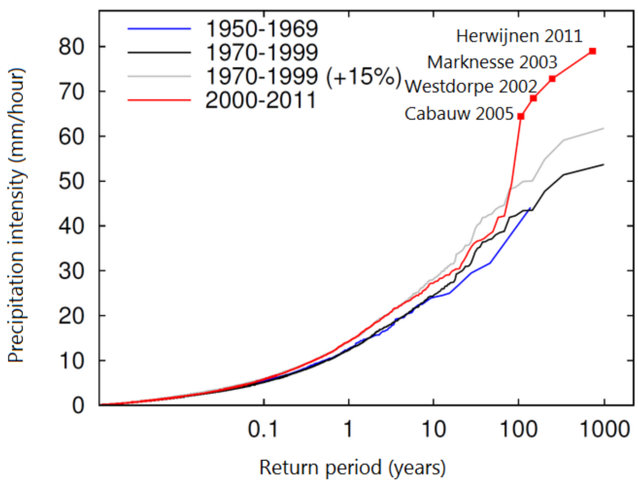
Figure 5: Relative change of precipitation extremes in summer (Willems, 2011)



4.4 Climate change

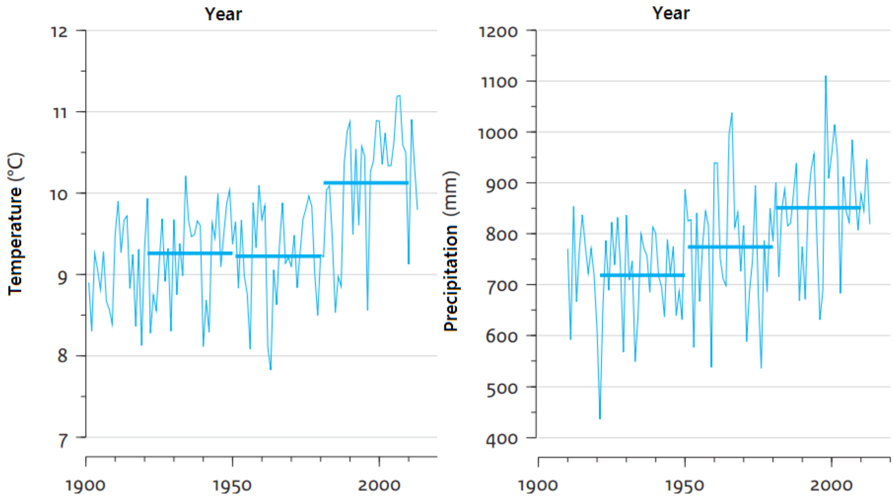
As the lifetime of sewer systems is approximately 60 till 80 years (Korving et al., 2009), changes in climate in the next decades, even till the end of this century, will be relevant when judging the performance of the system using design storms based on precipitation series. The IDF-relations used to construct the composite design storms for this research, are based on the period 1970 - 2007 (Willems, 2011). Due to climate change, it is expected that the intensity of extreme precipitation will increase, although it is very unsure how large the increase will be. It is striking to see that the most extreme hourly precipitation intensities in the Netherlands, are largely measured in the last 10 to 15 years, see figure 6 (KNMI, 2011).

Figure 6: Trend extreme rainfall events period 1950-2011



In figure 7 the change in temperature and amount of precipitation over the last decades is shown for the Bilt (KNMI, 2014). The bold lines represent the 30 years average. This figure shows that the average temperature over the last 30 years is nearly 1 °C higher than the preceding decades. Also the annual precipitation amounts are increasing over the last decades. These findings together with the finding that the most extreme hourly precipitation intensities are measured in the last 10 to 15 years, possibly indicates that the influence of climate change is already observable.

Figure 7: Change in temperature and annual precipitation amount in De Bilt



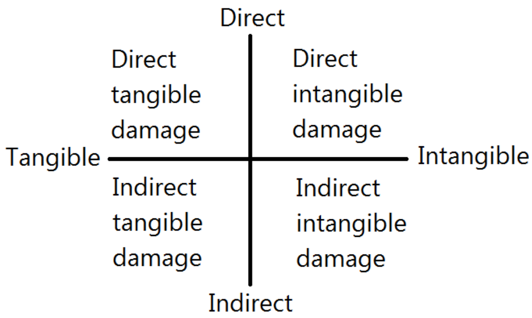
Due to an increase in temperature, the amount of water vapour in the air will increase, leading to increasing extreme precipitation intensities. Climate models are capable of indicating the change in frontal rain zones pretty well, however this is not valid for the extreme convective summer storms (KNMI, 2014). The amount of increase of extreme precipitation intensities in summer, which is important for the design of sewer systems, is therefore quite uncertain, although in general the climate models indicate that the intensities will increase.

The way in which the influence of climate change is taken into account is described in Appendix III. The storms that can be used for modelling the influence of climate change are presented in Appendix IV.

5 EFFECTS

In this chapter, the possible effects of flooding are described and classified. If the effect is of importance for urban pluvial flooding in the Netherlands, it is also described how the effect can be quantified. The classification system applied in this research is first published by Parker et al. (1987) and classifies flood damage first in direct and indirect damage and second as tangible and intangible damage (Freni et al., 2010). The difference between direct and indirect damage is if the damage is caused by immediate physical contact with the floodwater (direct), or not (indirect). The difference between tangible and intangible is if the damage is quantifiable in monetary terms (tangible) or not (intangible) (Hammond et al., 2015). This classification gives four types of damage, schematized in figure 8. These four types of damage are analysed in the next four paragraphs. In paragraph 5.5 an overview is given of the effects which are taken into consideration in the methodology.

Figure 8: Classification of flood damage



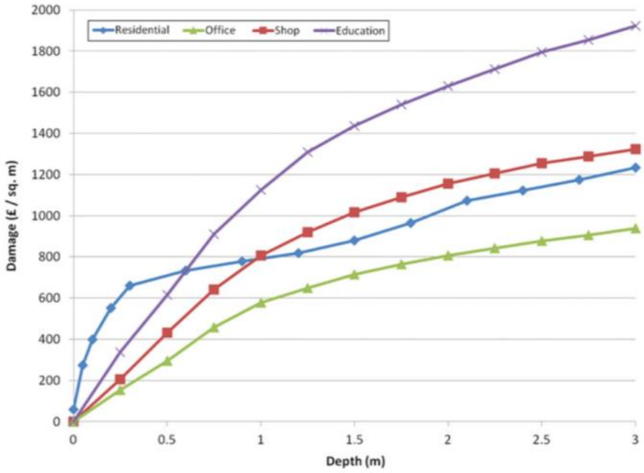
5.1 Direct tangible damage

Most flood damage assessments in literature focus on direct tangible damage. Direct tangible damage is mainly caused by flooding of buildings (Ten Veldhuis, 2010). To monetize direct tangible damage, damage functions are used. These functions can be divided into empirical and synthetic damage functions (Hammond et al., 2015). Empirical damage functions are based on real flood damage data; whereas synthetic damage functions are based on a hypothetical analysis of what damage would be caused by floods of a certain depth in a certain asset. Due to the lack of good quality damage data, most damage functions are synthetic (Freni et al., 2010), although some authors have argued that empirical damage functions are more accurate (Gissing & Blong, 2004). An advantage of synthetic damage functions is that they are easily transferable, as it is based on a general hypothetical analysis, whereas empirical damage functions are constructed for a certain location (Hammond et al., 2015). Two types of damage functions are used in literature, namely depth-damage curves and threshold functions. These two types will be elaborated in the next paragraphs.

Depth-damage curves

Most of the flood damage functions in literature are depth-damage curves (Freni et al., 2010; Hammond et al., 2015; Ten Veldhuis, 2010), a concept introduced by Gilbert F. White (1945). An example of such a curve is given in figure 9 (Penning-Rowsell & Parker, 1987), from which it can be seen that with increasing depth, the damage also increases. Depth-damage curves assume that the damage caused by flooding is almost only completely dependent on the flood depth. However, several studies have shown that damage is not only dependent on the flood depth, but also on other flood characteristics like flow velocity (Freni et al., 2010; Merz et al., 2004; Pistrika & Jonkman, 2009; Thieken et al., 2005). Besides that, these depth-damage curves are generally developed for flooding from rivers (fluvial flooding), with depths varying from 0-5m, making them unsuitable for urban pluvial flooding in the Netherlands, where flood depths only reach a few decimetres (Sušnik et al., 2014; Ten Veldhuis, 2010).

Figure 9: Example depth-damage curves



Threshold functions

An alternative for depth-damage curves is the most simple form of a flood damage function, namely a binary function, making a distinction between assets that are flooded or not (Hammond et al., 2015). Such a flood damage function is also called a threshold function, as the damage occurs once a certain threshold water depth is exceeded (Olsen et al., 2015; Sušnik et al., 2014; Zhou et al., 2012). Part of the threshold function is assigning a cost price to an object when it is flooded. These cost prices represent the amount of damage caused by flooding and are dependent on the type of object (Olsen et al., 2015). Spekkers (2015) analysed the relation between rainfall intensity and insurance claims related to water damage. From this analysis it follows that with increasing intensity, the amount of households claiming increases, but that the amount of damage per household stays the same. This finding supports the use of a threshold function to calculate flood damage due to urban pluvial flooding in the Netherlands.

Based on the literature study, threshold functions seem to be more appropriate to monetize direct tangible urban pluvial flood damage in the Netherlands than depth damage functions. However, it has to be mentioned that such threshold functions have never been compared to real damage data from pluvial flooding, so the reliability of this method is unknown (Spekkers, 2015).

A difficulty in the threshold method is the allocation of unit costs to flooded assets. In order to get an accurate flood damage assessment, it is important to divide the assets in homogeneous classes regarding flood damage (Merz et al., 2010). An example of a division into homogeneous classes is shops and houses. It is also possible to distinguish further, between for example single-storey and multi-storey dwellings or food shops and other shops. However, the amount of damage can differ a lot, even within homogeneous classes. To illustrate this, in table 8 the unit costs for a house as applied in different studies regarding urban pluvial flooding are given. The values are translated to 2016 values using an interest rate of 4 %.

Table 8: Unit cost house as applied in literature

Unit cost house	Explanation	Source
maximal € 4,600	1998 value: ƒ5000, translated into €	(Bolt & Kok, 2000)
€ 2,800	Based on water related insurance claims in the Netherlands.	(Sušnik et al., 2014)
€ 8,500 - € 20,000	Based on synthetic damage function, assuming a maximum flood depth of 0.3 m, for an average surface area of 50 m^2.	(STOWA, 2013)
€ 16,000	Based on insurance claims after extreme rainfall event in Copenhagen, 2011.	(Olsen et al., 2015)
€ 88,000	Based on national Danish databases on costs of damage and malfunctioning of important infrastructure.	(Arnbjerg-Nielsen & Fleischer, 2009)

The values show that there is a huge variety in unit costs applied. The unit costs applied in the Netherlands vary from € 2,800 to € 20,000. In Denmark values of € 16,000 and even € 88,000 are used. Besides that, Grothmann and Reusswig (2006) state that the behaviour of citizens during a flood event largely influences the amount of damage. This makes it very difficult to allocate a single unit cost value to flooded houses.

The problem becomes even larger when allocating a unit cost value to shops. The stock in the shop can be damaged by the flood. However, the value of the stock can differ widely among different kind of shops. For example, the flooding of a computer shop will lead to far more damage than flooding of a bicycle shop. A solution could be to Dividing shops into separate classes, to which a unit cost prices is allocated, is not a solution to the problem. This because sewer systems have a service life of several decades (Korving et al., 2009), which is in general much longer than the time a specific shop stays at the same location.

The preceding paragraphs have shown that quantifying direct tangible damage into monetary terms is possible, but comes with very large uncertainties. The proposed methods are quite suitable for fluvial flooding as in that type of floods a large area is flooded and the uncertainty in assessing damage decreases with an increasing amount of flooded assets (Merz et al., 2004). In urban pluvial flooding the amount of flooded assets is in most cases limited, introducing large uncertainties in monetizing flood damage. Therefore, it is questionable if it is useful to monetize direct tangible damage, although this type of damage is the most straightforward to monetize. An alternative would be to give the amount of houses and shops flooded and let the policy makers decide upon the severity of flooding of each type of asset. By using the local knowledge of the policy makers, the specific characteristics of the local situation can be incorporated more completely than when using a unit cost for a flooded asset for the complete country. This method still requires a threshold to indicate when a certain asset is flooded. Threshold values used in literature for different type of assets are presented in table 9.

Table 9: Threshold values used in literature

Asset: threshold value	Explanation	Source
House: 10 cm Shop: 10 cm	Assumption made by author	(Olsen et al., 2015)
House: 20 cm Basement: 0.3 cm	Assumption made by author	(Zhou et al., 2012)
House: 10 cm Basement: 0 cm	Average median of height of 4664 doorsteps in Rotterdam: 10 cm. Standard deviation: 15 cm.	(Stone et al., 2013; Veerbeek & Gersonius, 2010)

For houses the research of Veerbeek and Gersonius (2010) is the most detailed research regarding the height of doorsteps. Stone et al. (2013) used this research to obtain threshold values for houses with and without basements, see table 9. As this is the best available information it is recommended to use these threshold values when no additional information is available. Stone et al. (2013) advice to base the ratio of houses with and without a basement on expert judgment, as this ratio varies a lot throughout the Netherlands and within cities.

For shops the only threshold value found is 10 cm, used by Olsen et al. (2015), however this value is an assumption made by the author. In the Netherlands, public areas are since 1975 obliged to have no obstacles higher than 2 cm between the entrance and the adjacent areas (VNG, 1975). Larger height differences need to be bridged by a ramp or a lift. For shops and other public buildings like offices, town halls, etc., the vast majority has no ramp or lift. So in the absence of additional information, assuming a threshold value of 2 cm for all buildings other than houses seems very reasonable.

Besides using a fixed threshold value for different types of buildings, it is possible to measure the height of the doorsteps for buildings at risk of flooding based on the calculated flood extent with ICM. This could then immediately be combined with investigating if the building has a basement or not. The advantage of this procedure is that it is more accurate than assuming a fixed value, because the height of the doorstep can vary quite a lot, as shown by Veerbeek and Gersonius (2010). However, this procedure is quite laborious as the residents have to be asked for permission. Besides that it could be that water enters the building via for

example low windows or ventilation holes (Stone et al., 2013), which implies that it has to be checked what the lowest point is where water can enter the building.
Flooding can cause material damage to the public space, roads and other infrastructure. Due to the small water depths such damage is however expected to be negligible (Van Riel, 2011), therefore this type of direct tangible damage is left out of consideration.

Summary

Direct tangible damage can be monetized, but for urban pluvial flooding this involves great uncertainties, mainly because flood depths are rarely larger than a few decimetres and the number of buildings flooded is relative low. Therefore, it is not suggested to monetize direct tangible damage, but to present the number of flooded houses, basements, shops, offices and industries. To determine if a building is flooded or not, the standard threshold values determined in this paragraph can be used. If a more detailed investigation is desirable, the height of the doorstep or any lower point where water can enter the buildings at risk of flooding can be measured in the field. The ratio of buildings with a basement is area specific and has to be based on expert judgment or can be obtained through field investigation. Material damage to public space, roads and other infrastructure is left out of consideration as it is expected to be negligible due to the small flood depths.

5.2 Direct intangible damage

Direct intangible damage is non-material damage caused by direct contact with the floodwater. The following effects are analysed in this paragraph:

- casualties due to flooding;
- infected residents due to exposure to contaminated sewer water.

Casualties

In the Netherlands, urban pluvial flooding rarely results in casualties, although accidents can happen when manholes are lifted (Stone et al., 2013; Ten Veldhuis, 2010). In literature, it is assumed that a manhole is lifted if the pressure in the sewer system rises 10 cm above the manhole level (Olsen et al., 2015; Zhou et al., 2012). Using this assumption, the number of lifted manholes can be determined quite easy from the results of ICM.

In literature, information about the link between the number of lifted manholes and risk of casualties is absent. One could argue that a lifted manhole at a main road results in a higher risk of casualties than a lifted manhole at a street due to the higher traffic intensity. However, in a street the probability of children playing in the floodwater is higher, resulting in a higher risk of casualties. In absence of additional information, assuming an equal risk of casualties for all manholes, irrespective the location of the manhole seems quite reasonable. Using this assumption implies that flood locations can be compared regarding risk of casualties based on the number of manholes lifted.

Besides lifted manholes, flooding of tunnels results in a risk on casualties. For drivers it is hard to estimate how deep the flood water is in the tunnel. To account for this effect, the number of flooded tunnels with a water depth of more than 0.5 m is determined.

It has to be considered that the proposed method only gives a very rough indication of the risk of casualties. If additional information comes available in future, this has to be used to investigate if a more accurate assessment of the risk of casualties is possible.

Infection risk

A second intangible effect of direct contact with the floodwater is the risk of getting infected due to the contaminations in the floodwater. Different studies are performed to quantify the risk of getting infected in urban flood situations (De Man, 2014; Sterk et al., 2008). These studies used pathogens concentrations obtained by sampling of sewer floodwater combined with an assumed volume of floodwater swallowed by children or adults, to obtain the risk of getting infected making use of dose-response relationships. Such a procedure is called a quantitative microbial risk assessment and has to rely on assumptions that introduce significant uncertainties (De Man, 2014). The results should be regarded as an indication of the risk of getting infected and can be used by risk managers to judge proposed control measures (Medema & Ashbolt, 2006). The resulting infection risks due to exposure to sewer floodwater are given in table 10.

Table 10: Infection risks due to exposure to sewer floodwater

Type of sewer system	Infection risk	Source
Combined sewer	Gastrointestinal illness: > 10 %; Respiratory illness: > 3.9 %	(Sterk et al., 2008)
Combined sewer	Adults: 3.9 %; Children: 33 %	(De Man, 2014)
Separate sewer	Adults: 0.58 %; Children: 23 %	(De Man, 2014)

The results indicate that the risk of getting infected is not negligible, that flooding in areas with a combined sewer system involves a higher infection risk than flooding in areas with a separate sewer system and that children have a much higher infection risk than adults. As the infection risk is not negligible, it has to be incorporated in the risk assessment. As indicated by Medema and Ashbolt (2006), the results must not be used to calculate a precise amount of infected residents, but to compare different flood locations and judge proposed control measures. Therefore, it is proposed to categorize flood situations in a limited amount of possible scenarios and allocate an infection-score to these scenarios. The infection-score represents the relative risk of infection between the different scenarios. The characteristics of the scenarios and corresponding severity level are based on the information from literature. To categorize flood situations, the following considerations are made:

- As shown by De Man (2014), the infection risk of flooding of separate sewer systems is 6.7 times lower for adults and 1.4 times lower for children, compared to flooding of combined sewer systems. The average reduction in infection risk, assuming an equal ratio of adults and children exposed to sewer water, is 4 times. Therefore, the infection-score for a flood situation with the same characteristics except for the type of sewer system will be 4 times lower for an (improved) separate sewer system compared to a combined sewer system.
- The W+B-method makes only a distinction between situations where floodwater stays between the kerbs (small risk of infection) or where sidewalks are also flooded (large risk of infection). Based on this assumption a flood situation will get a twice higher infection-score when the sidewalks are flooded. The same increase is used for areas comparable with sidewalks, like shopping areas, squares, etc.
- A distinction between flood scenarios with a different amount of exposed residents, or a different ratio of adults and children is not made, as it is very difficult to make a well-founded assumption regarding these characteristics for flooded areas.

Based on these considerations, the scenarios as shown in table 11, with their accompanying infection-scores are obtained. The reference scenario is the situation where a combined sewer causes flooding of the road. This scenario has an infection-score of 1. The scores of the other scenarios are obtained by multiplication with the factors that result from the considerations described above.

Table 11: Flooding scenarios regarding infection risk

Type of sewer system	Flooded sidewalks / shopping areas / squares	Infection-score
Combined	No	1
Combined	Yes	2
Separate	No	0.25
Separate	Yes	0.5

For the determination of the infection risk, the same consideration has to be made as for the determination of the risk of casualties. The proposed method is based on currently available information and gives only a very rough indication that can be used to compare different flood locations. If in future new information comes available, the possibility of a more accurate assessment of the infection risk has to be investigated.

Summary

In the risk assessment direct intangible damage is incorporated regarding the risk of casualties due to lifted manhole covers and the risk of infection due to exposure to contaminated sewer water. The risk of casualties is represented by the number of lifted manhole covers and flooded tunnels. The infection risk is represented by the infection-score for different flood scenarios. Both representations are rough indications of the possible risk and can be used to compare different flood locations and judge proposed measures.

5.3 Indirect tangible damage

The third category of flood damage is indirect tangible damage. The effects of business interruption and traffic disruption are discussed in this paragraph.

Business interruption

Flooding will cause business interruption to industries and shops in the flooded area, but also to their suppliers and customers outside the flooded area. For fluvial and coastal foods, different methods are developed to quantify the amount of damage caused by business interruption, i.e. assuming a fixed percentage of the direct costs (James & Lee, 1971), a sector-specific unit loss value (Booysen, Viljoen, & Villiers, 1999), input-output modelling (Veen & Logtmeijer, 2005) and computable general equilibrium modelling (Rose & Liao, 2005). However, for urban pluvial flooding in the Netherlands, damage caused by business interruption is expected to be negligible, because of the small flood depths and relative short flood durations (Van Riel, 2011). Therefore, business interruption is left out of consideration in the risk assessment.

Traffic disruption

An often overlooked indirect impact of flooding is traffic disruption (Hammond et al., 2015). However, research has shown that traffic disruption is an important part of the damage caused by urban pluvial flooding in the Netherlands (Ten Veldhuis, 2010). To quantify the amount of traffic disruption caused by flooding, different methods are used in literature. Dutta et al. (2003) used a traffic model to determine the amount of delay if certain main roads in a river basin are flooded. Ten Veldhuis (2010) assumed the duration of traffic delay for an urban flood location to be equal to 5 minutes per vehicle, with the amount of vehicles being based on traffic counts for main roads. Zhou et al. (2012) applied the same method, only assuming 30, instead of 5 minutes delay per vehicle. The variety in assumptions of average delay per affected vehicle is quite high. Thereby this method doesn't take into account traffic disruption and inconvenience caused by flooding of secondary roads like residential streets. Using a traffic model would give more exact information about the amount of delay if a certain road is blocked due to flooding. This however requires external expert knowledge of using the traffic model, which is seen as unpractical and too costly. Therefore, it is proposed to represent traffic disruption by the time that main roads or secondary roads are blocked due to flooding. This approach requires assumptions for which roads are classified as main roads and which flood depth causes traffic disruption. In the Netherlands, roads in urban areas are divided into access roads and distributor roads. This classification is used to distinguish between main roads and secondary roads.

The minimal flood depth causing traffic disruption is in literature assumed to be 15 cm (Zhou et al., 2012). Another source shows that flood depths of 5 cm already cause loss of control of the vehicle (SmartDriving), thus requiring to drive very slow through the floodwater, making the road in practice inaccessible. In this research a threshold value of 10 cm, the average of the two values found, is used. Larger water depths will cause traffic disruption. Using these assumptions, it is possible to compare different flood locations and judge the expediency of measures regarding the reduction in traffic disruption.

Summary

Because business interruption is assumed to be negligible for urban pluvial flooding in the Netherlands, it is left out of consideration in the risk assessment. Traffic disruption is represented by the time that distributor roads or access roads are flooded with a depth of 10 cm or more.

5.4 Indirect intangible damage

The last category of flood damage that is taken into consideration is indirect intangible damage. The following effects are discussed in this paragraph:

- Psychological impact of flooding
- Flooding of public space

Psychological impact

The first effect of flooding that is categorised as indirect intangible damage is the psychological impact of flooding. Tapsell and Tunstall (2003) have shown that mental health effects due to flooding are not negligible for pluvial flooding in the United Kingdom. However, pluvial flooding in the United Kingdom is associated with much higher flood depths and flow velocities than in the Netherlands. There is no information about the mental health effects of urban pluvial flooding in the Netherlands. Thereby, the psychological impacts of flooding are complex and poorly understood (Hammond et al., 2015). Because of these reasons, it is decided to exclude mental health effects from the risk assessment.

Flooding of public space

Another indirect tangible damage is nuisance due to flooding of the public space. The effect of traffic disruption is already described in paragraph 5.3, however this effect does not account for nuisance in for example shopping streets, squares and other places that attract a lot of public. Although it is the case that during heavy rainfall people won't visit tourist attractions, it can occur that flooding of specific locations causes great nuisance, like the flooding of a railway station. This effect can be negligible for some municipalities, whereas for other municipalities this effect is of importance. To account for this effect, the duration of flooding of the specific locations in the public space can be obtained from the results of ICM. The threshold that is used to judge whether flooding of such a location causes nuisance is set at 10 cm, the same threshold as applied for traffic disruption.

5.5 Overview

In table 12 an overview is given of which effects are included in the risk assessment and the way in which they are quantified.

Table 12: Effects included in the risk assessment

Effect	Quantification method
Flooding of buildings	Number of flooded houses, basements, shops, offices and industries
Risk of casualties	Number of flooded manholes and number of flooded tunnels with flood depth > 0.5 m
Risk of infection	Infection-score, dependent on type of sewer system and flooding of sidewalks, shopping areas, etc.
Traffic disruption	Duration of blockage of distributor or access roads due to flooding
Flooding of public space	Duration of flooding of specific sites in public space

6 FINAL METHOD

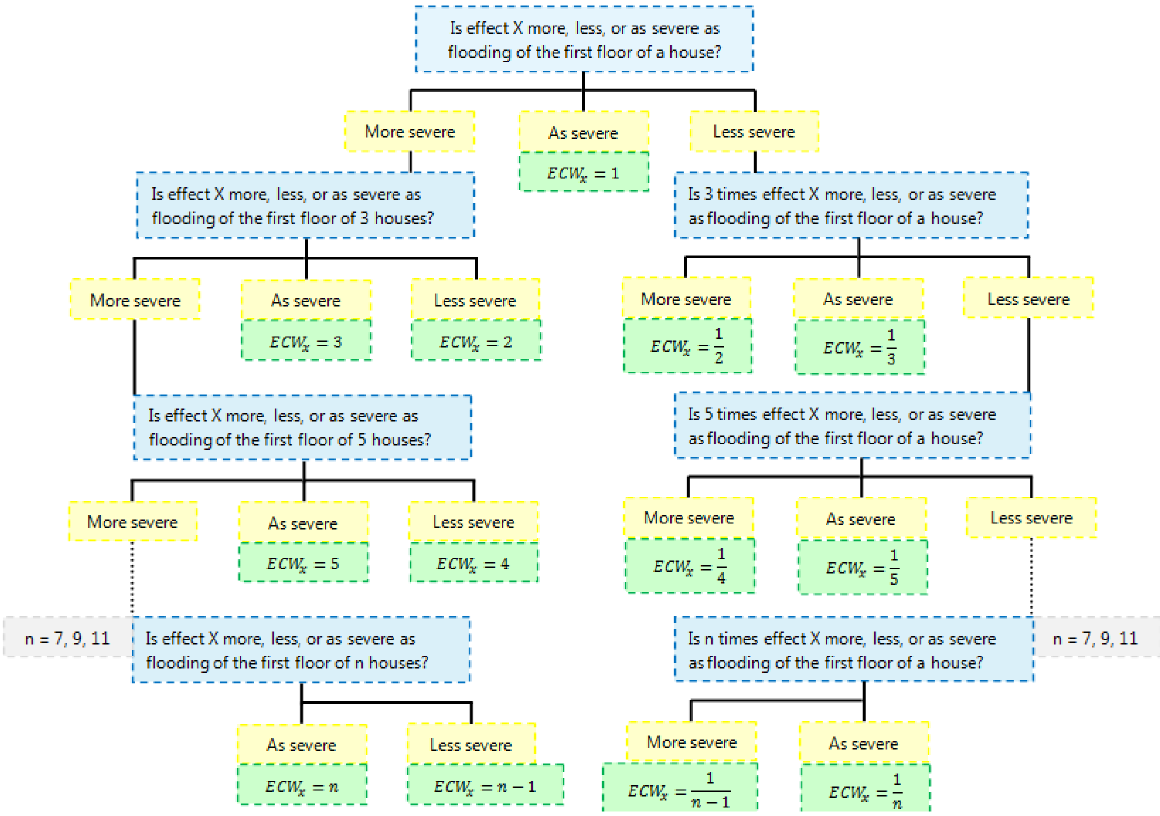
In chapter 3 adjustments to the method regarding the effect and risk matrix where proposed. The base is the Severity Score-method, described in paragraph 3.1. In addition to that, the expected annual severity score is calculated. Some aspects need to be elaborated in more detail, but are dependent on which effects are taken into account. These effects are described in chapter 5. With this knowledge, it is possible to complete the method and elaborate the following aspects in more detail:

- how the effect category weights (ECW's) are allocated;
- which range of severity scores is allocated to each box in the risk matrix;
- how the expediency of measures is determined.

6.1 Allocation of ECW's

In order to use the proposed method in chapter 3 the municipality has to assign a weight to each category based on their mutual severity for the society. The effect category that appeals most to the imagination of policy makers is probably the flooding of a house. Therefore, this effect category is used as a reference category with weight 1. For each effect category the procedure as schematized in figure 10 is followed in order to obtain the accompanying effect category weight. The blue boxes are questions that are asked to the policy makers, the yellow boxes are the possible answers which either result in an effect category weight, a green box, or are followed by a next question, a blue box.

Figure 10: Procedure to follow for each category in order to obtain the accompanying effect category weight



The effects, for which the procedure is followed, are given in table 13. These effects are in figure 10 called 'effect X'.

Table 13: Effects for which effect category weights are obtained

Effect	replace 'n times effect X' in questions with:
Flooded buildings: basements or shops, offices and industries	n flooded basements or shops / offices / industries
Risk of casualties: flooded manholes / tunnels	n flooded manholes or tunnels
Risk of infection	n locations where a combined sewer floods only the street
Traffic disruption: distributor roads or access roads	flooded distributor road or access road for n hours
Flooding of public space	flooded specific location for n hours

In order to enhance this procedure, the policy makers have to know as much information as possible about the consequences for the society of certain effects. The information that is presented to them is given in Appendix V.

The preceding described procedure will result in an effect category weight for every effect category. These weights can be used to obtain the severity score for different flood locations by multiplying them with the effects that occur. The allocation of effect category weights is quite subjective. The uncertainty involved in allocating the weights is quantified in chapter 8.

6.2 Risk matrix

Besides the allocation of effect category weights, it is required that a range of severity scores is allocated to each box in the risk matrix. In order to do this, first amount of risk levels has to be chosen. The choice is made to use 7 risk levels, see table 14. To each risk level a range of annual severity scores is allocated, see the second column in table 14. These ranges are based on the acceptance of the flooding of a house. Each municipality can choose their acceptable return period for flooding of a house; see the third column in table 14.

Table 14: Range of annual severity score allocated to risk levels

Risk level	Range of annual severity score	Accepted return period of flooding of a house
Negligible	0 - 0.02	≥ 50 years
Very low	0.02 - 0.05	20 - 50 years
Low	0.05 - 0.10	10 - 20 years
Moderate	0.10 - 0.20	5 - 10 years
High	0.20 - 0.50	2 - 5 years
Very high	0.50 - 1.00	1 - 2 years
Extremely high	> 1.00	< 1 year

Using these ranges, it is possible to calculate the range of severity scores that belongs to each box in the risk matrix. This is simply done by multiplying the ranges in table 14 with the accompanying return period. This results in the risk matrix presented in table 15.

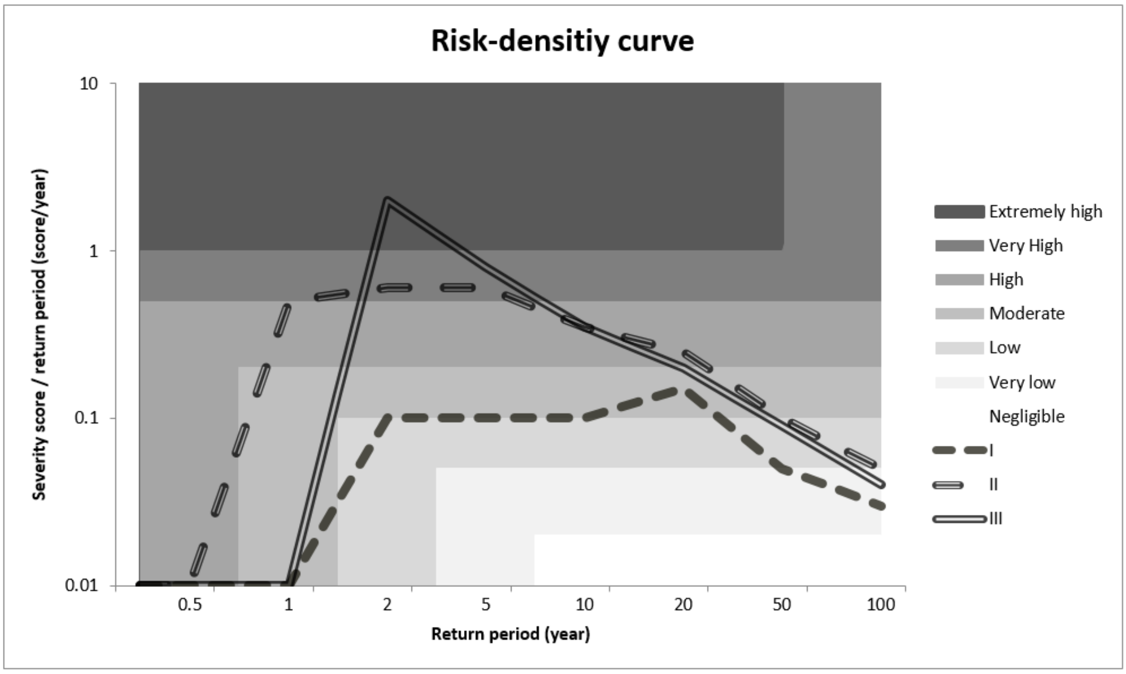
Table 15: Adapted risk matrix

Frequency of occurence (return period)							
T = 100 y	T = 50 y	T = 20 y	T = 10 y	T = 5 y	T = 2 y	T = 1 y	T = 0.5 y
Very high ≥ 50	Extremely high SS ≥ 50	Extremely high SS ≥ 20	Extremely high SS ≥ 10	Extremely high SS ≥ 5	Extremely high SS ≥ 2	Extremely high SS ≥ 1	Extremely high SS ≥ 0.5
High 20 ≤ SS < 50	Very high 25 ≤ SS < 50						
Moderate 10 ≤ SS < 20	High 10 ≤ SS < 25	Very high 10 ≤ SS < 20					
Low 5 ≤ SS < 10	Moderate 5 ≤ SS < 10	High 4 ≤ SS < 10	Very high 5 ≤ SS < 10				
Very low 2 ≤ SS < 5	Low 2.5 ≤ SS < 5	Moderate 2 ≤ SS < 4	High 2 ≤ SS < 5	Very high 2.5 ≤ SS < 5			
Negligible SS < 2	Very low 1 ≤ SS < 2.5	Low 1 ≤ SS < 2	Moderate 1 ≤ SS < 2	High 1 ≤ SS < 2.5	Very high 1 ≤ SS < 2		
	Negligible SS < 1	Very low 0.4 ≤ SS < 1	Low 0.5 ≤ SS < 1	Moderate 0.5 ≤ SS < 1	High 0.4 ≤ SS < 1	Very high 0.5 ≤ SS < 1	
		Negligible SS < 0.4	Very low 0.2 ≤ SS < 0.5	Low 0.25 ≤ SS < 0.5	Moderate 0.2 ≤ SS < 0.4	High 0.2 ≤ SS < 0.5	Very high 0.25 ≤ SS < 0.5
			Negligible SS < 0.2	Very low SS < 0.25	Low SS < 0.2	Moderate SS < 0.2	High SS < 0.25

The risk matrix in table 15 shows that for different return periods, different ranges of severity score are important. For a return period of 1 year, the range around 0.2 - 1 is of importance to determine the risk level. It doesn't matter if the severity score is 2 or 10, the risk level stays then 'Extremely high'. For a return period of 10 years, the range around 0.2 - 10 is of importance. This shows that for different return periods different effects will be of importance. For low return periods, an effect with a low effect category weight can be of importance, whereas for higher return periods, these effects only play a minor role. If an effect with a high effect category weight will occur at a low return period, it doesn't matter how many times it occurs, the risk level will stay extremely high, whereas for higher return periods, this does matter for the determination of the risk level. This characteristic of the risk matrix seems reasonable, as for example flooding of a house will be unacceptable with a return period of 1 year, regardless of the amount of houses flooded, whereas the acceptability of flooding of houses with a return period of 10 years will depend on the amount of houses that is flooded.
A disadvantage of this risk matrix is that there is still a border between different risk levels, for example if the severity score is 2.9 or 3.1 for a return period of 10 years, the risk level will be 'Moderate' or 'High' respectively. The border is however less problematic as the severity score is given, so it can easily be deduced if a situation results in a certain risk level because it just passed the border or if this situation belongs very obvious to a certain risk level.

Another advantageous feature of the allocation of a range of annual severity scores to each risk level is that a plot of the risk density curve can illustrate very well the lapse of the risk levels over the return periods. A fictive risk density curve for 3 locations is presented in figure 11. The background colour indicates the risk level, which makes it very clear if the normative risk level is reached for more than one return period (location I and II) or only for one return period (location III).

Figure 11: Risk density curve



6.3 Expediency of measures

The last aspect that has to be elaborated to complete the method is the way in which the expediency of measures is determined. The formula that is used to determine the expediency of measures is:

$$Expediency = \frac{(EASS_{cuurent} - EASS_{measure}) * LT_{measure}}{\frac{C_i + C_m * LT_{measure}}{100{,}000}} \quad (5)$$

where:

$$EASS_{current} = the\ expected\ annual\ severity\ score\ in\ the\ current\ situation,$$
$$EASS_{measure} = the\ expected\ annual\ severity\ score\ after\ implementing\ the\ measure,$$
$$C_i = the\ investment\ costs\ of\ the\ measure,$$
$$LT_{measure} = the\ expected\ lifetime\ of\ the\ measure\ in\ years,$$
$$C_m = the\ annual\ cost\ of\ maintenance\ of\ the\ measure.$$

The measure with the highest result from this formula is said to be the most expedient measure. In order to compare the measures not only regarding their expediency related to flooding effects, the relevant side effects are also determined, giving a more complete image of the effects a measure has. Besides that, a rough estimate of the range in which the cost-benefit ratio will be is calculated, in order to get an idea if the investment amount is justifiable. The reasons for using formula 5, the side effects which are possibly relevant and the way in which the cost-benefit ratio is determined is presented in appendix VI.

7 MODEL UNCERTAINTIES

The assessment of the effects caused by flooding requires the use of ICM, a 1D/2D hydrodynamic model. Using a hydrodynamic model introduces uncertainties, which can be divided into two categories, namely natural variability and epistemic uncertainty (Korving et al., 2009). Natural variability represents the spatial and temporal variability in nature, in the case of sewer systems the most important is the variability in rainfall (Ten Veldhuis, 2010). The epistemic uncertainty represents the lack of sufficient data and knowledge about the physical system. The natural variability in rainfall is already described and analysed in chapter 4. In this chapter, the epistemic uncertainties will be discussed.

The data set which is used to construct the sewer model, will never be entirely perfect (Korving et al., 2009). Errors in the geometric structure, catchment area, runoff parameters, etc. will considerably influence the calculation results (Clemens, 2001). In addition, estimation or calibration of model parameters and numerical calculation errors lead to model uncertainties (Clemens, 2001). In paragraph 7.1 the procedure used to quantify these uncertainties is described. In paragraph 7.2 the results of the procedure are presented.

7.1 Method

A method to quantify the uncertainties mentioned, was described and applied by Clemens (2001). In this methodology the parameters that are related to uncertainties are given a distribution of likely values. The uncertainty is assumed to be Gaussian, therefore a normal distribution with a certain average and standard deviation is used. A Monte Carlo simulation is used to quantify the influence of the variation of the parameter values on the model results. In each run of the Monte Carlo simulation, a value for each variable is picked from the given distribution. With these variable values, a simulation is performed. Repeating this procedure in the order of 10^2 times gives an indication of the mean and variance of the modelling results (Clemens, 2001). 2 case studies by Clemens (2001) showed that in order to get stable results for both the mean and standard deviation for a flat sewer system with some weirs, a Monte Carlo simulation with 400-700 runs was required.

The proposed method is developed for 1D-models. In 1D/2D-models additional uncertainty is introduced regarding the interaction between the 1D and 2D part of the model and in the 2D part of the model. To deal with these uncertainties the same principles can be applied. Instead of simple Monte Carlo sampling, Latin Hypercube Sampling is used. This method requires a lower amount of runs to get stable results. Additional information about Latin Hypercube Sampling can be found in Appendix VII.1.

7.1.1 System used

The system used to quantify the uncertainties is the sewer system of Wehl, a small town in the municipality of Doetinchem. Characteristics of the sewer system are given in table 16.

Table 16: Characteristics of the sewer system of Wehl

Paved area (ha)	Inhabitants	Ground level (m)	Storage volume (m^3)	Total sewer length (m)	Sewer diameters (m)
43.9 ha	4,516	10.9 - 14.7 m	8,940 m^3	31,052 m	0.15 - 1.45 m

7.1.2 Parameters taken into consideration

The sources of uncertainties that are incorporated in the 1D-part of the model are based on information from Clemens (2001). The following groups of parameters are taken into consideration:

A. runoff parameters
B. (sub)catchment area
C. hydraulic parameters

The applied mean values and standard deviations for the parameters are given in table 17. The standard deviations are based on information from Clemens (2001), whereas the mean values are the default values from the module C2100 from the 'Leidraad Riolering' (Stichting Rioned, 2004).

Table 17: Distribution used for 1D parameters in uncertainty analysis

Parameter	Description	Group	Mean	Standard deviation
$c_{l;f}$	Linear reservoir constant - flat surface	A	0.2 min^{-1}	30 % (0.06 min^{-1})
$c_{l;s}$	Linear reservoir constant - sloping surface	A	0.5 min^{-1}	30 % (0.15 min^{-1})
$b_{f;p}$	Initial and depression losses - flat pavement	A	0.5 mm	30 % (0.15 mm)
$b_{f;r}$	Initial and depression losses - flat roof	A	2.0 mm	30 % (0.6 mm)
i	Infiltration rate	A	2.0 mm/h	30 % (0.6 mm/h)
A	Change in (sub)catchment area	B	0 %	5 %
K_a	Hydraulic roughness	C	3.0 mm	1.0 mm
c	Weir coefficient	C	0.43	0.11
Q_{pump}	Change in pumping capacity	C	0 %	5 %

The runoff parameters (group A) are the same for the complete model, so each run a different value is used for these parameters. The catchment area is different for each manhole in the system. The relative change is different in each run, but is the same for all manholes. If the change would be different for each manhole, the effects would partly cancel each other out. In addition to that, this would result in a Latin Hypercube sample with an unpractical high amount of dimensions.

In the 2D-part of the model there will also be sources of uncertainty. In order to quantify a part of this uncertainty, the parameters given in table 18 are varied in the simulations.

Table 18: Distribution used for 2D parameters in uncertainty analysis

Parameter	Description	Model part	Mean	Standard deviation
c_m	Weir coefficient manholes	1D-2D interaction	0.50	0.20
n_m	Roughness of the mesh	2D part	$0.06\ s/m^{1/3}$	$0.02\ s/m^{1/3}$

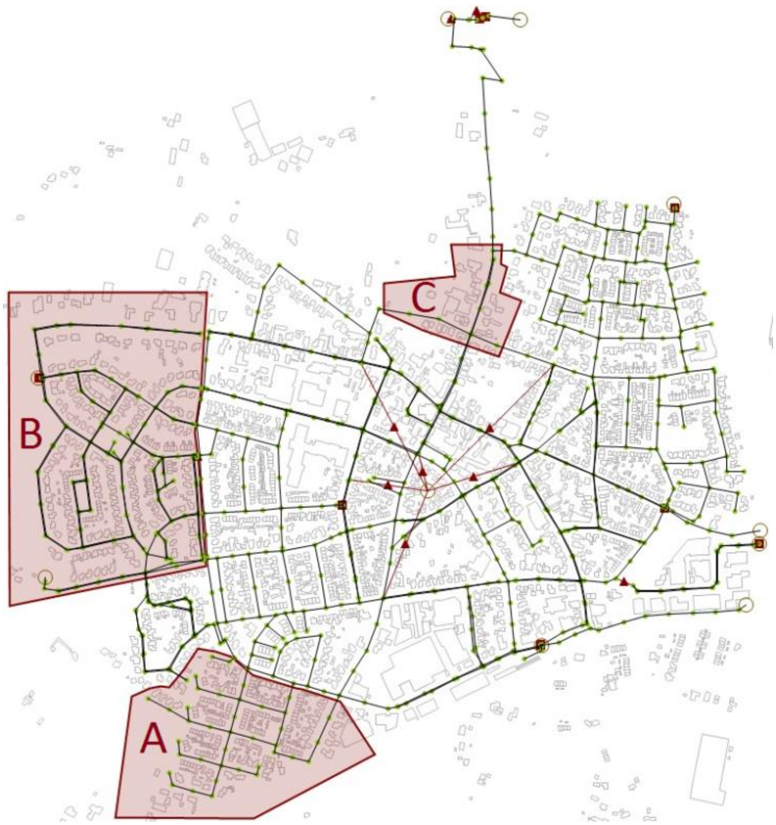
The interaction between the 1D and 2D part of the model goes via the manholes of the 1D-model. These manholes are modelled as weirs from which water will flow onto the surface if the water level in the manhole rises above surface level. The amount of water that flows onto the surface level depends on the weir coefficient of the manhole and on the shaft area of the manhole as the weir length is defined as the circumference of the shaft (Innovyze, 2015). In order to quantify the uncertainty of this part the model, the weir coefficient of the manholes is varied. A mean of 0.50 is assumed, as this value is used in literature (Russo et al., 2011). The standard deviation is nearly twice as large as the standard deviation used by Clemens (2001) for weirs, taking into consideration that less knowledge is available about the processes taking place at an overflowing manhole, compared to an overflowing weir.

The roughness of the mesh will also influence the modelled flow in the 2D part of the model. Values for manning's n to represent the roughness of the surface in urban areas vary from 0.02 (Boulos et al., 2006) to 0.1 $s/m^{1/3}$ (Chow, 1959). Therefore, in this research a mean value of 0.06 $s/m^{1/3}$ and a standard deviation of 0.02 $s/m^{1/3}$ are used in this analysis.

7.1.3 Quantifying uncertainty

Using the parameters given in tables 17 and 18, implies that 11 parameters are used, resulting in a Latin Hypercube with 11 dimensions. The parameter values for each run are obtained from this Latin Hypercube and the 1D/2D-model is simulated with each set of parameters. To quantify the uncertainty, the total flooded area for each run is obtained for the complete system and for 3 locations in the system where flooding occurs. The locations are given in figure 12. Location A is situated upstream in the system. Location B is also situated upstream in the system, but there are 2 weirs located in that part of the system. Location C is situated downstream in the system, close to the waste water treatment plant. These locations are used to investigate if the uncertainty in the results differs for different locations in the system. The return period used is 2 years.

Figure 12: Locations used for analysis



In order to characterize the results, the mean, standard deviation, coefficient of variation, skewness and kurtosis are calculated. The mean gives an indication of the average amount of flooded area. The standard deviation indicates the amount of spread in the results. A larger standard deviation indicates a larger spread in the results. The coefficient of variation is a standardized measure of the dispersion of the distribution. The skewness indicates the lack of symmetry of the distribution. A positive skewness indicates a long thin tail to the right, whereas a negative skewness indicates a long thin tail to the left. The kurtosis indicates if the peak of the distribution departs from the shape of a normal distribution. A positive value indicates a sharper peak, whereas a negative value indicates a broader peak. The skewness and kurtosis of the normal distribution are both 0. The calculations of the skewness and kurtosis have to be seen as indicative, due to the limited number of runs.

In order to get an indication of which variables are most important in explaining the variance in the results, a linear model is fitted to the results. The amount of variance that is explained by the linear model is calculated. The higher the value of $R^2_{adjusted}$, the better the model explains the variance of the data. A good model will be able to explain at least approximately 90% of the variance, so have a $R^2_{adjusted}$ value which is larger than 0.90 (Burgers et al., 2010).

If the model has a good fit, the relative variable importance can be derived, using the beta weight method (Nathans et al., 2012). The beta weight of an independent explaining variable indicates the expected change in the dependent variable in standard deviation units, given that the independent explaining variable is changed one standard deviation, while the other independent explaining variables are kept constant. The relative variable importance can then be derived from the absolute value of the beta weight, the larger this value, the more important the variable is. This method gives reliable results when the explaining variables are uncorrelated (Nathans et al., 2012). In order to check this, the correlation between the explaining variables is calculated.

The formulas used to calculate the parameters mentioned in the preceding paragraphs are presented in Appendix VII.2.

7.1.4 Sources of uncertainty not taken into consideration

Clemens (2001) also took into consideration the groups geometry and structural errors. Examples of structural errors are pipes which are missing, a different construction of the weir, etc. From the research it followed that the influence of structural errors on the model is limited when the structural errors are present in 1 % of the database. With a higher amount of structural errors, the influence increases. When a proper check is done on the database it is possible to obtain a low amount of structural errors (+/- 1 % is realistic), which implies that for such applications the influence of structural errors is limited. The influence of differences in geometry is not negligible. It mainly influences the maximum water depth at a manhole (Clemens, 2001). In this research however, taking into account this group is unpractical as this would result in a Latin Hypercube with an unpractical high amount of dimensions.

Another source of uncertainty in the 2D part of the model is the height of the 2D mesh, which is based on the Actueel Hoogtebestand Nederland 2 (AHN2). The standard deviation of the AHN2 is approximately 5 cm. In the interpolated parts of the AHN2 the standard deviation can be even much larger. Besides that, it could be that part of the surface area is given the wrong height because of unfiltered bushes or parked cars. Due to the presence of difference in standard deviation over the mesh and a wrong allocation of height to certain cells, varying the height of each cell independently, using the standard deviation of 5 cm will not give a good image of the uncertainty in the digital elevation model (Gallant & Hutchinson, 2006).

The sewer model itself is also not perfect as physical phenomena are not completely understood, or simplified in order to increase efficiency (Korving et al., 2009). An example of this is the use of the Preismann slot to model pressurized flow in pipes that generally act as open channel flows. Besides that, imperfect functioning of the sewer system due to asset failure (Ten Veldhuis, 2010) and sewer deterioration (Korving et al., 2009) is not taken into account. These sources of uncertainty are not taken into account because of the limited knowledge about which distributions to use for these sources of uncertainty.

7.2 Results

The results of the model uncertainty analysis are presented and analysed in this paragraph. In figure 13 the spread in the amount of inundated area is given for the total system and the 3 locations. In this figure it can be seen that the variety in the results is quite large. In order to check if 200 runs were enough to get stable results, after each run the mean of the inundated area computed till then is divided by the mean of the inundated area after the 200 runs. The same is done for the standard deviation, skewness and kurtosis of the inundated area. The results are shown in figure 14.

Figure 13: Distribution of amount of inundated area for the total system and 3 locations

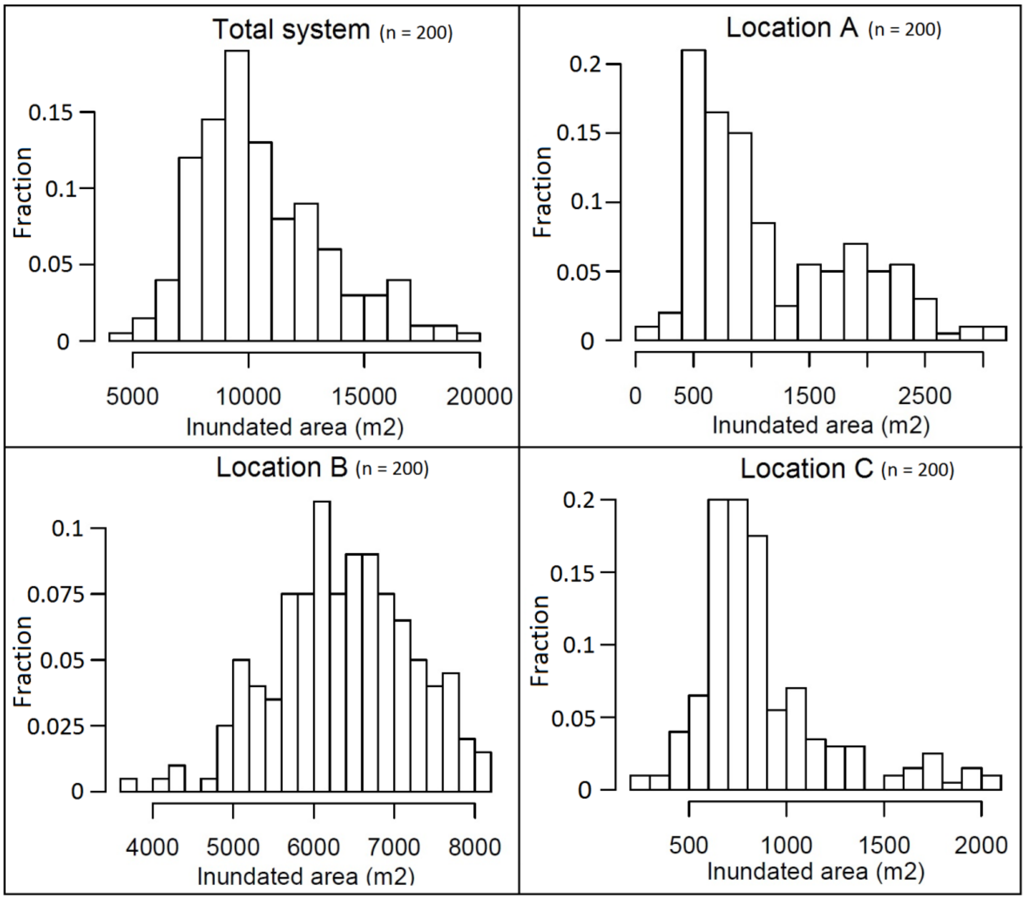
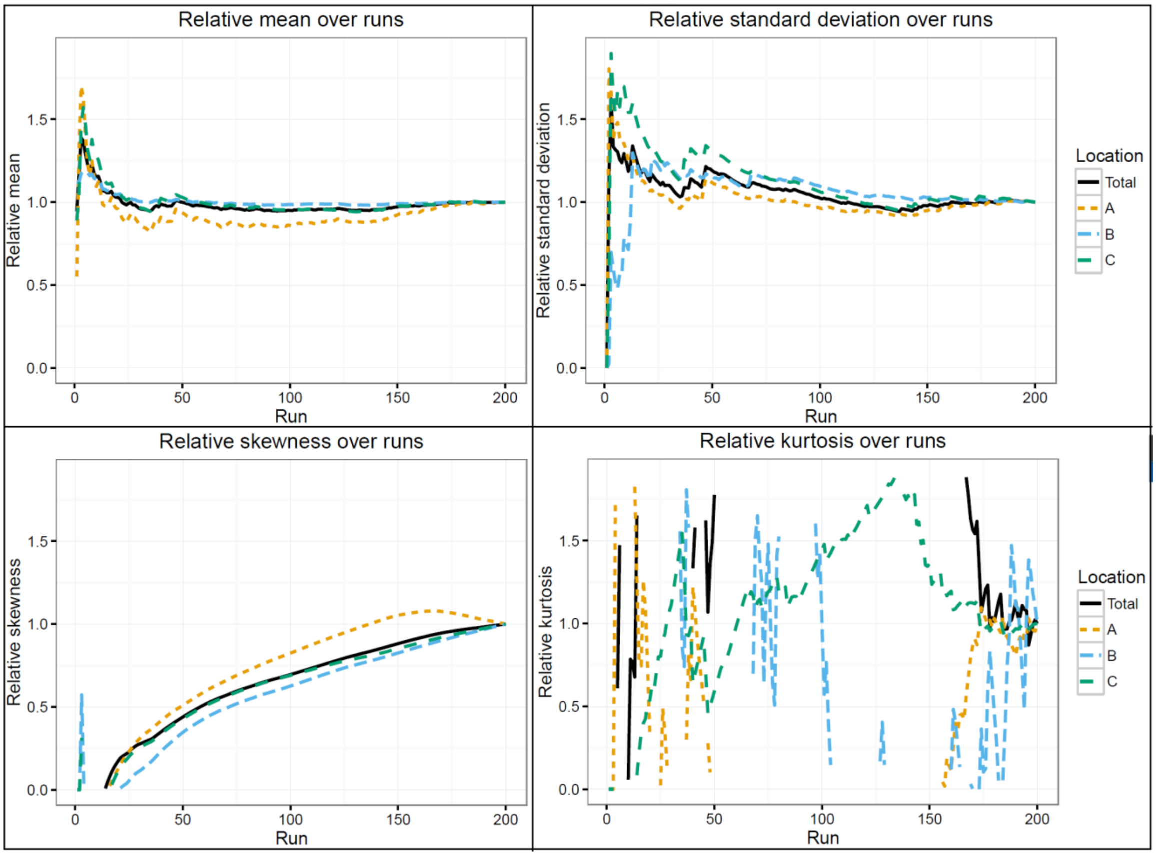


Figure 14: Relative mean, standard deviation, skewness and kurtosis over runs



From the results it can be concluded that the mean and standard deviation give stable results within the 200 runs. The skewness and kurtosis however don't. Therefore, the skewness and kurtosis given in table 19 have to be seen as indicative.

The characteristics of the distribution of the results are given in table 19. From these characteristics it can be seen that most flooding occurs at location B. The largest spread in the amount of flooding occurs at location A, as this location has the highest coefficient of variation. Location B coincides most with a normal distribution, as it has skewness and kurtosis values that are closest to 0.

Table 19: Characteristics of the distribution of the results

Location	Total	A	B	C
Mean ($\bar{x}$)	10,525 m^2	1,152 m^2	6,363 m^2	873 m^2
Standard deviation (s)	2,884 m^2	700 m^2	839 m^2	342 m^2
Coefficient of variation (CV)	0.27	0.61	0.13	0.39
Skewness (G_1)	0.76	0.77	-0.23	1.49
Kurtosis (G_2)	0.29	-0.45	-0.06	2.20

For each location, the fraction of variance that is explained by the linear model that is fitted to the results is given in table 20. For both the total system and location B, this fraction lies above the border of 90 %. When comparing the values of the skewness and kurtosis with the fraction of explained variance, it seems to be the case that more variance is being explained for the locations with a skewness and kurtosis closer to 0. The relative low amount of explained variance at location C is likely caused by its downstream position in the system. Due to the non-linear system response, the amount of flooded area at location C is also non-linear and has a long tail. As the fitted model is a linear model, it is not able to incorporate the non-linear behaviour.

Table 20: Amount of explained variance

Location	Total	A	B	C
Explained variance ($R^2_{adjusted}$)	0.90	0.86	0.96	0.78

The beta weight method is used to obtain the relative importance of the explaining variables for each location. In order to get a feeling of which variables are most important, the following parameter is given in the last column of table 21:

$$WW_i = |\beta_{i;T}| * R^2_{adjusted;T} + |\beta_{i;A}| * R^2_{adjusted;A} + |\beta_{i;B}| * R^2_{adjusted;B} + |\beta_{i;C}| * R^2_{adjusted;C} \quad (6)$$

where:

$$WW_i = the\ weighted\ beta\ weight\ of\ explaining\ variable\ i$$
$$|\beta_{i;T}| = the\ absolute\ value\ of\ the\ beta\ weight\ of\ explaining\ variable\ i\ for\ the\ total\ system$$
$$R^2_{adjusted;T} = the\ fraction\ of\ explained\ variance\ for\ the\ total\ system$$
$$|\beta_{i;A}| = the\ absolute\ value\ of\ the\ beta\ weight\ of\ explaining\ variable\ i\ for\ location\ A$$
$$R^2_{adjusted;A} = the\ fraction\ of\ explained\ variance\ for\ location\ A$$

The resulting weights are presented in table 21. The amount of correlation between the explaining variables is given in Appendix VI. The results show that there is no strong correlation between any of the explaining variables; the largest correlation coefficient is 0.41. Therefore, the resulting beta weights can be judged as a reliable representation of the importance of the explaining variables.

Table 21: Beta weights

Location	Total	A	B	C	WW_i
$c_{l;f}$	-0.59	-0.66	-0.41	-0.43	1.83
$c_{l;s}$	-0.01	0.00	-0.04	0.04	0.08
$b_{f;p}$	0.01	0.03	-0.01	0.04	0.08
$b_{f;r}$	-0.05	-0.03	-0.02	-0.08	0.15
i	-0.03	-0.02	-0.02	-0.06	0.11
A	0.73	0.61	0.89	0.77	2.64
K_a	0.27	0.30	0.23	0.13	0.82
c	-0.12	-0.06	-0.25	-0.07	0.45
Q_{pump}	-0.07	-0.01	-0.07	-0.15	0.26
c_m	-0.04	0.01	-0.06	-0.05	0.14
n_m	0.00	0.03	-0.01	-0.05	0.07

From these beta weights it can be concluded that the subcatchment area is the most important explaining variable, followed by the linear reservoir constant of flat surfaces. As the beta weight of subcatchment area is positive, an increase in area will lead to an increase of inundated area. The negative beta weight of the linear reservoir constant indicates that a faster discharge to the sewer will lead to an increase of inundated area.

From the remaining variables, the pipe roughness and weir coefficient have the largest influence on the amount of inundated area. The influence of the weir coefficient is by far the largest at location B, which is explained by the fact that at this location 2 weirs are present, while at location A and C no weirs are present. The influence of the pumping capacity at the locations is likely related to distance of the locations to the nearest pump; the influence at location A is relative low, as this location is relative far away from a pump, while the influence at location C is relative large, as this location is relative close to the waste water treatment plant.

The parameters in de 2D-part of the model have no a significant influence on the amount of inundated area. This shows that by far the largest part of the model uncertainty is situated in the 1D-part of the model.

8 UNCERTAINTY IN ALLOCATION OF EFFECT CATEGORY WEIGHTS

In this chapter the uncertainties in the allocation of the effect category weights are quantified. The questionnaire used to obtain the effect category weights is already described in paragraph 6.1. In paragraph 8.1 the method of quantifying the uncertainty in the allocation of the effect category weights is described. In paragraph 8.2 the results are presented and analysed.

8.1 Method

The questionnaire used to derive the effect category weights is described in paragraph 6.1. First some general questions are asked to be able to investigate the presence of differences in the perception of the effects of flooding between policy makers, advisors and citizens. After that each effect is compared to flooding of the first floor of a house, using the structure given in figure 10, paragraph 6.1. From the information in this part the effect category weights can be derived for each respondent. The resulting effect category weights from all respondents are used to construct an empirical cumulative distribution function (ECDF), a distribution function which is directly estimated from the sample data without assuming an underlying algebraic form (Everitt & Skrondal, 2010). The ECDF is defined as (van der Vaart, 1998):

$$\widehat{F}_n(t) = \frac{number\ of\ elements\ in\ the\ sample\ \leq t}{n} \quad (7)$$

where, in this case:

$$t = a\ value\ between\ \frac{1}{12}\ and\ 12$$
$$n = the\ number\ of\ respondents$$

The questionnaire is submitted to a limited number of policy makers, advisors and citizens. This is done to investigate if the different groups have a different perception of the severity of the effects of flooding.

In the last part of the questionnaire the effects other than flooding of the first floor of a house are compared mutually. This is done to investigate if the respondents are consistent in their comparative judgments. The degree of consistency of each respondent is calculated using the following formula (Kendall, 1948):

$$Degree\ of\ consistency = \zeta = 1 - \frac{number\ of\ inconsistencies}{possible\ number\ of\ inconsistencies} \quad (8)$$

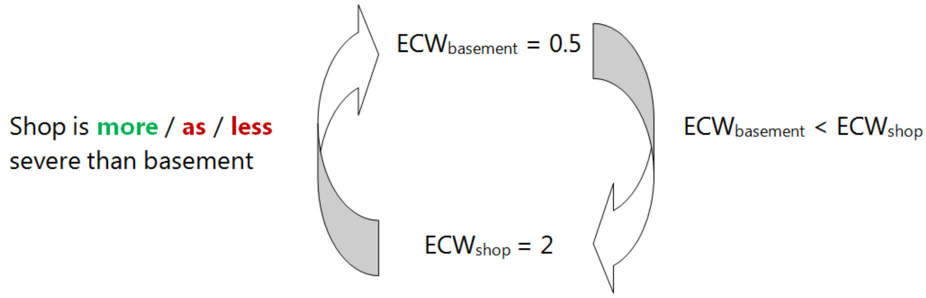
The method used by Kendall (1948) is developed for the situation where the respondents have two answer possibilities, less or more severe. In this questionnaire it is also possible to answer as severe. Therefore the number of inconsistencies has to be obtained in another way than done by Kendall (1948). In this case the consistency of the results is checked in two steps. First the effect category weights of two effects are compared, first column of table 22. After that the answer to the question which of these effects is more severe is checked for consistency, column 2 and 3 of table 22. The possible combinations of (in)consistent answers are given in table 22.

Table 22: Possible combination of (in)consistent answers

Comparison of effect category weights	Consistent answers	Inconsistent answers
	Effect A is .... severe as effect B	Effect A is ...... severe as effect B
$ECW_A > ECW_B$	more	as / less
$ECW_A = ECW_B$	as	more / less
$ECW_A < ECW_B$	less	more / as

In order to clarify this procedure, an example is schematized in figure 15. In this case the effect category weight of basement is smaller than the one of shop, thus the only consistent answer is that shop is more severe than basement.

Figure 15: Procedure of determining inconsistencies



As 8 effects are taken into consideration, the possible amount of inconsistent answers is:

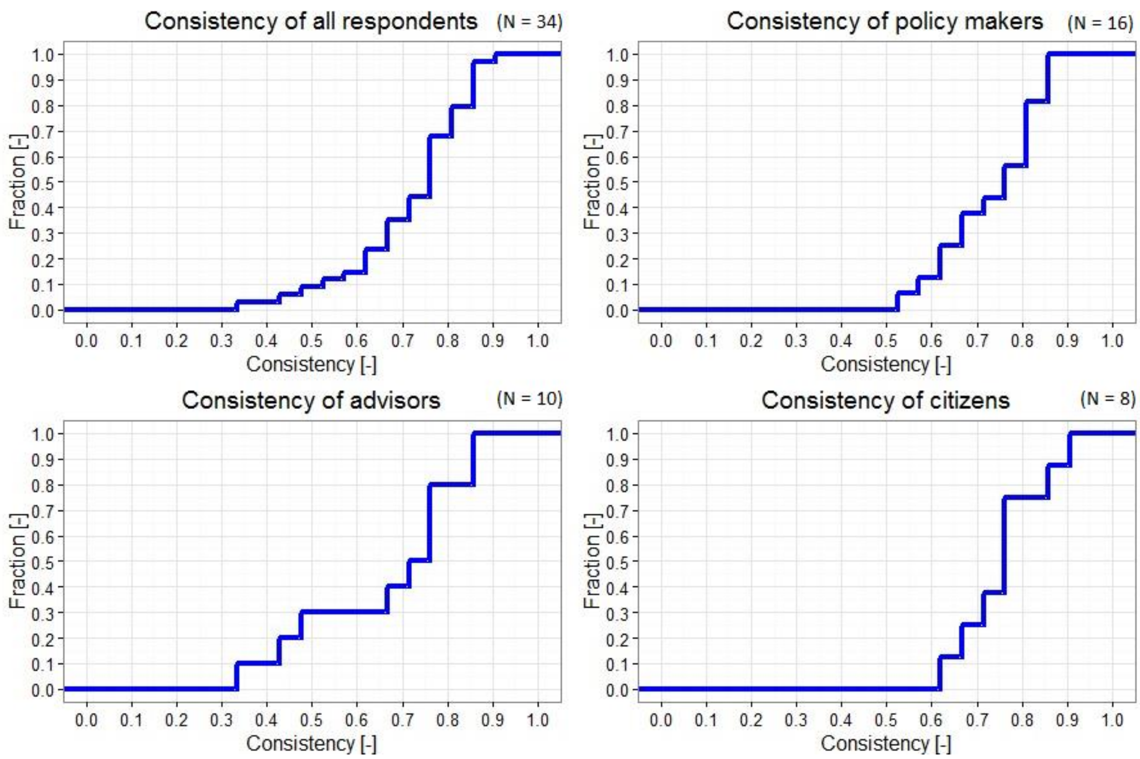
$$possible\ number\ of\ inconsistencies = 6 + 5 + 4 + 3 + 2 + 1 = 21 \quad (9)$$

In the next paragraph the results of the questionnaire are presented and analysed.

8.2 Results

The questionnaire is completed by 34 respondents, of which 16 policy makers, 10 advisors and 8 citizens. In figure 16 the consistency of the total group and the different subgroups is given. It can be seen that more than 90% of the respondents has a consistency of above 50 %. 50 % of the respondents have a consistency above 75 %. The respondents that have a consistency below 50 % are all advisors. The average consistency of the respondents is 71 %. Citizens have a distribution of consistency that is comparable with the one of policy makers. The results show that in general, the respondents have a pretty good feeling about the severity of different effects.

Figure 16: Consistency of respondents



The 5th, 50th and 95th-percentiles of the allocated effect category weights are given in table 23. Green boxes indicate an effect category weight lower than 1, yellow boxes indicate an effect category weight of 1 and red boxes indicate an effect category weight higher than 1.

Table 23: 5th, 50th and 95th-percentiles of allocated ECW's

Effect	5th-percentile ECW	50th-percentile ECW	95th-percentile ECW
Flooded basement	1/12	1/5	1
Flooded shop	1	1	5
Flooded manhole	1/10	1	12
Flooded tunnel	1/6	3	12
Infection risk	1/12	1/3	12
Blocked distributor road	1/12	1	12
Blocked access road	1/12	1/5	3

In appendix VIII the empirical cumulative distribution functions of all effects are presented. The following statements are based on information from table 23 and the ECDF's in appendix VIII:

- there are no respondents that judge that flooding of a basement is more severe than flooding of a house. A large majority (80 %) judges the flooding of a basement as less severe than flooding of a house;
- there are no respondents that judge that flooding of a shop is less severe than flooding of a house. Approximately half of the respondents judge the flooding of a shop as more severe than flooding of a house;
- the judgments about the severity of flooded manholes are very wide spread. This is possibly caused by the fact that some respondents took into consideration the occurrence of a wounded person, whereas others took into consideration the risk of getting wounded, which was also the goal of the survey;
- the judgments about the severity of flooded tunnels are also quite wide spread, although 55 % of the respondents judge that a flooded tunnel is more severe than a flooded house and only 20 % of the respondents judge this as less severe;
- the judgments about the infection risk are wide spread, possibly for the same reason as mentioned at the flooded manholes. The majority (55 %) judges infection as less severe than flooding of a house, while 30 % judges it as more severe than flooding of a house;
- the judgments about a blocked distributor road vary from the minimal (1/12) till the maximal (12) possible effect category weight. Both the group that judges a blocked distributor road as less severe than a flooded house, as the group that judges it as more severe, consist of 40 % of the respondents;
- the majority of the judgments (65 %) about a blocked access road judge it as less severe than a flooded house, although a small part (< 10 %) judges it as more severe than a flooded house.

Overall, it can be concluded that respondents are most consistent in their judgments about tangible effects (the flooding of buildings). The judgments about risks of injuries and infection are very wide spread. Judgements about traffic disruption are also quite wide spread. In paragraph 8.3 the uncertainty involved in allocating the effect category weights is compared to the uncertainty of the 1D/2D-model.

8.3 Comparison of uncertainties

The two sources of uncertainty are compared to each other in order to determine which source brings the largest uncertainty. In subparagraph 8.3.1 the method of the comparison is described. In subparagraph 8.3.2 the results are presented.

8.3.1 Method

In order to compare the two sources of uncertainty, the sewer system of Wehl is used. For the 3 locations analysed in chapter 7, a 90%-confidence interval of the risk level is obtained for both sources of uncertainty.

To construct the 90%-confidence interval of the risk level regarding model uncertainty, the following procedure is followed for each of the 3 locations:

1. Sort the 200 runs from chapter 7 in ascending order regarding the amount of flooded area.
2. Use the parameter values of the 11th and 189th ranked run (5th and 95th percentile) to obtain the lower and upper limit of the effects that occur.
3. Multiply the effects with the 50th percentile effect category weights from the questionnaire (table 24) to obtain the lower and upper limit of the 90%-confidence interval of the risk level.

To construct the 90%-confidence interval of the risk level regarding effect category weight uncertainty, the following procedure is followed for each of the 3 locations:

1. Use the default parameter values to obtain the effects that occur at the locations.
2. Multiply the effects with the 5th and 95th percentile effect category weight from the questionnaire (table 24) to obtain the lower and upper limit of the 90%-confidence interval of the risk level.

8.3.2 Results

In table 24, the 90%-confidence interval of the risk level regarding model uncertainty is shown. The more detailed results are presented in Appendix IX.

Table 24: 90%-confidence interval of the risk level regarding model uncertainty

Location	A		B		C	
Limit:	Lower	Upper	Lower	Upper	Lower	Upper
Normative risk level:	High	High	Extremely high	Extremely high	Very high	Extremely high
Exp. annual severity score:	0.5	1.4	8.7	13.1	1.9	3.9

From these results it can be seen that the model uncertainty has only a limited influence on the risk level obtained for the different locations. The influence on the expected annual severity score is larger. The largest relative influence is present at location A, where the upper limit of the expected annual severity score is nearly 3 times larger than the lower limit. An analysis of the detailed results presented in Appendix VIII shows that the differences largely depend on the amount of manholes flooded. This can be explained by the fact that the pressure height in the sewer system has a more direct relation with the model parameters than the effects that occur at the surface.

In table 25, the 90%-confidence interval of the risk level regarding effect category weight is shown. The more detailed results are presented in Appendix IX.

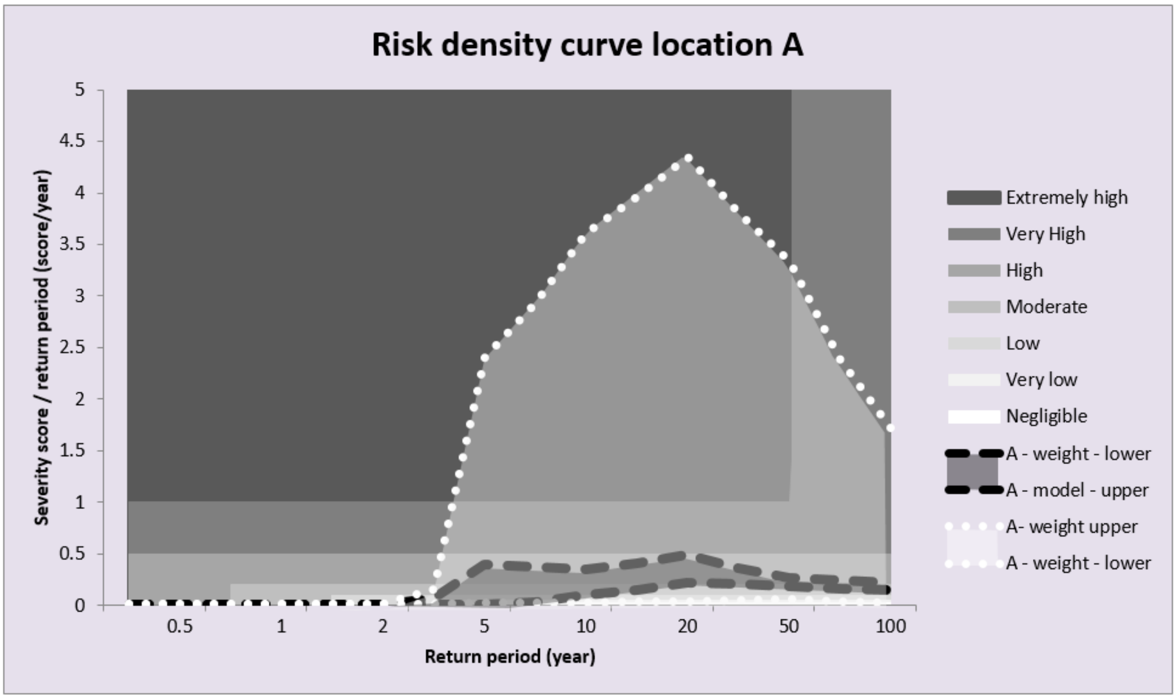
Table 25: 90%-confidence interval of the risk level regarding effect category weight uncertainty

Location	A		B		C	
Limit:	Lower	Upper	Lower	Upper	Lower	Upper
Normative risk level:	Low	Extremely high	Very high	Extremely high	Moderate	Extremely high
Exp. annual severity score:	0.1	12.8	1.5	164.2	0.6	37.3

From the results in table 25 it can be concluded that the uncertainty regarding the effect category weights is very large. The upper limit of the risk level is 'Extremely high' for each location. When looking solely to the risk level, the variation is limited for location B, but large for location A. The expected annual severity score does however vary approximately a factor 100 for both location A and B and a factor 60 for location C. The variation at location C is relative lower because a substantial part of the effects consists of flooding of houses, which has no varying effect category weight. The variation in risk level at location B is limited because the lower limit is just in 'Very high', whereas the upper limit is way up in 'Extremely high'. The large variation in risk level at location A is caused by the lower limit getting just in 'Low', while the upper limit is somewhat in 'Extremely high'.

When comparing the influence of both sources of uncertainty on the risk analysis, it can be concluded that, although the model uncertainty is not negligible, the influence of the effect category weight uncertainty is much larger. This uncertainty is even too large to make a reasonable judgment about the risk level of a certain location. To illustrate the difference in influence that both sources have, the 90%-confidence intervals of the model and weight uncertainty are plotted for location A in figure 17. This figure clearly shows the much larger variation of the weight uncertainty when compared to the model uncertainty.

Figure 17: 90%-confidence interval risk density curve model / weight uncertainty location A



9 DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

The goal of this thesis was to develop a general and straightforward applicable method for risk based flood management of sewer systems, giving an unambiguous result. From the content of the report it can be concluded that this goal is achieved, although there are some points of discussion. These points are described in paragraph 9.1. In paragraph 9.2 conclusions are drawn, while recommendations for how to apply the method and further research that can improve the method are given in paragraph 9.3.

9.1 Discussion

In the method risk is defined as the product of the severity and their frequency of occurrence. However, many scientists also emphasize the importance of further considerations, like a fair distribution of the risks and the available alternatives to a technology (Roeser, 2006). These considerations are already somewhat present in the developed method. An unfair distribution of the risks will lead to a high risk level of a certain location, implying that this location has priority in designing measures. The availability of alternatives can be accounted for by implementing measures. If a proper alternative is available, it will show up to be very expedient. Although the mentioned considerations are already somewhat present in the developed method, it is necessary to keep in mind that other considerations than solely taking into account the effects and their frequency of occurrence can be of importance to determine the risk level of certain locations.

The use of design storms instead of a full precipitation series is a weak point of the method. Among others, the design storms disregard the stochastic elements of precipitation and due to the non-linear system response, the return period of the effects can be different than the return period of the design storms. Besides that, applying uniform rainfall introduces additional uncertainty, as the direction of a storm is of significant importance for the effects that will occur during the storm.

The determination of which effects occur can be improved for several effect categories. There is great uncertainty about the damage that occurs if a building is flooded. Another aspect of the flooding of buildings which is not mentioned before, is who pays the damage that occurs. This could be the insurance company, the citizen or the municipality. In how far is the damage due to flooding relevant for the municipality when this damage is being paid by the insurance company? Also the infection risk and the risk of wounded are only roughly estimated in this method. Besides that, the traffic disruption depends not only on the type of road that is flooded, which is assumed in this method. Also the moment of flooding and the traffic intensity will determine the severity of traffic disruption.

Another point of discussion regarding the effects that occur is that the effects that are taken into consideration in the method do not fully represent all effects that can occur during an urban flood event. Flooding of specific locations in a city, like tunnels or main crossings, can lead to a huge accessibility problem. Also the accessibility of specific locations like hospitals and fire stations are of great importance. As it is difficult to judge the impact on accessibility of flooding of a specific road, only a distinction was made between access and distributor roads. In order to get a more precise image of the influence of flooding on accessibility, the use of a traffic model would be required.

The main point of discussion is the uncertainty involved in allocating the effect category weights of the different effect categories. The uncertainty is so large that it is not possible to make a reasonable judgment about the risk level of a location. This is mainly caused by the large spread in weights allocated to intangible effects, like infection risk, risk of wounded and traffic disruption. This large spread is possibly caused by the fact that some respondents took into account the occurrence, instead of the risk of infection and wounded, resulting in much higher weights. This explanation does not hold for the large spread in the weights allocated to traffic disruption.

As the uncertainty caused by allocating the effect category weights is very large, it is questionable in how far the method developed in this thesis gives more reliable results than alternative methods, like using depth-damage curves, etc. As long as it is not possible to reduce this uncertainty, one is not able to conclude that the alternative methods are less appropriate for performing a risk assessment.

9.2 Conclusion

The goal of this thesis was to develop a method for risk based flood management of sewer systems, which fulfils the following criteria:

- general applicability;
- only interpretable in one way. Risk assessment may not be dependent on the person using the method;
- required data has to be overall available / accessible / free;
- explainable in an easy way.

The method developed is general applicable. The design storms used are representative for the complete country. Every municipality can apply the method, by allocating the effect category weights. Once these weights are determined, the method is only interpretable in one way, as the risk is defined as the product of the effects and their frequency of occurrence. The data required for the method is freely available in the sense that the municipality does not need to buy additional data. Due to the straightforward structure of the method and the use of the risk matrix, the method is easily explainable. This shows that it can be concluded that the goal of the thesis is achieved.

In contrast with the Proeftuin-method, the developed method takes into account the full risk that occurs, as all effects are taken into consideration to determine the risk level, instead of only the normative effect category. Due to the calculation of the expected annual damage, also all return periods are taken into account, instead of only the normative return period as was the case in the Proeftuin-method.

Based on the underlying principles it can be concluded that the composite design storms are more appropriate for use in the method than the Dutch design storms. A comparison of the design storms with the full precipitation series shows that in general the Dutch design storms coincide better with the full precipitation series at low return periods, while the composite design storms do this at larger return periods. Overall, it is concluded that the composite design storms are more appropriate to use in this method than the Dutch design storms.

The amount of subcatchment area is the parameter that is responsible for by far the largest part of the model uncertainty. The influence of the parameters in the 2D-part of the model is only limited. The influence of model uncertainty on the results of the risk level is not negligible and mainly caused by the varying amount of flooded manholes.

Based on the results of the questionnaire it can be concluded that in general, the respondents have a pretty good feeling about the severity of different effects. Between the respondents, the judgments about the severity of the effects do however vary a lot; the spread in the weights that are allocated to the effect categories is very large. Due to this fact, the influence of the uncertainty caused by the allocation of the effect category weights on the risk analysis is huge and way larger than the influence of model uncertainty.

9.3 Recommendations

As the method had to be practical applicable, the use of design storms was preferred above a full precipitation series. This choice was made because using a full precipitation series requires much more computational power and a laborious process of statistical analysis of the results. For the same reason, uniform precipitation was applied instead of spatially varying precipitation. It is recommended to investigate if the processes of statistical analysis can be automated. If this is possible, the only obstacle is the large computational power that is required. This obstacle can be tackled by using powerful computers and accepting longer simulation times. As the use of spatially uniform design storms comes with considerable inaccuracy, the use of a full precipitation series and spatially varying precipitation will result in significant benefits.

If additional information or alternative methods become available, it is recommended to review the way in which the effects are determined. Other methods or additions to the current method could improve the determination of the effects. It is also recommended to include other effects if additional information shows that there are significant effects which are not taken into consideration in the current method.

The influence of model uncertainty on the risk analysis is not negligible and the amount of subcatchment area is the most influential parameter in the model uncertainty. When carrying out the risk analysis, it is therefore recommended to investigate the amount of subcatchment area that surcharges to the sewer and the location at which it surcharges to the sewer as detailed as feasible.

As mentioned before, the uncertainty in the allocation of the effect category weights is very large. This is shown to be a very weak point of the method. Therefore, the most important recommendation is to perform additional research that is aimed at finding alternative methods or gathering additional information to reduce this uncertainty.

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Appendices

APPENDIX: CAUSAL MATRIX PROEFTUIN-METHOD AND ADAPTED MATRICES W+B-METHOD

		waarden					
			kwaliteit leefomgeving				
		veiligheid & gezondheid	**bereikbaarheid** **A: erg belangrijk gebied** **B: tamelijk belangrijk gebied**	**leefbaarheid openbare ruimte** **(functie zoals bedoeld kan niet meer worden vervuld)** **tijdsduur is mede bepalend**	**leefbaarheid particulier terrein** **tijdsduur is mede bepalend**	**kosten en financien** **schade bedrag** **kapitaalvernietiging**	**imago**
ernstcategorie	**zeer ernstig**	Doordat de riolering overbelast is, staat er in meer dan 25% van de stad of op meer dan 10 hoofd-wegen gedurende meerdere uren (verdund) rioolwater op straat of ontstaat er een waterdiepte van meer dan 0,5m op doorgaande wegen, en/of komen op zeer grote schaal putdeksels omhoog (>1.000 over de gehele stad of >100 per buurt/wijk)	–	–	**gehele stad:** Er staat water in meer dan 10 gebouwen cat. A en/of er staat water in meer dan 100 gebouwen cat. B welke een peil hebben van enkele decimeters boven straatpeil en/of in 1.000 achtertuinen.	- kosten voor tijdelijk oplossen wateroverlast - gevolgschade die ontstaat door water-overlast.	**op stads- of wijkniveau:** uit het hele gebied klachten over zeer ernstige wateroverlast
	ernstig	Doordat de riolering overbelast is, staat er in 5-25% van de stad of op enkele hoofdwegen gedurende 1-2 uur (verdund) rioolwater op straat, of komen op grote schaal putdeksels omhoog (>100 over de gehele stad of >10 per buurt/wijk)	meerdere locaties **categorie A** niet meer bereikbaar door ernstige water op straat (>20cm) **stadsniveau:** op diverse locaties op doorgaande wegen water op straat (>20cm) waardoor grote delen van de stad niet of slecht bereikbaar zijn dat leidt tot blokkering van deze wegen	op diverse locaties in de stad is de openbare ruimte gedurende meer dan 1 dag niet bruikbaar door wateroverlast vanuit de riolering	**gehele stad:** Er staat water in 1-10 gebouwen cat. A en/of er staat water in 10-100 gebouwen cat. B , welke een peil hebben van enkele decimeters boven straatpeil en/of er staat water in 100-1.000 achtertuinen	- kosten voor tijdelijk oplossen wateroverlast - gevolgschade die ontstaat door water-overlast.	**op wijkniveau:** uit het hele gebied klachten over ernstige wateroverlast
	aanzienlijk	Doordat de riolering overbelast is, staat er in <5% van de stad of op enkele van de hoofd-wegen gedurende 30-60 minuten (verdund) rioolwater op straat (ook buiten de trottoirbanden), of komen op sommige plaat-sen putdeksels omhoog (>10 over de gehele stad of enkele per buurt/wijk)	**Stadsniveau:** Er staat gedurende 1-2 uur 10-20 cm water op straten in gebieden cat. A (omvang 300-1.000 gebouwen). **Wijkniveau:** Er staat gedurende meerdere uren 5-10 cm water in gebieden cat. A (300-1.000 gebouwen) en 10-20 cm water op straten in gebieden cat. B (omvang 300-1000 gebouwen).	op 1 locatie in de stad is de openbare ruimte gedurende meer dan 1 dag niet bruikbaar door wateroverlast vanuit de riolering	**op wijkniveau:** Er staat water in 1-10 gebouwen cat. B. , welke een peil hebben van enkele decimeters boven straatpeil en/of er staat water in 100 achtertuinen.	- kosten voor tijdelijk oplossen wateroverlast - gevolgschade die ontstaat door wateroverlast.	**op buurtniveau:** uit het hele gebied klachten over water-overlast
	matig	Doordat de riolering overbelast is, staat er in <5% van de stad of op enkele van de hoofdwegen gedurende minder dan 30 minuten (verdund) rioolwater op straat (wel tussen de trottoirbanden, opletten waar je rijdt of loopt), of komt er een enkele putdeksel omhoog,	**Wijkniveau:** Er staat gedurende 1-2 uur 5-10 cm water op straten in gebie-den cat. A (omvang 100-300 gebouwen) en 10-20 cm in gebieden cat. B (300-1000 gebouwen). **Buurtniveau:** Er staat gedurende meerdere uren <5 cm water op straten in gebieden cat. A (omvang 100-300 gebouwen) en <10 cm water op straten in gebieden cat. B (omvang <300 gebouwen).	op diverse locaties in de stad is de openbare ruimte gedurende enkele uren niet bruikbaar door wateroverlast vanuit de riolering	**op buurtniveau:** Water in 1 gebouw dat een aanlegpeil heeft van enkele decimeters boven het straatpeil of in 10 achtertuinen.	- kosten voor tijdelijk oplossen wateroverlast - gevolgschade die ontstaat door water-overlast.	**op straatniveau:** uit het hele gebied klachten over water-overlast
	klein	Er staat op meerdere plekken water op straat maar dat blijft binnen de trotoirbanden.	Buurtniveau/straatniveau: Er staat gedurende 1-2 uur <5 cm water op straten in gebieden cat. A (omvang <100 gebouwen) en <10 cm in gebieden cat. B (100-300 gebouwen).	op 1 locatie in de stad is de openbare ruimte gedurende enkele uren niet bruikbaar door wateroverlast vanuit de riolering	op straatniveau: Water in gebouwen en achtertuinen	- kosten voor tijdelijk oplossen wateroverlast - gevolgschade die ontstaat door water-overlast.	cluster van enkele woningen: lokaal zijn er klachten over wateroverlast
	verwaarloosbaar	Er staan op meerdere plekken kleine plassen op straat.	Cluster van enkele gebouwen: Er staan flinke plassen op straat.	door wateroverlast vanuit de riolering zijn enkele gebieden in de stad gedurende maximaal 1 uur niet te gebruiken voor de functie waarvoor zij zijn ingericht	cluster van enkele woningen: Water in gebouwen en achtertuinen	- kosten voor tijdelijk oplossen wateroverlast - gevolgschade die ontstaat door water-overlast.	enkele klachten en meldingen per jaar, verdeeld over de stad

Table I.1: Adapted risk matrix W+B-method

		Frequency of occurrence			
		Unlikely	**Possible**	**Likely**	**Regular**
		Return period: 100 y	**Return period: 10 y**	**Return period: 2 y**	**Return period: 1 y**
Level of severity	**Severe**	Moderate	High	Very high	Unacceptable
	Substantial	Low	Moderate	High	Very high
	Moderate	Negligible	Low	Moderate	High
	Small	Negligible	Negligible	Low	Moderate

Table I.2: Adapted effect matrix W+B-method

Level of severity	**Security and health**	**Quality of the living environment**			**Reputation**
		Accessibility	**Liveability private terrain**	**Companies (capital damage)**	
Severe	Diluted sewer water also outside the kerbs (large health risk) More than 10 manhole covers rise (large risk of wounded) Water depth > 0.5 m on a connection road (large risk of wounded)	Longer than 1 hour more than 10 cm water on a main road	Water enters more than 10 dwellings	Water enters a company or shop	From all over the area complaints about severe flooding
Substantial	Diluted sewer water also outside the kerbs (large health risk) Some manhole covers rise (risk of wounded)	For maximal 1 hour more than 10 cm water on a main road	Water enters some dwellings	Water doesn't enter companies or shops	From all over the area complaints about flooding
Moderate	Diluted sewer water on the road, inside the kerbs (small health risk)	For maximal 1 hour less than 10 cm water on a main road	Water doesn't enter dwellings	Water doesn't enter companies or shops	Locally complaints about flooding
Small	Water stays inside the kerbs, no diluted sewer water on the road	For maximal 1 hour less than 10 cm water on a main road	Water doesn't enter dwellings	Water doesn't enter companies or shops	A few complaints

II

APPENDIX: UNDERLYING PRINCIPLES DESIGN STORMS

II.1 Dutch design storms

The design storms that are generally used in The Netherlands, are described in the module C2100 of the Leidraad Riolering, developed by Stichting Rioned (2004). These design storms are based on a 25 year long precipitation series measured at De Bilt from 1955 till 1979 with an interval of 15 minutes. From this series extreme rainfall events are selected based on their maximal rainfall intensity in 15 minutes. A rainfall event is defined as a continuous period in which no dry periods longer than 5 hours are present. In the next subparagraphs the way in which different aspects of the design storms are obtained, is described.

Peak intensity

To obtain the peak intensity, first the selected rainfall events from the precipitation series are sorted in a descending order, based on their maximum precipitation intensity in 15 minutes. A return period is coupled to this list based on the rank of the intensities. The 5^{th} ranked intensity for this series of 25 years has then a return period of 5 years, the 25^{th} ranked intensity has then a return period of 1 year. The return periods for which events are constructed are 0.25, 0.5, 1, 2, 5 and 10 years. So to determine the peak intensity, respectively the 100^{th}, 50^{th}, 25^{th}, the average of the 12^{th} and 13^{th}, 5^{th} and the average of the second and third event are used. For the design storms a peak length of 10 minutes is assumed.

Length

Many of the rainfall events in the precipitation series have a long tail, which is not of importance for hydraulic calculations of sewer systems. To prevent very long simulation periods, the length of the event is limited to the time in which 85 % of the total precipitation quantity of that event has fallen. The length of the design storms is equal to the average length from all rainfall events that have an equal or larger return period than the concerning return period.

Precipitation amount

The total precipitation amount of the design storms is based on the average precipitation amount of 9 rainfall events, namely the rainfall event corresponding to the concerning return period, the 4 adjacent rainfall events with a larger return period and the 4 adjacent return periods with a smaller return period.

Shape

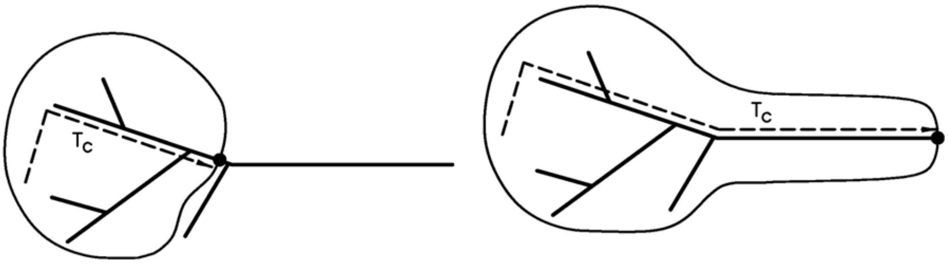
For each of the return periods 0.25, 0.5, 1, and 2 years, 2 design storms are developed, one with the peak at the front of the storm and one with the peak at the rear of the storm. A different place of the peak will lead to different effects, although the storm has the same return period. If the peak is at the rear of the storm, the sewer system will be filled more before the peak flow than when the peak is at the front of the storm. For the return periods 5 and 10 years only storms with a peak at the front of the storm are developed.

II.2 Composite design storms

Another type of design storm used in many countries is the composite design storm. The principles of the composite design storm are analogous to the Chicago design storm, developed by Keifer and Chu (1957). In this paragraph, the principles of constructing the Flemish composite design storms as described by Vaes et al. (2004) are described.

To determine which precipitation to use for hydraulic calculations of sewer systems, it is not enough to analyse only the precipitation, also the behaviour of the system is of importance. The most important parameter that determines this behaviour is the concentration time (Vaes et al., 2004). The concentration time is the time it takes for the most upstream precipitation to flow to a certain point. This time is different for different locations in the sewer system, see figure II.1.

Figure II.1 : Illustration of the physical meaning of concentration time (T_c) for two different locations (●) (Vaes, et al., 2004)

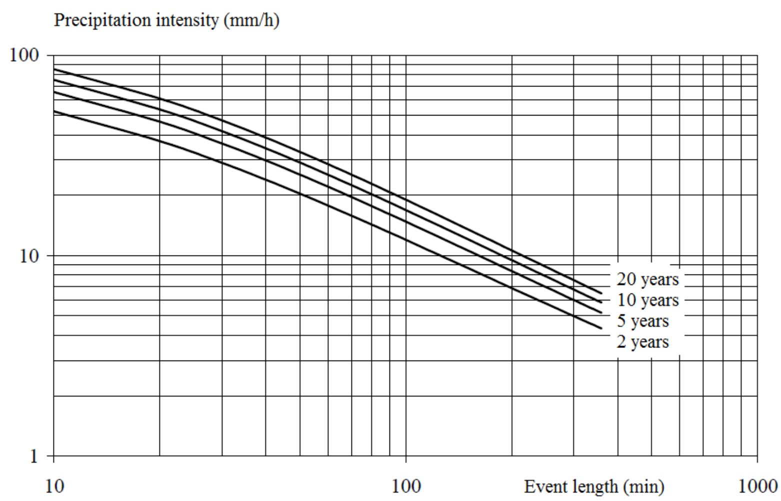


To incorporate the concentration time in the determination of the appropriate design storm, IDF-relations are a very useful tool. An IDF-relation describes the relation between the average precipitation intensity and frequency of occurrence for a range of event lengths. To construct an IDF-relation, the following procedure is followed:

1 For every event length taken into consideration, all high precipitation volumes are selected from a long precipitation series. The parameter event length, also called aggregation level, doesn't represent the total length of the event, but the width of the time frame that is moved over the precipitation series, in which the precipitation volume is accumulated. The corresponding precipitation intensity is than found by dividing the acquired volume by the width of the time frame, the precipitation is thus averaged over the aggregation level. This averaging also takes place in sewer systems, due to the distributed discharge. The minimum aggregation level is the frequency at which the precipitation is measured. The maximum aggregation level has to be at least equal to the maximum concentration time of the system.
2 The acquired precipitation intensities per aggregation level are ordered from high to low, in order to find the relation with the return period. Using just the rank in the ordered series to determine the return period, as done for the Dutch design storms, an accurate statement can be done till approximately 1/20 of the total length of the used precipitation series (Vaes et al., 1994a, 1994b). Therefore, extreme value analysis is required to find the corresponding precipitation intensities for larger return periods.
3 Using the acquired precipitation levels per aggregation level for the different return periods, IDF-relations are constructed. An example is given in figure II.2. For a more detailed description of the construction of IDF-relations, is referred to (Vaes et al., 2004) and (Vaes et al., 1994a).

Willems (2000) expanded this procedure by using a compound IDF-relationship for air mass thunderstorms and frontal storms. Besides that a correction for the intensities of the minimum aggregation level of 10 minutes was included, because in the original procedure these intensities where underestimated (Willems, 2000). This correction was justified based on 1-minute precipitation data from the city of Antwerp (Willems et al., 2002).

Figure II.2: IDF-relations based on Ukkel precipitation series (Vaes, et al., 2004)



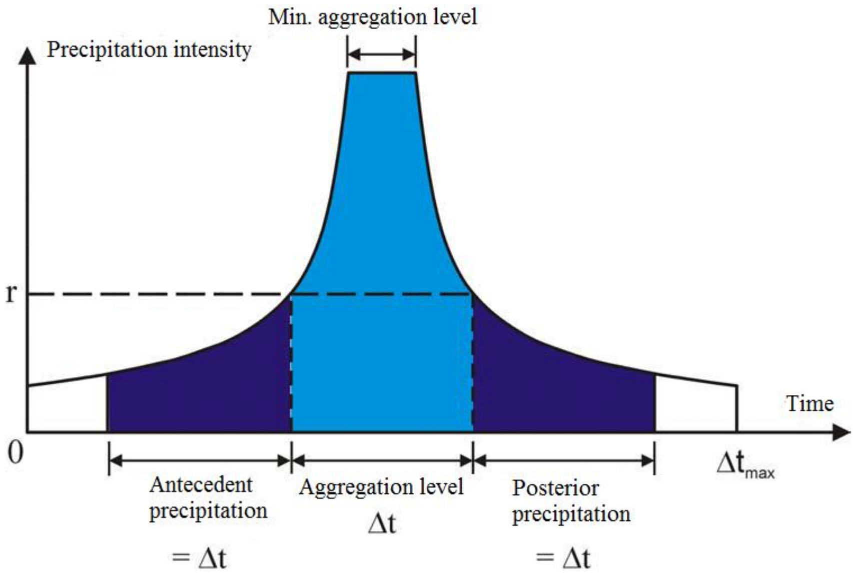
From the IDF-relations design storms are derived that have a uniform return period as function of the concentration time. These design storms are constructed in such a way, that the return period is equal for every event length. Assuming independency between the different aggregation levels, all event lengths between the minimum and maximum aggregation level for a specific return period are thus incorporated in one design storm, which is the reason why such storms are called composite design storms. To construct such a storm for a certain return period, the precipitation volumes from the IDF-relations are placed symmetric regarding the centre of the event, starting with the minimum aggregation level, see figure II.3. In this way, the volume of the design event is equal to the IDF-relations for every interval symmetric to the centre of the event. The symbols in figure II.3 represent:

- $\Delta t = aggregation\ level$
- $\Delta t_{max} = maximum\ aggregation\ level$
- $r = the\ precipitation\ intensity\ for\ \Delta t, obtained\ with\ the\ following\ eqaution:$

$$r = i(1 + \frac{d \log i}{d\ log \Delta t}) \quad \text{(II. 1)}$$

with $i = precipitation\ intensity\ from\ the\ IDF - relation\ corresponding\ to\ \Delta t$

Figure II.3: Schematization construction of a composite design storm (Vaes et al., 2004)



In figure II.3, also the terms antecedent and posterior precipitation are present. These types of precipitation incorporate the effect of the filling of the sewer system before the event and the effects of precipitation during the emptying of the system after the event (Vaes & Berlamont, 1996). The constructed composite design storms already have an intrinsic antecedent and posterior precipitation included, as shown in figure II.3. The significant antecedent precipitation for aggregation level Δt is the precipitation during the period Δt before the aggregation level Δt (Vaes, 1999). The same holds for the posterior precipitation, only now it is the precipitation that follows after the aggregation level Δt. Vaes (1999) compared the amount of antecedent and posterior precipitation from the composite design storms with the amount of precipitation before and after the events considered for the aggregation level in the historic precipitation series. From this comparison it follows that the antecedent and posterior precipitation in the composite design storms coincides well with the median value of those from the historic precipitation series (Vaes, 1999).

III

APPENDIX: CONSTRUCTING COMPOSITE DESIGN STORMS AND ACCOUNTING FOR CLIMATE CHANGE

III.1 Constructing composite design storms

The IDF-relation used for constructing the composite design storm is described by the following formula (Willems, 2011):

$$T = \frac{n}{m\left(p_a \exp\left(\frac{i_0 - i.C}{\beta_a}\right) + (1-p_a)\exp\left(\frac{i_0 - i.C}{\beta_b}\right)\right)} \quad \text{(III.1)}$$

with:

$$\log(\beta_a) = -0.05 - 0.58\log(D)$$
$$\log(\beta_b) = -0.55 - 0.58\log(D)$$
$$\log(p_a) = -1.35 - 0.58\log(D)$$
$$\log(i_0) = -0.55 - 0.58\log(D)$$
$$\log(p_1) = -1.12 - 0.50\log(D)$$

where:

$$T = return\ period\ [years]$$
$$i = precipitation\ intensity\ [mm/h]$$
$$n = the\ number\ of\ years\ in\ the\ original\ period\ 1967 - 1993 = 27$$
$$m = the\ number\ of\ values\ above\ the\ treshold = 1 + 54 * \frac{p_1}{p_1 - p_a}$$
$$i_0 = the\ treshold\ value\ [mm/h]$$
$$\beta = the\ average\ intensity\ of\ the\ exponential\ extreme\ value\ distribution$$
$$C = correction\ factor\ for\ the\ period\ 1970 - 2007\ relative\ to\ the\ original\ period\ 1967 - 2013$$
$$This\ factor\ is\ 0.93\ for\ aggregation\ levels\ till\ 1\ day\ and\ 1\ for\ aggregation\ levels\ larger\ than\ 1\ day$$
$$D = the\ aggregation\ level\ [days]$$

Using this relation, the intensities for every aggregation level and return period are obtained. The results are given in table III.1.

Table III.1: IDF-intensities for given return periods and aggregation levels [mm/h]

	Return period [years]					
aggregation level [minutes]	0.5	1	2	5	10	100
10	49.1	60.9	72.8	88.5	100.3	139.7
20	25.2	32.6	40.4	50.9	58.8	85.2
30	17.5	22.7	28.6	36.8	43.1	63.9
40	13.7	17.6	22.4	29.2	34.5	52.1
50	11.5	14.6	18.5	24.4	29.0	44.5
60	10.0	12.6	15.9	21.1	25.2	39.1
120	6.1	7.4	9.1	12.0	14.6	23.9
180	4.6	5.6	6.7	8.7	10.6	17.9
240	3.8	4.6	5.5	7.0	8.5	14.6
300	3.3	4.0	4.7	5.9	7.2	12.4
360	3.0	3.5	4.2	5.2	6.2	10.9
720	1.9	2.3	2.6	3.2	3.8	6.6
1440	1.2	1.5	1.7	2.0	2.3	3.9

From these intensities the intensities for the composite design storms can be acquired for a time step of 5 minutes using the formula (Vaes et al., 2004):

$$r = \frac{di}{d\Delta t} + i = i\left(1 + \frac{d\log i}{d\log \Delta t}\right) \quad \text{(III.2)}$$

where:

$$i = precipitation\ intensity\ [mm/h]$$
$$\Delta t = aggregation\ level\ [days]$$

The first step is to determine the peak intensity at the centre of the storm for the aggregation level of 10 minutes, using the formula:

$$R_{comp}(-5\text{ min} \le t \le 5min, T) = R_{idf}(D = 10min, T)$$

Where:

$$R_{comp} = precipitation\ intensity\ for\ the\ composite\ design\ storm\ [mm/h]$$
$$D = aggregation\ level\ [min]$$
$$T = return\ period\ [years]$$
$$R_{idf} = precipitation\ intensity\ from\ the\ IDF - relation\ at\ a\ given\ aggregation\ level\ and\ return\ period$$

The intensity for the next time steps of 5 minutes from the centre of the storm is obtained by:

$$R_{comp}(-10\text{ min} \le t \le -5min, T) = R_{comp}(5\text{ min} \le t \le 10min, T)$$
$$= R_{idf}(D = 20min, T) * 20min - R_{comp}(-5\text{ min} \le t \le 5min, T) * 10min$$

The intensity for the next time steps of 5 minutes from the centre of the storm is obtained by:

$$R_{comp}(-15\text{ min} \le t \le -10min, T) = R_{comp}(10\text{ min} \le t \le 15min, T)$$
$$= R_{idf}(D = 30min, T) * 30min$$
$$- \left(R_{comp}(-5\text{ min} \le t \le 5min, T) + R_{comp}(-10\text{ min} \le t \le -5min, T)\right) * 10min$$

In this way the intensities of all the time steps in the composite design storm can be obtained. The resulting composite design storms are presented in Appendix IV.

III.2 Climate change

To get insight in the possible effects of climate change on the extreme rainfall events that cause flooding in urban areas, Willems (2013) used different climate models to predict the change in the 10-minutes precipitation series from Ukkel in the period 2071 - 2100. This was done, using the quantile perturbation method, which is described by Willems and Vrac (2011): the quantile perturbation method applies the relative changes of rainfall intensities between the climate model run and the historical series dependant on the return period of the rainfall intensity. These changes are derived from daily rainfall intensities, but assumed to be the same for smaller time steps in the respective day, like hourly and 10-minute intensities. The validity of this assumption is questionable, as research by Loriaux et al. (2013) has shown that the increase in precipitation intensity due to an increasing dew point temperature is higher for hourly and 10-minute precipitation intensities than for daily precipitation intensities. However, this assumption has to be made, in order to transform the precipitation series and in that way get an indication of the influence of climate change. This procedure is followed for each climate model, per month, to change the 10-minute precipitation series of Ukkel (Willems, 2013).

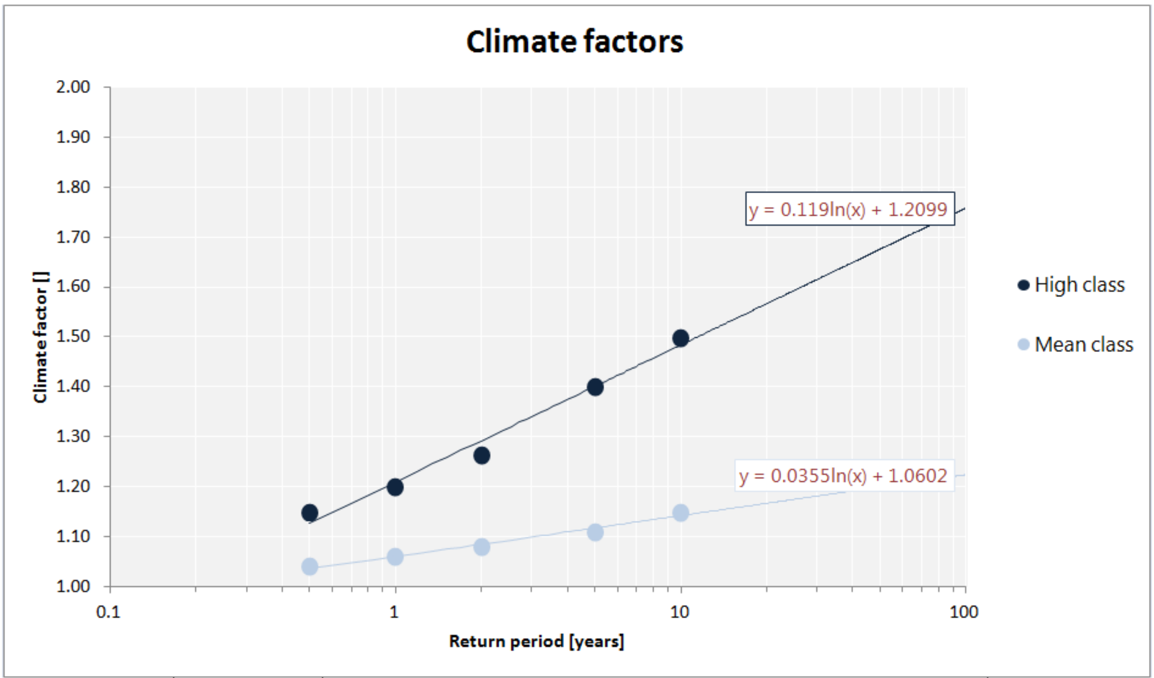
The transformed 10-minute precipitation series was then used to construct new IDF-curves. As there were more than 50 climate models, analysing the results of every climate model separate, would result in 50 new IDF-curves. To reduce this amount, the results of the 50 climate models where statistically processed in 3 classes, high, middle and low. These 3 classes then result in 3 new IDF-curves for each return period. The new IDF-curves are shifting more or less parallel upward when compared to the current IDF-curves on double logarithmic scale (Willems, 2013).

The scaling factor for each aggregation level is then equal to:

$$\frac{\log(precipitation\ intensity\ IDF - relation\ climate\ change)}{\log(precipitation\ intensity\ original\ IDF - relation)} \quad (\text{III}.3)$$

Such a factor is also called a climate factor. A parallel shift means that the intensities for all aggregation levels have the same relative shift. This makes it possible to deduce composite design storms for the climate scenarios from the original composite design storms by multiplying them with the climate factor for all time steps. The climate factors for the middle and high classes are given in figure III.1. The factors of the low class where almost equal to 1, indicating that climate change has no effect on the extreme precipitation intensities in the climate models from this class.

Figure III.1: Climate factors



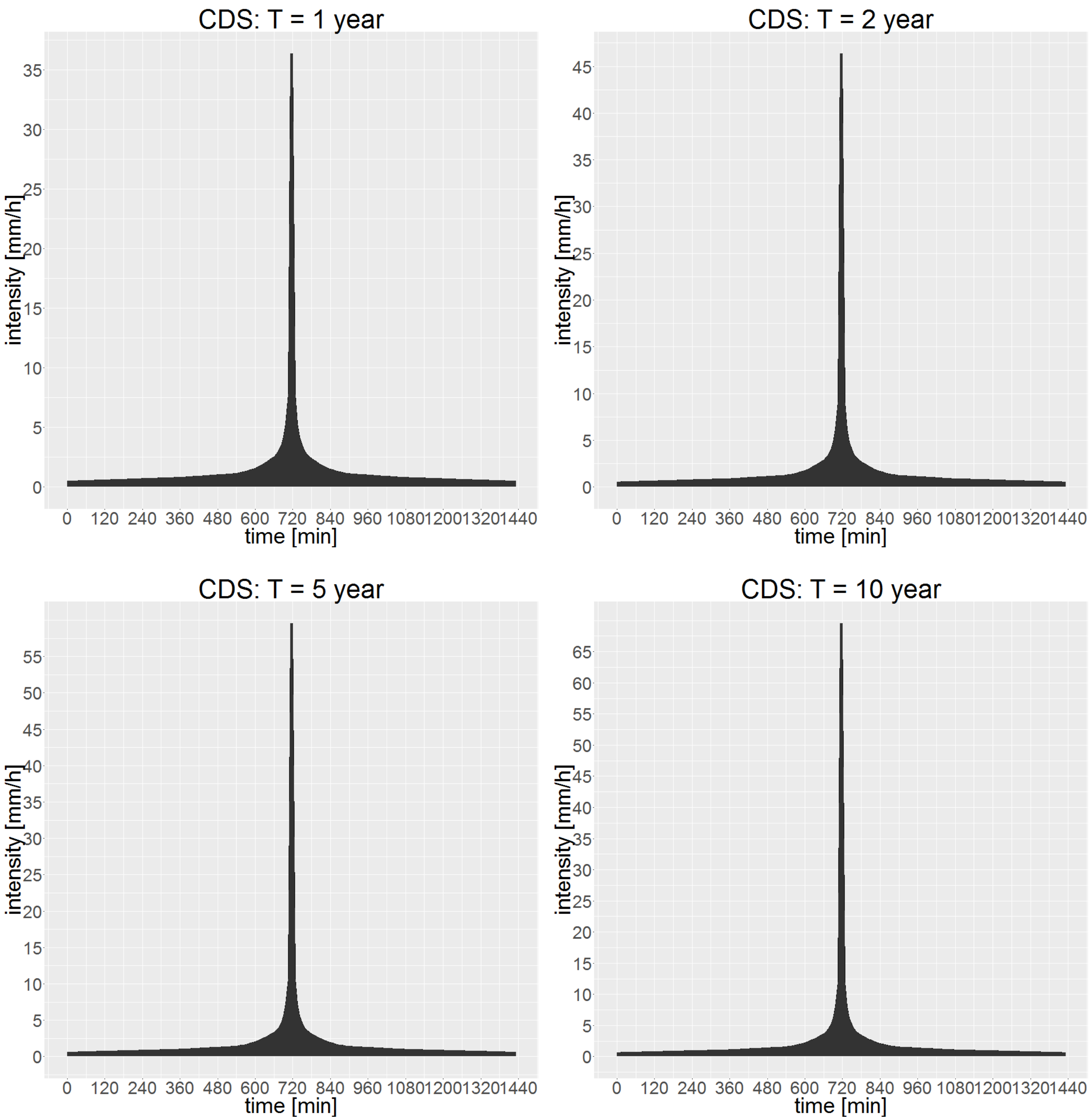
	Return period [years]							
Scenario	0.5	1	2	5	10	20	50	100
Mean	1.04	1.06	1.08	1.11	1.15	1.17	1.20	1.22
High	1.15	1.20	1.27	1.40	1.50	1.57	1.68	1.76

The climate factors for the return periods of 20, 50 and 100 years are not obtained in the research by Willems (2013), but deduced based on the logarithmic trend line. The choice is made to use the climate factors from the mean class to investigate the possible effects of climate change. The rainfall intensities of each time step from the original composite design storms are multiplied with the climate factor for the relevant return period. This results in a set of design storms that give an indication of the possible effects of climate change. It has to be noted that these factors only give a rough indication, due to the large uncertainties in the climate models. The storms are presented in Appendix IV.

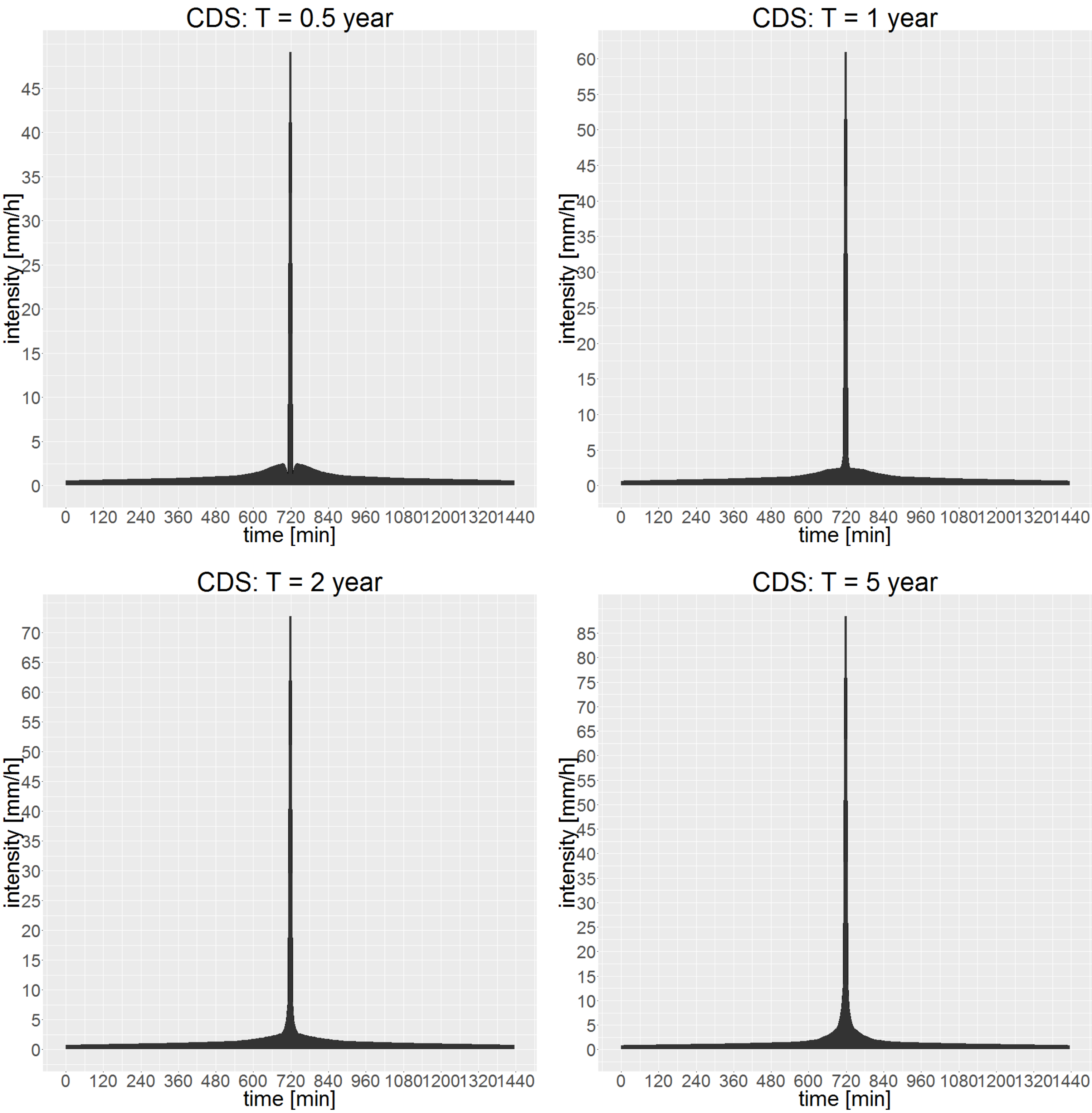
IV

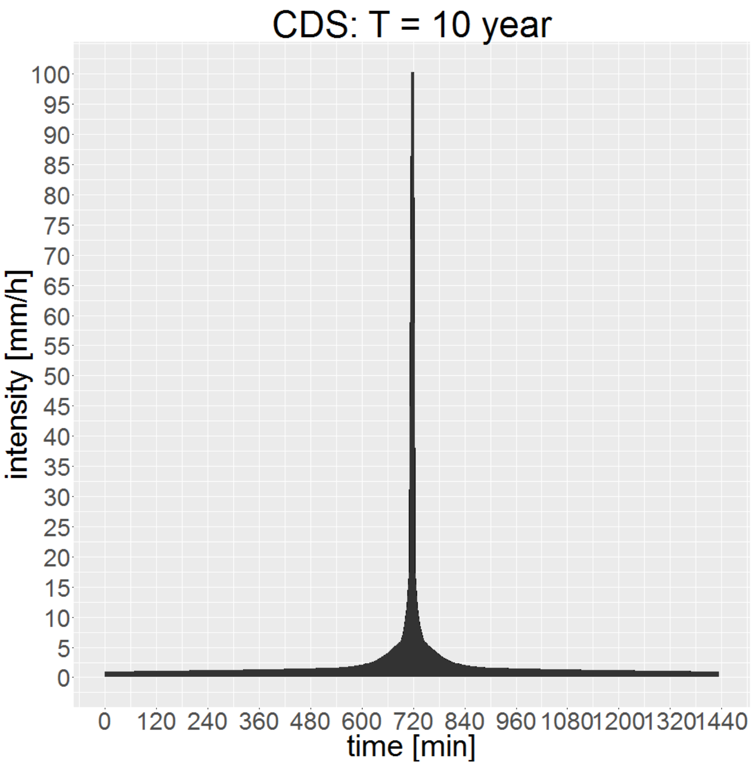
APPENDIX: PROFILES OF COMPOSITE DESIGN STORMS

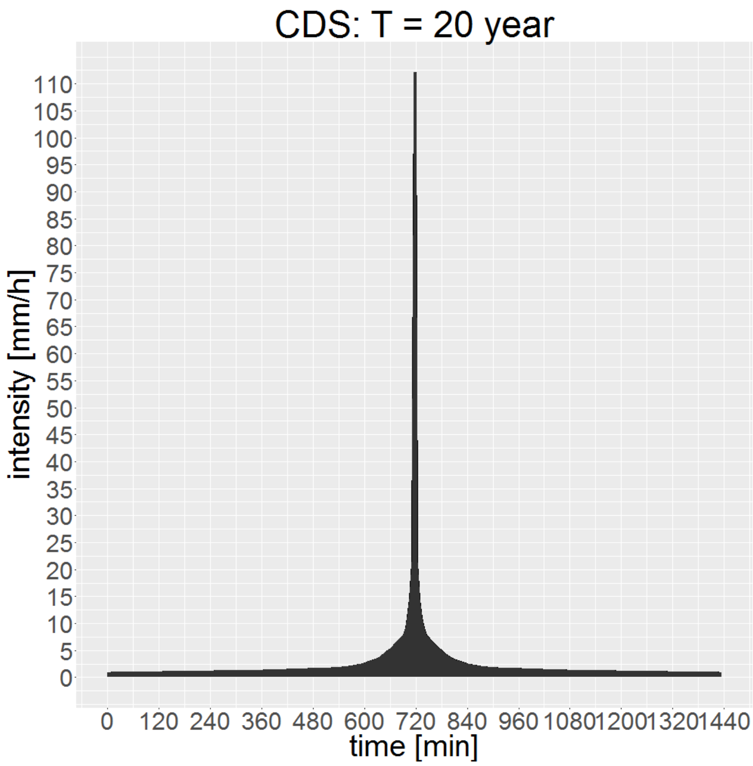
IV.1 Composite design storms based on De Bilt 1955-1979

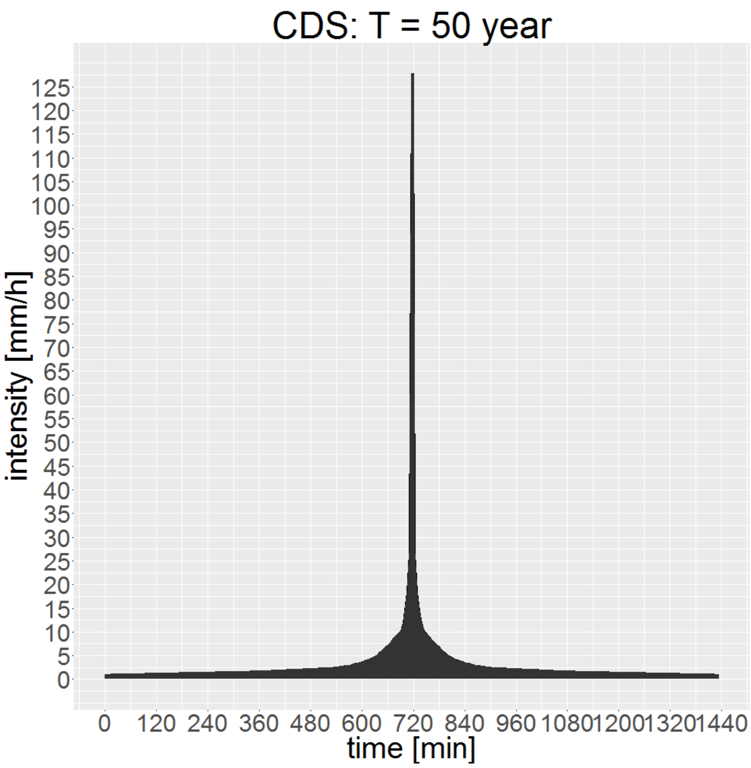


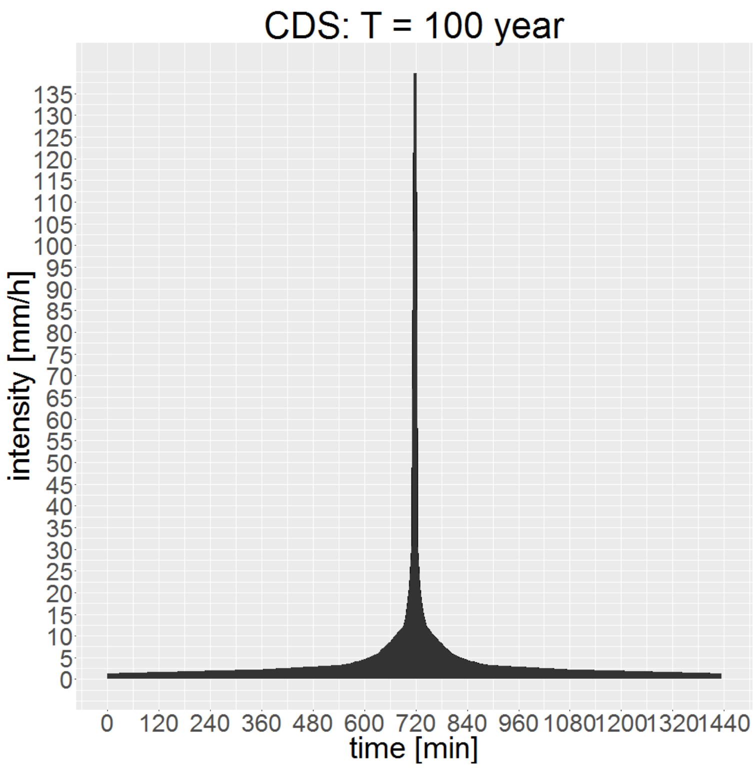
IV.2 Composite design storms based on Ukkel 1970-2007



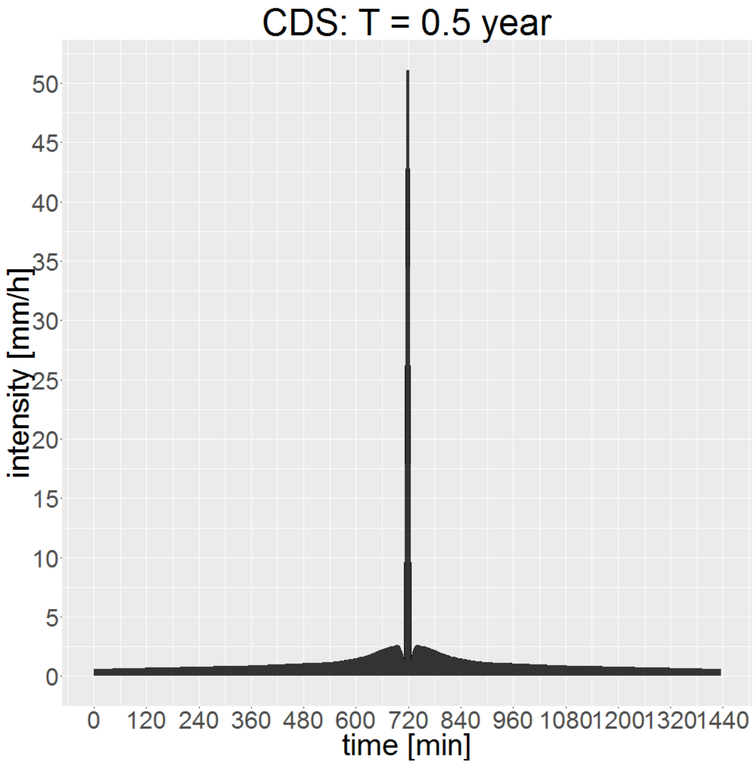

CDS: T = 10 year
intensity [mm/h]
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0
120
240
360
480
600
720
840
960
1080
1200
1320
1440
time [min]

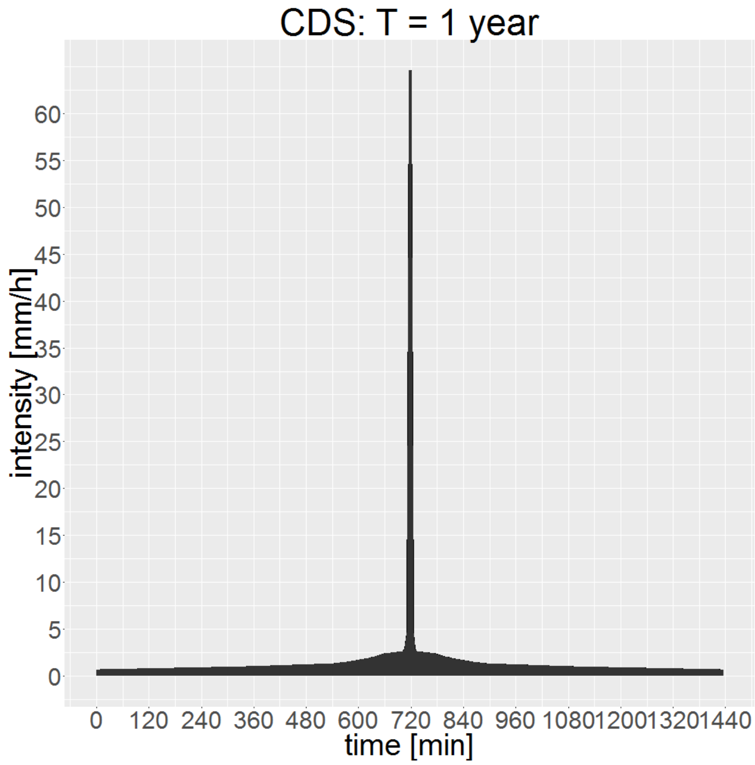

CDS: T = 20 year
intensity [mm/h]
110
105
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0
120
240
360
480
600
720
840
960
1080
1200
1320
1440
time [min]

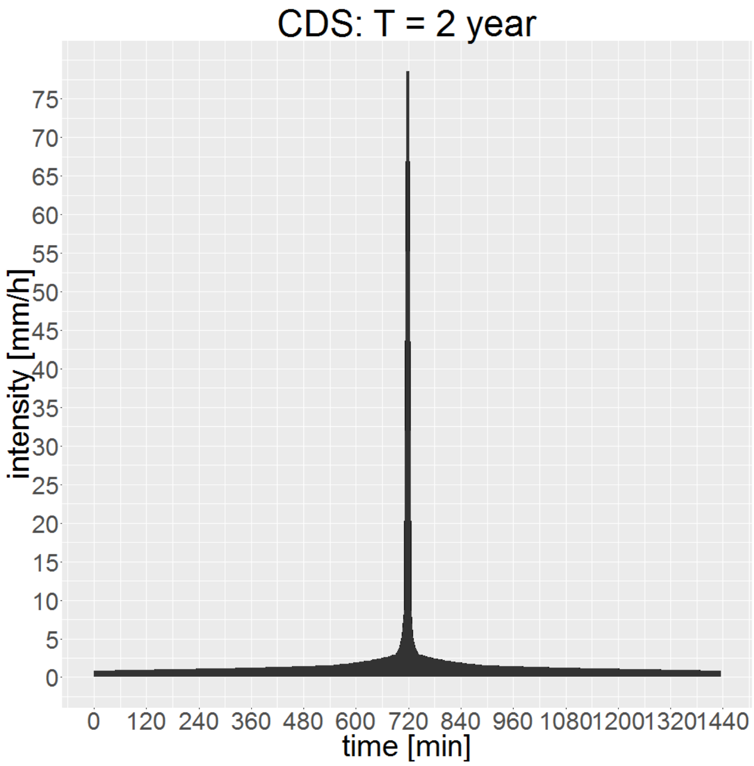

CDS: T = 50 year
intensity [mm/h]
125
120
115
110
105
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0
120
240
360
480
600
720
840
960
1080
1200
1320
1440
time [min]

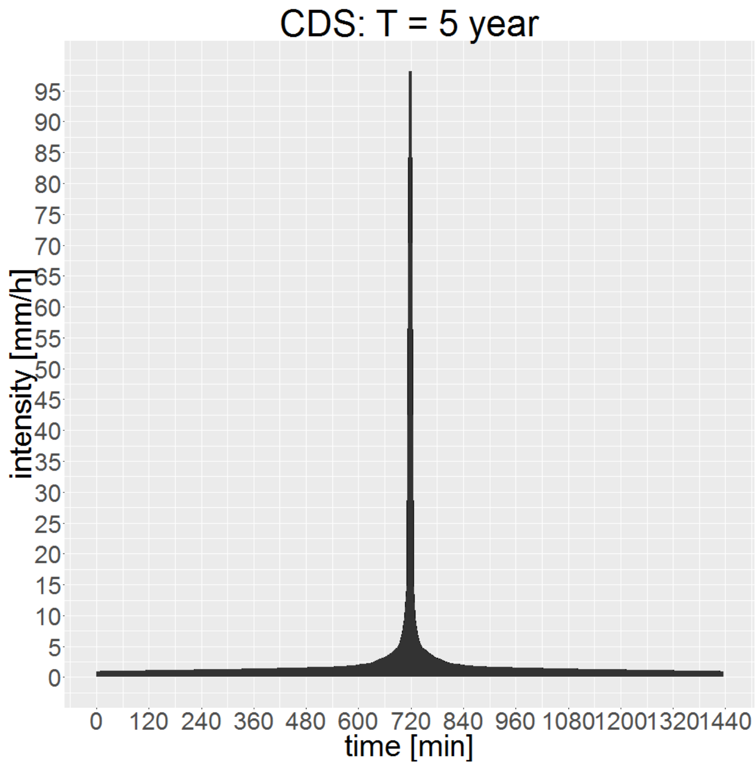

CDS: T = 100 year
intensity [mm/h]
135
130
125
120
115
110
105
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0
120
240
360
480
600
720
840
960
1080
1200
1320
1440
time [min]

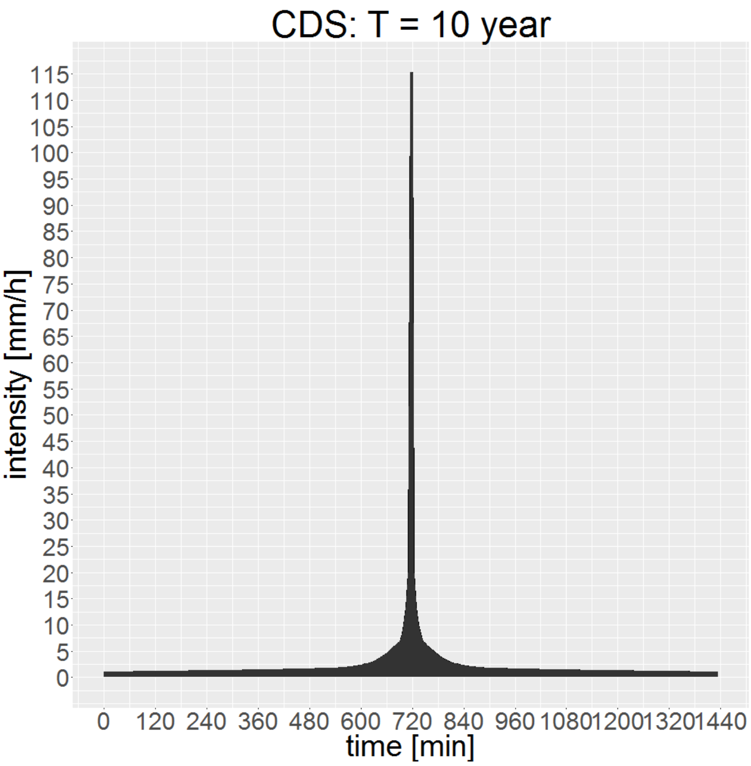
IV.3 Composite design storms based on Ukkel 1970-2007 (with climate change)

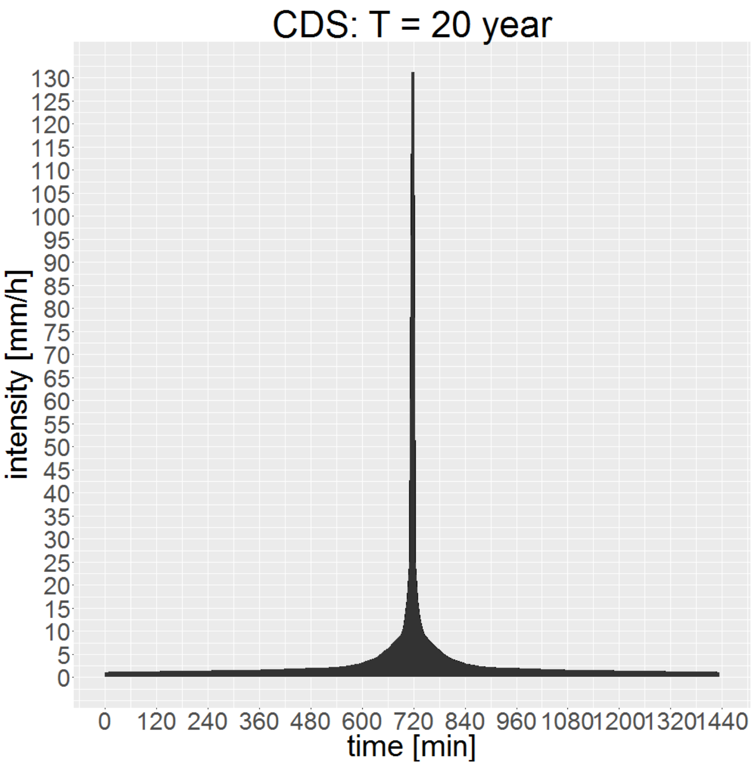


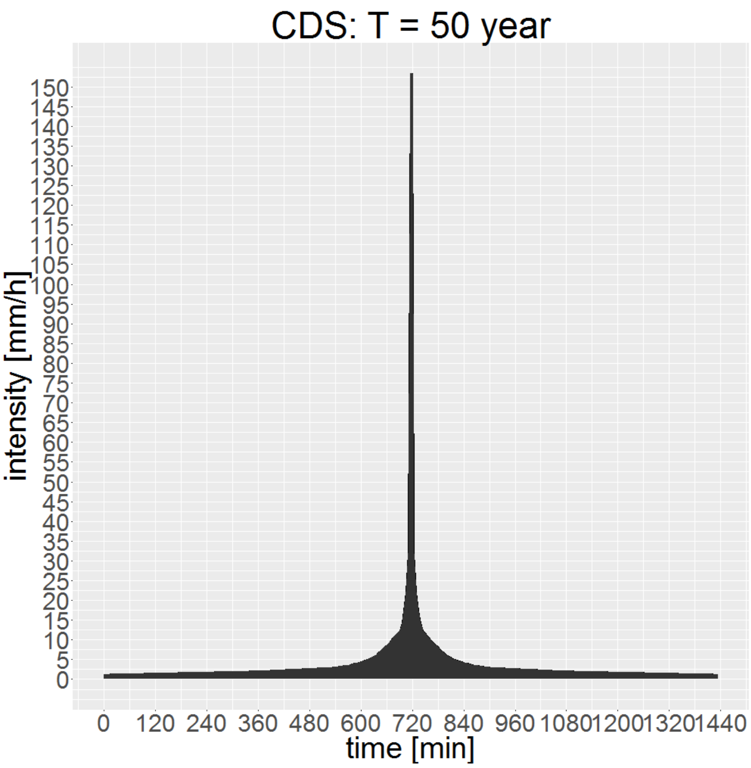


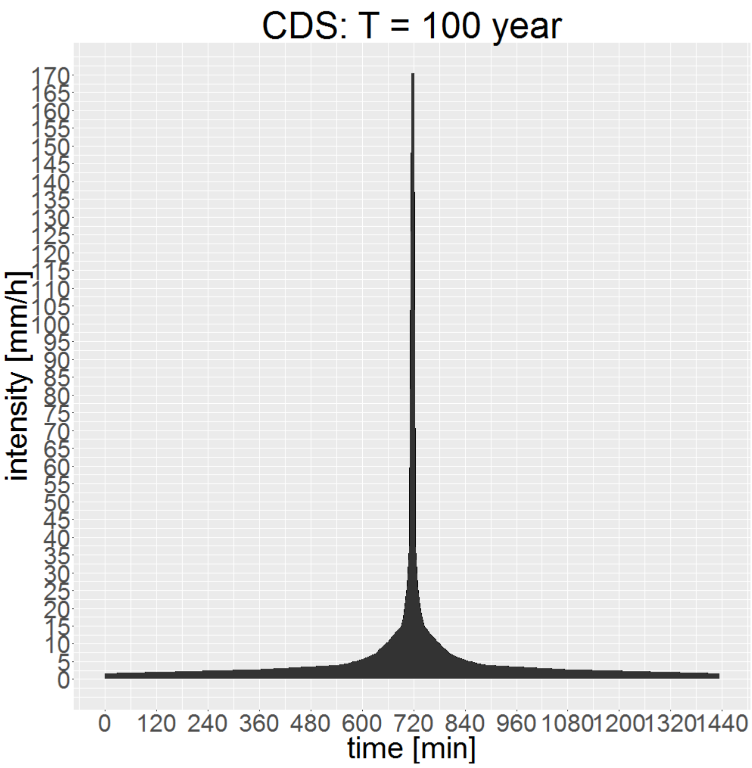





CDS: T = 10 year
intensity [mm/h]
time [min]


CDS: T = 20 year
intensity [mm/h]
time [min]


CDS: T = 50 year
intensity [mm/h]
time [min]


CDS: T = 100 year
intensity [mm/h]
time [min]

V

APPENDIX: INFORMATION TO SUPPORT ALLOCATION OF ECW'S

Effect	Possible consequences
Flooded buildings: houses	Requires removal of floodwater and humid with water vacuums and dehumidifiers. Requires cleaning of first floor. Possibly requires replacement of (part of) fixed inventory like furniture, kitchen cabinets, floor, etc.
Flooded buildings: basements	Requires removal of floodwater and humid by fire brigade and/or with water vacuums and dehumidifiers. Requires cleaning of the basement. Possibly requires replacement of (part of) fixed inventory, like the floor.
Flooded buildings: shops, offices and industries	Requires removal of floodwater and humid with water vacuums and dehumidifiers. Requires cleaning of the shop. Possibly requires replacement of (part of) fixed inventory like cabinets, floor, etc. Possibly requires replacement of store inventory, like food, electronic devices or other stock-in-trade.
Risk of casualties: flooded manholes	During flooding, people can drive/walk into flooded manholes, which are invisible due to floodwater. Example: During a heavy rainfall event, a lot of manholes where flooded in Oudenbosch. Three persons got seriously injured as consequence of an accident caused by a flooded manhole. The driver lost control of the car after she drove over the flooded manhole, which was invisible. ("Drie mensen gewond bij ongeluk in Oudenbosch," 2007) The change to get such an accident is very low, the consequences can however be quite severe.
Risk of casualties: flooded tunnels	During heavy rainfall, tunnels can get flooded with high water depths. For drivers it is difficult to estimate the depth of the water in the tunnel. Therefore, it could occur that a car drives into a tunnel and gets stuck in the tunnel, resulting in dangerous situations.
Risk of infection	People can get infected by the contaminated sewer water. For a combined sewer the change of infection for adults is approximately 4 % and for children 33 %. The infection can lead to gastrointestinal, respiratory illness, or other complaints like earache, itch, etc.
Traffic disruption: distributor roads and access roads	Due to flooding of the road, it is blocked for traffic, resulting in traffic delay. The amount of traffic delay will be larger for main roads than for streets, as alternative routes will be longer and/or traffic intensities are higher. A distinction is made between distributor roads and access roads. Distributor roads are more important roads and mainly used by through traffic. Access roads are mainly used by destination traffic.
Flooding of public space	Flooding of specific locations can lead to nuisance, think of flooding of the railway station or bus stations. For which location an effect category weight is obtained, is decided by the municipality.

VI

APPENDIX: EXPEDIENCY OF MEASURES

The last aspect that has to be elaborated to complete the method is the way in which the expediency of measures is determined. In the Proeftuin-method risk scores were allocated to each risk level in order to judge the expediency based on the reduction in risk score divided by the investment costs. The same principles can be applied to the risk matrix in table 14, which would result in the risk scores as presented in table VI.1.

Table VI.1: Risk scores per risk level

Risk level	Risk score	Risk level	Risk score	Risk level	Risk score	Risk level	Risk score
Negligible	0	Very low	0.01	Low	0.1	Moderate	1
High	10	Very high	100	Extremely high	1,000		

The advantage of this approach is that measures that reduce high risk levels will have a larger reduction in risk score and thus a higher expediency if the investment costs are equal. The disadvantage of this method is that the border between the different risk levels can lead to very different results for comparable situations. For example, a reduction from a severity score of 5.1 to 0.9 for a return period of 10 years would reduce the risk score by 99.9, whereas a reduction from a severity score of 4.9 to 0.9 for the same return period would reduce the risk score by only 9.9. This reduction is 10 times lower, although the reduction in severity score is comparable. To eliminate this problem, another approach, based on the reduction in expected annual severity score, is proposed.

The reduction in expected annual severity score can be calculated once the effects for all return periods with and without the measure are determined. The expediency of the measure can then be expressed by the reduction in expected annual severity score divided by the investment costs, in formula form:

$$Expediency = \frac{EASS_{cuurent} - EASS_{measure}}{\frac{C_i}{100{,}000}} \quad (VI.1)$$

where:

$$EASS_{current} = the\ expected\ annual\ severity\ score\ in\ the\ current\ situation,$$
$$EASS_{measure} = the\ expected\ annual\ severity\ score\ after\ implementing\ the\ measure,$$
$$C_i = the\ investment\ costs\ of\ the\ measure.$$

Using this approach, measures that reduce high risk levels, will still have a higher expediency if the investment costs are equal, while the problem with the border between the risk levels is eliminated. Therefore, this method is used to determine the expediency of measures.

In the proposed method and the Proeftuin-method, the following aspects are not taken into account:

- cost of maintenance;
- side effects, like changing combined sewer overflow volume or flexibility of the measure.

Cost of maintenance

Because the cost of maintenance is not incorporated in the current determination of the expediency of measures, a measure with low investment costs, but very high maintenance costs is said to be more expedient than a measure with somewhat higher investment costs, but much lower maintenance costs. This gives a distorted image of the reality. In order to account for the cost of maintenance, the following adjustment to formula VI.1 is proposed:

$$Expediency = \frac{(EASS_{cuurent} - EASS_{measure}) * LT_{measure}}{\frac{C_i + C_m * LT_{measure}}{100{,}000}} \quad (VI.2)$$

where:

$$LT_{measure} = the\ expected\ lifetime\ of\ the\ measure\ in\ years,$$
$$C_m = the\ annual\ cost\ of\ maintenance\ of\ the\ measure.$$

Using this formula, some additional information is required about the measures, namely their expected lifetime and the annual cost of maintenance of the measure. When this information is known, this approach will give a more complete image of the expediency of the measures.

Side effects

Besides the effects as considered in this report, measures will have other effects, like a change in combined sewer overflow volume or volume treated by the wastewater treatment plant (WWTP). Besides that, measures will differ in characteristics like flexibility and degree of nuisance during the construction. A possible way of incorporating these effects is using a multi criteria analysis. This will however add additional uncertainty as the weights given to the criteria will be subjective and can differ even within a municipality. For example, a reduction in combined sewer overflow volume on a small water body will come with more positive effects than the same reduction in spilled volume on a large water body. Therefore, it is decided to not use a general multi criteria analysis, but to present the relevant side effects for each location and let the municipality compare the different measures based on the calculated expediency and the additional information about the side effects.

Cost-Benefit ratio

In order to get a rough estimate if the investments required for the measure are justifiable, a cost-benefit ratio is calculated:

$$\text{Cost} - \text{Benefit ratio} \ = \frac{(EADD_{cuurent} - EADD_{measure}) * LT_{measure}}{(C_i + C_m * LT_{measure})} \quad (\text{VI}.3)$$

where:

$$EADD_{current} = the\ expected\ annual\ direct\ damage\ in\ the\ current\ situation$$
$$EADD_{measure} = the\ expected\ annual\ direct\ damage\ after\ implementing\ the\ measure$$

The direct damage for each of the return periods taken into consideration is calculated based on the number of houses, basements and shops flooded. The unit costs for flooding of each category are based on the WaterSchadeSchatter developed by STOWA (2013) and are given in table VI.2. The maximum and minimum values are used, as the actual damage is very uncertain.

Table VI.2: Unit costs flooded buildings

Category	Damage per m^2 min - max values (STOWA, 2013)	Average surface area	Costs per flooded object
Houses	€ 155 - € 365	50 m^2	€ 7,750 - € 18,250
Basements	€ 25 - € 75	50 m^2	€ 1,250 - € 3,750
Shops	€ 200 - € 600	250 m^2	€ 50,000 - € 150,000

The expected annual direct damage is calculated analogous to the expected annual severity score:

$$EADD_{cuurent} = 0.5 * \sum_{i=1}^{n-1} \left(\frac{1}{T_i} - \frac{1}{T_{i+1}}\right) * (DD_i + DD_{i+1}) \quad (VI.4)$$

where:

$$n = 6 = number\ of\ return\ periods\ taken\ into\ account$$
$$T_i = i^{th}\ return\ period\ taken\ into\ account$$
$$T_{i+1} = i + 1^{th}\ return\ period\ taken\ into\ account$$
$$DD_i = direct\ damage\ of\ i^{th}\ return\ period\ taken\ into\ account$$
$$DD_{i+1} = direct\ damage\ of\ i + 1^{th}\ return\ period\ taken\ into\ account$$

The procedure will result in a range of the cost-benefit ratio. If the ratio is larger than 1, the reduction in expected direct damage over the lifetime of the measure is higher than the total costs of the measure, thus the investment is justifiable. It has to be noted that the ratio only gives a very rough indication, so if the ratio is around 1, conclusions have to be drawn with certain caution.

As the cost-benefit ratio is based only on the damage caused by flooding of buildings, the remaining effects, risk of casualties, risk of infection and traffic disruption, are left out of consideration. In order to also incorporate these effects, the expected annual remaining severity score is calculated based on the severity score of only these effects.

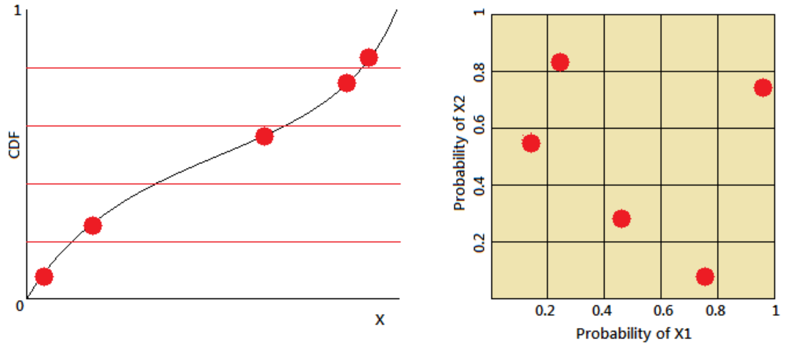
VII

APPENDIX: ADDITIONAL INFORMATION MODEL UNCERTAINTY

VII.1 Additional information Latin Hypercube Sampling

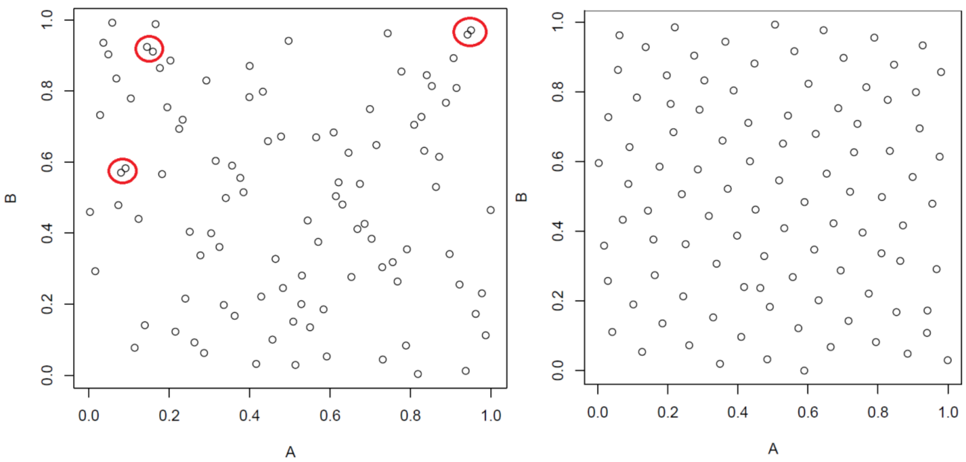
To reduce the amount of runs that is required to get stable results for both the mean and standard deviation, Latin Hypercube Sampling, instead of simple Monte Carlo sampling is used. Latin Hypercube Sampling divides the cumulative distribution function (CDF) of each parameter taken into consideration into n equal partitions, where n is the number of runs used. The algorithm facilitates that from each partition a value is randomly sampled (McKay et al., 1979). If more parameters are included, first for each parameter the CDF is partitioned into n parts, after which from each partition a value is randomly sampled. The obtained values from the separate variables are combined randomly, resulting in n sets of sampled parameter values. Due to this characteristic of the algorithm, a lower amount of runs is required to get stable results for the mean and standard deviation of the results. A schematization of 1D and 2D Latin Hypercube Sampling is shown in figure VII.1. The same principles can be applied to a problem with any amount of dimensions.

Figure VII.1: Schematization Latin Hypercube Sampling



Latin Hypercube Sampling can be done random, as described above. This can however result in clustering of points, because they are chosen close to the border. An example is given in the left plot in figure VII.2, where the points within the red circles are clustered. In order to obtain a more equally spread distribution of the sampling points, Beachkofski and Grandhi (2002) developed an algorithm, called Improved Distributed Hypercube Sampling, that constructs a Latin Hypercube in which the smallest distance between two sampling points is as large as possible. This results in a Latin Hypercube which is much more equally spread, see the right plot in figure VII.2. Improved Distributed Hypercube Sampling will be applied in this research to obtain an equally spread Latin Hypercube Sample. As Latin Hypercube Sampling gives more information with a reduced amount number of runs, 200 runs are used.

Figure VII.2: Difference between random (left) and improved (right) Latin Hypercube Sampling



In order to get parameter values, the values of the Latin Hypercube Sample are read from the CDF, as schematized in the left graph of figure VII.1. As mentioned, the variation in parameters is assumed to be Gaussian, so a normal distribution with mean μ and standard deviation σ is applied to all parameters.

VII.2 Formulas to calculate parameters for model uncertainty

The mean, standard deviation, coefficient of variation, skewness and kurtosis are calculated using the following formula's (Everitt & Skrondal, 2010):

$$mean = \bar{x} = \frac{\sum_{i=1}^{n} x_i}{n} \qquad (VII.1)$$

where:

$$x_i = the\ total\ flooded\ area\ during\ run\ i$$
$$n = the\ total\ number\ of\ runs = 200$$

$$standard\ deviation = s = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(x_i - \bar{x})^2} \qquad (VII.2)$$

$$coefficient\ of\ variation = CV = \frac{\bar{x}}{s} \qquad (VII.3)$$

$$skewness = G_1 = \frac{n^2}{(n-1)(n-2)} * \left(\frac{\frac{1}{n}\sum_{i=1}^{n}(x_i - \bar{x})^3}{\left(\frac{1}{n-1}\sum_{i=1}^{n}(x_i - \bar{x})^2\right)^{\frac{3}{2}}} \right) \qquad (VII.4)$$

$$kurtosis = G_2 = \frac{(n+1)n}{(n-1)(n-2)(n-3)} \frac{\sum_{i=1}^{n}(x_i - \bar{x})^4}{\left(\sum_{i=1}^{n}(x_i - \bar{x})^2\right)^2} - 3\frac{(n-1)^2}{(n-2)(n-3)} \qquad (VII.5)$$

The amount of explained variance is calculated with the following formula (Ezekiel, 1930):

$$R^2_{adjusted} = 1 - \left(1 - \frac{\sum_{i=1}^{n}(f_i - \bar{x})^2}{\sum_{i=1}^{n}(x_i - \bar{x})^2}\right)\frac{n-1}{n-p} \qquad (VII.6)$$

where:

$$f_i = the\ modelled\ total\ flooded\ area\ during\ run\ i$$
$$p = the\ number\ of\ explaining\ variables = 11$$

The correlation between the explaining variables is calculated using the following formula (Everitt & Skrondal, 2010):

$$correlation = r = \frac{\sum_{i=1}^{n}(x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_{i=1}^{n}(x_i - \bar{x})^2(y_i - \bar{y})^2}} \qquad (VII.7)$$

where:

$$x_i = the\ i^{th}\ value\ of\ explaining\ variable\ x$$
$$\bar{x} = the\ average\ value\ of\ explaining\ variable\ x$$
$$y_i = the\ i^{th}\ value\ of\ explaining\ variable\ y$$
$$\bar{y} = the\ average\ value\ of\ explaining\ variable\ y$$

VII.3 Correlation between explaining variables

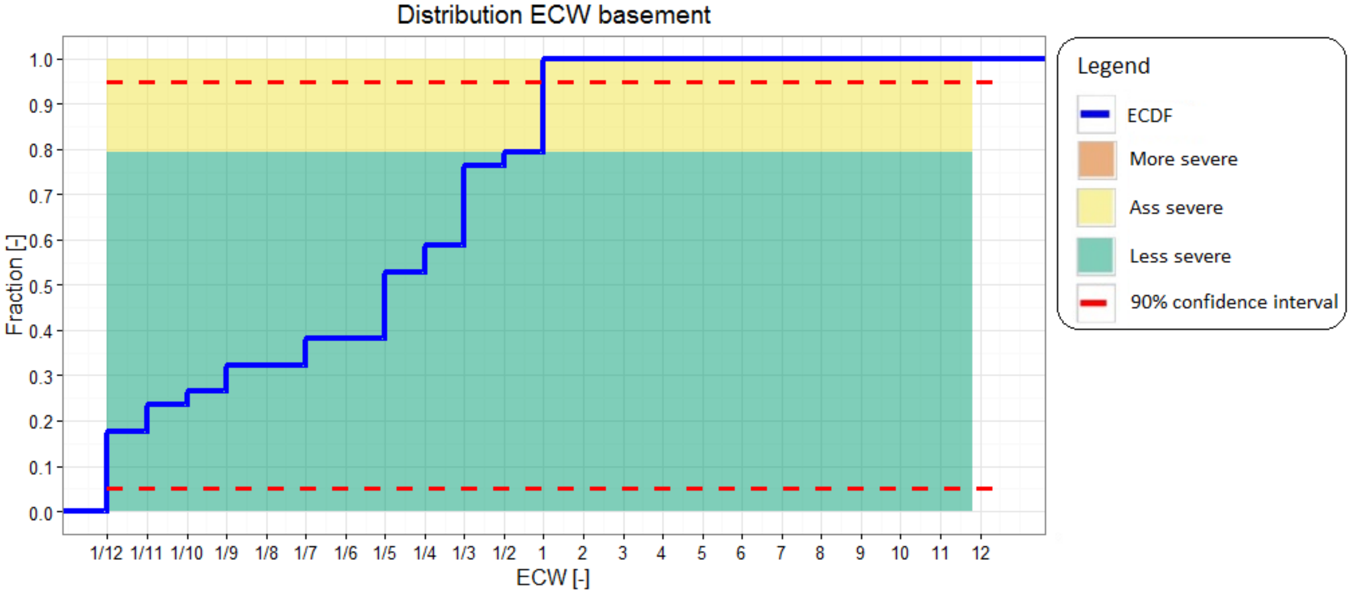
Table VII.1: Correlation between explaining variables

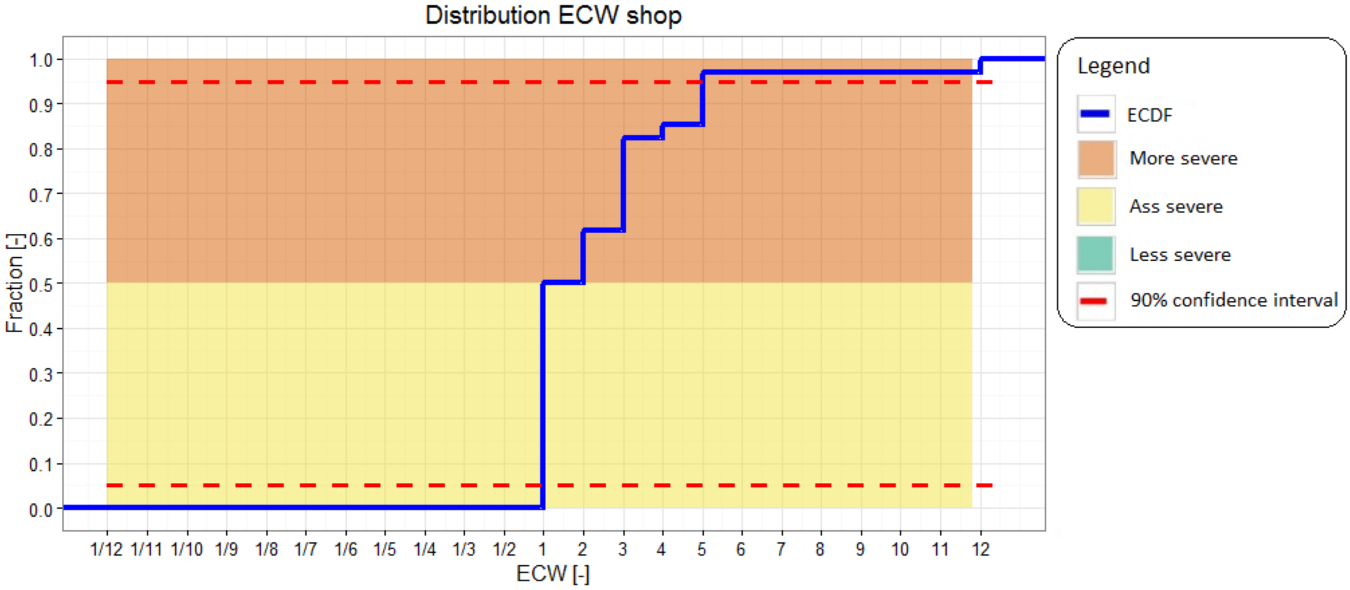
Variable	$c_{l;f}$	$c_{l;s}$	$b_{f;p}$	$b_{f;r}$	i	A	K_a	c	Q_{pump}	c_m	n_m
$c_{l;f}$	1										
$c_{l;s}$	0.03	1									
$b_{f;p}$	-0.03	0.08	1								
$b_{f;r}$	-0.05	0.11	-0.28	1							
i	0.20	-0.01	0.05	-0.19	1						
A	0.03	0.05	0.07	0.02	-0.10	1					
K_a	-0.03	0.00	-0.08	0.04	0.18	-0.01	1				
c	-0.09	0.23	0.39	-0.18	0.09	0.14	0.05	1			
Q_{pump}	0.07	0.02	-0.01	0.11	-0.03	0.12	0.06	0.08	1		
c_m	0.16	0.29	-0.17	-0.14	0.01	0.09	0.18	0.11	-0.09	1	
n_m	0.01	0.17	0.41	-0.26	0.06	-0.01	-0.02	0.25	-0.12	0.13	1

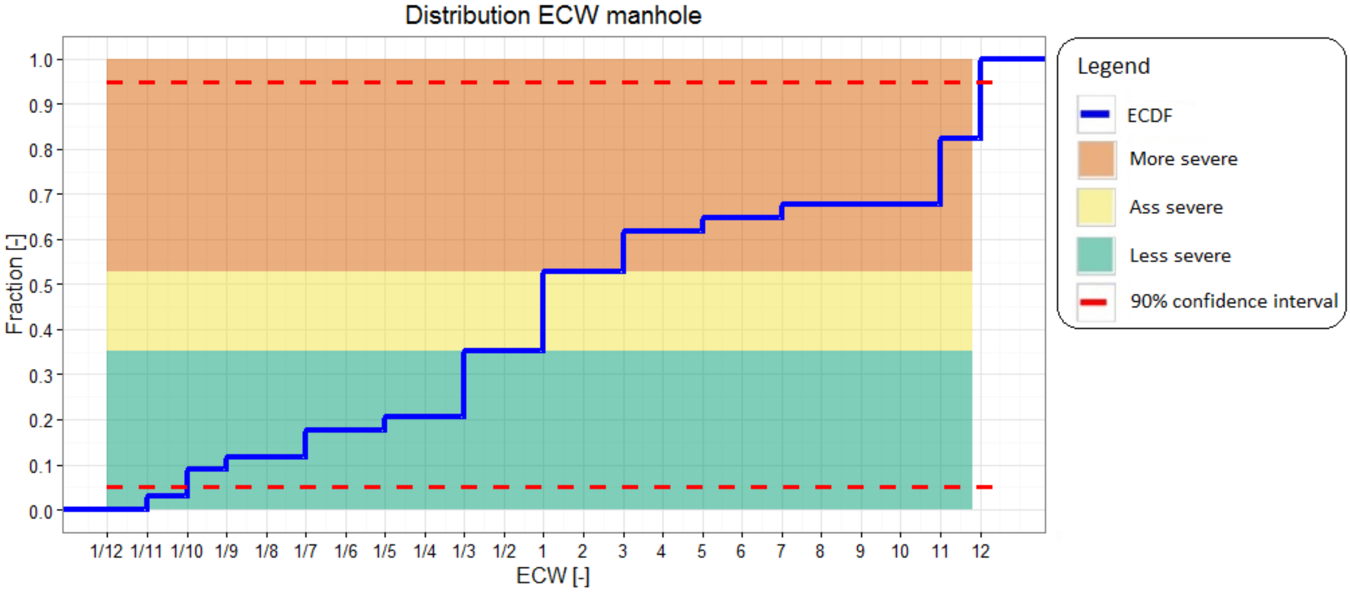
VIII

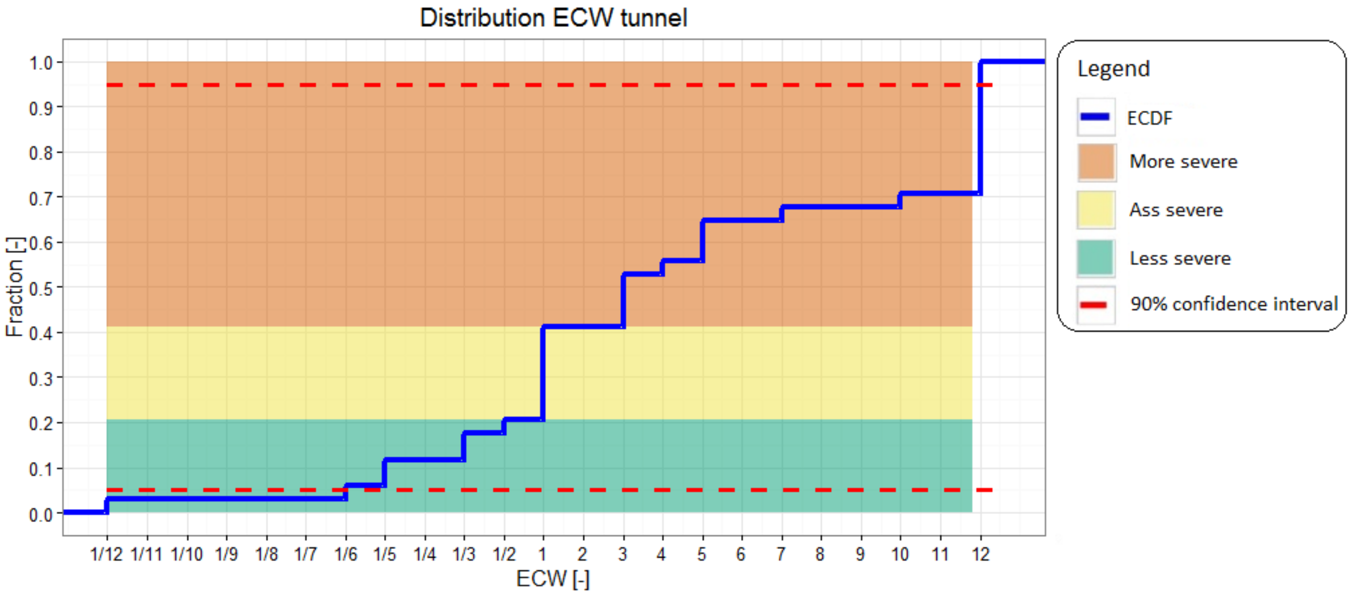
APPENDIX: DISTRIBUTION OF EFFECT CATEGORY WEIGHTS QUESTIONNAIRE

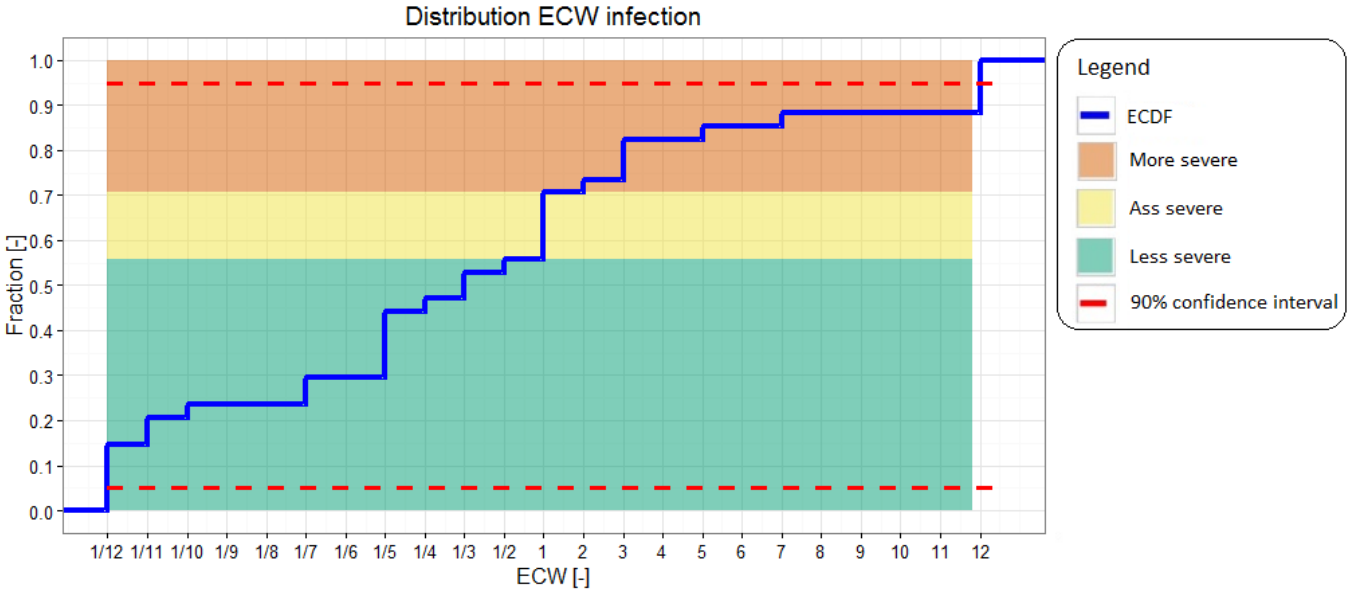
VIII.1 Distribution of all respondents (n = 34)

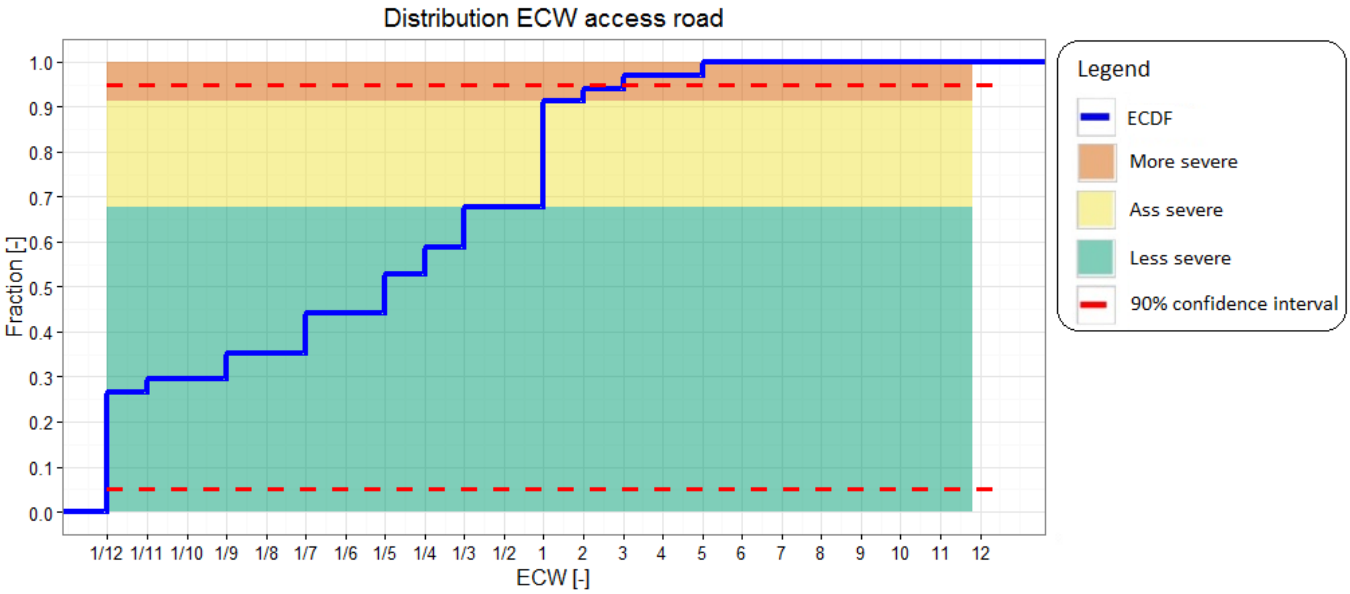


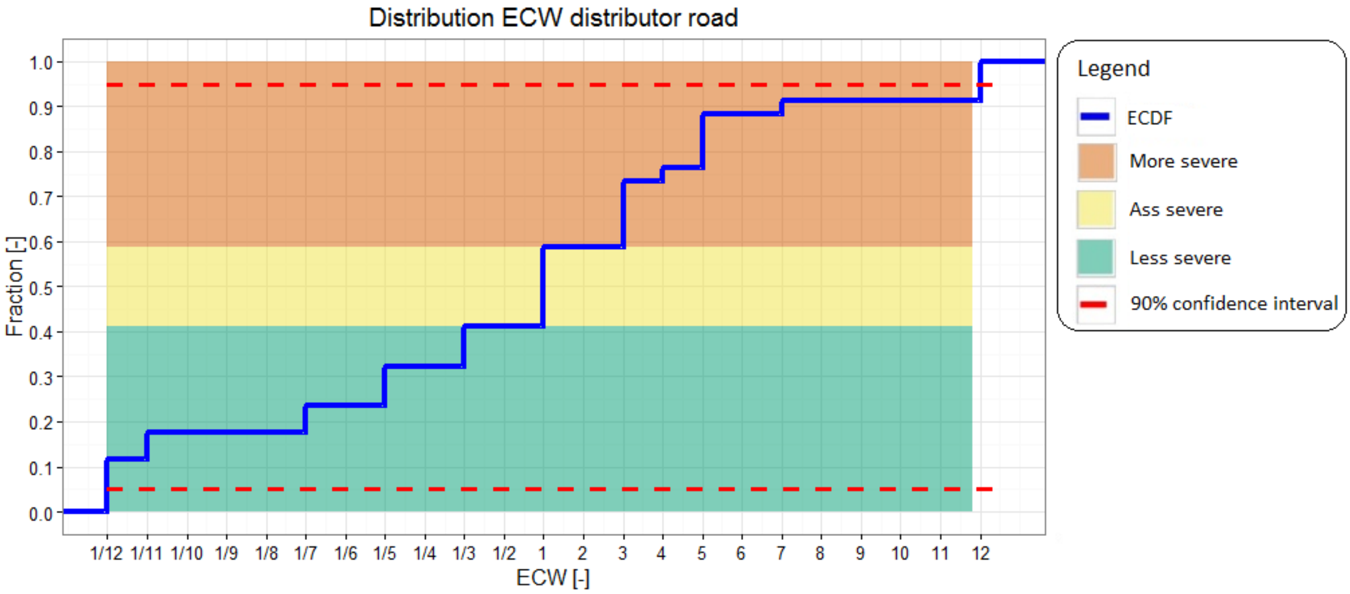




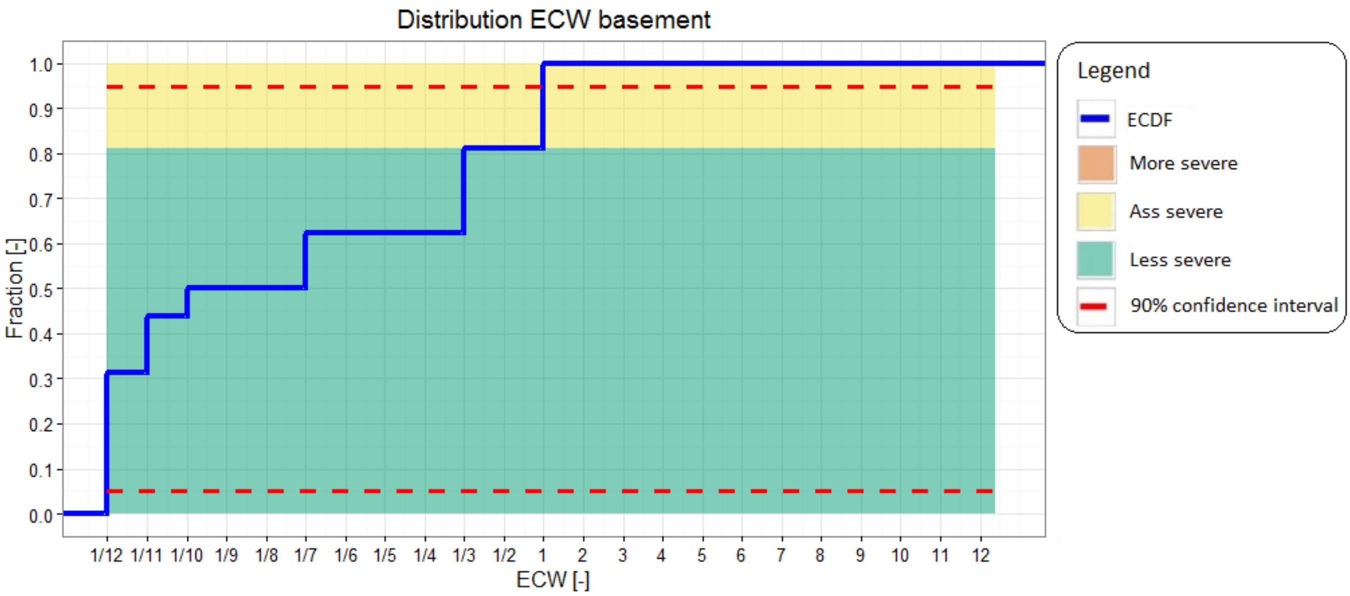

Distribution ECW tunnel
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

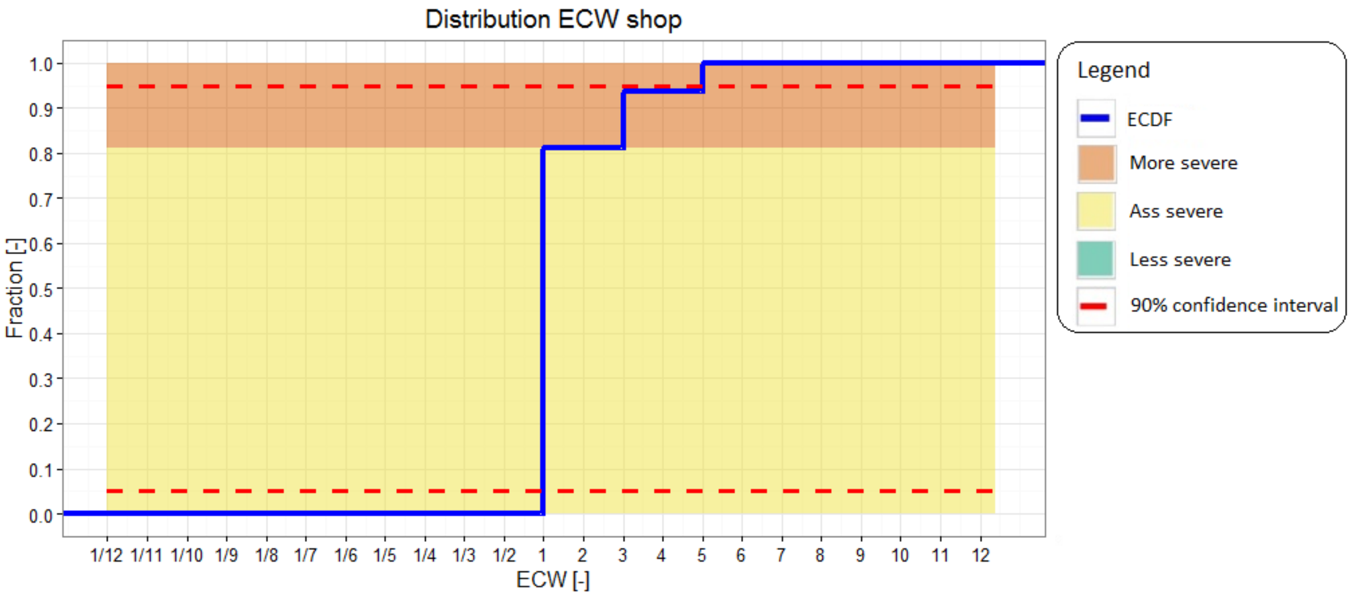

Distribution ECW infection
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

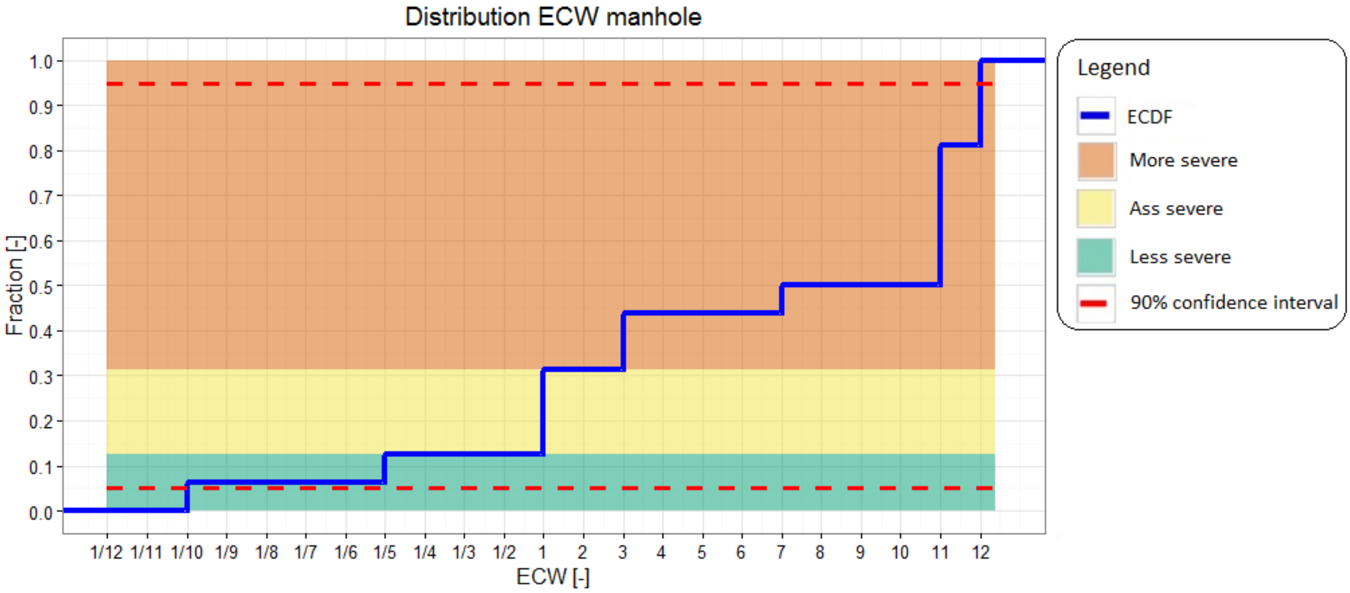

Distribution ECW access road
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

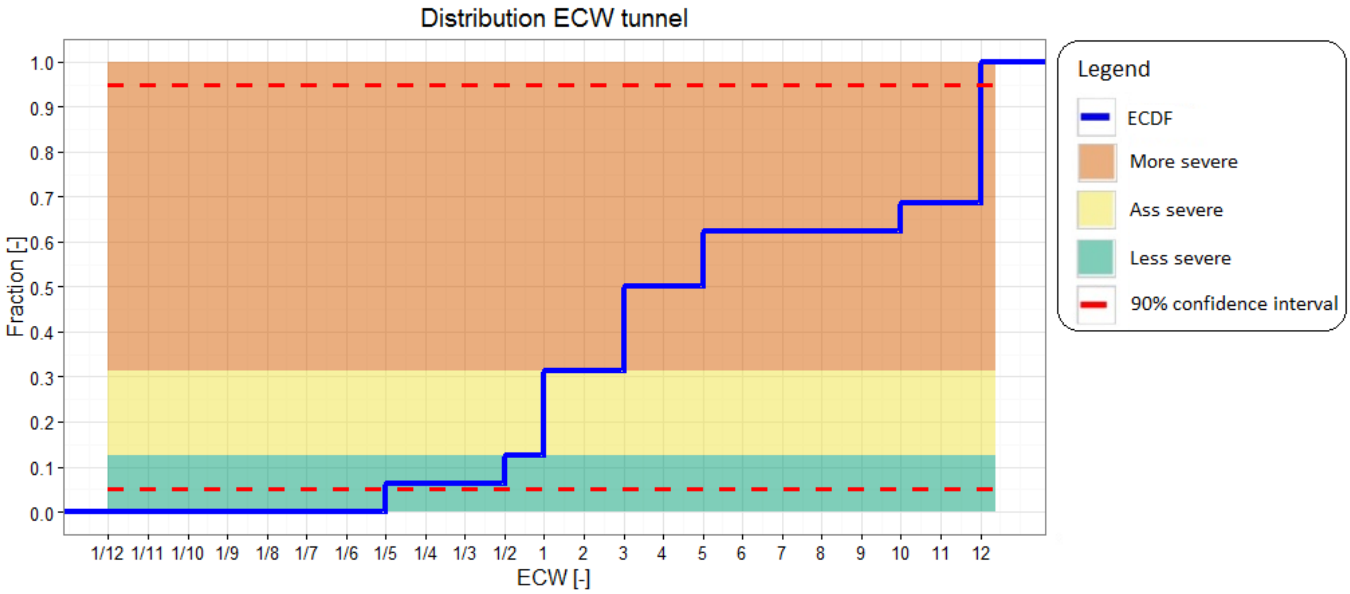


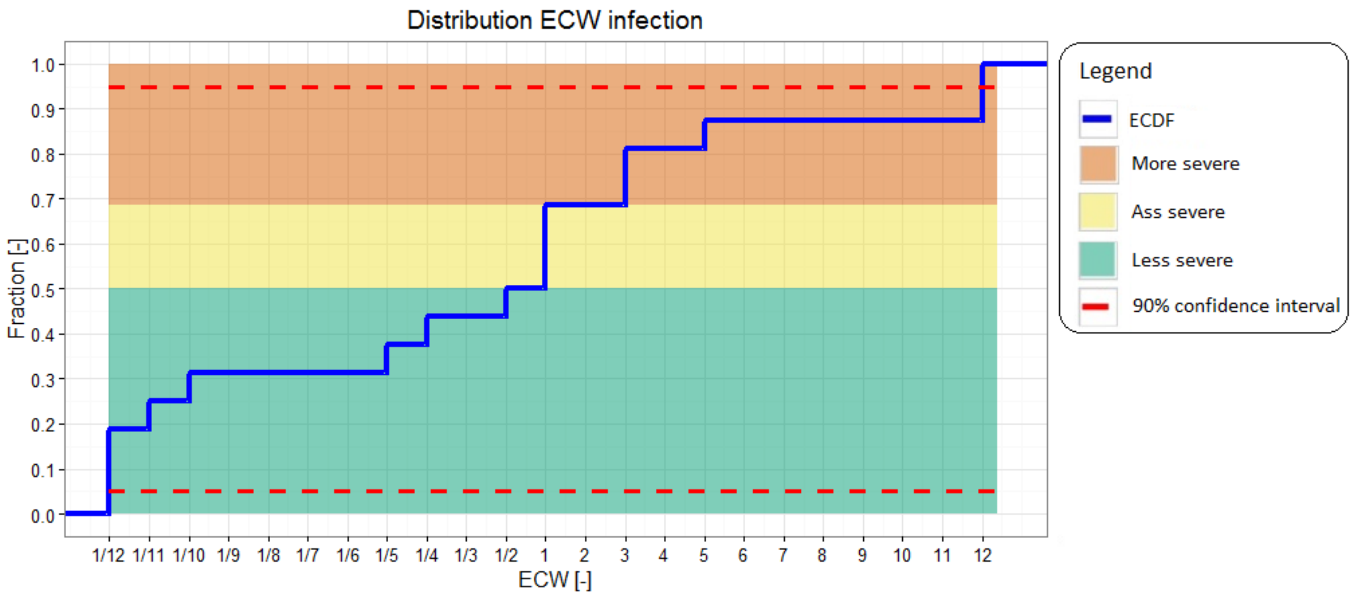
VIII.2 Distribution policy makers (n = 16)

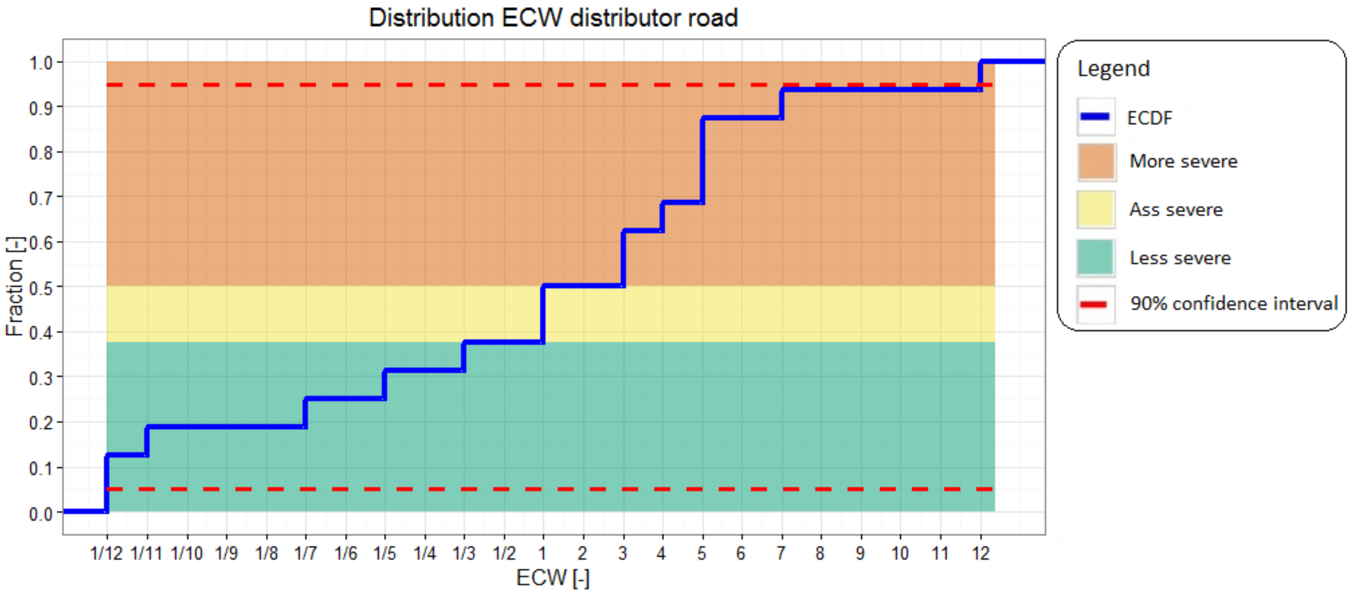


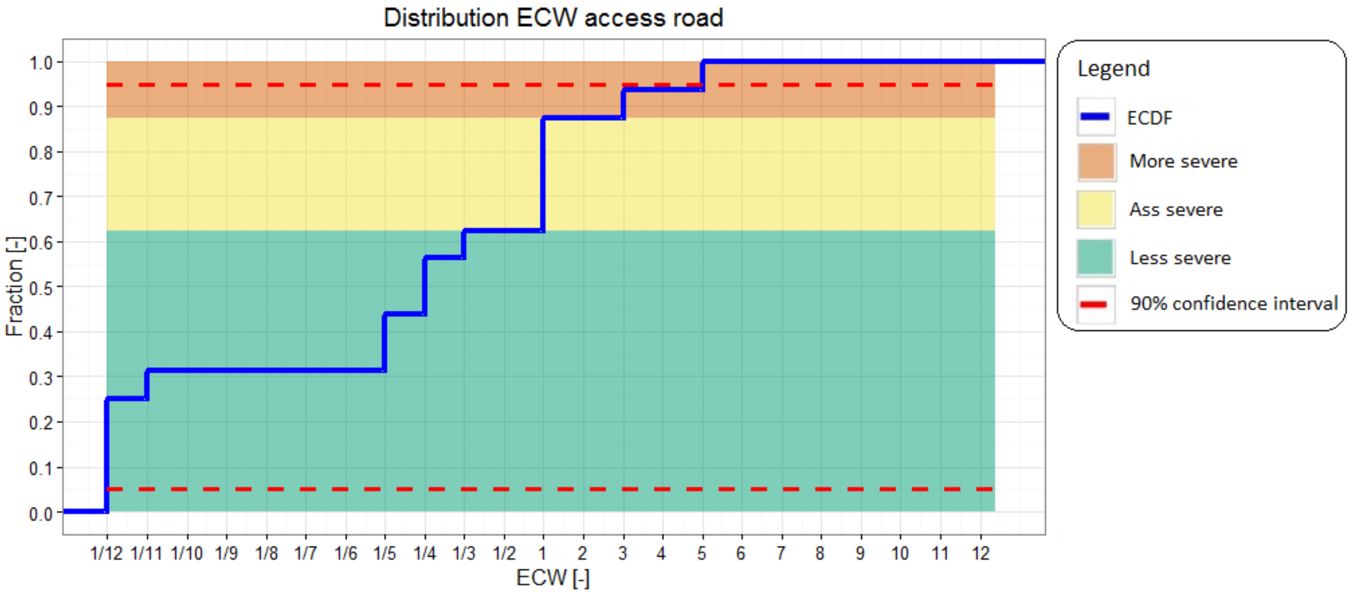



Distribution ECW manhole
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

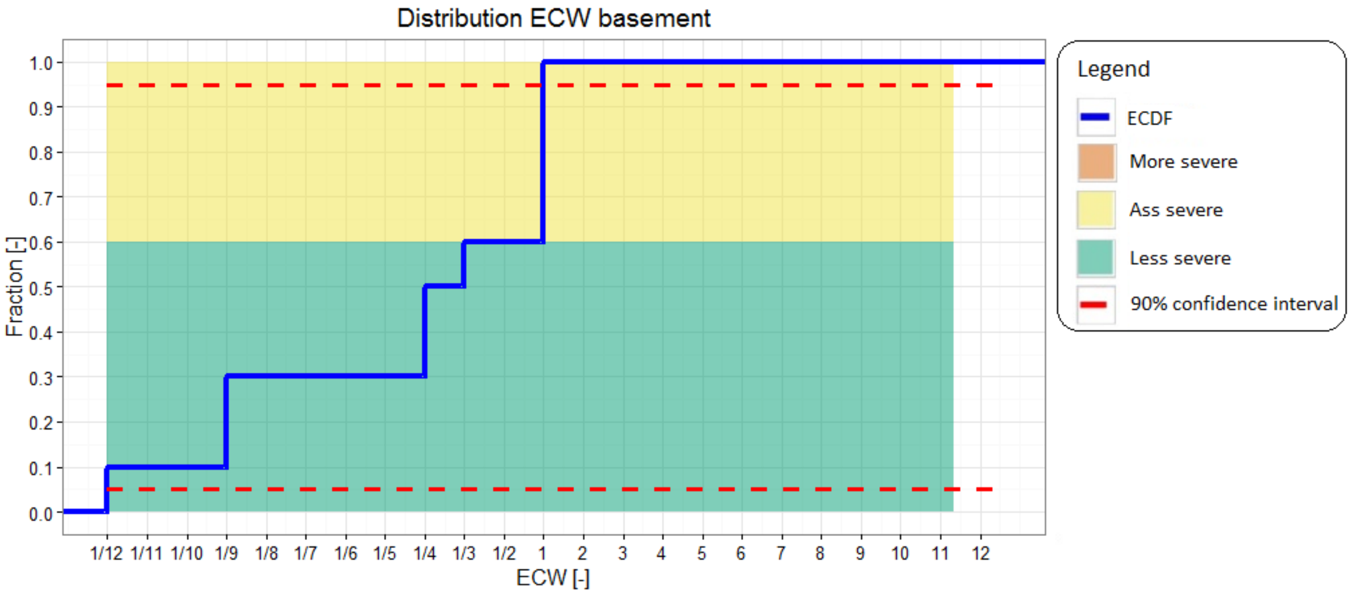

Distribution ECW tunnel
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

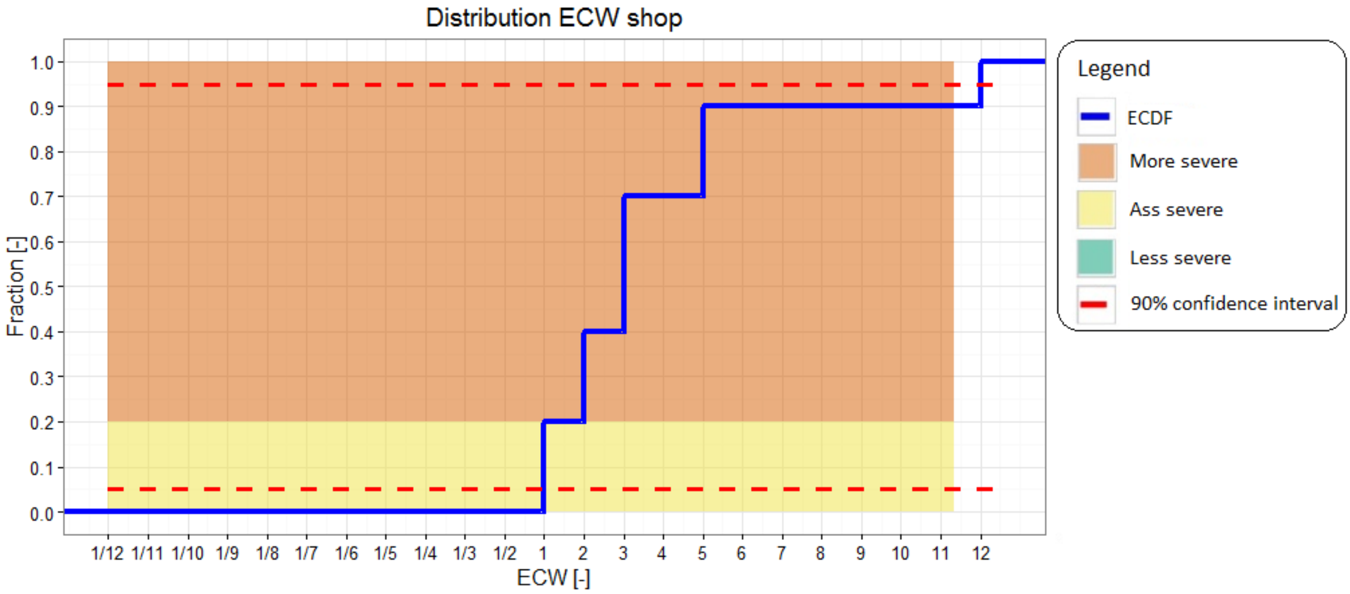

Distribution ECW infection
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

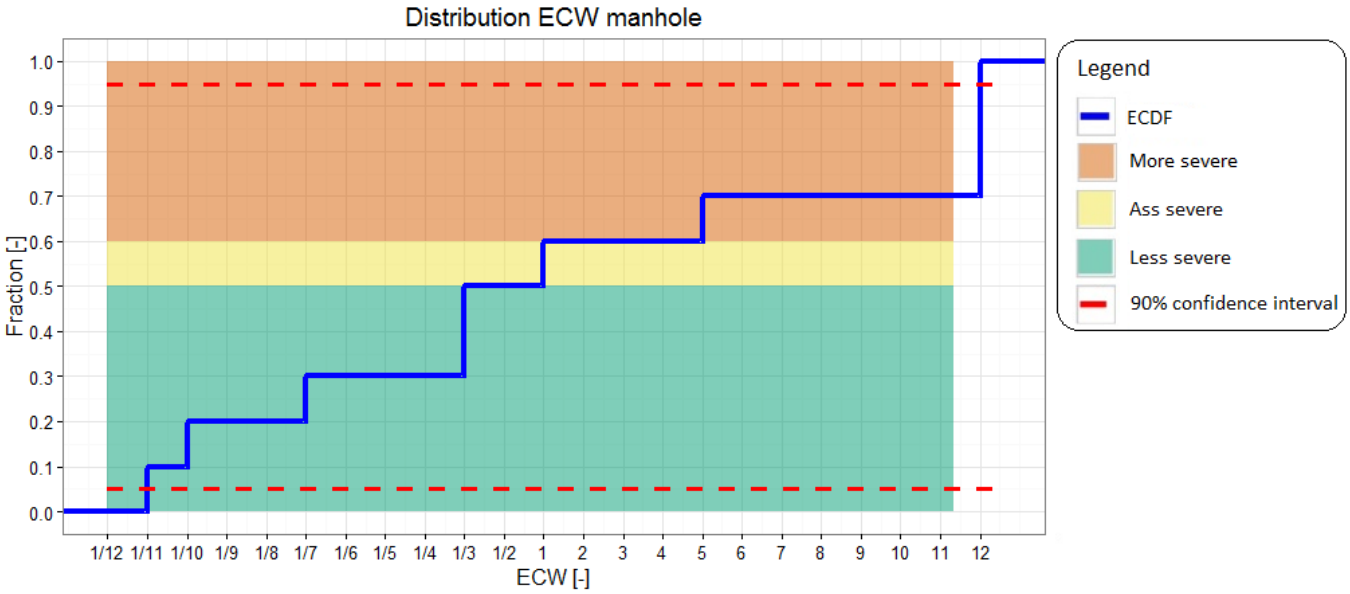


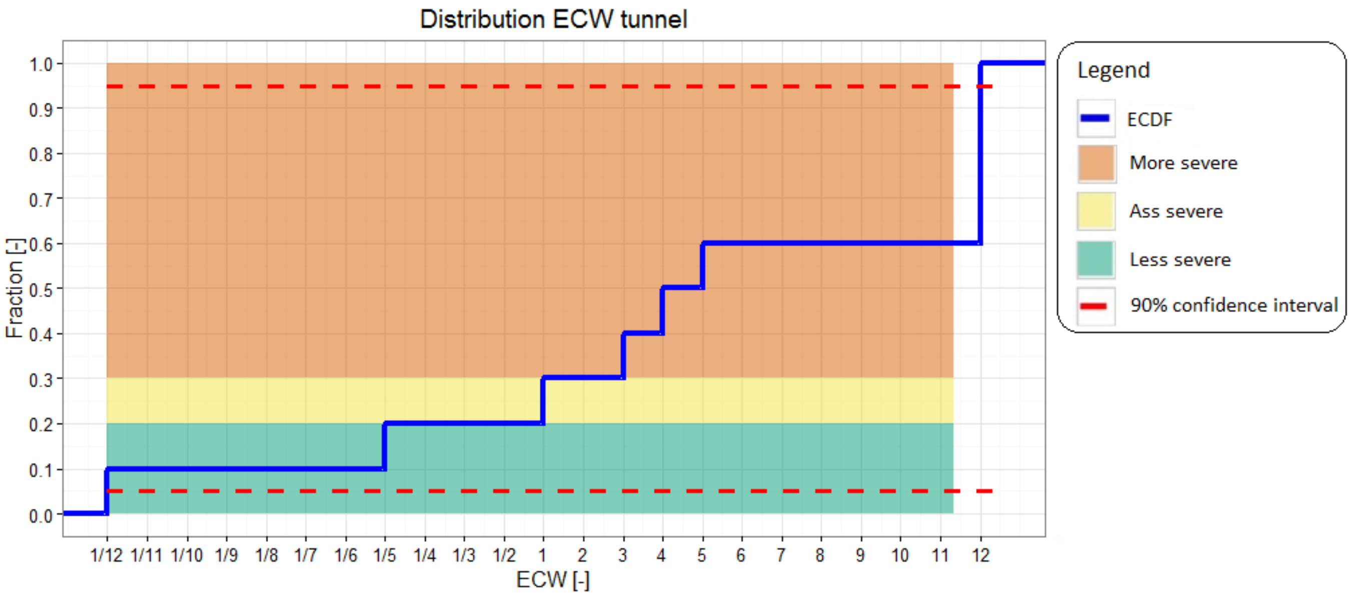


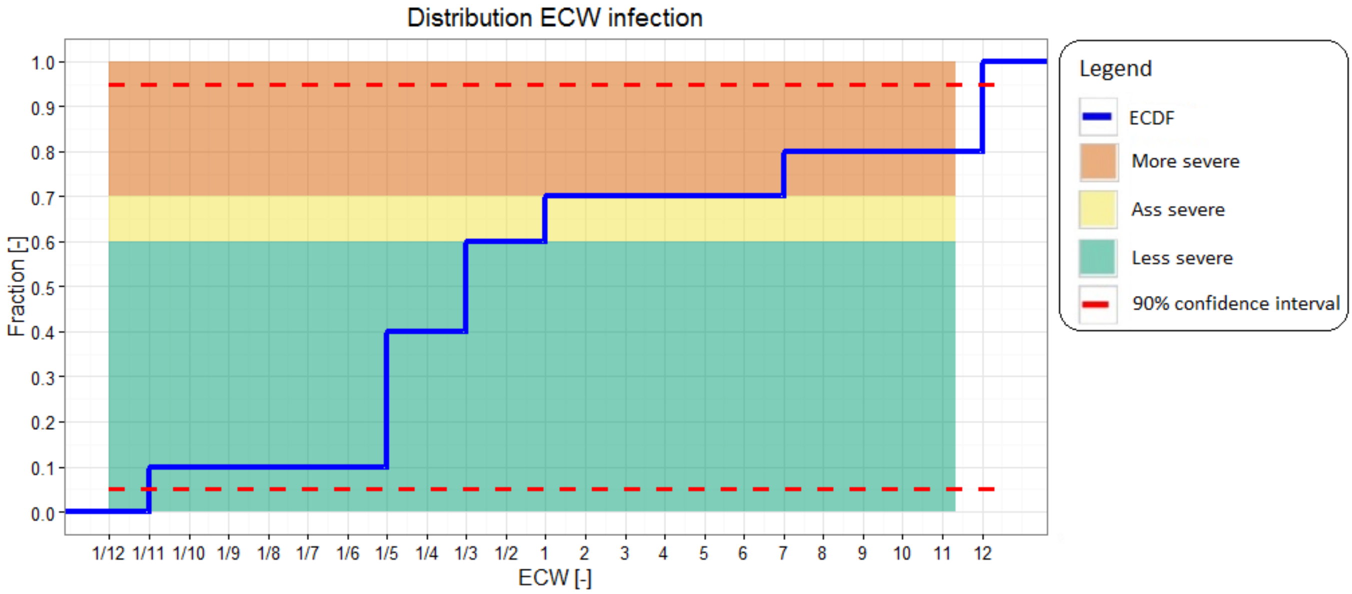
VIII.3 Distribution advisors (n = 10)

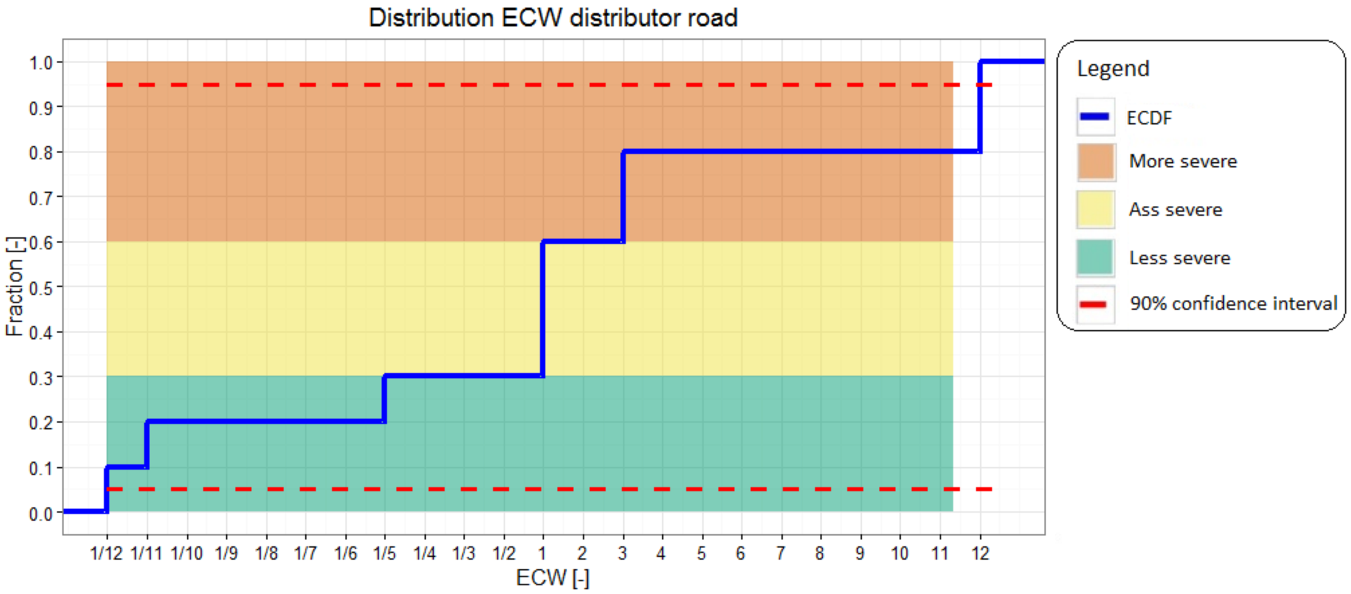


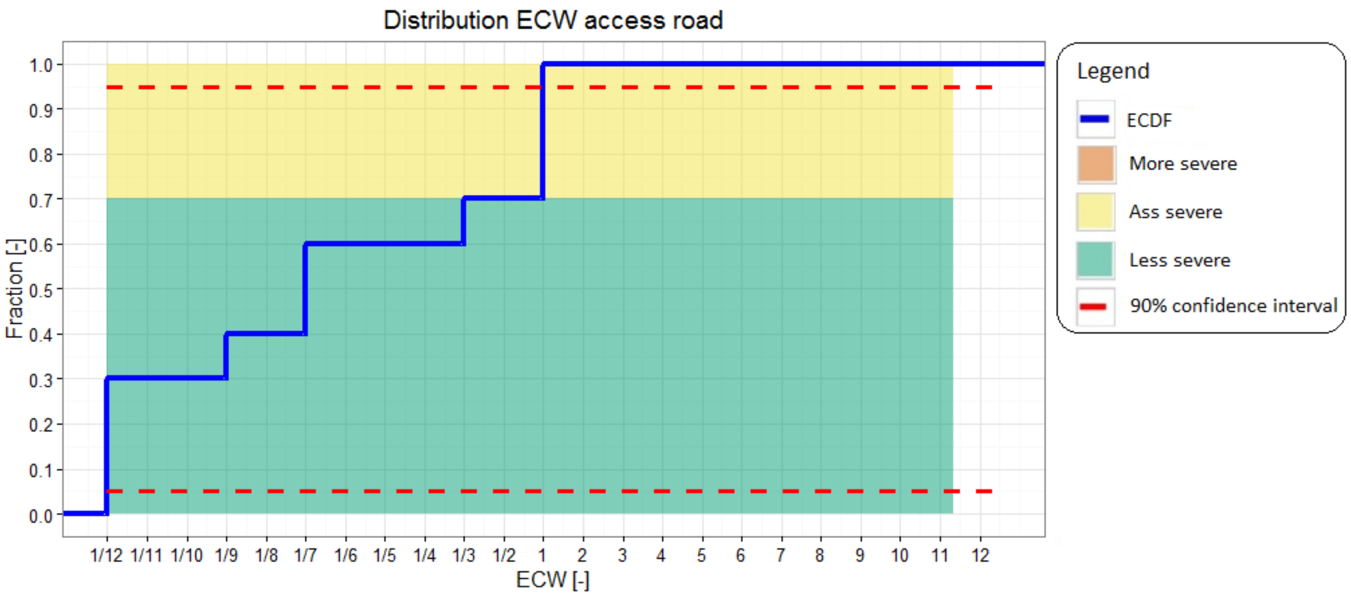

Distribution ECW shop
Fraction [-]
ECW [-]
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval


Distribution ECW manhole
Fraction [-]
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval


Distribution ECW tunnel
Fraction [-]
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

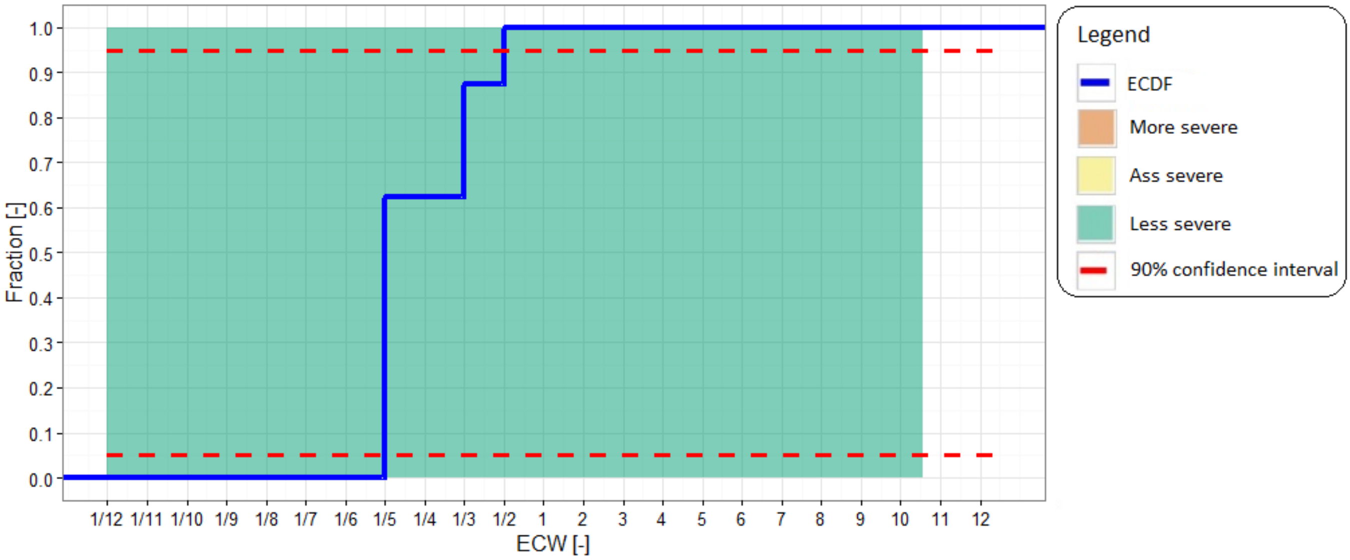

Distribution ECW infection
Fraction [-]
ECW [-]
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval


Distribution ECW distributor road
Fraction [-]
ECW [-]
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

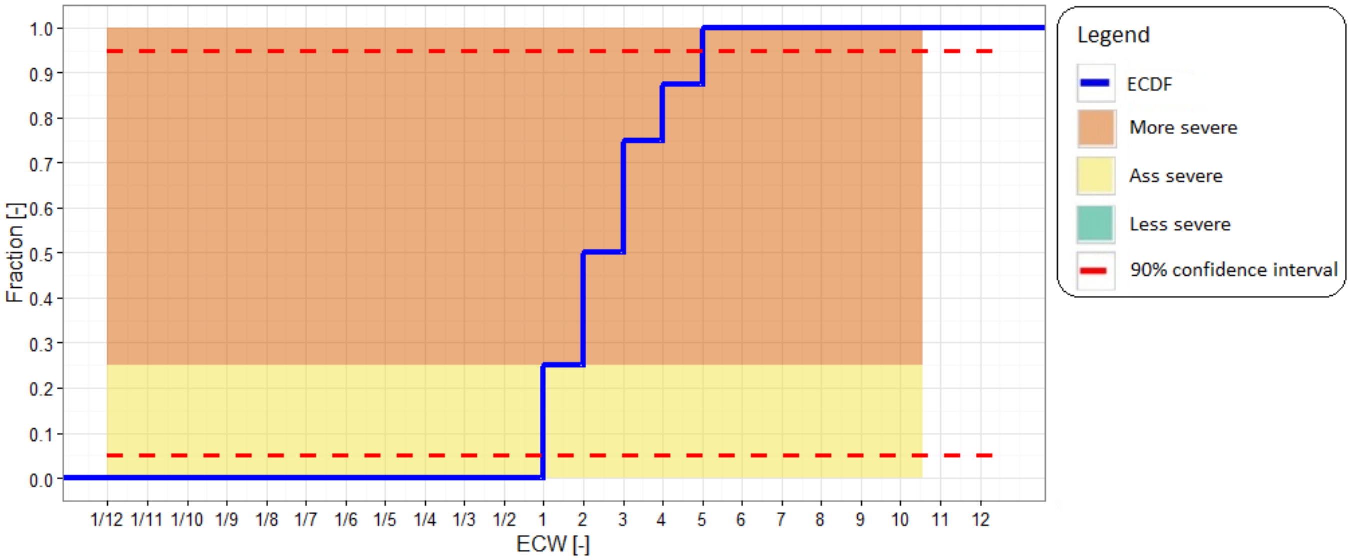

Distribution ECW access road
Fraction [-]
ECW [-]
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

VIII.4 Distribution citizens (n = 8)

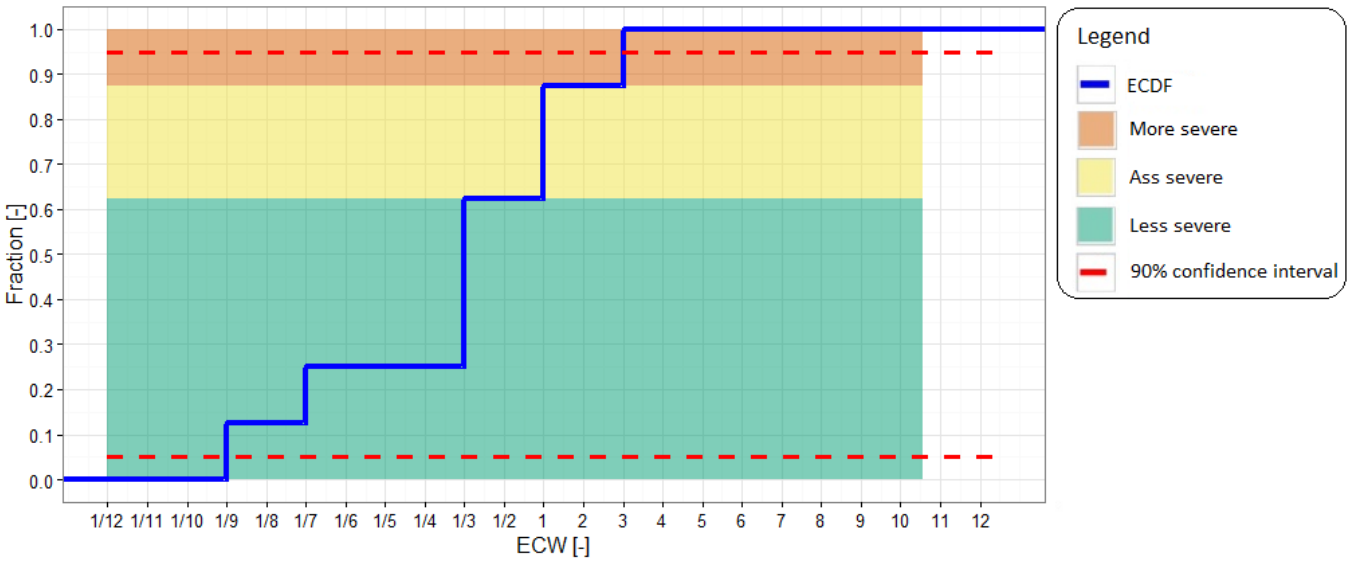
Distribution ECW basement

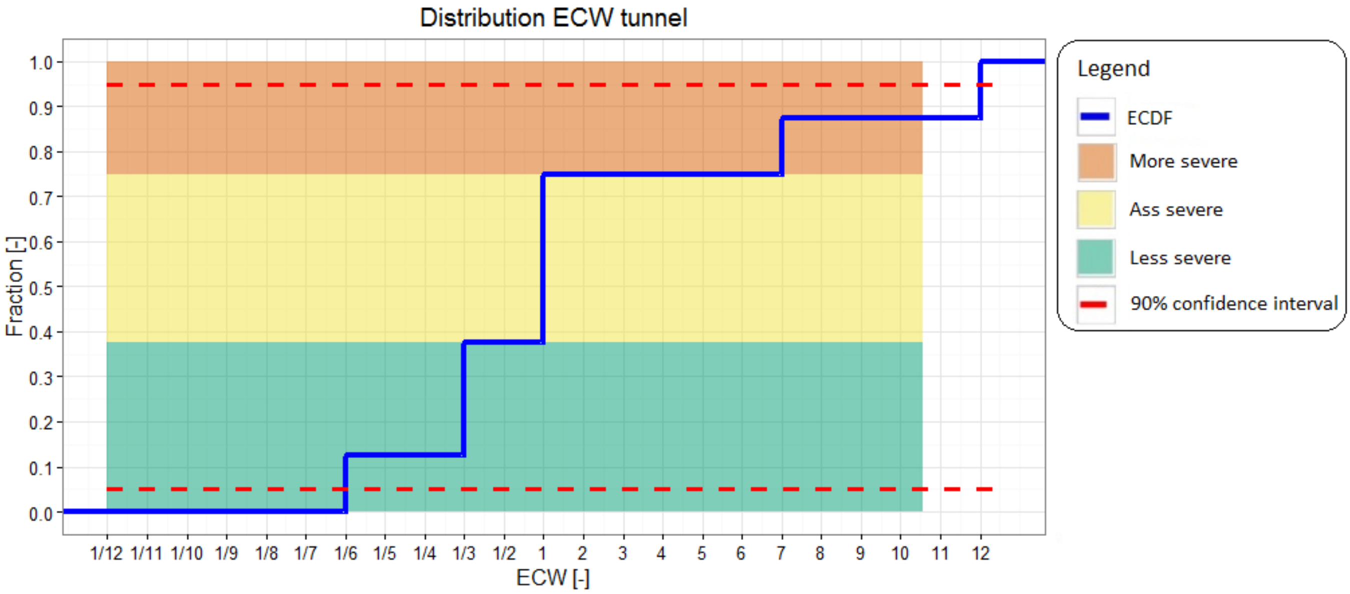


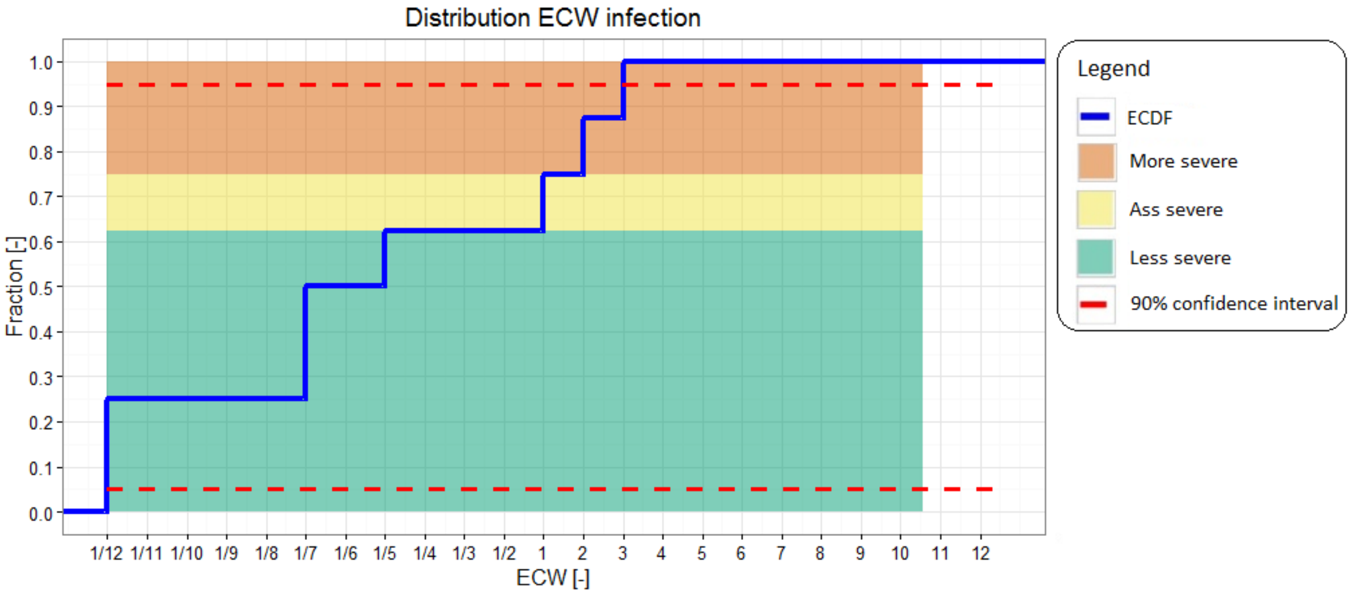
Distribution ECW shop

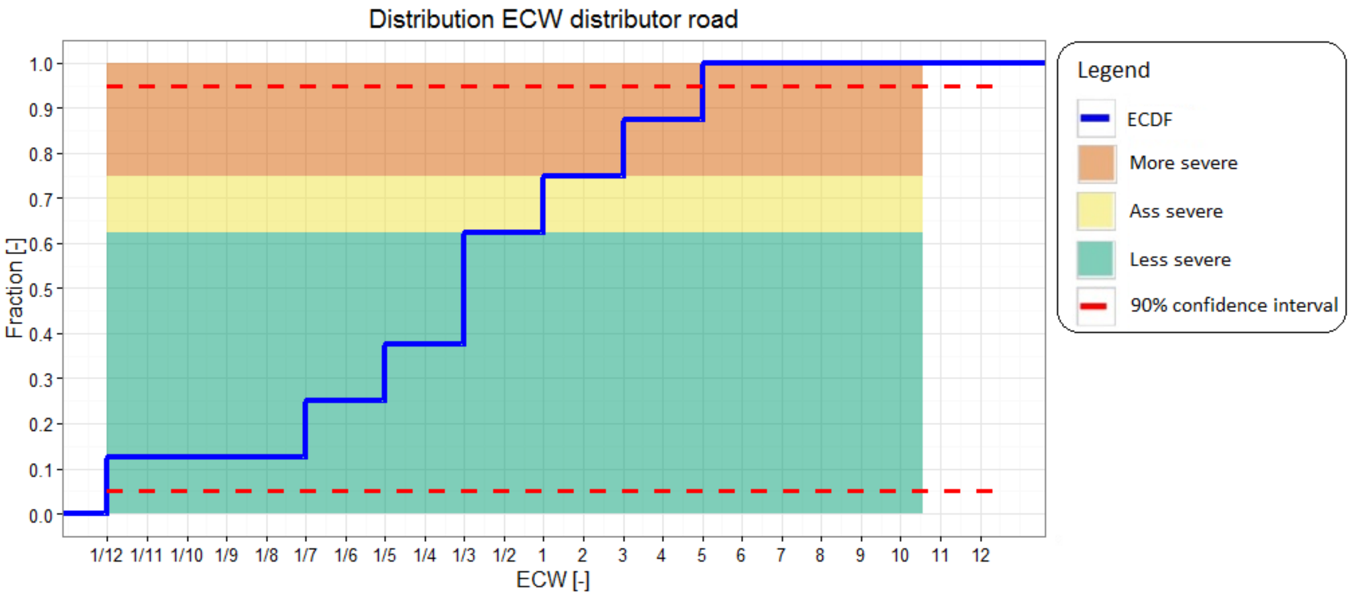


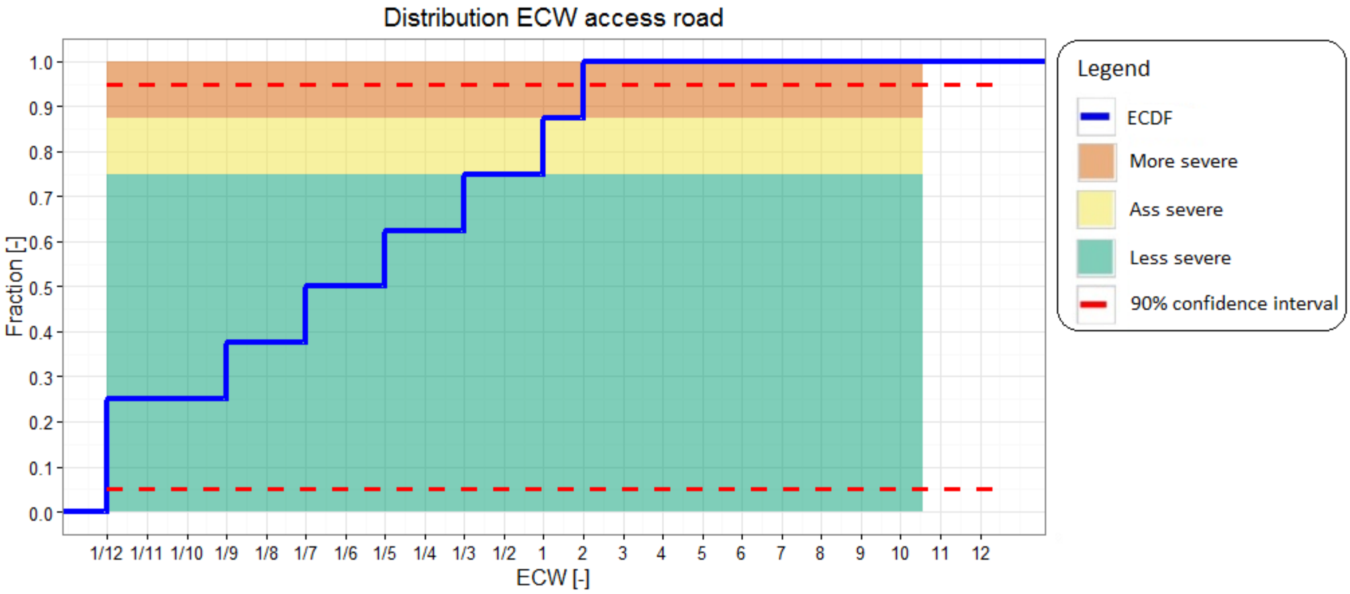
Distribution ECW manhole




Distribution ECW tunnel
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval


Distribution ECW infection
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval


Distribution ECW distributor road
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval


Distribution ECW access road
Fraction [-]
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
1/12 1/11 1/10 1/9 1/8 1/7 1/6 1/5 1/4 1/3 1/2 1 2 3 4 5 6 7 8 9 10 11 12
ECW [-]
Legend
ECDF
More severe
Ass severe
Less severe
90% confidence interval

IX

APPENDIX: DETAILED RESULTS UNCERTAINTY ANALYSIS

IX.1 Detailed results 90%-confidence interval model uncertainty

Table IX.1: Effects location A: lower / upper limit

		Return period [year]							
	ECW	0.5	1	2	5	10	20	50	100
Flooded houses	1						0 / 1	1 / 1	2 / 2
Flooded basements	0.2							2 / 4	6 / 6
Flooded shops	1								
Flooded manholes	1				0 / 2	1 / 3	4 / 8	7 / 10	10 / 17
Flooded tunnels	3								
Flooded pedestrian area's									
Flooded roads						0 / 1	1 / 2	2 / 3	3 / 3
Infection risk-score	0.33					0 / 1	1 / 2	2 / 3	3 / 3
Blocked distributor road (hours)	1								
Blocked access road (hours)	0.2					0 / 0.5	0.5 / 1	1 / 2	2 / 3
Total severity score		0 / 0	0 / 0	0 / 0	0 / 2	1 / 3.4	4.4 / 9.9	9.3 / 13.2	14.6 / 21.8
Risk level (lower)		No	No	No	No	Moderate	High	Moderate	Moderate
Risk level (upper)		No	No	No	High	High	High	High	High
Normative risk level (lower)	High	An. exp. severity score (low)	0.5						
Normative risk level (upper)	High	An. exp. severity score (high)	1.4						

Table IX.2: Effects location B: lower / upper limit

		Return period [year]							
	ECW	0.5	1	2	5	10	20	50	100
Flooded houses	1								
Flooded basements	0.2								
Flooded shops	1								
Flooded manholes	1		0 / 3	8 / 10	11 / 12	13 / 18	17 / 24	23 / 30	26 / 33
Flooded tunnels	3								
Flooded pedestrian area's									
Flooded roads			0 / 1	2 / 2	3 / 3	3 / 3	3 / 3	3 / 4	4 / 4
Infection risk-score	0.33		0 / 1	2 / 2	3 / 3	3 / 3	3 / 3	3 / 4	4 / 4
Blocked distributor road (hours)	1								
Blocked access road (hours)	0.2			1 / 1.5	3 / 4	6 / 8	8 / 10	12 / 14	15 / 17
Total severity score		0 / 0	0 / 3.3	8.9 / 11.0	12.6 / 13.8	15.2 / 20.6	19.6 / 27.0	26.4 / 34.1	30.3 / 37.7
Risk level (lower)		No	No	Extr. high	Extr. high	Extr. high	Very high	Very high	High
Risk level (upper)		No	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high	Very high	High
Normative risk level (lower)	Extremely high	An. exp. severity score (low)	8.7						
Normative risk level (upper)	Extremely high	An. exp. severity score (high)	13.1						

Table IX.3: Effects location C: lower / upper limit

	ECW	Return period [year]							
		0.5	1	2	5	10	20	50	100
Flooded houses	1					0 / 2	1 / 3	3 / 3	3 / 3
Flooded basements	0.2								
Flooded shops	1								
Flooded manholes	1		0 / 1	1 / 3	3 / 4	4 / 4	4 / 4	4 / 4	4 / 4
Flooded tunnels	3								
Flooded pedestrian area's						1 / 1	1 / 1	1 / 1	1 / 1
Flooded roads					1 / 1				1 / 1
Infection risk-score	0.33				1 / 1	2 / 2	2 / 2	2 / 2	2 / 3
Blocked distributor road (hours)	1								
Blocked access road (hours)	0.2				0.5 / 1	1 / 1	1 / 1.5	1.5 / 2	2 / 2.5
Total severity score		0 / 0	0 / 1	1 / 3	3.4 / 4.5	4.9 / 6.9	5.9 / 8.0	8.0 / 8.1	8.4 / 8.5
Risk level (lower)		No	No	Very high	Very high	High	High	Moderate	Low
Risk level (upper)		No	Extr. high	Extr. high	Very high	Very high	High	Moderate	Low

Normative risk level (lower)	Very high	An. exp. severity score (low)	1.9
Normative risk level (upper)	Extremely high	An. exp. severity score (high)	3.9

IX.2 Detailed results 90%-confidence interval effect category weights

Table IX.4: Effects location A: lower / upper limit

	ECW	Return period [year]							
		0.5	1	2	5	10	20	50	100
Flooded houses	1							1	1
Flooded basements	0.08 / 1							4	6
Flooded shops	1 / 5								
Flooded manholes	0.1 / 12				1	2	5	10	10
Flooded tunnels	0.17 / 12								
Flooded pedestrian area's									
Flooded roads						1	2	3	3
Infection risk-score	0.08 / 12					1	2	3	3
Blocked distributor road (hours)	0.08 / 12								
Blocked access road (hours)	0.08 / 3						1	2	3
Total severity score		0 / 0	0 / 0	0 / 0	0.1 / 12.0	0.3 / 36.0	0.8 / 87.0	2.8 / 167.0	3.0 / 172.0
Risk level (lower)		No	No	No	Very low	Very low	Very low	Low	Very low
Risk level (upper)		No	No	No	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high

Normative risk level (lower)	Low	An. exp. severity score (lower)	0.1
Normative risk level (upper)	Extremely high	An. exp. severity score (upper)	12.8

Table IX.5: Effects location B: lower / upper limit

	ECW	Return period [year]							
		0.5	1	2	5	10	20	50	100
Flooded houses	1								
Flooded basements	0.08 / 1								
Flooded shops	1 / 5								
Flooded manholes	0.1 / 12		3	10	11	16	22	27	29
Flooded tunnels	0.17 / 12								
Flooded pedestrian area's									
Flooded roads				2	3	3	3	4	4
Infection risk-score	0.08 / 12			2	3	3	3	4	4
Blocked distributor road (hours)	0.08 / 12								
Blocked access road (hours)	0.08 / 3			1.5	4	7	8	10	12
Total severity score		0 / 0	0.3 / 36.0	1.3 / 148.5	1.7 / 180.0	2.4 / 249.0	3.1 / 324.0	3.9 / 402.0	4.2 / 432.0
Risk level (lower)		No	High	Very high	High	High	Moderate	Low	Very low
Risk level (upper)		No	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high

Normative risk level (lower)	Very high	An. exp. severity score (lower)	1.5
Normative risk level (upper)	Extremely high	An. exp. severity score (upper)	164.2

Table IX.6: Effects location C: lower / upper limit

	ECW	Return period [year]							
		0.5	1	2	5	10	20	50	100
Flooded houses	1					1	2	3	3
Flooded basements	0.08 / 1								
Flooded shops	1 / 5								
Flooded manholes	0.1 / 12		1	1	4	4	4	4	4
Flooded tunnels	0.17 / 12								
Flooded pedestrian area's						1	1	1	1
Flooded roads					1				1
Infection risk-score	0.08 / 12				1	2	2	2	3
Blocked distributor road (hours)	0.08 / 12								
Blocked access road (hours)	0.08 / 3				1	1	1.5	2	2.5
Total severity score		0 / 0	0.1 / 12.0	0.1 / 12.0	0.6 / 63.0	1.7 / 76.0	2.7 / 78.5	3.7 / 81.0	3.9 / 94.5
Risk level (lower)		No	Moderate	Low	Moderate	Moderate	Moderate	Low	Very low
Risk level (upper)		No	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high	Extr. high

Normative risk level (lower)	Moderate	An. exp. severity score (lower)	0.6
Normative risk level (upper)	Extremely high	An. exp. severity score (upper)	37.3