FLOOD RISK OF REGIONAL FLOOD DEFENCES

APPENDIX

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APPENDIX

FLOOD RISK OF REGIONAL FLOOD DEFENCES TECHNICAL REPORT

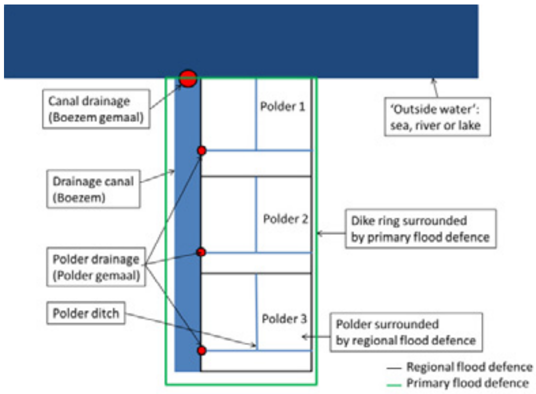
FLOOD RISK OF REGIONAL FLOOD DEFENCES

CONTENT

SUMMARY

Historically the Netherlands have always had to deal with the threat of flooding, both from the rivers and the sea as well as from heavy rainfall. The country consists of a large amount of polders, which are low lying areas of land protected from flooding by embankments. These polders require an extensive water storage and drainage system to discharge excess water to the surrounding 'outside water'. Through a large system of ditches water is pumped onto large storage canals which in turn drain in to the 'outside water': in the sea or in rivers. In Dutch these drainage canals are called 'Boezems'. The embankments which enclose the storage canals inside the polders are called Regional Flood Defences.

FIGURE 1 LEFT: TYPICAL STORAGE CANAL (RIJKSWATERSTAAT), RIGHT: SCHEMATIC VIEW OF POLDER, CANAL AND OUTSIDE BODY OF WATER



The objective is to determine whether or not the flood risk approach can be applied to a system of regional flood defence systems, given the existing data and 'state of the art' methods. The proposed methodology will provide a basis for more thorough assessment of the regional flood defences, not only based on the safety standards, but including the failure probabilities of all relevant geotechnical failure mechanisms and corresponding consequences of flooding. <u>The project focuses on 'Boezemkaden', which will be addressed as regional flood defences in the remainder of this document.</u> These flood defences typically retain lower hydraulic heads than primary flood defences.

RISK ASSESSMENT METHODOLOGY

Flood risk is assessed by the annual expected damage due to flooding, which is estimated by multiplying the probability of flooding with the consequences of flooding. The largest knowledge gaps exist in the calculation of probabilities, because several 'state of the art' models are available to determine the flood consequences due to breaches in a flood defence system.

PROBABILITY OF FLOODING

The probability of failure of a system of flood defences is determined based on a schematization of the system, which divides it in sections with similar strength properties. Probabilistic methods are used to determine the probability of failure of one section. Flood scenarios are defined as groups of sections which have similar consequences during a flood: these are chosen such that every breach in this group, regardless of its location, will lead to the same flood consequences.

FIGURE 2 COMPONENTS OF A FLOOD RISK ASSESSMENT

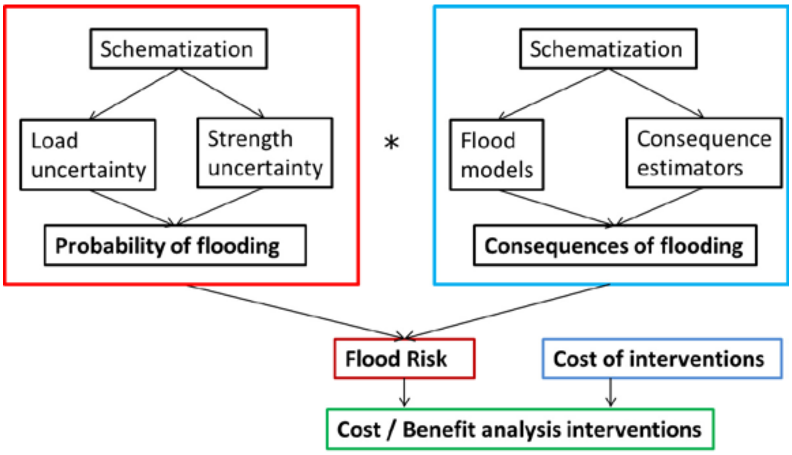
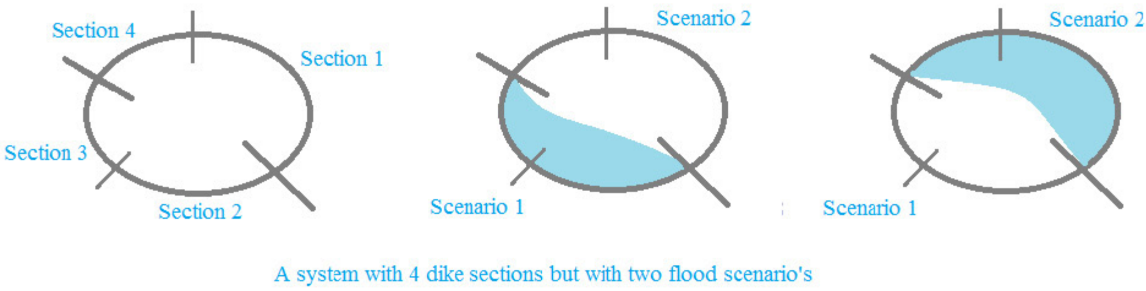


FIGURE 3 A SYSTEM WITH 4 SECTIONS BASED ON STRENGTH PARAMETERS (LEFT) BUT TWO FLOOD SCENARIOS' (MIDDLE AND RIGHT)



To obtain the probability of one flood scenario, the failure probabilities of individual dike sections are combined. The probability of flooding of the whole system can be determined by combining the scenario probabilities. The manner in which the failure probabilities are combined depends on the occurrence of relief in the system:

- **Relief**: if failure of one section results in lower loads on the other sections relief is taken in to account. This can be done in two ways: by assuming that the weakest section fails first or by assuming that the first loaded section fails first;
- **No relief**: if failure of one section does not have any effect on the loads of other sections, no relief is taken in to account.

The occurrence of relief strongly depends on the volume of water inside the canal compared to the size of the inundated area after a breach. The extent of relief in canal systems requires careful investigation for every case study. With relief, we can assume only one breach is possible within one canal system. However, when there is no relief multiple breaches are possible. This is an important part of the schematization of the system, as the occurrence of relief has large effect on flood risk.

FLOOD CONSEQUENCES

It is assumed that floods resulting from a breach in a regional flood defence system only have inundation depths in the order of decimetres, except for the occasional deeper polders. Due to the low expected inundation depths no loss of life is taken in to account. The economic consequences are determined with HIS SSM and WSS.

The HIS SSM model provides an estimate of the economic consequences and loss of life for large floods, with inundation depths of several meters. A disadvantage of the model is that it is inaccurate for low inundation depths. The 'Water Schade Schatter' is developed to determine the consequences of small floods due to heavy rainfall in polders. This model uses more accurate consequence functions for low inundation depths of several decimetres, providing more accurate estimations for small floods. A major disadvantage is that the consequence functions are limited to inundation depths below 0.3 meter. To account for larger inundation depths, the consequence functions for buildings specifically have to be changed.

Both methods are used and compared in this report; the difference between both estimators can reach up to 20%. No clear distinction can be made of which estimator provides an upper or lower limit.

UNCERTAINTIES IN LOADS ON REGIONAL FLOOD DEFENCES

To determine failure probabilities, insight is required in the statistical distribution of the governing loads, which for regional flood defences are:

1 Hydraulic loads: water levels inside the canals and resulting groundwater level;
2 Traffic loads: vertical loads on top of the flood defence;

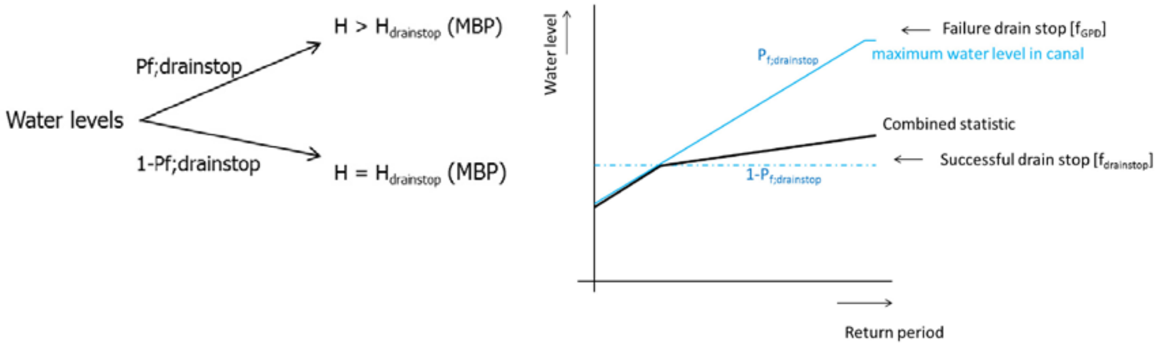
Waves in these canals are neglected. Currently a research program is undergoing on the stability of peat dikes during droughts. This load is not taken in to account at this stage, because the results of this research are expected to largely influence the assessment of regional flood defences. Furthermore, a case study is chosen for a region where earthquake loading is not present.

HYDRAULIC LOADS

We consider the volume of water in the canals being governed by the inflow from the polder drainage stations and the outflow to the outside water; neglecting rainfall, seepage and local wind set up. These water levels are regulated by the water boards. During extreme rainfall events, the canals have a certain storage volume available for storage of excess water out of the polders pumped on to the canals, which is determined by the difference between the target water level and the 'drain stop level'; the maximum allowed water level on the canals.

Once the 'drain stop level' is reached, the polder drainage stations are not allowed to keep pumping water on to the canals. This event may only occur with a probability of 1/100 per year. Whether or not the drain stop is successful depends on the way these are managed. During heavy rainfall events, water boards may have to choose between having to exceed the 'drain stop level' on the canals to keep polders dry, or vice versa. The drain stop may fail due to factors which cannot be influenced by the waterboard, or because a certain part of a polder needs to remain dry. The event where the water levels exceed the drain stop level is defined as 'failure of the drain stop'.
Water level observations are used to determine the water level statistic of the regulated system. A Generalized Pareto Distribution is fitted through independent peaks of water levels in the canals. This distribution is modified, to account for the regulation of water levels in the system, by making a distinction successful and unsuccessful drain stop, see Figure 21.

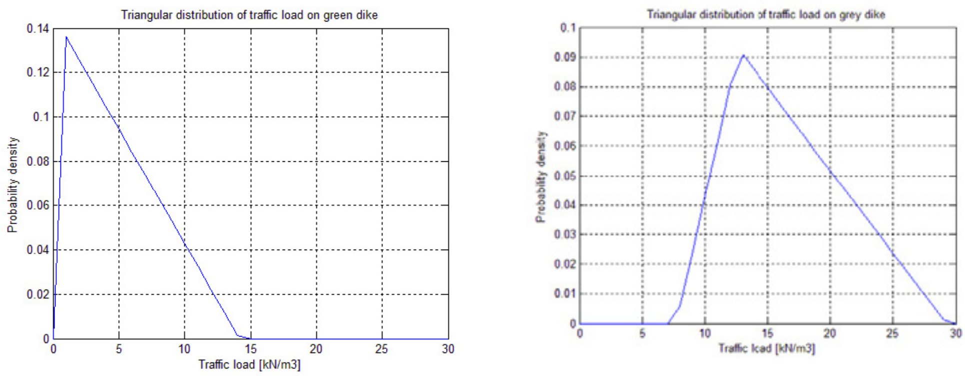
FIGURE 4 METHOD FOR WATER LEVEL STATISTIC (LEFT) AND RESULTING ANNUAL EXCEEDANCE LINES (RIGHT)



TRAFFIC LOADS

The combination of hydraulic loads (high water levels) and traffic loads is governing for the stability of the flood defence. Expert ellicitation was used to determine the statistical distribution of traffic loads. Water board employees responsible for the assessment of the regional flood defences were asked to provide estimates of the 5th, 50th and 95th quantiles of the statistical distribution of the traffic loads. Furthermore, they were asked to provide an estimate of the correlation between the traffic load and water level. The resulting traffic load distribution is shown in Figure 25, for green and grey flood defences. No correlations between the traffic load and water levels was expected with average water levels; the experts all agreed on this point. However, they did not agree on the correlation between the traffic load and the extreme water levels, which was either positive or negative.

FIGURE 5 TRIANGULAR DISTRIBUTIONS OF TRAFFIC LOADS ON GREEN (LEFT) AND GREY (RIGHT) FLOOD DEFENCES



Different combinations of hydraulic and traffic loads are possible, which depend on the management of the water board. Specifically the policy regarding traffic loads on a regional flood defence determines which combinations of loads are most likely to occur. We determined the probability of failure of the regional flood defences with and without traffic loads. Using this methodology, we showed the influence of the traffic loads on the failure probability and risk of regional flood defences, which is significant.

UNCERTAINTIES IN STRENGTH OF REGIONAL FLOOD DEFENCES

The following failure mechanisms are governing for regional flood defences: Overflow, Piping and Instability of the inner slope. FORM reliability calculations are used to determine the probability of failure for each mechanism.

Overflow will occur when the water levels in the canal exceed the retaining height of the surrounding flood defence. The limit state function of overflow is based on a critical overflow amount, which can lead to erosion of the inner slope and breaching.

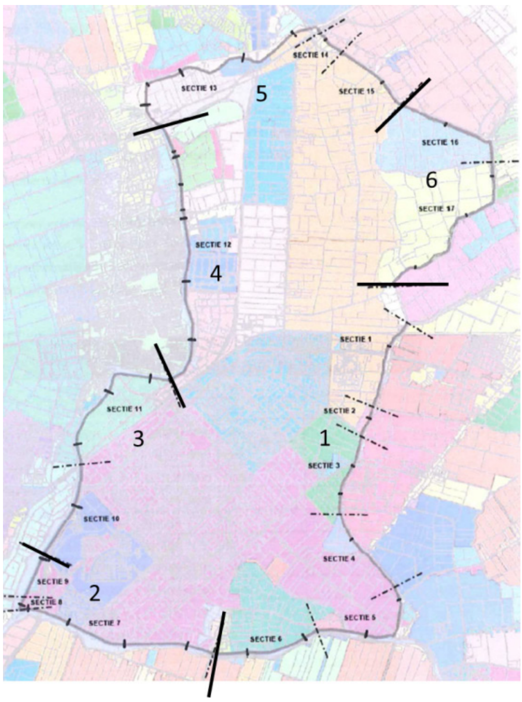
The stability for piping is calculated with the updated Sellmeijer formula. 'Hydraulic short circuiting' is required for piping to develop under regional flood defences. Recent research has shown that hydraulic short circuiting is likely to occur when there is an aquifer below the canals. The response of the water pressure behind the dike to intrusion of water from the canal in the aquifer depends on the thickness of the aquifer. Field tests are required to determine the reduced hydraulic head over the flood defence, due to reduced infiltration of water from the canal to the aquifer.

D-Geo Stability is used to determine the probability of failure for inner slope instability. The failure probability of critical slip circles is calculated with Bishop, for several combinations of water levels, piezo metric lines and traffic loads. Only slip circles which will lead to breaching of the flood defence are taken in to account (i.e. slip circles which protrude the crest of the flood defence). Finally, these are combined to obtain the failure probability of instability. Due to the absence of data on probabilities of phreatic lines, we assumed a distribution. We recommend to perform field tests to determine the actual distribution of the phreatic line.

PROVEN STRENGTH

Proven strength has high potential for updating failure probabilities of regional flood defences. In these canal systems, the difference between average and maximum water levels is very low which results in high potential for proven strength. Especially for overflow and piping the potential is great; the main uncertainty for these failure mechanisms lies in the water levels. The potential of proven strength for the instability failure mechanism is much lower, because not only the water level load determines the stability, but also the phreatic line and traffic loads; these are not always known for the survived load cases.

FIGURE 6 **FLOOD SCENARIOS FOR RISK ASSESSMENT OF HEERHUGOWAARD POLDER**



CASE STUDY: HHNK HEERHUGOWAARD

The ‘Heerhugowaard’ polder is considered in our case study. It is surrounded by two canal systems: the Schermerboezem and the VRNK-boezem. The city of Heerhugowaard lies within the polder, on the Western side. The flood defence system was divided in 17 sections, based on strength properties. The water board made simulations of flooding for a number of breach locations along the flood defence system with Sobek. These were used to schematise the flood defence system in six flood scenarios, each consisting of a group of dike sections (Figure 42).

FAILURE PROBABILITIES

Overflow: The probability of overflow in this flood defence system is negligible; the retaining height of the flood defences is well above the water levels in the canals, which correspond with very low return periods (< 10^{-6} per year).

Piping: The hydraulic heads over the considered regional flood defences result in rather high failure probabilities. However, there is no direct contact between the water in the canals and the aquifer, due to impermeable layers on the bottom of the canal. Therefore a reduced hydraulic head is taken in to account, which is based on field tests of the intrusion resistance of these layers. When taking the reduced hydraulic head in to account, more accurate failure probabilities are found (considering these flood defences have not failed in the last decennia).

TABLE 1 PIPING FAILURE PROBABILITIES

Piping	Section 4	Section 9	Section 11	Section 12	Section 13	Section 17
Pf [yr-1] With new sellmeijer	0.0089	0.1583	0.8529	0.1210	0.0129	0.0199
Pf with reduced head [yr-1] With new sellmeijer	0.0005	$6.4 * 10^{-5}$	0.0178	0.0019	0.0.0004	0.0004

Note that these flood defences may still be at risk for piping if the geological profile is changed, for example due to dredging works or erosion of the bottom of the canals. This may expose the flood defence to the maximum hydraulic head, due to intrusion of the canal water in to the aquifer below, which results in high probability of piping. In follow up research, we recommend including the effect of these events. The piping probability can be computed for scenarios with and without the reduced head, and then combined.

Instability: The probability of failure for instability largely depends on the combination of the phreatic line and top loads. The influence of the outer water level for a given phreatic line is very low. Traffic loads reduce the reliability of the flood defence considerably. Due to the absence of data on probabilities of phreatic lines, we assumed a distribution based on expert judgement. We recommend performing field tests to determine actual distribution of the phreatic line in the defence more accurately.

TABLE 2 INSTABILITY FAILURE PROBABILITIES

Instability	Section 4	Section 9	Section 11	Section 12	Section 13	Section 17
Pf with traffic load [yr-1]	0.0033	0.0007	0.0254	0.0001	0.0004	< 10^{-5}
Pf without traffic load [yr-1]	0.0013	< 10^{-5}	0.0065	0.0001	0.0001	< 10^{-5}

Several experts have stated that the current approach to for including traffic loads does not model the actual situation correct. We therefore recommend discussing the impact of having to include traffic loads on the strength of regional flood defences more thoroughly. The total failure probability of the regional flood defence system surrounding the Heerhugowaard polder is shown in Table 3.

TABLE 3 RESULTING FAILURE PROBABILITIES WITHOUT UPDATING PIPING WITH PROVEN STRENGTH

Flood scenario	Critical section	Overflow [yr-1]	Piping [yr-1]	Instability (with traffic load) [yr-1]	Total failure probability [yr-1]
1	4	$< 10^{-5}$	0.0005	0.0033	0.0038 (1/26)
2	9	$< 10^{-5}$	$6.4 * 10^{-5}$	0.0007	0.0007 (1/1400)
3	10	$< 10^{-5}$	0.0178	0.0254	0.0428 (1/23)
4	12	$< 10^{-5}$	0.0019	0.0001	0.0020 (1/500)
5	13	$< 10^{-5}$	0.0004	0.0004	0.0008 (1/1250)
6	17	$< 10^{-5}$	0.0004	$< 10^{-5}$	0.0004 (1/2500)

PROVEN STRENGTH

We used First Order Survival Updating to update the failure probabilities found for piping. The probability of piping reduced to below 10^{-6}, partly because the probability of water levels higher than the maximum survived water level is very small. However, if we analyse the equations used in this method we conclude that, for this specific case, the method is unrealistic, because the water level uncertainty has little influence on the failure probability (alpha values of 0,05). This results in an error in the formulas, with very low failure probabilities as a result. We therefore recommend to apply an exact method of Bayesian Updating for proven strength, which is described in (Schweckendiek, 2014). This will provide better estimates of the posterior failure probability.

FLOOD RISK

HIS SSM and WSS were used to compute the consequences of flooding for each flood scenario. The calculated consequences both lie in the same order of magnitude (except for scenario 5, due to large difference in the damage to industry). We assume the WSS model to determine the flood damages for regional flood defences more accurately than HIS SSM, because, in general, these floods have lower inundation depths.

The flood risk of each scenario is shown in Table 34. The largest flood risk is determined by scenario 3, or section 11, which has a large probability of flooding combined with high flood consequences. Moreover, the Schermer canal has higher flood risks than the VRNK canal.

TABLE 4 OVERVIEW OF FLOOD PROBABILITY, CONSEQUENCES AND RISK PER SCENARIO

Flood scenario	Section	Canal	Probability of flooding [yr-1]	Damage [mln euro]	Flood risk [mln euro/yr]
1	4	VRNK	0.0038	15	5.75
2	9	Schermer	0.0007	266	0.20
3	11	Schermer	0.0428	431	18.5
4	12	Schermer	0.0020	482	0.95
5	13	VRNK	0.0008	93	0.07
6	17	VRNK	0.0004	1	$4.3 * 10^{-4}$

COST EFFECTIVENESS

The results of a flood risk assessment for regional flood defences can be used to make cost benefit assessment for interventions in the system. Currently interventions in the system are based on the assessment of regional flood defences, wherein weakest sections are prioritized over stronger sections. However, the weakest sections within a system may very well not be the sections where interventions are most cost effective. Note that the results in this assessment are presented to illustrate the method. The expected total costs of several interventions aiming to reduce flood risk in a system of regional flood defences were compared:

- Reducing the hydraulic loads on regional flood defences is not cost effective, as the influence on the flood risk of reducing the drain stop level is negligible;
- Restricting traffic loads on regional flood defences can be cost effective, if instability is the governing failure mechanism for the considered section.
- Compartmentalization of canals, to reduce the consequences after a flood, can be a cost effective intervention.
- Reinforcements prove to be the most cost effective measure.

DISCUSSION AND CONCLUSIONS

We conclude that the flood risk approach can be applied to regional flood defence systems using the data used in the safety assessment of regional flood defences. The approach not only provides insight in the failure probabilities of the flood defences, but also in the corresponding consequences of flooding and therefore the flood risk. The results can be used to compare the flood risk within the system and prioritize interventions based on the expected risk reduction and cost effectiveness.

To obtain more accurate results we recommend investigating how the data obtained in the assessment can be used more effectively in the flood risk approach and/or proven strength assessments. For example, more insight in the relation between the outer water level and rainfall on the phreatic line may provide better estimates of the probability density function of the phreatic line. For these assessments, insights obtained from the water board dike supervisors can play a useful rule.

According to the IPO safety standards, the probability of flooding for these flood defence system is required to be 20% of the probability of a drain stop, which is 1/500 per year. The probabilities found for overflow comply with these requirements. However, several sections do not comply with the safety standard for piping and instability, which was also concluded in the safety assessment of the flood defence system.

Temporary measures to increase the strength of flood defences during calamities have not been considered in this report. We recommend investigating the potential effectiveness of these measures and comparing this with more traditional reinforcements of the flood defences and consequence reducing measures, such as compartmentalization of the canals.

The results of a flood risk assessment of regional flood defence systems can be compared with the results of flood risk from primary flood defences. To do so, more research is required in system behaviour of several regional flood defence systems, as one primary flood defence system often surrounds several of these systems.

This report focussed on the flood risk from regional flood defences loaded by canal systems; however, these flood defences are also used to protect polders from flooding from the larger lakes and several ‘regional rivers’ (e.g. the Dommel). The load uncertainty (i.e. water level difference) in these systems is larger, which can result in different conclusions than those obtained in this report. This also requires further research.

1

INTRODUCTION

1.1 INTRODUCTION

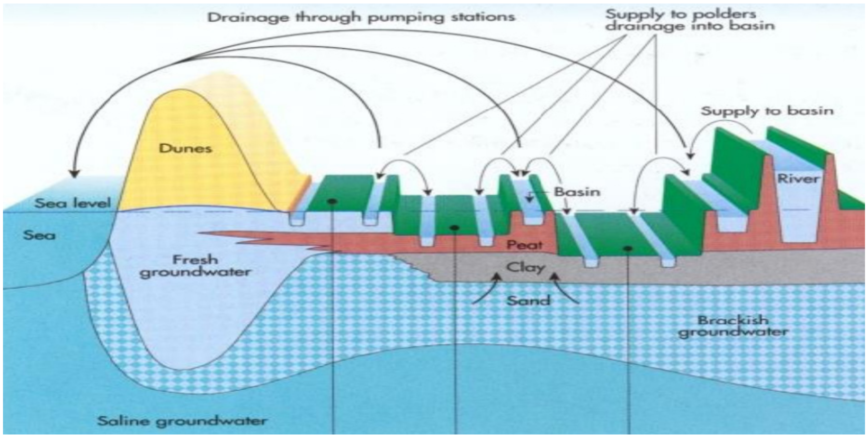
This report is the result of a research project at the Delft University of Technology, funded by the STOWA. It covers the results of research with respect to flood risks of regional flood defences in the Netherlands. The research team consists of researcher Kasper Lendering (Flood defences and Flood risk), Professor Matthijs Kok (Flood risk) and Professor Bas Jonkman (Integral Hydraulic Engineering). In addition, dr. Timo Schweckendiek (Flood defences) provided very useful comments and advice on the subject. Several water boards have also contributed to this report, including Hollands Noorderkwartier and Groot Salland.

Deliverables for 2014 consist of a written report, a management summary in Dutch and a Powerpoint presentation. The results of this research project will be presented in a symposium organised by the STOWA.

1.2 PROBLEM DESCRIPTION

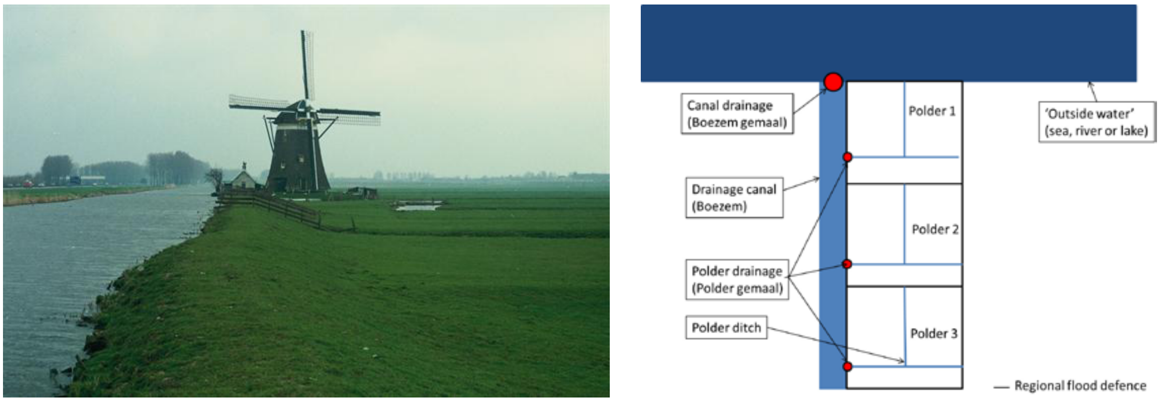
Historically the Netherlands have always had to deal with the threat of flooding, both from the rivers and the sea as well as from heavy rainfall. The country consists of a large amount of polders, which are low lying areas of land. These polders are surrounded by embankments which protect the polder from flooding by ‘outside water’, from rivers or the sea (Figure 7). As a result, the water inside the polder has no connection with the ‘outside water’. Water enters the polders through groundwater flow (seepage) or rainfall. Excess water in the polder is drained out with large drainage pumps.

FIGURE 7 CROSS SECTION OF A DUTCH POLDER (FLOOD DEFENCES, 2014)



Polders require an extensive water storage and drainage system to discharge excess water to the surrounding ‘outside water’. Through a large system of channels water is pumped onto large storage canals which in turn drain in to the ‘outside water’: in the sea or in rivers. This often occurs in several steps, because the level difference between the inner and outer water level is too large to drain in one step. An intermediate level is therefore required, which is why the large storage canals lie above the surrounding polder land, see for example Figure 8. In Dutch these drainage canals are called ‘Boezems’. The embankments which enclose the drainage canals inside the polders are called Regional Flood Defences. In comparison, the embankments surrounding the polder are called Primary Flood Defences; these protect the polder from flooding with ‘outside water’. Floods may occur, among other causes, when either one of these flood defences fail.

FIGURE 8 **LEFT: TYPICAL STORAGE CANAL (RIJKSWATERSTAAT), RIGHT: SCHEMATIC VIEW OF POLDER, CANAL AND OUTSIDE BODY OF WATER**



In recent years, methods have been developed to assess the flood risk of an area based on flooding probabilities and flood damage estimates (R. Jongejan, Maaskant, & Horst, 2013; R. Jongejan & Maaskant, 2013; VNK, 2005). Flood risk is determined by multiplying the annual probability of flooding with the consequences of flooding, which consist of economic consequences as well as loss of life. The consequences are estimated based on detailed flood simulations and damage models. These methods were applied to the primary flood defences in project ‘Veiligheid Nederland in Kaart’. In this project, the flood risk dike rings in the Netherlands is determined. The results have formed the basis for development of new safety standards for the primary flood defences, where the ‘traditional’ probability of exceedance is replaced by the probability of flooding.

A flood risk assessment, such as ‘Veiligheid Nederland in Kaart’, is yet to be made for the regional flood defences. Flood events, such as the floods in New Orleans and Germany, have shown that dikes can also fail before they are overtopped (Ellenrieder & Maier, 2014). Mechanisms such as geotechnical instability and piping have led to several breaches in both primary and regional flood defences. Specifically for regional flood defences, the dike failure at Wilnis in 2003 showed that geotechnical failure mechanisms cannot be ignored (see cover page).

Currently, the safety of regional flood defences is assessed every six years (Stowa, 2007). The assessment provides insight in whether or not the flood defences comply with the safety standards. Interventions are required when a flood defence does not comply with the safety standards, examples are further research or dike reinforcements. However, the assessment does

not provide a method to prioritize the required reinforcements based on cost effectiveness.
A flood risk assessment of the regional flood defences provides insight in the failure probabilities and flood risk of these systems and also provides a basis for prioritizing interventions in the system based on cost benefit analyses. The benefits in this case being a reduction of the flood risk (or reduction of avoided damages). These insights are required to effectively distribute the available budget over the proposed interventions in the system.

1.3 RESEARCH OBJECTIVE

The objective is to determine whether or not the flood risk approach can be applied to a system of regional flood defence systems, given the existing data and 'state of the art' methods. New methods will be added if necessary.
The proposed methodology can provide a basis for more thorough assessment of the regional flood defences, not only based on the safety standards, but including the failure probabilities of all relevant geotechnical failure mechanisms and corresponding consequences of flooding. The research aims to show how such a framework can be used to make cost benefit analyses of proposed interventions and prioritise these according to their cost effectiveness.

Furthermore, the flood risk of a system of regional flood defences can be compared to the flood risk of the primary system. These methods can also be used to determine if the safety standards require revision. This is considered beyond the scope of this project, but can be the focus of follow up research.

RESEARCH QUESTIONS

Specifically, the following research questions will be addressed:

- How can we assess the flood risk of a regional flood defence system?
    - How can the flood probability of regional flood defence system be determined?
    - How can a system of regional flood defences be schematized?
    - How can the joint statistical distribution of the loads be estimated?
    - What are the dominant failure mechanisms and how can their failure probability be estimated?
    - Can these probabilities be validated / verified using 'proven strength'?
- How can the consequences of a flood in a regional system be estimated?
    - How can the resulting flood depths and current velocities be determined?
    - How can the resulting consequences be estimated?
- How can the cost effectiveness of interventions in the system to reduce flood risk be assessed?

1.4 RESEARCH METHODOLOGY

The research team collaborates with a Dutch water board in order to apply the theoretical framework to a practical case. The water board provided the research team with relevant data and was available for discussion and advice regarding the research subject. The research project was divided in several phases, which are explained below:

I **Literature study**: Investigating existing literature and previous research;

II **Framework**: Developing a framework to estimate the flood risk;

III **Case study:** Application of the theoretical framework to the Heerhugowaard polder;

IV Analyses: Analysis of the results resulting in conclusions and recommendations for further research.

At several moments during the project meetings were organized with an advisory board, consisting of participants of the TU Delft, the water board Hollands Noorder Kwartier and STOWA. During these meetings the progress of preliminary results and the required steps to finish the project were discussed.

1.5 REPORT STRUCTURE

Chapter one contains the introduction of the problem and the research objectives. In chapter two a description of regional defences in the Netherlands is given, including an explanation of the flood hazards within these systems and a comparison with primary flood defences in the Netherlands.

The methodology used to determine the flood risk in a system of regional flood defences is described in chapter three. It is concluded that the method to determine probability of flooding of regional flood defences requires further research. The load uncertainty is discussed in chapter four, which explains how the water level statistics can be derived. Furthermore, expert ellicitation was used to generate traffic load statistics for regional flood defences. The strength uncertainties are discussed in chapter five. The limit states of governing failure mechanisms are described, as well as the computation of flood probabilities for every flood scenario within the system.

The methodology is applied to a case study at a Dutch water board: Hoogheemraadschap Hollands Noorder Kwartier. A specific polder system is chosen, the Heerhugowaard. For this polder the flood probability and consequences are estimated. Following that, a cost benefit assessment is made in chapter 7, to demonstrate how the flood risk within such a polder can be reduced. Finally, the proposed methodology is discussed in chapter 8 and concluding remarks are given.

2

REGIONAL FLOOD DEFENCE SYSTEMS

2.1 DESCRIPTION OF REGIONAL FLOOD DEFENCES

A general description of regional flood defences was given in the last chapter. This chapter will further discuss specific aspects of a system of regional flood defences and how these are related to the primary flood defences.

In the Netherlands there are about 50 to 100 independent canal systems (boezems), which are protected by regional flood defences (Maas et al., 2004). These flood defences are often also used for other purposes than simply to retain water; for example as roads, quays for recreational and/or commercial shipping or as meadows for sheep. According to the guide for assessment of regional flood defences (Stowa, 2007), several types of regional flood defence systems can be distinguished:

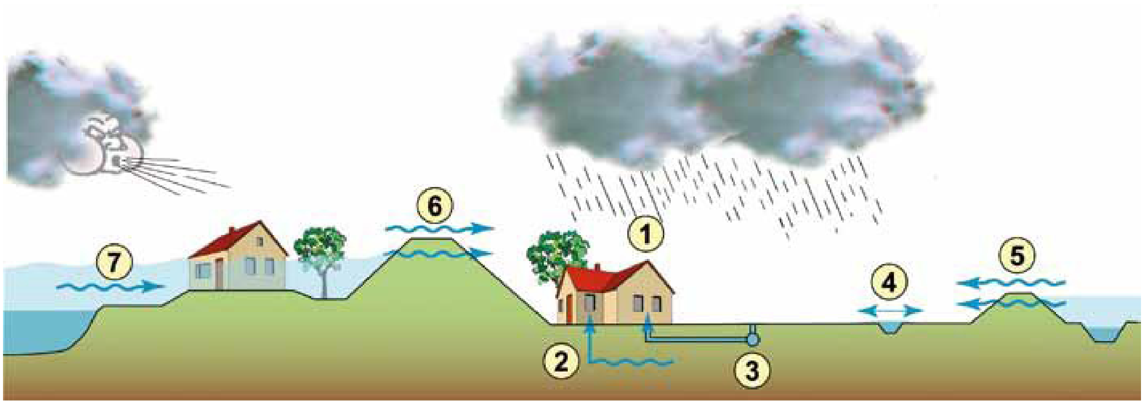
- 'Boezemkaden';
- Flood defences along regional rivers;
- Compartment dikes, secondary dikes and 'sleeper dikes';
- Summer dikes.

This project focuses on 'Boezemkaden', which will be addressed as regional flood defences in the remainder of this document. These flood defences can consist of earthen dikes or hydraulic structures. We focus on earthen structures; a separate analysis is required for hydraulic structures.

2.2 FLOOD HAZARDS IN POLDERS

Flooding can be caused by a lot of events. An overview is given in the following figure, an explanation of the numbers used in the figure follows below.

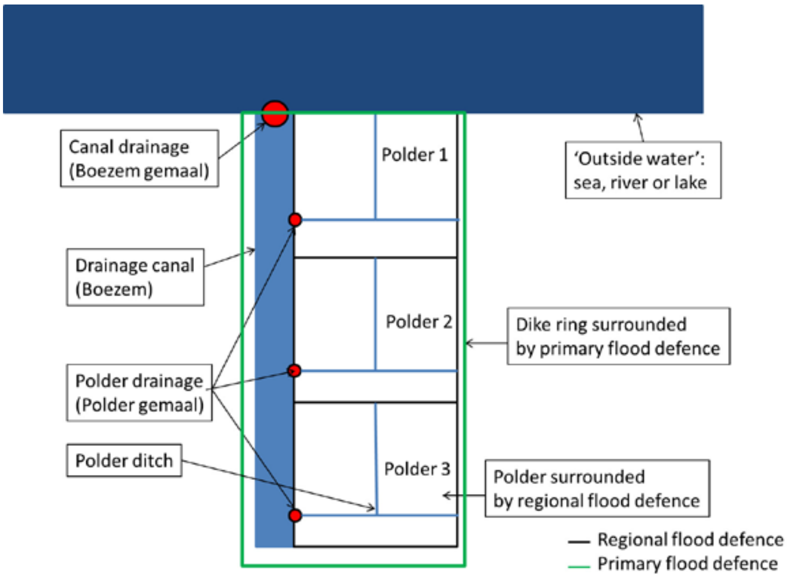
FIGURE 9 FLOODING CAUSES IN THE FLOOD PLAINS OF THE NETHERLANDS (KOK & KLOPSTRA, 2010)



1 Flooding inside a building due to rainfall or for example a leaking aquarium;
2 Flooding due to high ground water levels;
3 Flooding of the sewerage system;
4 Flooding of regional water;
5 Flooding due to breaches in the regional flood defence system;
6 Flooding due to breaches in the primary flood defence system;
7 Flooding of unembanked areas.

On a larger scale (larger than a single house), the major flood hazards are numbers four through seven. Several of these hazards has been investigated in recent years; see (Wolthuis, 2011) for unembanked areas, (VNK, 2005) for primary flood defences and (Hoes, 2006) for regional water. Insight in the flood risk of regional flood defences is still lacking. These hazards cannot be seen as independent events, because they are often the result of a combination of the same loads: wind and rain.

FIGURE 10 SCHEMATIC VIEW OF POLDERS



For example consider Figure 10, a storm at sea will bring heavy rainfall and storm surge. The heavy rainfall will result in excess waters in polders, which may already lead to flooding from regional water. This water will then be pumped on to the drainage canals, which, in combination with rainfall, may result in extreme water levels on the canals. This water will have to be drained to the 'outside water', to sea, which may be difficult or even impossible because the outer water level is too high due to the storm surge. Now both the primary and regional flood defences are loaded by extreme water levels from the storm surge and drainage canals, which may lead to breaching and flooding. The dependency between the outer and inner water levels in Dutch polders is shown in the following table.

TABLE 5 DEPENDENCY BETWEEN OUTSIDE AND INSIDE WATER LEVELS

Outer body of water	Dependency	Explanation
Coast (e.g. North sea)	Positive or negative	A storm can lead to storm surges at sea dependent on the wind direction and heavy rainfall
Lake (e.g. Ijssel lake)	Positive or negative	See above.
River (e.g. Ijssel)	No dependence	The largest part of the river catchment of major rivers in the Netherlands lies outside the catchment of Dutch polders; the peaks in the resulting water levels do not occur at the same simultaneously.

2.3 COMPARISON REGIONAL AND PRIMARY FLOOD DEFENCE SYSTEMS

The following paragraph will elaborate on the main differences between regional and primary flood defences. Typical cross sections of both flood defences are shown in Figure 11.

FIGURE 11 COMPARISON OF REGIONAL FLOOD DEFENCES (LEFT) WITH PRIMARY FLOOD DEFENCES (RIGHT)

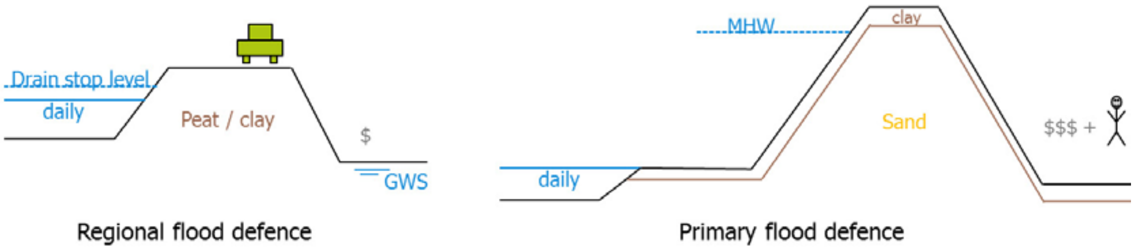


TABLE 6 COMPARISON OF TYPICAL ASPECTS OF REGIONAL VERSUS PRIMARY FLOOD DEFENCES

Aspect	Regional flood defence	Primary flood defence
Geometrical dimensions	Low retaining height (+/- 3m). Steep slopes.	High retaining height (+/- 5m). Mild slopes.
Material	Peat, clay.	Sand core with outer layer of clay.
Water level	'Inside water' in canals: Regulated, constant.	'Outside water' at sea, river or lakes: irregular.
Top loads	Traffic loads	No traffic loads
Protected area	Low economic damage, no loss of life	High economic damage, loss of life
Safety standards	1/10 ~ 1/1,000 per year	1/1,250 ~ 1/10,000 per year

2.3.1 GEOMETRICAL DIMENSIONS

Regional flood defences typically have lower retaining heights (about 3 meters) than primary flood defences (about 6 meters). As a result, the design hydraulic head over a primary flood defence is typically larger than a regional flood defence, which results in larger structures. Furthermore, regional flood defences often have steeper slopes than primary flood defences.

2.3.2 MATERIAL

Regional flood defences often consist of peat and clay layers, because these structures were traditionally constructed with the material present in the area. Currently, the majority of reinforcements of regional flood defences are made with clay. In contrast, primary flood defences are generally constructed with a sand core, covered by an impermeable clay layer.

2.3.3 LOADS

Primary flood defences protect the surrounded area from ‘outside water’: river floods or storm surge at sea or on lakes. Extreme water levels along primary flood defences are natural events, resulting from river floods or storm surge. The difference between the daily average water levels and extreme water levels can reach up to several meters. For example: the decimation height along part of the Ijssel 0.3 meter. Depending on the height and duration of the extreme water levels certain failure mechanisms become dominant.

Regional flood defences protect the surrounded area from ‘inside water’, which is excess water from the polder pumped on to the storage canals. The water levels inside the canals are regulated by human intervention. Water enters the canals through rainfall and inflow from the surrounding polders, water flows out by drainage to the outside bodies of water and seepage. The difference between the average water level and extreme water level is limited to several decimetres. For example: the decimation height in the Schermer boezem is 0.03 meter. The large difference between the water loads in primary and regional flood defence systems results in very different water statistics, which are treated in more detail in the next chapter.

TRAFFIC LOADS

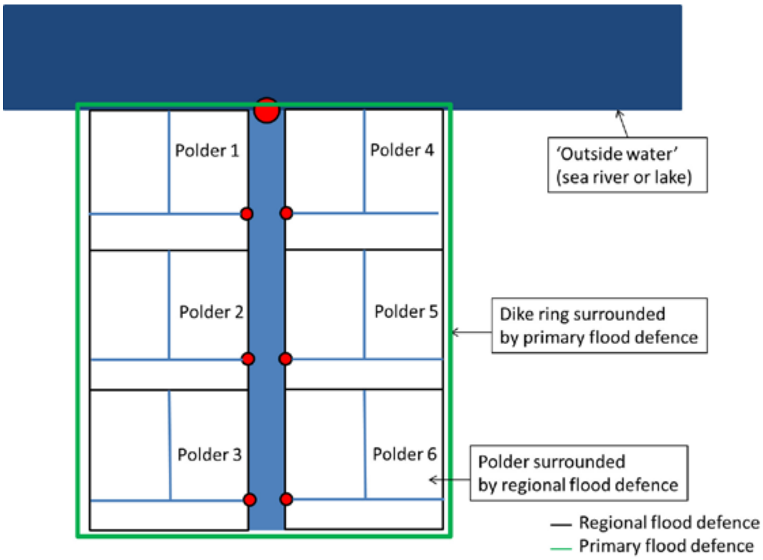
Traffic loads play an important role in the stability of regional flood defences, because a lot of roads have been built on top of these structures. As a result, traffic loads are taken in to account in the design and assessment of these flood defences, according to (Stowa, 2007).

2.3.4 PROTECTED AREA

The protected area of a regional flood defence system is often smaller than the area protected by primary flood defences. A dike ring of primary flood defences often protects several polders surrounded by regional flood defences. Therefore, the flood risk within one dike ring is not only determined by the primary flood defences, but also by the regional flood defences. If a primary flood defence fails, the inflow of water in the protected area is assumed to be significantly larger than the volume of the dike ring (R. Jongejan et al., 2013). This will result in complete flooding of the dike ring with large inundation depths, high economic damage and loss of life as a result.

For regional flood defences this can be very different, because the volume of water in the drainage canals is limited, because these are closed systems. Thus, the volume of water flowing in to a polder after a breach in the regional flood defence is limited. This will result in lower inundation depths, lower economic damage and no loss of life. No loss of life is expected because of the low inundation depths and current velocities. There are exceptions; some polders in the Netherlands are very deep and are surrounded by large drainage canals (e.g. the Haarlemmermeerpolder or the Beemster), which may result in higher inundation depths.

FIGURE 12 **DIFFERENCE BETWEEN PROTECTED AREA OF PRIMARY AND REGIONAL FLOOD DEFENCE SYSTEM**



2.3.5 SAFETY STANDARDS

The safety standards of regional flood defences are lower than those of primary flood defences, due to lower expected economic damages and loss of life. The probability of exceedance of water levels inside regional systems varies between 1/10 to 1/1,000 per year, These depend on the expected damage during a flood. In comparison, those of the primary flood defences vary between 1/1,250 and 1/10,000 per year.

FIGURE 13 **SAFETY STANDARD CLASSES FOR REGIONAL FLOOD DEFENCES ACCORDING TO IPO** (IPO, 1999)

IPO-klasse	herhalingstijd*	schade bij falen (in miljoenen euro)
I	1/10	<8
II	1/30	8 - 25
III	1/100	25 - 80
IV	1/300	80 - 250
V	1/1000	>250

* Overstromingen met een afvoer die eens in de 'n' aantal jaar voorkomen.

2.3.6 SAFETY ASSESSMENT AND MANAGEMENT

The management and maintenance of both primary and regional flood defences in the Netherlands is done by the regional water authorities, or 'Water Boards'. Part of the task is to perform an assessment of all flood defences, every six years, to determine whether or not they comply with the safety standards. The assessment of both the primary and regional flood defences is very similar. The main difference between both assessments is the distribution of the loads: for the primary flood defences uniform hydraulic boundary conditions are determined by the national government (Rijkswaterstaat) based on the maximum river discharges and/or storm surges.

For regional flood defence systems, the local regional water authorities (water boards) determine the hydraulic boundary conditions, because there are large differences in the water levels and discharges in regional systems. Another result of the different loading systems is that other failure mechanisms become dominant, as explained in the previous section. The loads of regional flood defences are treated in detail in chapter 4.

2.4 CONCLUDING REMARKS

Primary flood defences protect the surrounded area from ‘outside water’, while regional flood defences protect the surrounded area from ‘inside water’. This project focuses on ‘Boezemkaden’, specifically earthen structures, which will be addressed as regional flood defences in the remainder of this document.

To determine the flood risk of a regional flood defence the influence of the inner and outer water levels of the polder cannot be neglected, therefor requiring an integral approach. When insight in the flood risk of regional flood defences is obtained, an integral assessment of the flood risk in a polder can be made taking all possible causes of flooding in to account.

FIGURE 14 FLOODING CAUSES IN THE FLOOD PLAINS OF THE NETHERLANDS (FLOOD DEFENCES 2014)

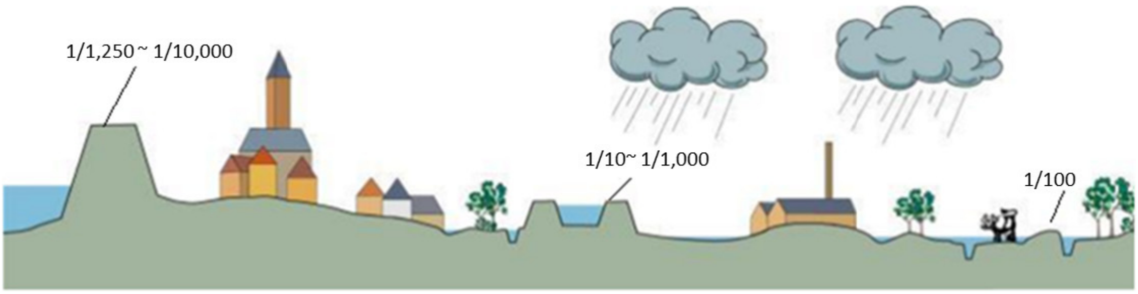


Figure 14 gives an example of a polder showing the safety standards for flooding due to breaches in the primary flood defences (e.g. 1/10.000 per year) or regional flood defences (e.g. 1/1.000 per year) and flooding due to excess water in polders (1/100 per year). After this project, a comparison can be made of the risk of flooding in polders, taking all these flood hazards in to account.

Regional flood defences typically retain lower hydraulic heads than primary flood defences. The cross section of these regional flood defences often consists of a mixture of clay and peat, whereas that of a river dike consists of sand core covered by a clay layer. Regional flood defences are often used for roads which results in a different top load than primary flood defences. Traffic loads are important in regional flood defences.
The protected area of a regional flood defence system is often smaller than the area protected by primary flood defences, which can result in lower consequences during flooding. It is assumed that there will be no loss of life due to flooding after breaching of a regional flood defence. The safety standards of regional flood defences are less stringent than those of the primary flood defence systems.
The main difference in assessment of the regional flood defences, compared to the primary flood defences, is the distributions of the loads. For the primary flood defences, uniform hydraulic boundary conditions are determined by the national government. Due to large differences in the local conditions of regional systems, the hydraulic boundary conditions are determined by the regional water authorities / water boards. For every individual regional flood defence system, the local hydraulic boundary conditions need to be determined separately.

3

RISK ASSESSMENT METHODOLOGY

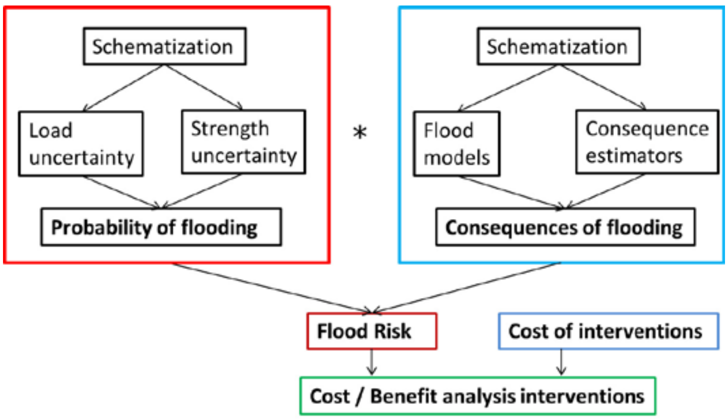
3.1 PROBLEM APPROACH

This chapter will give an overview of all components required to make a flood risk assessment. The application of each of the components to a regional flood defence system will be discussed in separate paragraphs. Flood risk is described by the annual expected damage of flooding, which is estimated by multiplying the probability of flooding with the consequences of flooding. A flood defence system can be modelled as a series system which fails when one of the sections fails.

Failure is defined as the loss of one or more functions of a structure; a failure mechanism is therefore defined as the mechanism which results in the loss of one or more of the functions of a structure. In case of flood defences, failure is defined as loss of water retaining function, with flooding as a result.

Decomposition of the flood defence system in sections is required to determine the failure probabilities of several parts of the system. This is done in the 'schematization', see Figure 15. For the whole system, the load and strength uncertainties need to be determined which may lead to failure of the flood defences. For these uncertainties the probability of failure of the sections can then be calculated using probabilistic methods.

FIGURE 15 COMPONENTS OF A FLOOD RISK ASSESSMENT



The consequences of a breach in the flood defence system depend, among others, largely on the characteristics of the system: the hydraulic loading conditions, the location and amount of breaches, the topography and vulnerability of the protected area (R. Jongejan & Maaskant, 2013). The resulting risk of flooding cannot be estimated simply by multiplying the probability of failure with the consequences.

In the 'Veiligheid Nederland in Kaart' project this issue is resolved by defining a set of flood scenarios; each separate flood scenario has similar consequences irrespective of breach location. For every scenario the probability of flooding is determined, after which all scenario probabilities are combined with the consequences. The cumulative flood risk is determined by combining the risk of each scenario. The obtained insight in the flood risk of the system can be used to make cost benefit analyses of interventions aimed to reduce flood risk.

3.2 PROBABILITY OF FLOODING

The probability of failure of a system of flood defences is determined based on a schematization of the system, which divides it in several sections. Probabilistic methods are then used to determine the probability of failure of the section, see for example (Bischiniotis, 2014; Meer, 2009). The failure probability of the whole system is determined by combining the failure probabilities of all sections for each failure mechanism.

1 **Schematization**: a schematization of the system is made based on sections with identical strength parameters.
2 **Load and strength uncertainty**: the uncertainties in load and strength parameters is expressed in statistical distributions, which are determined based on site investigation, observations of water levels or other methods.
3 **Probability of failure**: the failure probability is estimated based on limit state functions of the governing failure mechanisms. This is done for all governing loads.
4 **Combining probabilities**: The probability of failure of every failure mechanism is then combined to determine the probability of failure of one dike section or several dike sections.
5 **'Proven strength'**: using information of survived loads and 'proven strength' techniques the calculated failure probabilities can be updated to account for these survived loads.

The following sections will elaborate further on several of these components. 'State of the art' methods are available for several of these components. The uncertainties in load and strength will be discussed separately in the following chapters.

3.2.1 SCHEMATIZATION

The considered system of regional flood defences will have to be schematised to be able to compute flood risks. Different types of schematizations are required, such as:

- Schematization of the flood defence system in sections and flood scenarios;
- Schematization of the loads acting on the flood defences (see chapter 4);
- Schematization of the failure mechanisms considered (see chapter 5).

This paragraph will discuss the schematization of the flood defence system in sections and/or scenarios. The other two forms are discussed in the corresponding chapters. A system of regional flood defences can consist of several types of flood defences and/or hydraulic structures. These different types of flood defences and/or hydraulic structures can have very different strength parameters, which will make an assessment of a single cross section within the system unrealistic and unreliable. Therefore a decomposition of the system in several sections is required, which can be done in two ways:

- Define sections based on identical strength parameters: for every section a representative cross section is chosen for the whole section based on the available data. This method is used to determine the failure probability of the flood defence.
- Define sections based on similar consequences during a flood: groups of sections are chosen such that every breach in this group, regardless of its location, will lead to the same flood consequences.

SIMILAR STRENGTH PARAMETERS: DIKE SECTIONS

To determine the failure probability of a flood defence it is decomposed in representative sections with similar strength parameters: a dike section is characterized by the following features (Stowa, 2007):

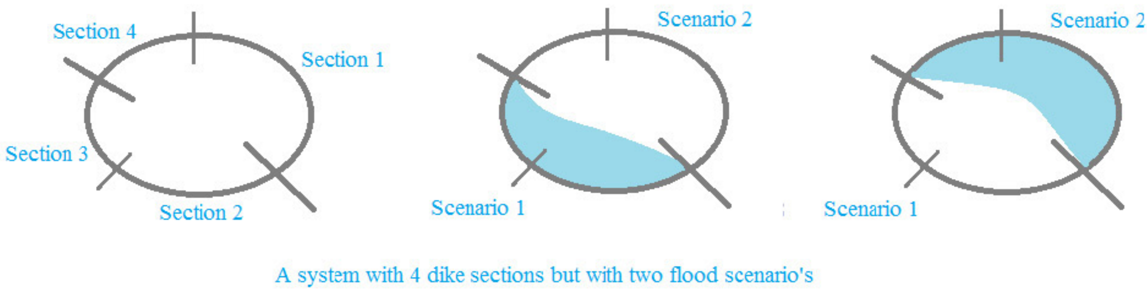
- Uniform loads;
- Homogeneous cross section: geometrical profile, inner and outer layer protection and objects in the flood defence;
- More or less homogeneous geotechnical profile in the flood defence and subsoil;
- The same IPO safety class, see Figure 13.

Note that within one dike section different cross sections can be used for different failure mechanisms, depending on which cross section is normative for the given failure mechanism.

SIMILAR FLOOD CONSEQUENCES: FLOOD SCENARIOS

A flood scenario is composed of a group of dike sections, wherein a breach will result in similar flooding irrespective of its location within the group of sections. In Veiligheid Nederland in Kaart', these groups of sections are called 'ring sections' (VNK, 2005).

FIGURE 16 A SYSTEM WITH 4 SECTIONS BASED ON STRENGTH PARAMETERS (LEFT) BUT TWO FLOOD SCENARIOS' (MIDDLE AND RIGHT)



For every group of sections the probability of flooding is obtained by combining the probability of flooding of the individual dike sections. The probability of flooding of the whole system is obtained by combining the probabilities of every flood scenario.

3.2.2 PROBABILITY OF FAILURE

The following step to determine the probability of flooding consists of the calculation of the failure probability of every failure mechanism given a certain load. The probability of failure of the flood defence depends on the difference between the load (solicitation) and strength (resistance), which is described by limit state functions. The general form of a limit state function is shown in equation 1, where the loads are described by the variable S (Solicitation) and the strength by the variable R (Resistance). The flood defence fails when the solicitation exceeds the loads (i.e. when the limit state function is smaller than zero).

$$Z = R - S \tag{1}$$

The failure probability is found by the probability that the limit state function is smaller than zero; the probability that the solicitation exceeds the resistance:

$$P_f = P(S > R) = P(Z = 0) \tag{2}$$

Probabilistic calculation methods, such as FORM and Monte Carlo simulation, can be used to calculate the probability of failure. For both the resistance and solicitation, probability density functions are required, which describe the uncertainty in the load and strength parameters. The calculation methods will be described in more detail in the next chapter.

FRAGILITY CURVES

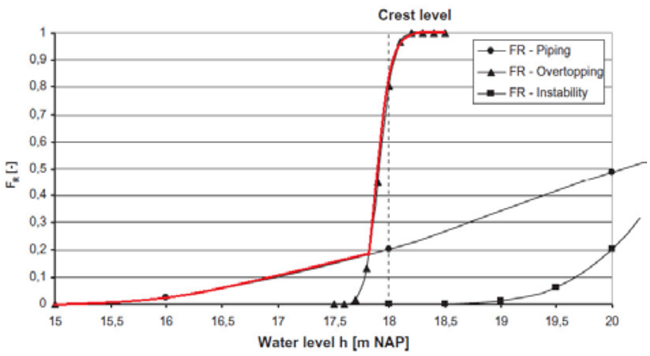
Fragility curves illustrate the failure probability of the flood defence conditional on the load. They represent the cumulative density function of the strength $F_r(s)$, given a certain load. The fragility curves can also be used to compute the total failure probability of the flood defences, by solving equation 4. This is often done numerically, with the probabilistic methods explained in the last section, as they can seldom be solved analytically.

$$P_f = \int_{s=-\infty}^{s=\infty} \int_{r=-\infty}^{s} f_s(s) \cdot f_r(r) drds \tag{3}$$

$$P_f = \int_{s=-\infty}^{s=\infty} f_s(s) \cdot F_r(s) ds \tag{4}$$

In equation 4, $f_s(s)$ represents the probability density function of random load variables and $F_r(s)$ represents the cumulative density function of the strength given that load, i.e. the conditional failure probability given a certain load.

FIGURE 17 EXAMPLE OF A FRAGILITY CURVES FOR PIPING, OVERTOPPING AND INSTABILITY, CONDITIONAL ON THE WATER LEVEL (MEER, 2009)



The fragility curves can be constructed for every failure mechanism and then be combined, providing insight in which mechanism is governing for a given load. The water boards can use this information in their day to day management of the flood defences, because these provide insight in the fragility of a flood defence for a given load. Suppose a certain extreme water level is predicted which, according to the fragility curve, will result in high failure probability of a flood defence. The water board managers can decide upon required measures for specific failure mechanisms of the flood defence based on the insights provided by the fragility curves.

3.2.3 COMBINING FAILURE PROBABILITIES

For every section the failure probability is calculated for each failure mechanism, after which these can be combined to obtain the failure probability per section. This can be done using fault tree analysis. In section 3.2.1 the flood defence system is decomposed in groups of sections corresponding with flood scenarios. Each group of sections is chosen such that a breach within this group will lead to similar flooding regardless of the location of the breach, thus representing one flood scenario.

COMBINING SCENARIO PROBABILITIES

Groups of sections are modelled as a series system, where the upper limit of the failure probability is the summation of the individual failure probabilities. The lower limit is the maximum of the failure probabilities of the individual sections. The actual failure probability will lay somewhere between these limits (VNK, 2005), this is treated in more detail in chapter 5.

$$\underset{i=1}{\overset{N}{\mathrm{MAX}}}\, P_{f_i} \le P_f \le \sum_{i=1}^{N} P_{f_i} \tag{5}$$

The probability of flooding of the whole system can be determined by combining the scenario probabilities. The manner in which the failure probabilities are combined depends on the occurrence of relief in the system. Two cases are distinguished (R. Jongejan et al., 2013):

- **No relief**: if failure of one section does not have any effect on the loading of other sections no relief is taken in to account.
- **Relief**: if failure of one section results in lower loading on the other sections relief is taken in to account. This can be done in two ways:
  - By assuming that the weakest section fails first.
  - By assuming that the first loaded section fails first.

For primary flood defence systems the occurrence of relief is investigated for each specific case, because a breach in the system may result in a reduction of the loads on other parts of the system. Often, the total volume of inflow through a breach into a dike ring is low compared to the total flow of a river or volume of the sea, which may result in a reduction of the water levels on the river.

The canal systems surrounding polders are closed systems regulated by the inflow from the polders, the outflow to the ‘outside’ water (river or sea) and rainfall. Whether or not relief can be taken in to account after breaches occur in these systems depends on the volume of water in the canals in relation to the size of the protected polder. For example, if the outflow of water through a breach results in a significant drop in water level of the canal surrounding the polder relief can be taken in to account. However, a different situation occurs when the outflow through a breach does not lead to a significant drop in water levels in the canals. This may occur when the volume in the canals is very large compared to the outflow in the polder. In that case no relief is can be taken in to account.

Concluding, the extent of relief in canal systems requires careful investigation for every case study. The size of the canal system (and corresponding volume of water) will determine whether or not relief can be taken in to account. This is an important part of the schematization of the system, as the occurrence of relief will have large effect on the flood risk. This is explained in the following section:

WITHOUT RELIEF

If a breach in the system does not result in a significant drop in water levels in the canal, no relief has to be taken in to account. In this case, multiple breaches may occur in a regional flood defence system as the water levels will remain extreme after the initial breach. This will not have consequences for the damage in the polder which has already flooded, but can lead to breaches in other polders.

WITH RELIEF

Relief can be taken in to account if a breach in the system results in a significant drop of the water levels in the surrounding canal. In this case, the probability of multiple breaches due to extreme water levels in one polder can be neglected, because these water levels will not occur after the breach. The maximum amount of breaches in the system is then equal to the amount of canal systems surrounding the polder.

Note that within one canal system several breaches may still occur when the canal system surrounds several polders. Relief is only taken in to account in one polder. Furthermore, due to the reduced water levels after the initial breach, the outer slopes of the flood defences may become unstable and slide. This is an extra form of damage, which has to be taken in to account on the consequence side.

Moreover, if the outer slope of flood defences of other polders fail due to the reduced water levels, these polders may also flood if the water levels increase again. This may happen if the canal is compartmented after an initial breach. Compartment works need to be chosen outside the influence area of the initial breach so no water will overflow locations where the outer slope has failed due to relief after the initial breach. This is an important point that has for flood defence managers deciding upon breach closure measures or compartments in canals during a calamity, this will be treated in more detail in chapter 7.

3.3 CONSEQUENCES OF FLOODING

Consequences of flooding consist of economic damage and life loss. The consequences are estimated by combining flood prediction models with consequence estimators. They depend on the inundation depth and flow through the inundated area. Several 'state of the art' methods are available, which will be explained in the following section. The following sub components are required in the assessment of the consequences of flooding.

1 **Schematization of flood scenarios**: as explained in the last paragraph distinction is made between groups of sections which together lead to similar flood damage, irrespective of the breach location within the group.
2 **Flood modelling**: for every flood scenario the resulting inundation is modelled with flood maps showing the water levels in the flooded area. Flood prediction models, such as SOBEK 2D and Delft 3Di, are used to model the water levels after flooding.
3 **Consequence estimates**: based on the resulting inundation depth of the flooded area estimates are made of the consequences of the flood. Methods such as the 'Schade en Slachtoffer Module' (HIS SSM) and the 'Waterschade Schatter' (WSS) are available to determine these consequences.

3.3.1 FLOOD MODELLING

The amount of water flooding the protected area is dependent on the total volume in the surrounding canal. It can be assumed that floods resulting from a breach in a regional flood defence system only have depths in the order of decimetres, except for the occasional deeper polders. The resulting inundation of the protected area is dependent on the surface area levels. Several methods are available to determine the resulting water levels in the protected area. The most widely used method in the Netherlands is SOBEK. The following information is required to determine these water levels:

- The location and size of the breach, which will determine which canal systems supply water to the breach.
- The size and growth over time of the breach determines the flow through the breach.
- The volume of water in the canal systems which supply water to the breach.
- Geo-information of the protected area: the surface area, surface levels and land use.

Compartment works can be used to close off large parts of the canal system limiting the inflow of water and reducing the flood inundation. At this stage these compartment works will not be included to make a comparison between both options possible.

WILNIS BREACH

An example of a breach in a regional system is the dike breach at Wilnis, where a regional flood defence failed due to horizontal sliding of part of the dike. The inflow of water in the protected area was stopped after 4 hours by successfully closing of the canals at three locations. The closure locations were chosen such that no other polders were flooded, even though several outer slopes along the system had failed due to the reduced water levels. The resulting inundation was limited to several decimetres, this still resulted in damages of 16 million euro (Wolthuis, 2011): 15 million euro to the infrastructure and 1 million euro to houses. 1500 inhabitants were successfully evacuated, there were no casualties.

FIGURE 18: REGIONAL FLOOD DEFENCE BREACH AT WILNIS, 2003 (SOURCE: KOENSUYK PHOTOGRAPHY)

3.3.2 CONSEQUENCE ESTIMATES

The consequences of floods depend on the inundation depths in the protected area and the flow through the protected area, both horizontally and vertically. Due to the low expected inundation depths no loss of life is taken in to account at this stage. The economic consequences are divided in two groups:

1. **Direct consequences,** which is material damage due to the contact with water, such as damage to cars, buildings and infrastructure.
2. **Indirect consequences,** which are losses due to down time of the affected industries. Both the industries inside the flooded area as well as industries outside the flooded area affected by the floods are taken in to account.

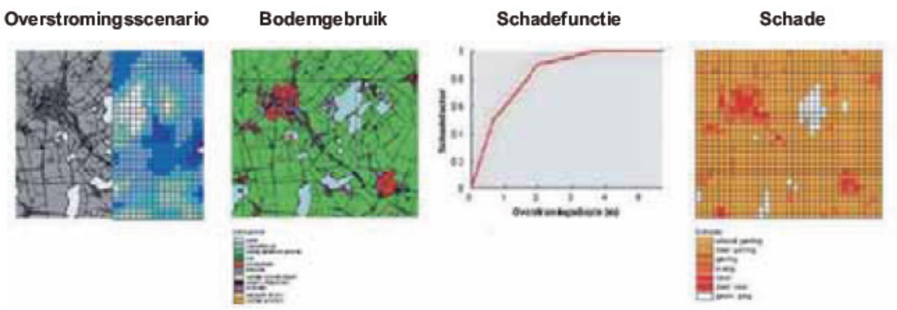
In the Netherlands two tools are available to make consequence estimates of floods: HIS SSM and WSS. When using the same flood simulations the difference in consequence estimates is a direct result of the difference between both estimators. A comparison of both methods is made.

HIS SSM

HIS SSM is used in 'Veiligheid Nederland in Kaart' to determine the consequences of floods due to breaches in the primary flood defence system. A grid of 25m • 25m is used wherein the resulting water levels are determined. Consequences functions are made to determine the consequences for different types of land use in the affected area (M Kok, Lammers, Vrouwenvelder, & van den Braak, 2006).

The HIS SSM model provides an estimate of the economic consequences and loss of life for large floods, with inundation depths of several meters. A disadvantage of the model is that it is inaccurate for low inundation depths.

FIGURE 19 COMPONENTS TO DETERMINE FLOOD CONSEQUENCES WITH HIS SSM (M KOK ET AL., 2006)



WSS

The 'Water Schade Schatter' is developed to determine the consequences of small floods due to heavy rainfall in polders. Compared to HIS SSM, a smaller grid of 0.5m • 0.5m is used. This grid is combined with more accurate consequence functions for low inundation depths of several decimetres. A major disadvantage of this model is that the consequence functions are limited to inundation depths below 0.3 meter. For infrastructure and crops, the WSS assumes that the maximum damage to infrastructure and crops occurs at inundation depths of 0.05 meter combined with a duration of minimal 48 hours.
To correct the method for larger inundation depths, the consequence functions of buildings need to be changed, as the current functions only account for damages to floors and limited damage to paintwork. With larger inundation depth more damage will occur to buildings than currently estimated.

CONCLUSIONS

Both methods will be used and compared in this report. Comparisons with similar flood simulations of SOBEK 2D showed that the difference between both estimators can reach up to 20%. No clear distinction can be made of which estimator provides an upper or lower limit.

3.4 FLOOD RISK ASSESSMENT AND COST BENEFIT ANALYSES

The flood risk for the different scenarios is obtained by combining the scenario probabilities with the scenario consequences. Cost benefit analyses have long been used in the Netherlands to inform policy debates about the current and optimal safety levels of flood defences (R. B. Jongejan, Jonkman, & Vrijling, 2012)(Eijgenraam, 2006). Furthermore, cost benefit analyses can also be used to prioritize interventions in the system (R. Jongejan & Maaskant, 2013), as the weakest sections within a system may very well not be the sections where interventions are most cost effective. These methods are currently being used to determine new safety standards for the primary flood defences in the Netherlands.

Similar analyses can be made to determine the optimal safety levels of regional flood defences and/or prioritize required interventions in the system based on cost effectiveness. Currently interventions in the system are based on the assessment of regional flood defences which only determines which dike sections do not comply with the current safety standards. The calculated safety factor gives insight in the size of the safety gap of the sections which do not comply; interventions in the system are often based on the size of these safety gaps. However, the weakest sections within a system may very well not be the sections where interventions are most cost effective, which is a major disadvantage of the current approach. To make cost benefit analyses insight is required in the cost of interventions in the system and the benefits of these interventions. In these cases, the benefits consist of a reduction of the flood risk. In regional flood defence systems several interventions are possible which are partly described in (Keizer, 2008). Interventions in the system can aim to reduce the probability of a flood or to reduce the damages of flooding.

Examples are:
- **'Classical' reinforcements**: these interventions aim to reduce the failure probability of the flood defence by increasing the strength;
- **Increase drainage capacity**: by increasing the drainage capacity in the canals lower water levels can be maintained resulting in lower loads on the flood defences;
- **'Emergency' measures**: these are temporary measures aimed to increase the strength or reduce the loads of a flood defence during extreme situations. Examples are placing sand bags on top of the dike to increase the retaining height, reducing water levels in the canals through compartment works and/or retention ponds or restricting traffic loads on top of flood defences (Lendering, Kok, & Jonkman, 2013);
- **Consequence reducing measures**: compartment works in the canals can also limit the amount of inflow through a breach resulting in less damage. Another consequence reduction measure consists of moving critical infrastructure and buildings to higher ground to avoid these being flooded after a breach;

Combinations of the proposed interventions are also possible. The framework developed in this report provides insight in the flood risk of regional flood defence systems. These insights can be used to determine the risk reduction of these interventions. Based on the results of the case study some examples of cost benefit analyses of several interventions will be demonstrated. These analyses will be based on cost estimates, as a complete study of the effectiveness of these measures is beyond the scope of this project.

3.5 CONCLUDING REMARKS

Flood risk is assessed by the annual expected damage due to flooding, which is estimated by multiplying the probability of flooding with the consequences of flooding. Therefore, insight is required in the failure probability of the flood defence system as well as the consequences after flooding. The largest knowledge gaps exist on the probability side, because several 'state of the art' models are available to determine the flood consequences due to breaches in a flood defence system.

The probability of failure of a system of flood defences is determined based on a schematization of the system, which divides it in several sections with similar strength characteristics. The statistical properties of the reliability function are assumed fully correlated over the length of one section. This allows us to assume that the failure probability calculated for one cross section is representative for the whole dike section. Probabilistic methods are then used to determine the probability of failure of the sections, taking uncertainties in load and strength in to account. These uncertainties will be discussed in the next two chapters.

For every dike section the failure probability is calculated for each failure mechanism, after which these can be combined to obtain the failure probability one section. To obtain the probability of one flood scenario, the failure probabilities of individual dike sections are combined. Groups of sections are modelled as a series system, where the upper limit of the failure probability is the summation of the individual failure probabilities. The lower limit is the maximum of the failure probabilities of the individual sections.

The occurrence of relief in the system needs to be investigated when combining failure probabilities of dike sections. This depends strongly on the volume of water inside the canal compared to the size of the inundated area after a breach. If a breach in the system does not result in a significant drop in water levels in the canal, no relief has to be taken in to account. In this case, multiple breaches may occur in a regional flood defence system as the water levels will remain extreme after the initial breach.

Relief can be taken in to account if a breach in the system results in a significant drop of the water levels in the surrounding canal. In this case, the probability of multiple breaches due to extreme water levels in one polder can be neglected, because these water levels will not occur after the breach. The maximum amount of breaches in the system is then equal to the amount of canal systems surrounding one polder.

It can be assumed that floods resulting from a breach in a regional flood defence system only have inundation depths in the order of decimetres, except for the occasional deeper polders. To determine flood consequences, both HIS SSM and WSS will be used and compared. No loss of life is taken in to account due to the low expected inundation depths and flows through breaches in regional flood defences.

The methodology used in this report provides insight in the flood risk of regional flood defence systems; it can be used to make cost benefit analyses of proposed interventions, which aim to reduce flood risk.

4

UNCERTAINTY IN LOADS ON REGIONAL FLOOD DEFENCES

4.1 INTRODUCTION

The probability of failure of structures is determined based on uncertainties in both the load and strength parameters. This chapter discusses the uncertainties in loads on regional flood defences, which consist of hydraulic loads (water and waves) and other non-hydraulic loads. This paragraph discusses all loads and determines which are governing. In the following paragraphs, statistical distributions will be determined for the governing loads, which are used to determine the failure probability of the structures. Finally, the joint probability of the governing loads is discussed.

4.1.1 HYDRAULIC LOADS

Hydraulic loads consist of the static water pressure in the canal, determined by the water levels, waves in canals and ice loads. These are treated in the following bullets:

- **Water levels** in the canals are influenced by:
    - Inflow from the polder pumping stations and rainfall. The inflow is largely dependent on the discharge from the polder pumping stations, which in turn depends on the amount of rainfall in the polder.
    - Outflow from the canals through the larger pumping stations, to the 'outside water': rivers or sea. Outflow to groundwater is also possible, which is called 'hydraulic short circuiting'. This occurs when the water in the canal is in direct contact with the groundwater in the aquifer below the canals (Kwakman, Doeke Dam, & van Hemert, 2013). In general, the bottom of the canals consist of impermeable soil (clay or peat), resulting in little outflow to the groundwater.
    - Wind set up can result in locally increased or reduced water levels. This may occur in longitudinal or transversal direction (over the length or width of the canal).
- **Ground water levels** in the earthen dikes influence the stability of the inner and outer slope of these dikes due to reduced soil pressures. These are influenced by the type of soil in the dike, the water levels in the canals and rainfall.
- **Waves** on the canals can be the result of wind and/or shipping. The size of wind waves depends on the wind speed, wind direction and the geometry of the canals: in particular the fetch and depth determine the size of wind waves. It is assumed that waves in regional canals are small; the resulting loads are negligible and will not lead to breaches in the flood defences.
- **Drought**: peat dikes suffer from a loss of volumetric weight as a result of long periods of drought (peat drying out). The weight of peat is largely influenced by the presence of water in the pores; long dry periods thus result in a loss of strength of the peat soil and as such form a threat to the stability of the dike.

- **Ice**: ice on top of the canals can cause damage to the outer layers of the flood defences. They are not taken in to account in the assessment of regional flood defences, because of the low (negligible) probability of occurrence of ice loads during high water levels on the canals.

4.1.2 NON-HYDRAULIC LOADS

- **Wind** loads do not have a direct impact on a regional flood defence. Wind can cause wind waves, which were treated in the section 4.1.1.
- **Traffic loads** on top of regional flood defences have to be taken in to account, as roads are often built on top of these defences. An equivalent load of 5 kN/m^2, for small cars, or 13 kN/m^2, for large trucks, is taken in to account in the assessment of these flood defences (Stowa, 2007)
- **Earthquakes** can be harmful for regional flood defences, as was concluded recently in (Visschedijk et al., 2014) for regional flood defences in Groningen (near the gas extraction plants). Currently research on this subject is undergoing ((Visschedijk et al., 2014)), this load case is only considered in areas where earthquakes are likely to occur.
- **Biological degradation**: as a result of biodegradation holes can develop in either the inner or outer (protection) layers of the flood defence. These are not taken in to account in the assessment, as it is assumed water boards can repair these damages before they lead to instability of the dike.
- **Ship collision**: canals are used for commercial shipping resulting in the possibility of a collision with the regional flood defence. In the assessment, ship collision is only taken in to account in outer turns of larger canals for ships larger than 1,000 tons.
- **Calamities**, such as terrorism, vandalism or attacks during wars (deliberate breaching of flood defences), are not taken in to account in the assessment.

4.1.3 GOVERNING LOADS

Only the governing loads are taken in to account in the in the assessment of regional flood defences (Stowa, 2007), which are:

1. Hydraulic loads: water levels inside the canals and resulting groundwater level;
2. Traffic loads: as vertical loads on top of the flood defence;
3. Drought: due to long dry periods peat soil partly loses its strength.

Hydraulic and traffic loads are treated in the following paragraphs. Currently a research program is undergoing on the stability of peat dikes during drought. This load is not taken in to account at this stage, because the results of undergoing research are expected to largely influence the assessment of regional flood defences.

Even though research concluded that earthquakes form a threat to the stability of regional flood defences, this load case will not be considered in this report. A case study is chosen for a region where earthquake loading is not present. Furthermore, ship collision is considered an unlikely event and will not be taken in to account at this stage of the project. Whether or not this results in unreliable estimates requires further investigation.

4.2 HYDRAULIC LOADS

This paragraph discusses the statistical models used to describe the water levels in the canals. The probability distribution of water levels along the primary flood defences is estimated based on data of observed water levels and/or river discharges. A type 1 extreme value distribution (Gumbel distribution) is used to describe probability that the water level exceeds a certain threshold value within a certain period, which is usually an annual maximum (Meer, 2009), see equation 10.

$$F_y(X) = P(X \leq x) = e^{-e^{\frac{-(x-\alpha)}{\beta}}} \tag{6}$$

x is the water level

a is a location parameter on the horizontal x axis

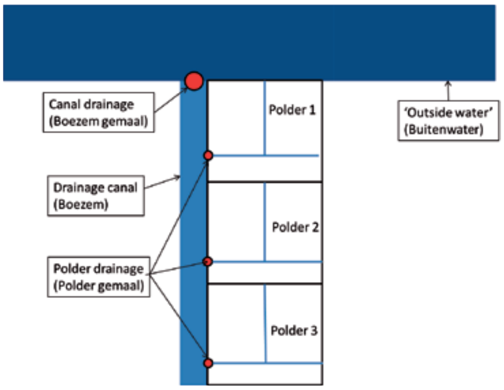
b is a measure for the spread of the distribution

This approach is used for water levels which are caused by natural events. The water levels in the canals treated in this report are regulated and therefore not completely caused by natural events. A different approach is required to determine the statistical distribution of these water levels, which will be explained in the following sections.

4.2.1 SYSTEM DESCRIPTION OF WATER LEVEL REGULATION IN CANALS

The volume of water in the canals is determined by the inflow from the polder drainage stations, the amount of rainfall on the canals, outflow through the canal drainage stations and outflow due to seepage. Wind set up can cause local increased or reduced water levels.

FIGURE 20 SCHEMATIC VIEW OF POLDER CANAL SYSTEM



We consider the volume of water in the canals being governed by the inflow from the polder drainage stations and the outflow to the outside water, neglecting rainfall, seepage and local wind set up. The remaining aspects which determine the volume of water, and thus the water levels, in the canals are inflow and outflow through the pumping stations. These water levels are maintained at a certain target level by the water boards.

The governing water level, corresponding with the required safety standard of the regional flood defence, is determined with a statistical analysis of observed water levels. During extreme rainfall events, the canals have a certain storage volume available for storage of excess water out of the polders pumped on to the canals. This volume is determined by the difference between the target water level and the governing water level. This level is also called the 'drain stop level' and is defined as the maximum allowed water level on the canals. As explained earlier, these water levels are regulated by the water boards. Once the 'drain stop level' in the canals is reached, the polder drainage stations are not allowed to keep pumping water on to the canals. This may result in flooding of the polder, if it has insufficient storage capacity. The probability of these floods in the polders is bound by safety standards; this event may only occur with a probability of 1/100 per year. During heavy rainfall events, water

boards may have to choose between having to exceed the ‘drain stop level’ on the canals to keep polders dry, or vice versa.
In the past, there have been cases where the water levels on the canals exceeded the ‘drain stop level’; apparently the polder drainage stations were not turned off resulting in higher water levels in the canals. This event will be called ‘failure of the drain stop’ and has to be taken in to account in a risk assessment. In the following paragraph, a statistical analysis is made of the water levels in the canals taking ‘failure of the drain stop’ in to account.

4.2.2 STATISTICAL ANALYSIS OF CANAL WATER LEVELS

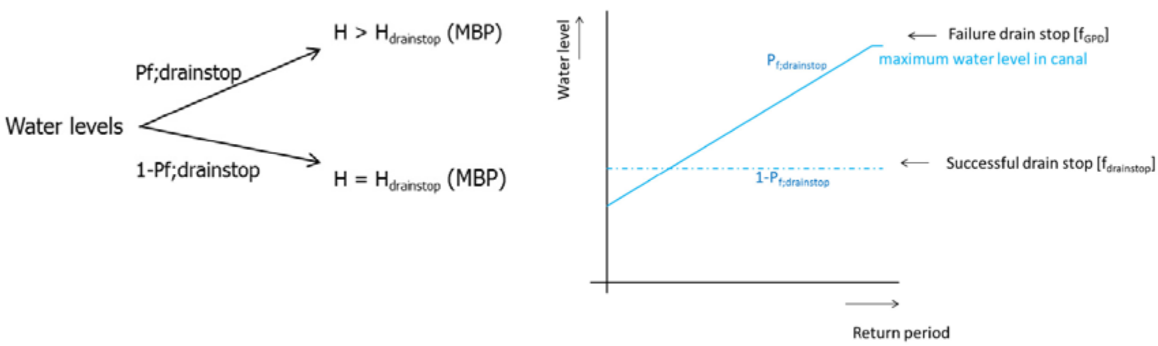
In summary, the water levels are influenced by several factors such as the inflow from the polders, outflow to the outside water, rainfall, seepage, wind set up and waves. Depending on the considered canal system, several of these factors may or may not be governing. The probability of flooding is represented by an annual probability, which means that the annual exceedance frequencies of the water levels on the canals are required to determine this probability of flooding. These will be determined using data of observed water levels in the system. For these canal systems, hourly water level observations are available. These will be used to generate independent peaks of the observed water levels, with a Peaks Over Threshold method. A Generalized Pareto Distribution is then fitted through the data, which provides an initial probability distribution of the water levels.

As explained earlier, the water levels in the canal may not exceed the ‘drain stop level’. In the following statistical analysis distinction will be made between two events (see Figure 21):

- Successful drain stop: the water levels remain at or below the drain stop level
- Failure of the drain stop: the water levels exceed the drain stop level;

During successful drain stop events, the water levels will not exceed the drain stop level. This is represented by the dotted blue line ($f_{drainstop}$) in Figure 21; the distribution is truncated at this level. If the drain stop fails, the water levels will keep increasing above the ‘drain stop level’, represented by the straight blue line (f_{GPD}). This straight blue line is actually the GPD fit through the data of observed water levels. Water levels keep increasing to a certain maximum, which is the maximum water level in the canals; higher water levels will lead to overflow over the regional flood defences.

FIGURE 21 METHOD FOR WATER LEVEL STATISTIC (LEFT) AND RESULTING ANNUAL EXCEEDANCE LINES (RIGHT)

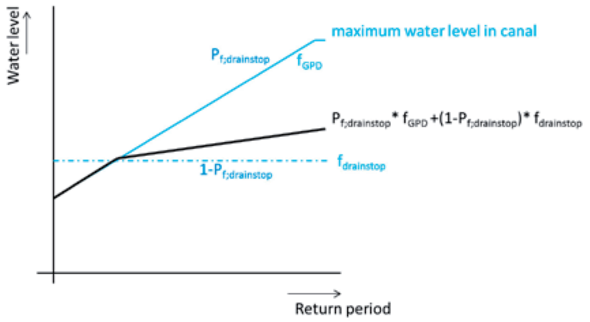


The probability of failure of the drain stop [$P_{f;drainstop}$] can be estimated with the water level observations on the canal. The amount of times the water level exceeded the ‘drain stop level’ provides information about the probability of failure. This information is used to generate a combined probability of exceedance line ($f_{h;regulated}$) for the regulated water levels. The two fits are combined, each with their probability of occurrence, according to equation 7.

$$f_{h;regulated} = P_{f;drain\ stop}\, g f_{GPD} + \left(1 - P_{f;drain\ stop}\right) g f_{drain\ stop} \qquad (7)$$

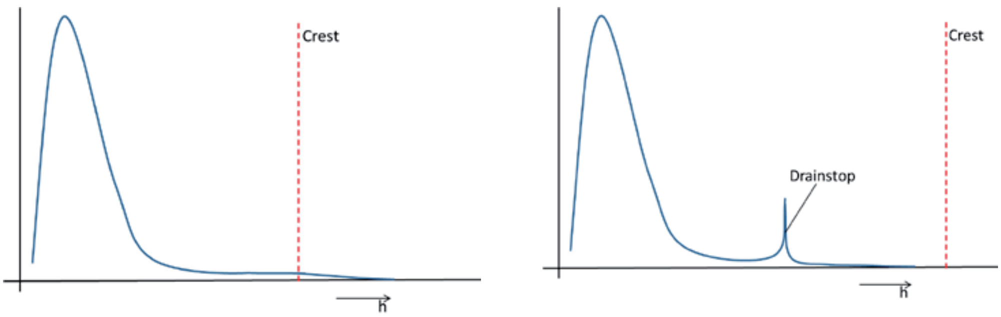
The resulting exceedance line is shown in black in Figure 22.

FIGURE 22 **COMBINED EXCEEDANCE FREQUENCIES OF REGULATED WATER LEVELS ON CANALS**



The resulting empirical distribution of the regulated water levels will have a peak at the drain stop level, because a large percentage of the original distribution remains at this level for a successful drain stop. The following figure shows the difference between the probability density function of water levels on rivers or at sea, compared to the empirical distribution of the regulated water levels in the canals. Notice the second peak in the pdf of the canals, at the drain stop level and the location of the dike crest.

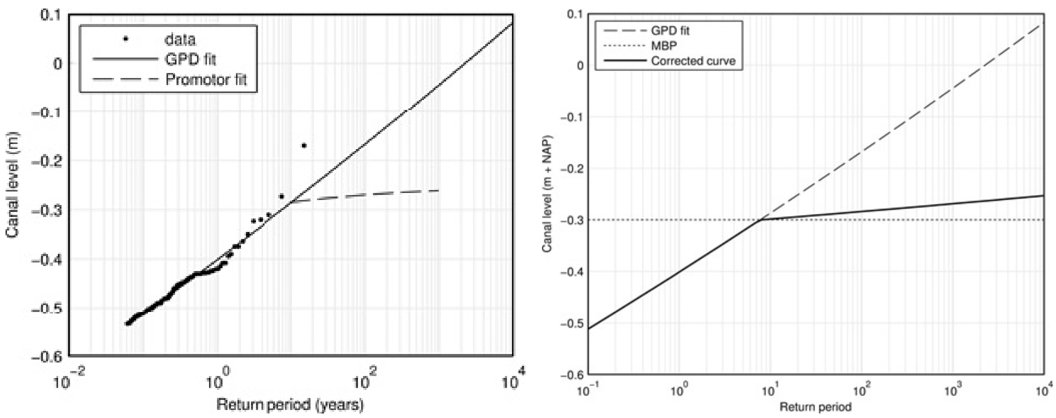
FIGURE 23 **DISTRIBUTION OF WATER LEVELS PRIMARY FLOOD DEFENSES (LEFT) AND REGIONAL FLOOD DEFENSES (RIGHT)**



GOVERNING WATER LEVELS

In (Kramer & van Veen, 2013) a comparison is made of several methods to determine the governing water level on the canal. One method often used is ‘Promotor’ , which is a probabilistic model developed by HKV (Kramer & van Veen, 2013). In this model rainfall and wind are taken in to account to predict the probability of extreme water levels in a system of regional flood defences. However, failure of the drain stop is not included. The data obtained with Promotor will be compared with the exceedance lines determined with the previously described method in the case study. An example is shown in the following figures.

FIGURE 24 WOGMEER STATION GPD FIT (LEFT) AND RESULTING EXCEEDANCE LINE FOR WATER LEVELS



4.3 TRAFFIC LOADS

The combination of hydraulic loads (high water levels) and traffic loads can be governing for the stability of the flood defence. According to the assessment of regional flood defences, traffic loads are advised to be taken in to account as a permanent vertical load on top of the flood defence (Stowa, 2007). This load can lead to water overpressure inside the flood defence, which results in a decrease of the effective soil pressures. The extent to which the water over pressure develops depends on the type of flood defence and the duration of the loads.

4.3.1 ONGOING RESEARCH ON TRAFFIC LOADS

The STOWA is currently doing research on the traffic loads on regional flood defences, some preliminary conclusions are included below (Kwakman, 2013):

- The increase of water pressure in historically heavy loaded defences only gradually builds up in time and reached its maximum after about 7 to 9 hours;
- Moving vehicles have no influence on the flood defences, dynamic forces due to moving vehicles can therefore be omitted in the reliability assessment;
- For peat dikes a degree of consolidation of 60% can be used, for clay dikes 80%.

According to the assessment of regional flood defences, a temporary top load with a magnitude of 13,3 kN/m^2 over a width of 2.5 meter and an infinite length has to be taken in to account. This represents 40 ton trucks standing in line on top of the flood defence, each with a length of 12 meter. Preliminary results indicate that this top load is not realistic. Trucks will not stand in line on the flood defence, as was assumed in the original assessment, because they have to travel back and forth on the flood defence. A temporary top load with the same magnitude, but over a finite length of 12 meters is assumed to be more realistic. This represents one truck on top of the flood defence (Kwakman, 2013).

4.3.2 STATISTICAL DISTRIBUTION OF TRAFFIC LOADS

Statistical distributions of the traffic loads are required to determine the failure probability of the flood defence. The influence of traffic loads on other civil structures (e.g. bridges) has been studied by Morales-Napoles et al (2014), but flood defences were not considered in these studies. Moreover, these studies consider dynamic (moving) traffic loads whereas the traffic load on regional flood defences is modelled as a temporary (static) top load.

Expert ellicitation was used to determine the statistical distribution of traffic loads, due to lack of other sources of data for the traffic loads. Water board employees responsible for the assessment of the regional flood defences were interviewed to obtain estimates of the statistical distribution of the traffic loads. They were asked to provide estimates of the 5th, 50th and 95th quantiles of the statistical distribution of the traffic loads. Furthermore, they were asked to provide an estimate of the correlation between the traffic load and water level. The questionnaires are included in appendix A.

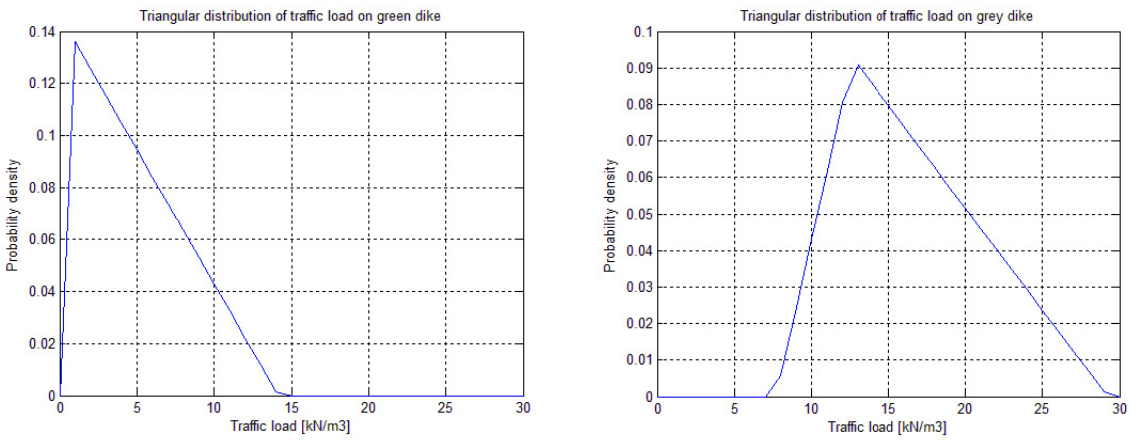
The resulting traffic loads and correlations are shown in Table 7. The experts all agreed that distinction has to be made between a green and grey flood defence, meaning a flood defence without a road on top and one with a road on top. Green defences will only have small traffic loads, corresponding with small cars for maintenance. On grey defences large traffic loads can be expected, corresponding with the loads advised by the assessment. No correlations between the traffic load and water levels is expected with average water levels; the experts all agreed on this point. However, they did not agree on the correlation between the traffic load and the extreme water levels, as can be seen in the table. The correlations and joint probability of all loads acting on regional flood defences is further discussed in the next paragraph.

TABLE 7 **RESULTS EXPERT ELLICITATION TRAFFIC LOADS**

Type	Average water level	Extreme water level	5th quantile kN/m^2	50th quantile kN/m^2	95th quantile kN/m^2
Green flood defence	No correlation	No / negative correlation	0.5	2	5
Grey flood defence	No correlation	No / positive correlation	10	15	25

The expected quantiles of the statistical distributions of traffic loads provided a basis to determine triangular distributions for these loads, which are show in Figure 25.

FIGURE 25 **TRIANGULAR DISTRIBUTIONS OF TRAFFIC LOADS ON GREEN (LEFT) AND GREY (RIGHT) FLOOD DEFENCES**



4.3.3 APPROACH TO INCLUDE TRAFFIC LOADS

The experts are aware of the fact that traffic loads reduce the stability of the flood defences. They argue that it is not so much the combination of water levels and traffic loads leading to instability, but the combination of high phreatic lines and traffic loads. Extreme water levels lead to high phreatic lines, but rainfall has much more influence on the phreatic line. Rainfall may lead to high phreatic lines even when the water level in the canal is still at its target level. This may lead to instability when traffic is allowed on top of the flood defence. This is very different from the primary flood defences, where the phreatic lines only increase during extreme water levels, when traffic is restricted on primary flood defences.
In conclusion, we will determine the probability of failure of the regional flood defences for two situations: one with traffic load and another without. Using this methodology, we hope to show the influence of the traffic loads on the failure probability and risk of regional flood defences.

4.4 CORRELATIONS BETWEEN LOADS

To properly calculate failure probabilities for regional flood defences, insight in the correlations between the governing loads is required. The stability of the dike is influenced by several loads, which are summed up below:

- Water level;
- Traffic load;
- Drought;
- Rainfall;
- Phreatic line.

Different combinations of these loads are possible, which partly depend on the management of the water board. Specifically the policy regarding traffic loads on a regional flood defence determines which combinations of loads are most likely to occur. According to the assessment of regional flood defences traffic loads are advised to be taken in to account, to allow trucks to supply sand bags to weak sections (i.e. sections with insufficient retaining height) of the dike. However, several water boards have acknowledged that they restrict traffic loads on defences for which the stability becomes critical if they are allowed. Which policy is chosen depends on the management of the water board; we therefore describe both situations in the following sections.

4.4.1 ALLOWING TRAFFIC LOADS

This section describes the load combinations which are most likely to occur in an area managed by a water board who allows traffic loads on regional flood defences. The following table shows a qualitative analysis of correlations between the governing loads, which is meant to provide insight. No data was available to compute joint probabilities (until now).

TABLE 8 CORRELATION MATRIX OF LOADS ON REGIONAL FLOOD DEFENCES WHEN ALLOWING TRAFFIC LOADS

	Average water level	Extreme water level	Traffic load	Rainfall	Drought	Phreatic line / dike saturation
Average water level						
Extreme water level	+					
Traffic load	0	+				
Rainfall	+	+	0			
Drought	-	-	0	-		
Phreatic line / dike saturation	+	+	+	+	-	

- *Extreme water* levels are positively correlated with average water levels.
- Following the results of the expert ellicitation session on *traffic loads*, no correlation is expected between traffic loads and average water levels. However, we do expect a positive correlation with extreme water levels, as in this case it can be expected that sand bags will be required on top of the flood defence resulting in traffic loads.
- *Rainfall* is positively correlated with the water levels, as rainfall will lead to an increase of water in the canals. The occurrence of rainfall is not correlated with the occurrence of traffic loads, as these are seen as independent events.
- *Droughts* are the inverse of rainfall, as they occur when there is little to no rainfall. Therefore, droughts are negatively correlated with water levels and rainfall. Furthermore, droughts are not correlated with the occurrence of traffic loads, as these are independent load cases.
- The *phreatic line / saturation* of the flood defence is positively correlated with the water levels in the canal. Furthermore, it is also positively correlated with the traffic load, as these will take place when the dike is loaded by extreme water levels, which lead to saturation of the dike. Following the argumentation of droughts given before, the phreatic line is negatively correlated with this load case.

4.4.2 RESTRICTING TRAFFIC LOADS

If a water board's policy is to restrict traffic loads in regional flood defences, a different correlation matrix is found, as can be seen in Table 9. In this case traffic loads are negatively correlated with extreme water levels and the phreatic line / saturation of the dike. During these events the water board will restrict traffic on the flood defence, because it reduces the stability of the flood defences.

TABLE 9 CORRELATION MATRIX OF LOADS ON REGIONAL FLOOD DEFENCES WHEN RESTRICTING TRAFFIC LOADS

	Average water level	Extreme water level	Traffic load	Rainfall	Drought	Phreatic line / dike saturation
Average water level						
Extreme water level	+					
Traffic load	0	-				
Rainfall	+	+	0			
Drought	-	-	0	-		
Phreatic line / dike saturation	+	+	-	+	-	

We will determine the failure probability of regional flood defences for both situations, to show the effect traffic loads have on the flood risk of the system.

4.5 CONCLUDING REMARKS

Hydraulic and traffic loads are taken in to account in our methodology. Currently a research program is undergoing on the stability of peat dikes for droughts and earthquakes. These loads are not taken in to account at this stage. To determine failure probabilities of structures, insight is required in the statistical distribution of the loads.

Only the water levels are taken in to account, as waves are neglected due to their small size. The water level inside the canals in polders is regulated by the inflow from the polder, the outflow to the outside water and wind setup. Each canal has a 'drain stop level'; once this level is reached no more water is pumped on to the canal. Water level observations are used to generate independent peaks of water levels in these canals, through which a Generalized Pareto Distribution is fitted. This distribution is modified, to account for the regulation of water levels in the system, by making a distinction between two events:

- Successful drain stop: the water levels remain at or below the drain stop level
- Failure of the drain stop: the water levels exceed the drain stop level;

The probability of failure of the drain stop [$P_{f;drainstop}$] is estimated with the amount independent water level peaks which exceeded the 'drain stop level'. This probability is used to generate a combined distribution ($f_{h;regulated}$) for the regulated water levels, see equation 7 and Figure 21.

The combination of hydraulic loads (high water levels) and traffic loads is governing for the stability of the flood defence. Expert ellicitation was used to determine a triangular statistical distribution of traffic loads on regional flood defences. The experts agreed that these loads can threaten the stability of regional flood defences during both average and extreme water levels. Different combinations of these loads are possible, which partly depend on the management of the water board. Specifically the policy regarding traffic loads on a regional flood defence determines which combinations of loads are most likely to occur.

We will determine the probability of failure of the regional flood defences for situations with and without traffic loads, to show the influence of these loads on the probability of failure and flood risk.

5

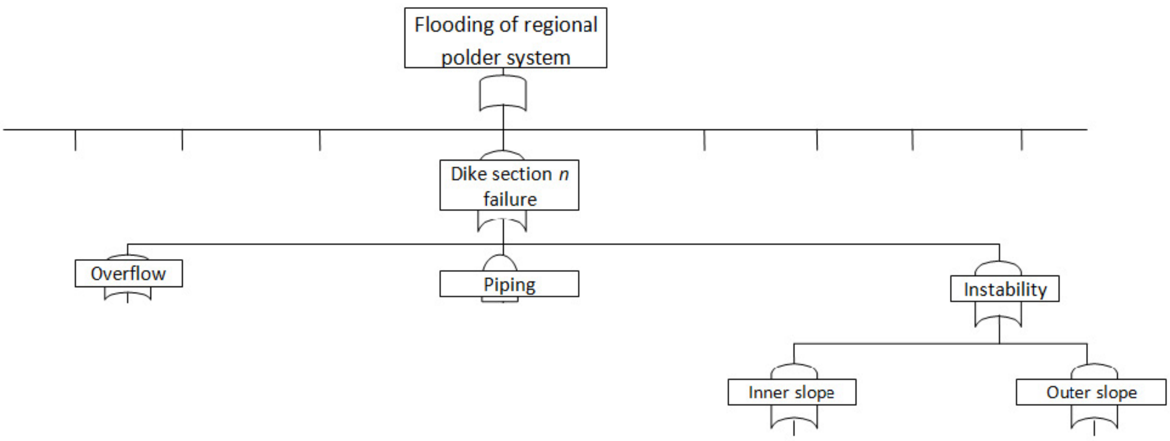
UNCERTAINTY IN STRENGTH OF REGIONAL FLOOD DEFENCES

5.1 PROBLEM APPROACH

This chapter discusses a methodology to determine the probability of flooding of a regional flood defence system, which depends on several failure mechanisms (see Figure 26). Fault tree analysis and probabilistic calculations will be used.

In primary flood defences, the 'Overtopping', 'Piping', 'Instability of the outer protection' and 'Inner slope instability' failure mechanisms are governing. Due to the different loads and strength properties (e.g. soil type) of regional flood defences, other failure mechanisms are governing, as is concluded with a quick scan of failure mechanisms in appendix B. Overflow, piping, horizontal sliding and instability of the inner slope will be taken in to account in the reliability analysis of regional flood defences, see Figure 26.

FIGURE 26 FAULT TREE FLOODING REGIONAL FLOOD DEFENCE SYSTEM



Methods to calculate the failure probability of dike sections are explained in the following paragraphs, as well as methods to combine failure probabilities. A bottom up procedure is followed:

- The probability of failure of the governing failure mechanisms overflow, piping and instability is determined for single dike sections (paragraph 5.2 and 5.3).
- The probability of failure of every dike section is combined to determine the probability of failure of a group of dike sections, based on similar consequences of flooding: the flood scenarios (paragraph 5.4).

5.2 LIMIT STATES OF GOVERNING FAILURE MECHANISMS

This section discusses the limit state functions of the governing failure mechanisms, which are used to make reliability calculations according to the methods explained in the following paragraphs. A typical cross section of regional flood defences ('boezemkaden') is shown in Figure 27; the geometrical parameters are discussed in Table 10. The uncertainty distributions used in VNK will also be used in this report (VNK, 2005). The variables required to solve the limit state functions are explained in the corresponding sections.

FIGURE 27 **GEOMETRICAL PARAMETERS OF TYPICAL REGIONAL FLOOD DEFENCE**

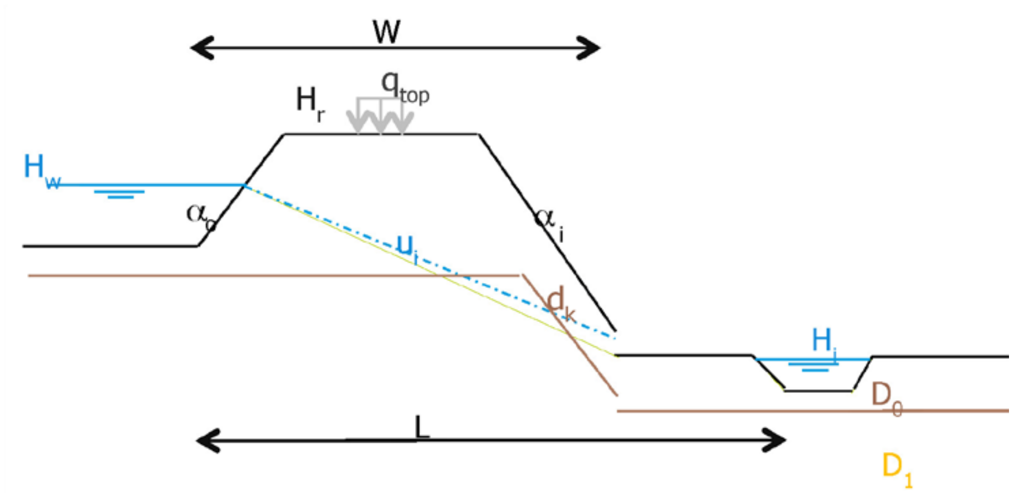


TABLE 10 **VARIABLES REGIONAL FLOOD DEFENCES** (VAN DER WOUDEN & GRASHOFF, 2009)

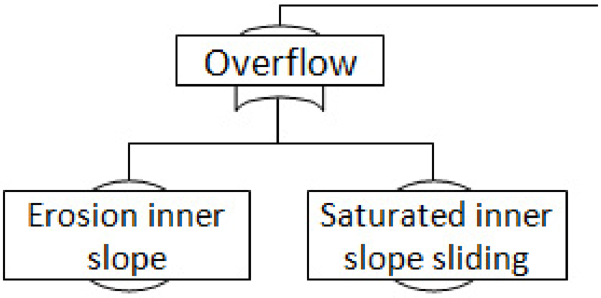
Variable	Type	Explanation	Unit	Distribution	Spread	Spatial scatter
H_w	Load	Water level in canal	m	Empirical	-	-
u_i	Load	Phreatic level inside flood defence	m	Empirical	-	-
H_i	Load	Water level in polder	m	Normal	CV = 0.1	
q_{top}	Load	Top load (e.g. traffic load)	kN/m²	Empirical	-	-
H_r	Geometry	Retaining level flood defence	m	Normal	CV = 0.1	300 m
H_t	Geometry	Level of toe flood defence	m	Normal	CV = 0.2	300 m
W	Geometry	Width of flood defence	m	Normal	CV = 0.15	300 m
L	Geometry	Seepage length	m	LogNormal	CV = 0.1	3000 m
d_k	Geometry	Thickness of cover layer inner slope	m	LogNormal	CV = 0.1	300 m
D_0	Geometry	Thickness of blanket layer	m	LogNormal	CV = 0.1	200 m
D_1	Geometry	Thickness of aquifer	m	LogNormal	CV = 0.1	200 m
α_1	Geometry	Slope of inner slope	-	Normal	CV = 0.05	150 m
α_1	Geometry	Slope of outer slope	-	Normal	CV = 0.05	150 m

5.2.1 OVERFLOW

Overflow will occur when the water levels in the canal exceed the retaining height of the surrounding flood defence, as is shown in Figure 28. When a flood defence is overflown, inner slope erosion or instability due to saturation of the dike can lead to breaching, see Figure 29. Instability due to saturation of the dike is taken in to account in the failure mechanism inner slope instability, so only erosion of the inner slope will be treated in this section.

FIGURE 28 OVERFLOWN DIKE IN ALPHEN AAN DE RIJN ON JULY 28TH 2014 (HART VAN NEDERLAND)

FIGURE 29 FAULT TREE FOR BREACHING DUE TO OVERFLOW OF A FLOOD DEFENCE



The limit state for inner slope erosion is described by equation 8.

$$Z_{erosion} = H_r + \Delta h_c - H_w \tag{8}$$

with: $\Delta h_c = \sqrt[3]{\frac{q_c^2}{0.36g}}$ (*critical depth* (van der Wouden & Grashoff, 2009)) (9)

The critical overflow amount depends on the overflow resistance of the inner slope cover layer. According to the assessment, a maximum amount of 0.1 litres per meter per second is allowed. However, recent tests have proved that well developed grass cover layers can resist much more. A critical overflow amount of 5 liter per meter per second will be used, leading to a critical overflow depth of 0.02 meter.

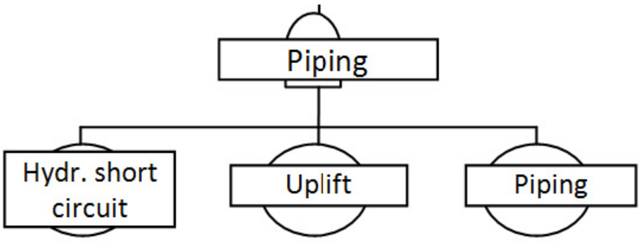
TABLE 11 OVERFLOW VARIABLES LIMIT STATE

Variable	Explanation	Unit	Distribution	Spread	Spatial scatter
h_c	Critical overflow depth	m	Deterministic	-	-
q_c	Critical overflow amount	$m^3/m/s$	Deterministic	-	-
g	Gravitational acceleration	m^2/s	Deterministic	-	-

5.2.2 PIPING

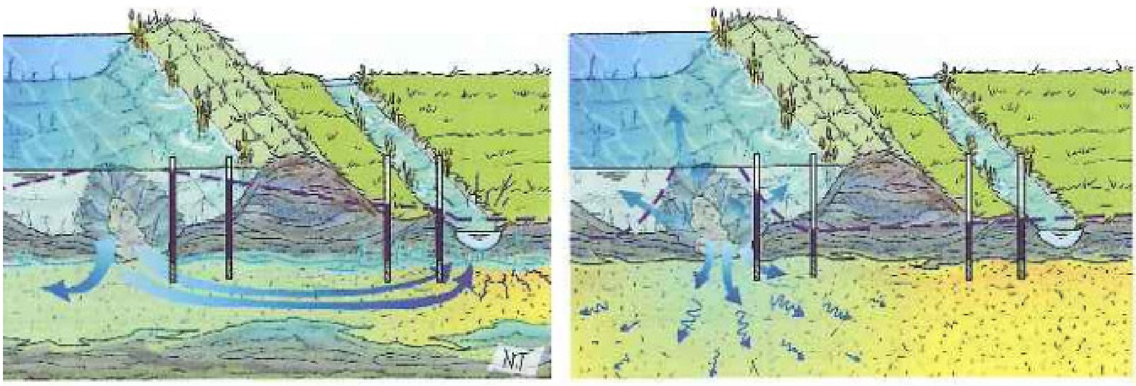
Piping occurs when a head difference over a flood defence causes uplift of the impermeable layer on the inland side, after which backward erosion forms channels or pipes in the aquifer under the flood defence. These channels can undermine the flood defence when they become so long that they connect the inner and outer water level. The water in the canals protected by regional flood defences is not always in direct contact with the aquifer, due to the presence of impermeable layers on the bottom of the canals. This may impede the occurrence of piping.

FIGURE 30 FAULT TREE FOR BREACHING DUE TO PIPING OF A REGIONAL FLOOD DEFENCE



The presence of these impermeable layers increase the resistance of intrusion of water from the canals to the aquifer below. For piping to develop, 'Hydraulic short circuiting' is required, which occurs when the impermeable layer is not present resulting in no increased resistance of intrusion from the canal to the aquifer. Hydraulic short circuiting may be the result of dredging works, uplift of peat layers in the bottom of the canal, leakage along revetments and/or horizontal displacements of the flood defence. These events may lead to a possible entry point of water in the canals to the aquifer under the canal, see Figure 31.

FIGURE 31 HYDRAULIC SHORT CIRCUITING IN REGIONAL FLOOD DEFENCES (KWAKMAN ET AL., 2013)



The response of the water pressure behind the dike to intrusion of water from the canal in the aquifer depends on the thickness of the aquifer. In thin aquifers (left picture in Figure 31) the response to intrusion of water from the canal in the aquifer is rather large, whereas in thick aquifers the response is small due to the size of the aquifer (right picture in Figure 31). We recommend measuring the resistance of intrusion due to the impermeable layers in the aquifer, to include the effect on the hydraulic head in the piping models. It is taken in to account as a reduction of the hydraulic head with a variable H_{ir} in the Z function for piping, which is based on the results of field tests, see equation 12.

UPLIFT

The limit state for uplift of the cover layer is described by equation 10. It is based on the balance of the weight of the cover layer with the upward water pressure due to the head difference over the flood defence.

$$Z_{uplift} = m_0 \cdot h_u - m_h \cdot (H_w - H_i) \tag{10}$$

$$with:\ h_u = \frac{\gamma_{nat} - \gamma_w}{\gamma_w} \cdot D_0 > 0 \tag{11}$$

PIPING

The limit state for piping is described by equation 12. The critical head difference $[H_p]$ can be determined with Bligh or Sellmeijer, equations 13 or 14.

$$Z_{piping} = m_b \cdot H_p - (H_w - 0.3 \cdot D_0 - H_i - H_{ir}) \tag{12}$$

$$H_{p;Bligh} = \frac{L}{C_{creep}} \quad (Bligh) \tag{13}$$

$$H_{p;Sellmeijer} = F1 * F2 * F3 * L \quad (Sellmeijer) \tag{14}$$

$$with \quad F1 = \eta\left(\frac{\gamma_s}{\gamma_w} - 1\right)\tan\theta_0 \ (15) \quad F2 = \frac{d_{70m}}{\sqrt[3]{\frac{\nu k L}{g}}}\left(\frac{d_{70}}{d_{70m}}\right)^{0.4} \ (16) \quad F3 = 0.91\left(\frac{D_1}{L}\right)^{\left[\frac{0.28}{(D_1/L)^{2.8}-1}+0.04\right]} \tag{17}$$

Bligh uses an empirical approach to determine the critical head difference, whereas Sellmeijer is a more elaborated formula based on material characteristics of the subsoil in the flood defence. In this report the updated Sellmeijer formula will be used, because it provides more accurate estimates of the failure probability. The uncertainty distributions of the variables are given in Table 12.

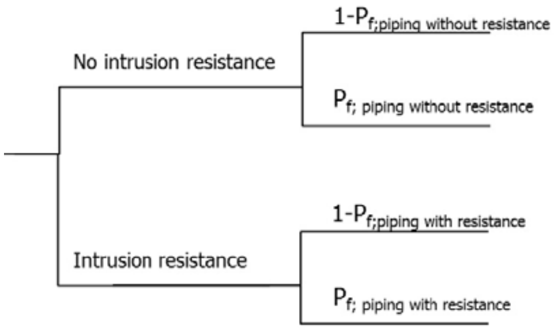
TABLE 12 **PIPING VARIABLES LIMIT STATE**

Variable	Explanation	Unit	Distribution	Spread	Spatial scatter
m_0	Model factor uplift	-	LogNormal	μ = 1 CV = 0.1	-
m_b	Model factor piping	-	LogNormal	μ = 1 CV = 0.12	-
y_s	Volumetric weight sand	kN/m^3	Deterministic	26.5	-
γ_{nat}	Wet volumetric weight subsoil	kN/m^3	Normal	CV = 0.05	300 m
γ_w	Volumetric weight water	kN/m^3	Deterministic	10	-
k	Permeability	m/s	LogNormal	CV = 1	600 m
d_{70}	Particle diameter top 70 % of subsoil	m	LogNormal	CV = 0.15	180 m
d_{70m}	Reference value for d70	m	Deterministic	2.08e-4	
θ_0	Angle of friction	°	LogNormal	37	-
η	White's constant	-	Deterministic	0.25	-
ν	Kinematic viscosity	m^2/s	Deterministic	$1.33*10^{-6}$	-
H_{ir}	Intrusion resistance	m	LogNormal	See case study	-

RISK ANALYSIS

In follow up research, we recommend including the effect of hydraulic short circuiting due to events such as dredging works. The piping probability can be computed for scenarios with and without the reduced head due to resistance of intrusion. With estimates of the probability of hydraulic short circuiting one can then compute the actual piping probability with the event tree shown in Figure 31.

FIGURE 31 MODEL FOR INCLUDING HYDRAULIC SHORT CIRCUITING

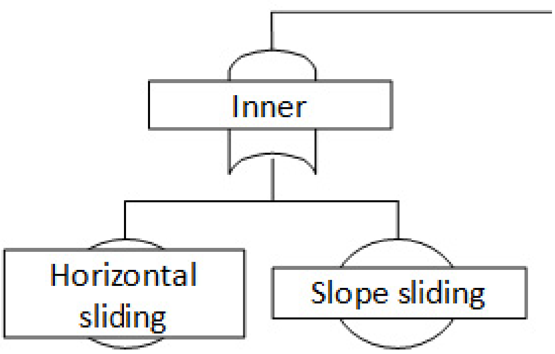


Considering the fact that failure probabilities are computed per year, we expect the contribution of these events to be low.

5.2.3 INNER SLOPE INSTABILITY

Inner slope instability occurs when major soil masses slide of the inner slope of the dike. Distinction is made between horizontal sliding of the dike and sliding of the inner slope, see Figure 32. The stability of the dike is defined by the shear resistance of the soil masses. The governing loads consist of a combination of extreme water levels and rainfall, which result in an increase of the phreatic line in the dike, and traffic loads on top of the dike. The limit state functions are discussed in the following section.

FIGURE 32 FAULT TREE FOR BREACHING DUE TO INNER INSTABILITY



HORIZONTAL SLIDING

The limit state for horizontal sliding is described by equation 18. The structure is stable when the friction force due to self-weight of the structure is larger than the horizontal water pressure on the structure. Note that a surcharge from traffic loads will increase the horizontal stability.

$$Z_{horizontal} = f \cdot \Sigma V - \Sigma H \tag{18}$$

$$\text{with } f = \tan(\theta) \tag{19}$$

INNER SLOPE SLIDING

Inner slope sliding may occur in circular slip circles or along straight planes, depending on the subsoil see also appendix B. Instability can be calculated with various methods, which all determine the balance between resistance moments and driving moments, for a large number of slip circles. The limit state is described with equation 20.

$$Z_{inner} = M_{resistance} - M_{driving} \tag{20}$$

In the assessment of regional flood defences, both Bishop and Uplift-Van are used. Bishop calculates the stability along circular sliding planes, whereas Uplift-Van also takes sliding along horizontal planes in to account. The calculation time of Uplift-van is much longer than that of Bishop. To avoid long calculation times, Bishop will be used to assess the stability for inner slope sliding. The results will be discussed with experts to assess whether or not these are reliable.
Bishop uses the 'method of slices'; the soil mass is divided in a number of vertical slices; the equilibrium of moments of each of these slices is then calculated. The driving moments are described by equations 21 and 22, the resistance moments by equation 24.

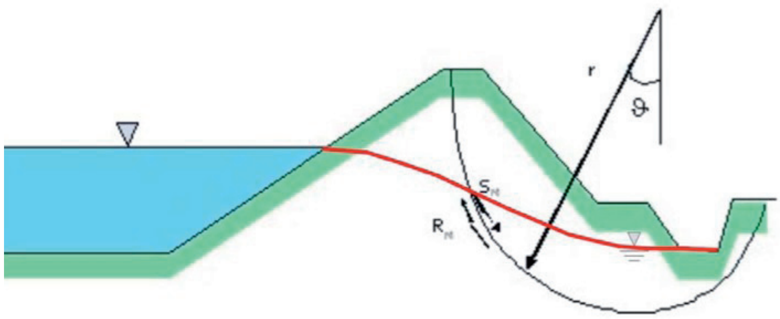
$$M_{driving} = r \cdot \Sigma G_i \cdot \sin(\vartheta_i) \tag{21}$$

$$\text{with } G_i = y_i \cdot A_i (26) \quad L_i = \frac{b_i}{\cos(\vartheta_i)} \tag{22}$$

$$M_{resistance} = r \cdot \Sigma T_i \tag{23}$$

$$M_{resistance} = r \cdot \sum \frac{(G_i - u_i \cdot b_i) \cdot \tan(\varphi_i) + c_i \cdot b_i}{\cos(\theta_i) + \frac{1}{F_s} \cdot \tan(\varphi_i) \cdot \sin(\theta_i)} \tag{24}$$

FIGURE 33 **BISHOP METHOD FOR CALCULATIONS OF MACRO INSTABILITY** (BISCHINIOTIS, 2014)



The ‘method of slices’, upon which the Bishop method is based, uses effective soil stresses to compute the stability (van der Wouden & Grashoff, 2009). The water pressures in the flood defence are required to determine the effective soil stresses, which can be specified by piezo metric lines. As was explained in chapter 4, both the water levels and traffic loads are governing for the stability of the inner slope. Traffic loads cause overpressures in the flood defence. The extent to which the overpressure develops depends on the consolidation of the soil in the flood defence. A consolidation factor of 70% will be taken in to account, which corresponds with the results of recent research (Kwakman, van Veen, & van Soest, 2012).

D-Geo Stability will be used to determine the probability of failure for inner slope instability. This program contains a FORM calculation, which determines the failure probability conditional on the specified piezo metric line, water level and traffic load. Standardized uncertainty parameters of strength variables are used. The failure probability of critical slip circles is calculated for several combinations of water levels, piezo metric lines and traffic loads. Only slip circles which will lead to breaching of the flood defence are taken in to account (i.e. slip circles which protrude the crest of the flood defence). The results are presented in fragility curves for each load, to show which is load case is dominant. Finally, these are combined to obtain the failure probability of instability.

TABLE 13 **INSTABILITY INNER SLOPE VARIABLES LIMIT STATE (NEN6740 TABLE 1)**

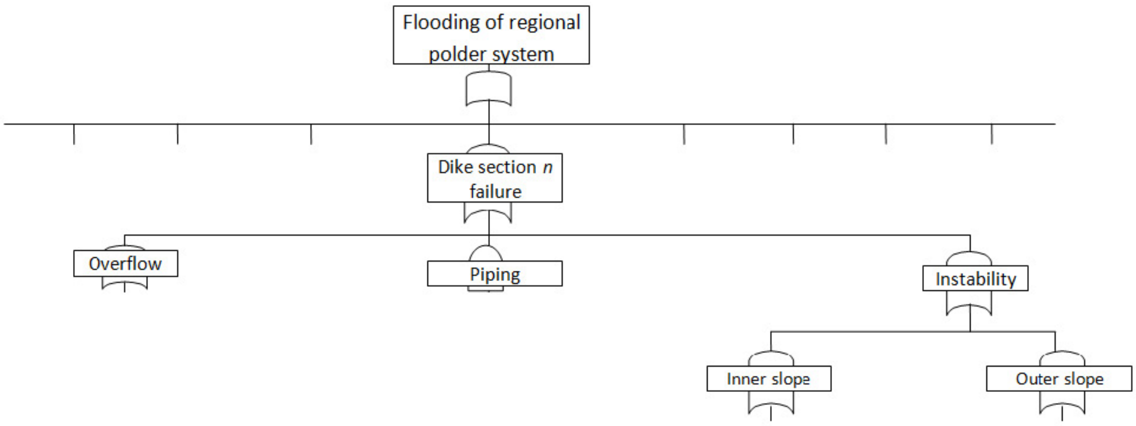
Variable	Explanation	Unit
Σ_V	Summation of vertical forces	kN
Σ_H	Summation of horizontal forces	kN
$M_{resistance}$	Moment of resistance	kNm
$M_{driving}$	Driving moment	kNm
r	Radius of slip circle	M
ϑ_i	Radial angle of slip circle	°
G_i	Weight of element i	kN
L_i	Length of element i	m
b_i	Width of element i	m
h_i	Height of element i	m
A_i	Area of element i	m^2
θ_i	Angle of internal friction	°
T_i	Shear resistance of element i	kN/m^2
u_i	Inner water pressure of element i	kN/m^2
c_i	Cohesion of element i	kN/m^2

5.3 FAILURE PROBABILITY OF DIKE SECTIONS, INCLUDING PROVEN STRENGTH ASSESSMENTS

Several methods exist to determine the probability that a limit state function is smaller than zero, each with advantages and disadvantages; examples are Numerical integration, Monte Carlo simulations, FORM and SORM analysis. We will use both FORM and Monte Carlo simulation to determine the failure probabilities, as these are widely used in reliability assessments in the Netherlands. A description of both methods is given in ((van der Wouden & Grashoff, 2009)). The limit state functions for the governing failure mechanisms are defined in the last paragraph. For every failure mechanism the limit state function is derived and the strength variables required to solve the limit states are defined.

Fault tree analysis is used to determine the failure probability of one dike section, following the approach illustrated in Figure 32.

FIGURE 34 FAULT TREE FLOODING REGIONAL FLOOD DEFENCE SYSTEM



The three failure mechanisms are connected through an OR-gate; for a first estimate they are considered to be completely independent. The combined failure probability can be found with the formula for a series system of independent variables, see equation 25.

$$P_f = 1 - \prod_{i=1}^{n} (1 - P_i) \tag{25}$$

5.3.1 PROVEN STRENGTH UPDATING

Information of survived loads can be used to validate / update the calculated failure probabilities estimated for specific failure mechanism; such an assessment is called a ‘proven strength’ assessment. The applicability of these assessments largely depend on the availability, accuracy and reliability of data of historically successfully survived loads (STOWA, 2009).

In primary flood defences the availability, accuracy and reliability of data required for a proven strength analysis is limited, mainly due to the limited amount of survived loads. Regional flood defences, however, retain water levels close to the maximum design water levels at a daily basis, as is shown in Figure 23. Extreme water levels on the canal systems occur more frequent than they do for primary flood defences, making a proven strength assessment possible.

Several methods exist for proven strength assessments, ranging from very elaborate to simple methods. As a first estimate, First Order Survival Updating will be applied in this report, which is described by (Calle, 1999, 2005). It is based on performance functions for both the future conditions as well as the survived conditions. The key to the updating procedure are the correlations between the two functions, which are derived from the FORM results (Schweckendiek, 2014). More elaborate methods are explained in detail in (Schweckendiek, Vrouwenvelder, & Calle, 2014), which is based on Monte-Carlo simulations.

5.3.2 POTENTIAL OF PROVEN STRENGTH IN REGIONAL FLOOD DEFENCES

The potential of proven strength assessments depends on the correlation between the survived event and the future event. The updating procedure will have large impacts on the estimated reliability of the flood defence if the strength properties of the survived event are highly correlated with the future event. In other words, if the same strength properties are present in the survived event as in the future event; the only uncertainty that remains is the survived load. The proven strength assessment will eliminate this uncertainty and therefore reduce the failure probability of the flood defence. Thus, the updating of failure probabilities with proven strength depends on correlations of the uncertainties of the considered failure mechanism:

OVERFLOW

The uncertainty of overflow lies in the water level, as the strength of the flood defence is determined by the retaining height. The retaining height for the survived and future load can be easily deduced, leaving the hydraulic load as the only uncertainty. In conclusion, proven strength has high potential for overflow.

PIPING

The strength properties of piping lie in the properties of the aquifer under the flood defence (permeability, thickness and d70 of aquifer, seepage length etc.). It can be assumed that these properties will not change over time, except when large excavations are done to remove the aquifer. The remaining uncertainty lies in the water levels, which allows for high potential of proven strength for piping.

INSTABILITY

The strength properties of instability are determined by the geometrical and geological properties of the flood defence. These can be assumed to be the same for survived events and future events, if reinforcements or changes have been made to the flood defence. However, large uncertainties lie in the development of the phreatic line in the flood defence, the amount of top load and the occurring water level. Information of survived water levels will therefore only have large impacts if for that water level the corresponding phreatic line and top load are also known. This is often not the case. Proven strength is therefore not expected to have large impacts for instability. However, if the top load and phreatic line are chosen optimistic, survived water loads may still have some impact. This requires more investigation.

5.3.3 ROLE OF WATER BOARD MANAGERS

Flood defence managers play an important role in the assessment of regional flood defences ('beheerdersoordeel'), see (Stowa, 2007). For every dike section, these managers give their opinion regarding the safety of all failure mechanisms of the considered section. Their judgment is based on their experience with considered dike section, for example based on observations of successfully retained water levels. If the opinion of the manager does not coincide with the result of the technical assessment, more research is recommended.

Insight in proven strength possibilities and fragility curves of failure mechanisms can aid flood defence managers in making well though-out decisions regarding the assessment of flood defences, as well as the required strategy during extreme events. We therefore recommend fragility curves to be constructed for each dike section of regional flood defences. Flood defence managers will then have insight in the fragility of the considered flood defences, and can compare field observations with the theoretical fragility. This will also provide more information for proven strength assessments, especially if the field observation does not correspond with the theoretical fragility. In this case, we recommend water boards to collect the necessary data to update the calculated fragility with proven strength methods.

5.4 FAILURE PROBABILITY OF FLOOD SCENARIOS

The failure probability of the individual dike sections can be combined to obtain the failure probability of a group of dike sections and/or the whole system. Each group of dike sections represents one flood scenario, see chapter 3. The manner with which the failure probabilities of individual sections are combined depends on the correlations between the dike sections. A group of dike sections is modelled as a series system. Depending on the correlation between dike sections the probability of failure of the system can be found with the relations in Table 14. For complete independent sections the probability of failure can be calculated analytically, the same holds for fully dependent dike sections. The correlations between the dike sections depend on the uncertainties in the parameters. When these are taken in to account, are failure probability needs to be estimated numerically (Ditlevsen or Hohenbichler), see (Stichting CUR, 1997).

TABLE 14 ELEMENTARY BOUNDS FOR PROBABILITY OF A SERIES SYSTEM FAILURE

	Mutually exclusive sections	Independent sections	Fully dependent sections
Series system	Upper boundary		Lower boundary
	$P_{f;system} = \sum_{i=1}^{n} P_{f;i}$	$P_{f;system} = 1 - \prod_{i=1}^{n}(1 - P_{f;i})$	$P_{f;system} = MAX(P_{f;i})$

In the schematization of the flood defence sections are chosen based on similar strength properties; over the length of one section the statistical properties of the reliability function are assumed fully correlated. This allows us to assume that the failure probability calculated for one cross section is representative for the whole dike section. Whether or not this is true requires further research.

TABLE 15 CORRELATION DISTANCES VARIABLES PRIMARY FLOOD DEFENCES

Parameter	Correlation length
Water level	50 to 100 km
Crest height	0.2 to 0.5 km
Slope gradient	0.2 to 0.5 km
Grain diameter (D_{n50})	1 to 10 km
Width	0.5 to 5 km

Correlations between dike sections depend on the correlation distances of the relevant variables; typical correlation distances of important variables for primary flood defences are given in Table 15. Detailed correlation distances were given in paragraph 5.2, where specific failure mechanisms were treated. The average length of dike sections in the case study (chapter 6) varies between 0.5 and 4.5 km with an average of 1.8 km. Based on the correlation distances shown in the table it is assumed that the water level is fully correlated along the dike sections.

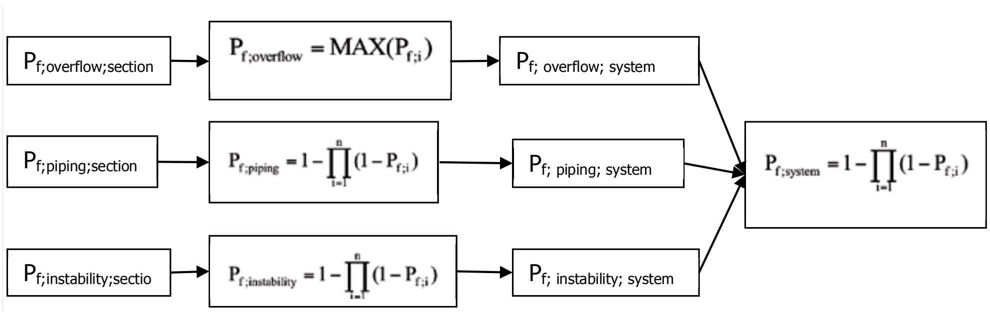
The strength properties cannot be assumed fully correlated along the dike sections, as there are large uncertainties in the subsoil. The correlation distances are often in the same order or even smaller than the average length of the dike sections. Based on these distances, it can be assumed that the correlations between sections are small. Small correlations, or independency, between sections result in an increase of the failure probability of the system with increasing length. At this stage the following assumptions are made for combining the failure probabilities of dike sections:

TABLE 16 **METHOD FOR COMBINING FAILURE PROBABILITIES OF DIKE SECTIONS**

Failure mechanism	Dependency	Method
Overflow	Fully dependent	$P_{f;system} = MAX(P_{f;i})$
Piping	Independent	$P_{f;system} = 1 - \prod_{i=1}^{n}(1 - P_{f;i})$
Instability inner/outer slope	Independent	$P_{f;system} = 1 - \prod_{i=1}^{n}(1 - P_{f;i})$

For overflow full dependency is assumed because the water levels along a flood defence are fully correlated and the difference in retaining height along a flood defence is small. For piping and instability the dike sections are assumed independent, because of the possibility of large variations in the subsoil characteristics between sections. This may result in an overestimate of the failure probability as correlations between sections cannot be neglected. However, it is expected that such an assessment together with the results of a 'proven strength' analysis will provide reliable first estimates of the failure probabilities. The resulting approach to combine failure probabilities for one flood scenario is shown in Figure 35.

FIGURE 35 **FLOW CHART METHOD OF COMBINING FAILURE PROBABILITIES**



FAILURE PROBABILITY OF THE WHOLE SYSTEM

The probability of flooding of the whole system can be determined by combining the scenario probabilities determined with the last paragraph. The manner with which the scenario probabilities are combined depends on the occurrence of relief in the system, which strongly depends on the schematization of the system. When relief is taken in to account, the probability of multiple breaches in one canal system is negligible, due to a drop in water levels after the initial breach. In that case, multiple flood scenarios in one canal system for one polder are not possible.

A regional flood defence system can be loaded by several canal systems. The maximum amount of breaches in the polder is equal to the amount of canal systems surrounding it. The probability of flooding of the whole system is therefore a combination of the probabilities of flooding of the flood scenarios of each canal system. The probability of two scenarios occurring simultaneously depends on the correlation between both canal systems:

- On the load side, both canal systems are correlated because water levels in the canals are the result of meteorological events in the area;
- On the strength side, the correlations can be much lower due to differences in the considered flood defences along the canal systems.

For complete independent scenarios the combined probability is the summation of the individual probabilities, for dependent scenarios the combined probability is equal to the highest probability of the individual scenarios.

$$\underset{i=1}{\overset{N}{\mathrm{MAX}}}\, P_{f_i} \le P_{f;system} \le 1 - \prod_{i=1}^{n} (1 - P_{f;i}) \quad (26)$$

To determine which correlations have to be taken in to account, we need to determine which uncertainty governs the failure probability: the strength or load uncertainty. In the case study discussed in chapter 6, the load uncertainty has very little effect on the failure probability (see chapter 6). In this case we can assume both canal systems to be independent. The flood probability of the polder is therefore estimated by the summation of the probability of flooding of each canal system.

5.5 CONCLUDING REMARKS

This chapter discusses the methodology to determine the probability of flooding of a regional flood defence system. The following failure mechanisms are governing: Overflow, Piping and Instability of the inner and outer slope (see Figure 26).

The limit state function of overflow is based on a critical overflow amount, which can lead to erosion of the inner slope and breaching. The stability for piping is calculated with the updated Sellmeijer formula. The presence of impermeable layers on the bottom of the canals increase the resistance of intrusion of water from the canals to the aquifer below. We recommend measuring the resistance of intrusion due to the impermeable layers in the aquifer, to include the effect on the hydraulic head in the piping models. The failure probability of instability is determined with D-Geo Stability for several combinations of water levels, phreatic lines and traffic loads. The results will be presented with fragility curves, to show which loads are dominant.

In follow up research, we recommend including the effect of hydraulic short circuiting due to events such as dredging works. The piping probability can be computed for scenarios with and without the reduced head due to resistance of intrusion. With estimates of the probability of hydraulic short circuiting one can then compute the actual piping probability with the event tree shown in Figure 31.

FORM reliability calculations will be used to determine the failure probability of overflow and piping; fragility curves will be made. The resulting failure probabilities of each failure mechanism are combined through fault tree analysis to obtain the failure probability of the dike section. For a first estimate, we assume the failure mechanisms to be independent.

'Proven strength' will be used to update the failure probabilities estimated, based on survived loads in the past. First Order Survival Updating is used for a first estimate. Proven strength will have high potential for updating failure probabilities of overflow and piping; the main uncertainty for these failure mechanisms lies in the load, which is the survived water level. The expected potential of proven strength for the instability failure mechanism is lower, because not only the water level load determines the stability, but also the phreatic line and traffic loads. More research on this subject is recommended.

Groups of sections are combined in flood scenarios based on similar flood consequences irrespective of breach location within one scenario. Scenario probabilities are determined by modelling groups of sections as a series system. For overflow full dependency between sections is assumed; for piping and instability the dike sections are assumed independent.

The probability of flooding of the whole system can be determined by combining the scenario probabilities. The manner with which the scenario probabilities are combined depends on the occurrence of relief within one canal system *and* the correlations between the canal systems surrounding the polder. When relief is taken in to account, the probability of multiple breaches in one canal system is negligible. Then, the maximum amount of breaches in the polder is equal to the amount of canal systems surrounding it.

For complete independent canal systems the combined probability of flooding is the summation of the individual probabilities, for dependent scenarios the combined probability is equal to the highest probability of the individual scenarios. Whether or not the canal systems can be modelled as dependent systems depends on which uncertainty governs the failure probability: the strength or load uncertainty.

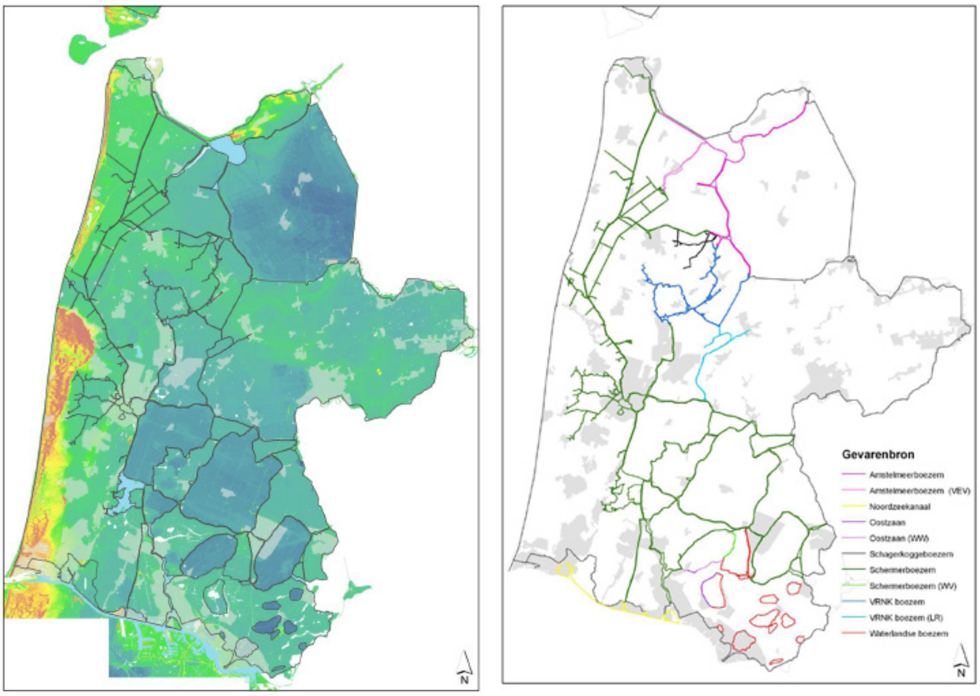
6

CASE STUDY HEERHUGOWAARD

6.1 AREA DESCRIPTION

The developed methodology is applied to a case study in the managed area of water board HHNK (Hoogheemraadschap Hollands Noorder Kwartier). This water board is responsible for the management and maintenance of flood defences in a large area in the northern part of Holland, 45 kilometres north of Amsterdam.

FIGURE 36 MANAGED AREA OF WATER BOARD HHNK (LEFT); CANAL SYSTEMS IN THE AREA (RIGHT): THE SCHERMERBOEZEM IS SHOWN IN GREEN AND THE VRNK BOEZEM IN BLUE (ARCADIS, 2011)



Several canal systems run through the area: the Schermerboezem, the Amstelboezem, the VRNK-boezem, the Schagekogge boezem and the Waterlandse boezem. The water levels in these canals are shown in Table 17. A total of 500 kilometres of regional flood defences protect the surrounding polders from flooding from these canals, of which 270 kilometres require reinforcement according to the last safety assessment.

TABLE 17 **CANAL SYSTEMS IN HHNK AREA**

Canal	Average water level [m +NAP]	Drain stop level [m +NAP]	Drains water to
Schermer	-0,5	0	Noordzee kanaal, Markermeer, Waddenzee
Amstel	-0,4	+0,6	Waddenzee (Schermerboezem)
VRNK	-0,6	-0,3	Amstelmeerboezem
Schagekogge	-0,85	-0,59	Schermerboezem
Waterlands	-1,54	-	Markermeer, Noordzee kanaal

The ‘Heerhugowaard’ polder is considered in our case study. It is surrounded by two canal systems: the Schermerboezem and the VRNK-boezem, see Figure 37. The city of Heerhugowaard lies within the polder, on the Western side. It is surrounded by 32 kilometres of regional flood defences.

FIGURE 37 **CANAL SYSTEMS SURROUNDING THE HEERHUGOWAARD POLDER (ARCADIS, 2011)**

SCHEMATISATION OF SYSTEM: DIKE SECTIONS

The flood defence system around the Heerhugowaard polder was divided in 17 sections for the last safety assessment, see Figure 39. The distinction was made based on:

- Geometry of the dike;
- Polder water level;
- Subsoil;
- IPO safety standard.

FIGURE 38 **DIKE SECTIONS ALONG HEERHUGOWAARD POLDER (ARCADIS, 2011)**

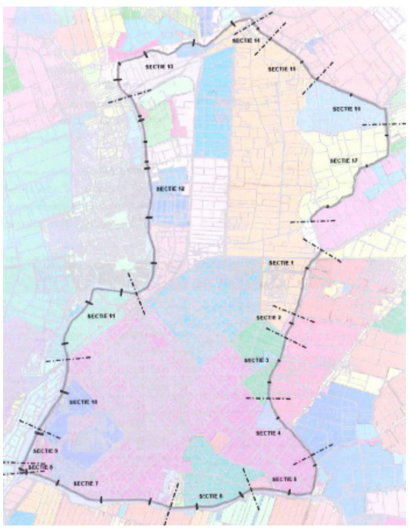
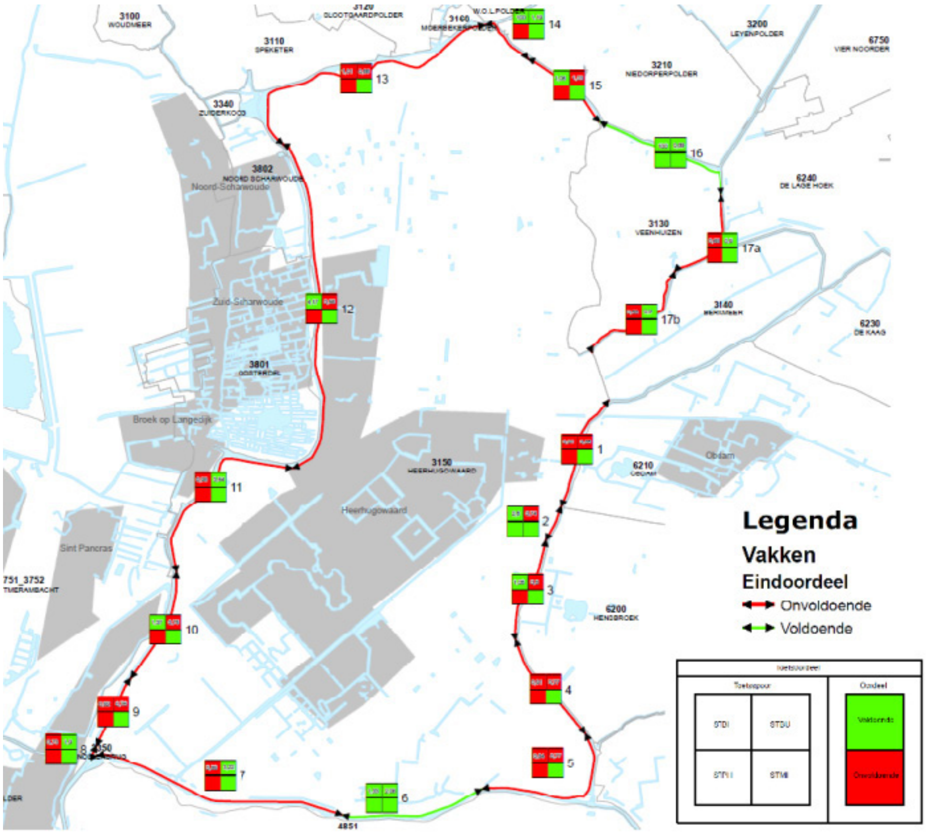


FIGURE 39 RESULTS OF SAFETY ASSESSMENT (ARCADIS, 2011): TOP LEFT CORNER OF THE SQUARE REPRESENTS INNER SLOPE INSTABILITY; TOP RIGHT CORNER REPRESENTS OUTER SLOPE INSTABILITY; BOTTOM LEFT CORNER REPRESENTS PIPING AND BOTTOM RIGHT CORNER REPRESENTS MICRO INSTABILITY. A GREEN COLOR INDICATES COMPLIANCE WITH THE SAFETY STANDARD, RED INDICATES NONCOMPLIANCE WITH THE SAFETY STANDARD



For the assessment, a representative cross section is chosen, which represents the strength of the whole section. This cross section is analysed to verify if the section complies with the current safety standards. The results of the assessment are summarised in the right map of Figure 39. Each square in the small boxes represents a failure mechanism of the flood defence:

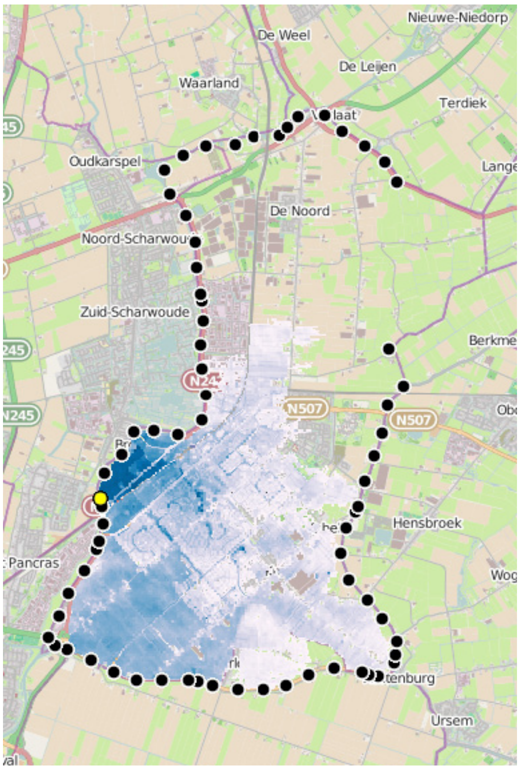
- Top left: Inner slope instability;
- Top right: Outer slope instability;
- Bottom left: Piping;
- Bottom right: Micro instability.

A green colour indicates that the section complies with the safety standard; a red colour indicates failure to comply with the safety standard. The same division in sections will be used in the flood risk assessment. From the results we conclude that all of the sections comply with the standard for Micro Instability, on the other hand none of the sections comply with the standard for Piping. This clearly requires more investigation.

SCHEMATISATION OF SYSTEM: FLOOD SCENARIOS

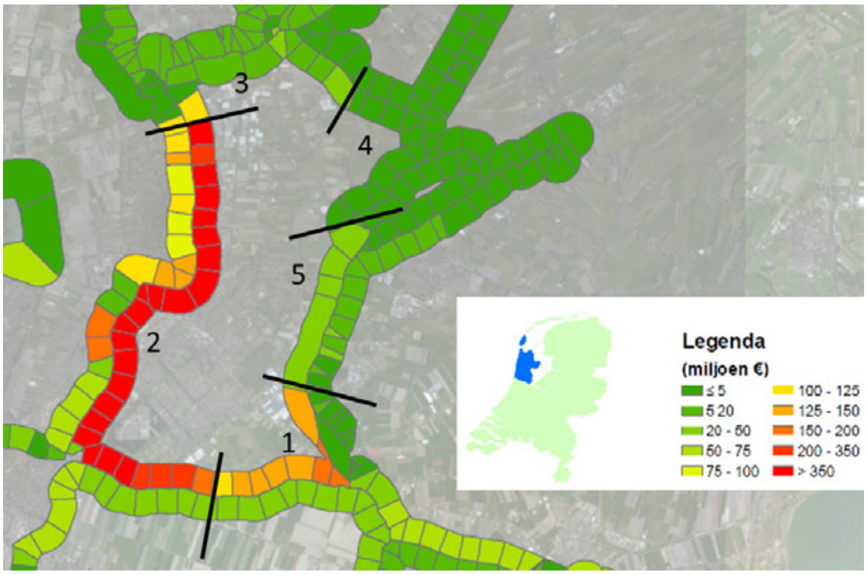
As stated in chapter 2, flood scenarios are composed of a group of dike sections, wherein a breach will result in similar flooding irrespective of its location within the group. The water board has made simulations of flooding for a number of breach locations along the flood defence system (points in Figure 40). The simulations have a maximum duration of 48 hours and a breach size of 10 meters (this will have effect on the development of the flood over time but not on the end result).

FIGURE 40 EXAMPLE OF FLOODING AFTER A BREACH AT LOCATION SHOWN IN YELLOW (LEFT) (ARCADIS, 2011)



These flood simulations were used to compute damage estimates with HIS SSM, see Figure 41. The colours along the flood defence system indicate the damage if that location breaches. Notice that no distinction is made for flood damages exceeding 350 million euro.

FIGURE 41 DAMAGE ESTIMATES WITH HIS SSM AND PRELIMINARY DIVISION IN FLOOD SCENARIOS (ARCADIS, 2011)



Based on these estimates, we divided the flood defence system in 5 groups of sections, each representing one flood scenario, see Figure 41. For each flood scenario, we calculated the corresponding flood damage with both HIS SSM and WSS (see chapter 3). After analysing the results, we concluded that a different division in scenarios would better represent the actual floods. The final flood scenarios used in the risk assessment are shown in Figure 42, which include 6 scenarios. Each scenario consists of a group of dike sections, as shown in Figure 42.

FIGURE 42 FLOOD SCENARIOS FOR RISK ASSESSMENT OF HEERHUGOWAARD POLDER

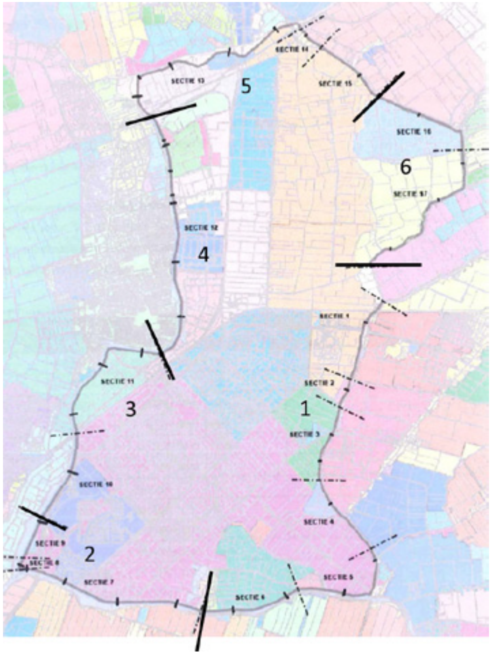


TABLE 18 FLOOD SCENARIOS AND CORRESPONDING DIKE SECTIONS

Scenario	Dike sections	Weakest section according to the assessment
1	1,2,3,4,5,6	4
2	7,8,9	9
3	10,11	11
4	12	12
5	13,14,15	13
6	16,17	17

Table 18 show which sections together form one flood scenario. The weakest sections within each group are also shown, according to the results of the safety assessment. These will be used to estimate the probability of flooding for each scenario. This method is assumed to provide a reliable first estimate of the probability of flooding for each scenario.

Whether or not the first estimate is accurate requires more research. It will be correct if the largest uncertainty is determined by the load, because the sections can then be modelled dependent. If the largest uncertainty is determined by the strength of the flood defences, the sections have to be modelled more independent, to account for the length effect. The first estimate calculated in this approach will then provide an underestimate of the actual probability of flooding.

6.2 LOAD UNCERTAINTY

The loads consist of the external water levels and traffic loads. The uncertainty distributions of the traffic loads were determined in chapter 4, with expert ellicitation. The results are summarized in Table 7. Water level observations are available for four locations along the flood defence system surrounding the Heerhugowaard, each location represents a polder drainage station. The four locations are called ‘Wogmeer’, ’Heerhugowaard’, ‘Speketer’ and ‘Berkmeer’, they are shown in Figure 43 and described in Table 19.

FIGURE 43 LOCATIONS OF WATER LEVEL OBSERVATIONS

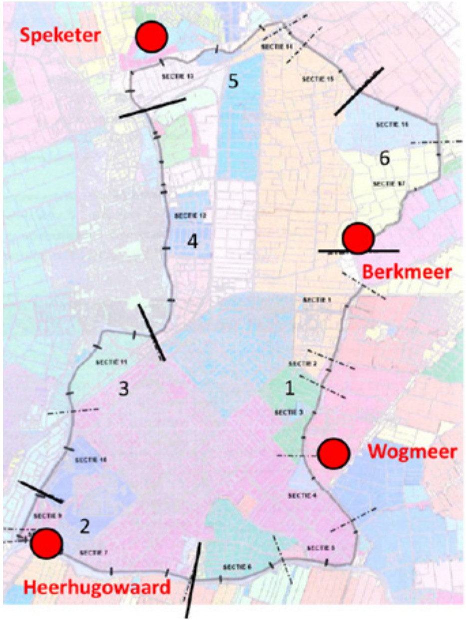


TABLE 19 STATISTICS FROM FEWS FOR THE HEERHUGOWAARD POLDER

Location	Canal	Length of dataset [yr]	Distribution	Mu	Sigma	Decimate height
Wogmeer	VRNK	20	Normal	-0.57	0.067	0.05
Heerhugowaard	Schermer	8	Normal	-0.444	0.039	0.03
Speketer	VRNK	20	Normal	-0.583	0.078	0.06
Berkmeer	VRNK	20	Normal	-0.593	0.041	0.03

The water board fitted normal distributions through the observed hourly water levels, which are shown in Table 19. However, we are interested in the annual probabilities of extreme water levels. Therefore a different statistical analysis of the water levels is required, based on the approach in paragraph 4.2.2. For three locations 20 years of data was available, but for the Heerhugowaard location only 8 years of data was available.

GENERALIZED PARETO DISTRIBUTION FIT

We first analysed the water level observations at each location, which are shown in appendix C. We filtered out the independent peak water levels above a certain threshold and plotted them against their return period, after which the GPD fit was made. The following figures show the resulting fit, including the fit which was obtained by Promotor.

FIGURE 44 WOGMEER STATION (LEFT) AND HEERHUGOWAARD STATION (RIGHT)

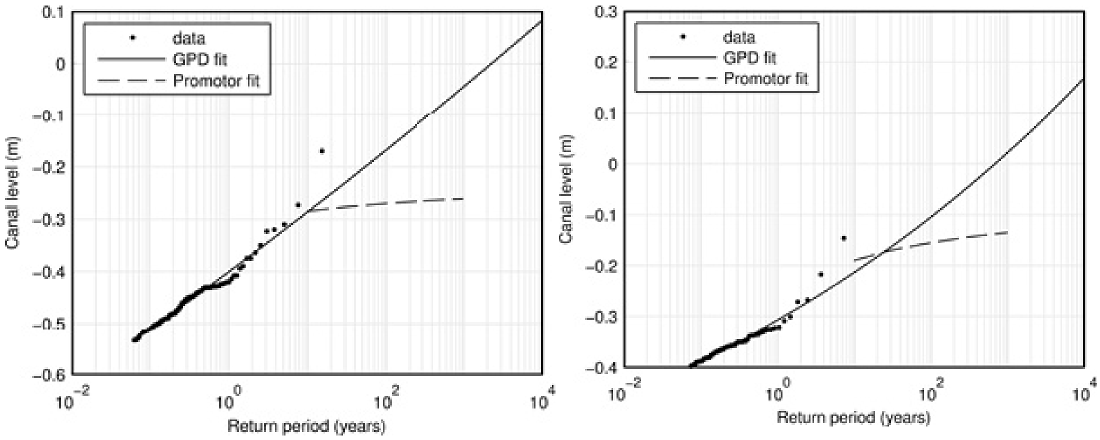
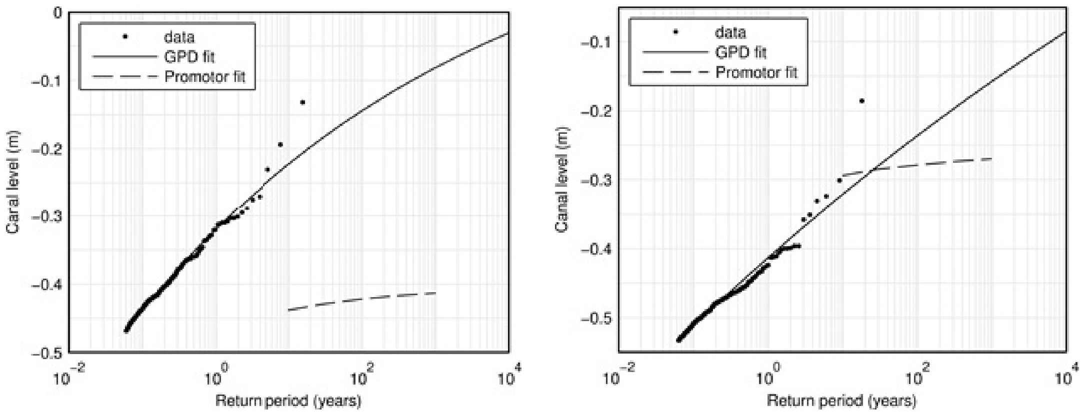


FIGURE 45 SPEKETER STATION (LEFT) AND BERKMEER STATION (RIGHT)



From the figures we can conclude the following:

- VRNK canal: the water levels in the VRNK canal have exceeded the drain stop level, which is 0.3 meter below NAP, a number of times. Especially at the Speketer location, there have been 9 events in 24 years of observed data where the water level exceeded the drain stop level;
- Schermer canal: the water levels in the Schermer canal have never exceeded the drain stop level, which is exactly 0m NAP.

We can also see that the promotor fit does not correspond with the observed water levels, which can be explained by the fact that it does not take failure of regulation of the water levels (drain stop) in to account. In Promotor, water levels higher than the drain stop level can only occur due to rainfall, wind or wave set up; this results in an exceedance line which only slightly exceeds the drain stop level. This is seen for the stations of Wogmeer and Berkmeer. The promotor fit at the Speketer location appears to have an error, as it is well below the average daily water level. For the Heerhugowaard station the promotor fit does not exceed the drain stop level, which corresponds with the water level observations.

STATISTICAL FIT, INCLUDING FAILURE OF DRAIN STOP LEVEL

The maximum observed water levels at each station are given in the following table.

TABLE 20 **MAXIMUM OBSERVED WATER LEVEL**

Location	Canal	Drain stop level [m +NAP]	Maximum observed water level [m +NAP]	Date
Wogmeer	VRNK	-0.3	-0.170	October 29th 1998
Heerhugowaard	Schermer	0	-0.14	January 3rd 2003
Speketer	VRNK	-0.3	-0.132	October 29th 1998
Berkmeer	VRNK	-0.3	-0.186	April 29^{t}h 2001
			-0.301	October 29th 1998

The maximum observed water levels which exceeded the drain stop level provide insight in the probability that the water levels exceed the drain stop level. For each station the frequency of events where the water levels exceeded this level is given in Figure 46. This information is used to compute the annual probability that the drain stop level is exceeded, using a Poisson distribution. This distribution gives the probability that during a random period of time the drain stop level is exceeded exactly n times, which is different from an estimate based on the frequency of the drain stop level.

However, we are not interested in the exact amount of times the drain stop level is exceeded, as one exceedance equals failure of the drain stop. Instead, we are interested in the annual probability that the drain stop level is exceeded. This is computed with the complement of the probability that the drain stop level is never exceeded, see equation 27.

$$P_f = 1 - e^{-\lambda t} \tag{27}$$

λ represents the frequency of the considered water level
t represents the time period (in this case 1 year to obtain the annual probability)

TABLE 21 **PROBABILITY OF WATER LEVELS EXCEEDING THE DRAIN STOP LEVEL**

Location	Canal	Frequency H>MBP [yr^{-1}]	P (H>MBP) [yr^{-1}]]
Wogmeer	VRNK	0.1303	0.1144 (1/9)
Heerhugowaard	Schermer	0	<<<<1
Speketer	VRNK	0.4454	0.2853 (1/3.5)
Berkmeer	VRNK	0.0551	0.0521 (1/19)

We should note that the data set for the Heerhugowaard location, where the drain stop level was never exceeded, was relatively short (8 years). This resulted in a negligible probability of water levels exceeding the drain stop level. Another method to compute the probability of failure of the drain stop level is by performing a complete risk assessment (fault tree analysis) of the drainage stations responsible for regulating the water levels in the canal, an example of such an analysis is given by Lendering et al. (2013) for emergency measures. This may result in different estimates of the probability of failure of the drain stop level, which may be more realistic than those found with the current dataset.

The resulting combined statistical distribution of the water levels, at all stations, is shown in the following figures. The probability tables are included in appendix D.

FIGURE 46 **COMBINED STATISTIC OF WOGMEER (LEFT) AND HEERHUGOWAARD (RIGHT)**

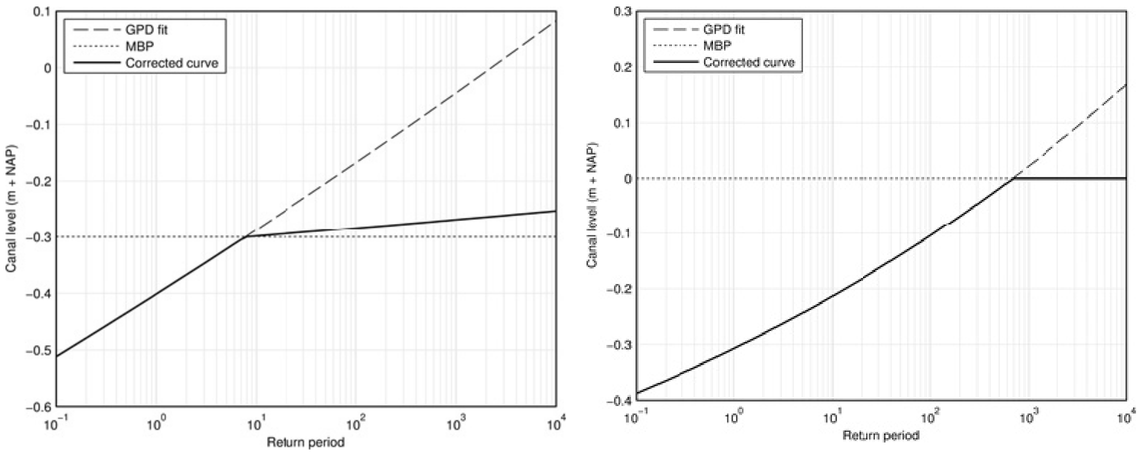
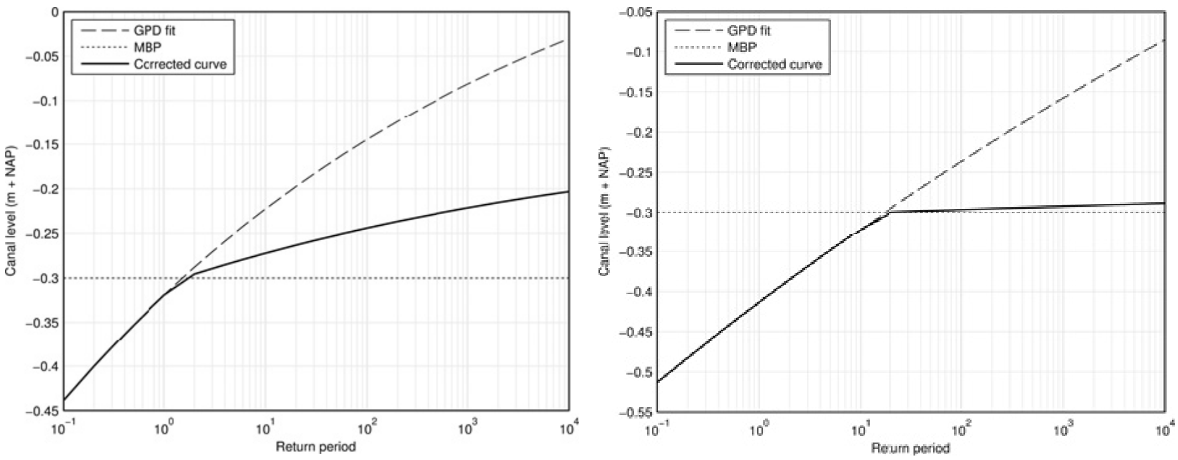


FIGURE 47 **COMBINED STATISTIC OF SPEKETER (LEFT) AND BERKMEER (RIGHT)**



We should mention that during the used dataset changes might have been made to the regulation system of the canals, which may result in some errors in the data. For example, several water boards regulated the water levels in the considered canal systems before 1998, each responsible for a certain area. During the heavy rainfall event in 1998 several peaks of water levels were observed, which were higher than the drain stop level. The water boards acknowledged that this is dangerous and decided to join together to manage the water levels more centrally. However, at this stage we do not include these changes in governance. It is recommended to investigate the possible effects of these changes on the failure probabilities.

6.3 STRENGTH UNCERTAINTY: PROBABILITY OF FAILURE MECHANISMS

This paragraph discusses the results of reliability calculations for the governing failure mechanisms at the critical dike sections determined in the last paragraph. For piping, proven strength will be used to update the estimated probability of failure with survived loads. The fragility curves for each failure mechanism are constructed, to illustrate the fragility of the dike sections for each failure mechanism.

The resulting failure probabilities of each failure mechanism are combined to determine the failure probability of each dike section (which represents the failure probability of the corresponding flood scenario). In the last section, these will be combined to determine the probability of failure in each canal system and finally the probability of failure of the whole regional flood defence system surrounding the Heerhugowaard polder.
Note that the probability of failure represents the probability of flooding.

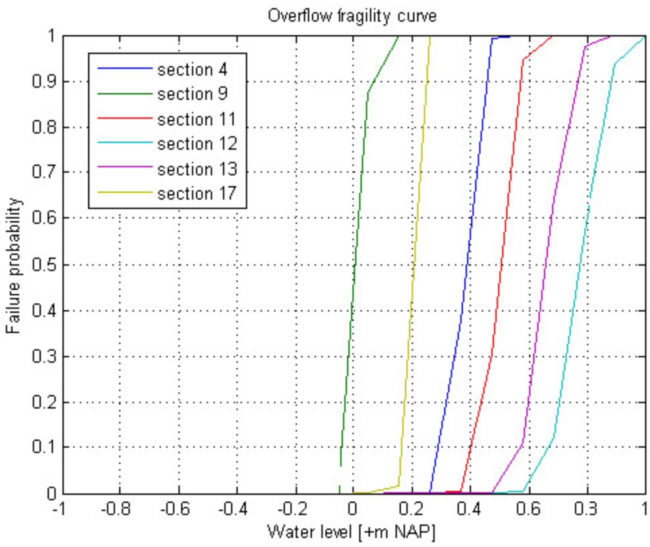
6.3.1 OVERFLOW

The strength for overflow is determined by the retaining height of the flood defence. The minimum retaining height of every section, according to the safety assessment, is given in Table 22. These were used to compute the failure probability of overflow, which is also included in the same table. We concluded that the probability of overflow is negligible in this system; the retaining height of the flood defences is well above the water levels in the canals, corresponding with return periods below 1/100,000 years. The fragility curves are shown in Figure 48.

TABLE 22 RETAINING HEIGHT OF CRITICAL SECTIONS FOR OVERFLOW

Flood scenario	Critical section	Mean of retaining height [m NAP]	Standard deviation of retaining height [m]	Probability of failure [yr-1]
1	4	0.38	0.1 * μ	Pf < e-5
2	9	0.047	0.1 * μ	Pf < e-5
3	10	0.499	0.1 * μ	Pf < e-5
4	12	0.775	0.1 * μ	Pf < e-5
5	13	0.66	0.1 * μ	Pf < e-5
6	17	0.201	0.1 * μ	Pf < e-5

FIGURE 48 OVERFLOW FRAGILITY CURVES



6.3.2 PIPING

The failure probability of piping is determined by three failure mechanisms, according to the approach described in chapter 5: hydraulic short circuiting, uplift and piping. For a first estimate, we assumed that there is no resistance between the intrusion of water from the canal to the aquifer. The hydraulic head taken in to account is determined by the difference of water levels between the canal and the polder. The variables used in the piping calculations are included in appendix E.

In all cases, no blanket layer was present behind the flood defence, which results in a probability of 1 for uplift (see variable H_i in appendix E). We will therefore omit the influence of uplift in the remaining reliability analysis and determine the probability of piping with the revised Sellmeijer formulas. The results are shown in the first row of the following table.

TABLE 23 PIPING FAILURE PROBABILITIES

Piping	Section 4	Section 9	Section 11	Section 12	Section 13	Section 17
Pf [yr-1] With new sellmeijer	0.0089	0.1583	0.8529	0.1210	0.0129	0.0199

The calculated failure probabilities for sections 9, 11 and 12 are high; according to these probabilities we would have to see signs of piping along the regional flood defence. The first signs of piping are uplift and/or heave, which is visible in the form of boils (uplift) and sand boils (heave). These results were discussed with experts of the water board, who state that they have not seen these signs, which suggests that piping has not occurred. These piping failure probabilities are therefore believed to be unrealistic.

In these calculations we assumed direct contact between water levels in the canals and the polders. In most cases, however, the bottom of the canals consists of impermeable layers which increase the resistance of intrusion of water to the aquifer below. The actual hydraulic head over the flood defence is lower than the modelled hydraulic head.
Field tests at the considered case study, at section 11, proved that a reduced hydraulic head can be taken in to account (Kwakman et al., 2013). The intrusion resistance was measures for several scenarios, which are shown in Table 24.

TABLE 24 INTRUSION RESISTANCE IN SECTION 11 (ARCADIS, 2011)

Scenario	Reduced head due to intrusion resistance [m]
Average water level without short circuiting	2.7
Average water level with large circuiting (MHW)	1
Drain stop level without short circuiting	3.2
Drain stop level with small short circuiting	2.4

These results were believed to be representative for all sections along the Schermer canal, which are sections 9,11 and 12. Based on these tests we assumed a reduction of the hydraulic head for both the Schermer and VRNK canals, which are shown in Table 25.

TABLE 25 REDUCED HYDRAULIC HEAD TAKEN IN TO ACCOUNT IN PIPING MODELS FOR BOT CANAL SYSTEMS

Scenario	Distribution	Average	Coefficient of variation	Standard deviation
Schermer canal	LogNormal	2.7	0.22	0.6
VRNK canal	LogNormal	1.0	0.22	0.22

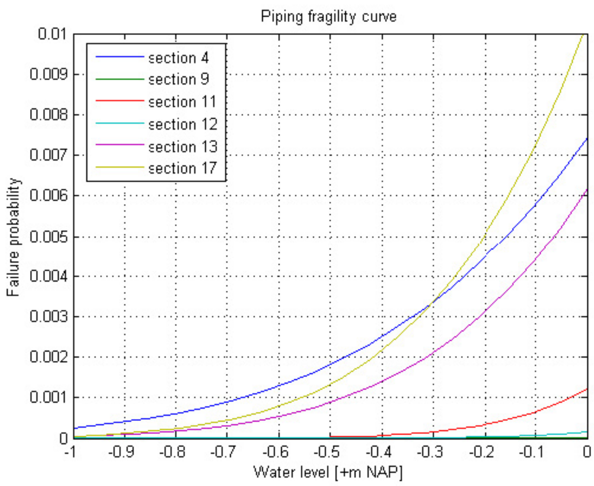
A LogNormal distribution was assumed, as the reduced hydraulic head cannot become negative. The coefficient of variation was chosen based on the results of the field tests at section 11. With this distribution the probability large short circuiting is 1/1,000 per year. The resulting failure probabilities are shown in the table below, for comparison purposed the failure probabilities without a reduced head are also shown. The fragility curve is shown in Figure 49.

TABLE 26 PIPING FAILURE PROBABILITIES

Piping	Section 4	Section 9	Section 11	Section 12	Section 13	Section 17
Pf [yr-1] With new sellmeijer	0.0089	0.1583	0.8529	0.1210	0.0129	0.0199
Pf with reduced head [yr-1] With new sellmeijer	0.0005	$6.4 * 10^{-5}$	0.0178	0.0019	0.0.0004	0.0004

Considering the absence of signs of piping (e.g. boils and sand boils) and no failure in recent years, we can conclude that the impermeable layers on the bottom of the canals reduce the hydraulic head significantly. Proven strength can have high potential for updating the failure probabilities calculated. This will be done in paragraph 6.3.4.

FIGURE 49 FRAGILITY CURVE FOR PIPING WITH REDUCED HEAD



6.3.3 INSTABILITY

The failure probability of instability of the inner slope is computed with D-Geo Stability, using the data obtained from the safety assessment of the water board. The Bishop probabilistic analysis is used, with design parameters as default input values. As explained in chapter 4, several loads influence the stability of the inner slope:

- Water levels in the canal;
- Phreatic line in the flood defence;
- Traffic loads.

To gain insight in the failure probabilities, several combinations of these loads were used. We first determined the governing slip circle for each dike section. The failure probability computed with D-Geo Stability for this slip circle is assumed to represent the probability of breaching due to instability (i.e. the probability of breaching after sliding along this slip circle is 1).

Each slip circle is shown in appendix F. For every dike section we computed the failure probability for four water levels, see Table 27:

- The lowest observed water level;
- The average water level;
- The drain stop level;
- The maximum observed water level.

TABLE 27 WATER LEVELS USED IN INSTABILITY CALCULATIONS

Location	Lowest [m NAP]	Average [m NAP]	Drain stop level [m NAP]	Maximum observed level [m NAP]
Wogmeer	-0.68	-0.59	-0.3 (1/10 yr^1)	-0.17 1/100 yr^1)
Heerhugowaard	-0.56	-0.45	0 (<10-5 yr^1)	-0.14 (1/50 yr^1)
Speketer	-0.71	-0.58	-0.3 (1/3 yr^1)	-0.13 (<10-5 yr^1)
Berkmeer	-0.65	-0.59	-0.3 (1/20 yr^1)	-0.19 (<10-5 yr^1)

These water levels were combined with three phreatic lines inside the flood defence, representing a drought, a daily event and an extreme event. Furthermore, the top load on top of the flood defence was varied according to the empirical distribution of a 'grey flood defence', see Table 7. In this paragraph we will summarize the important conclusions drawn from the reliability calculations.

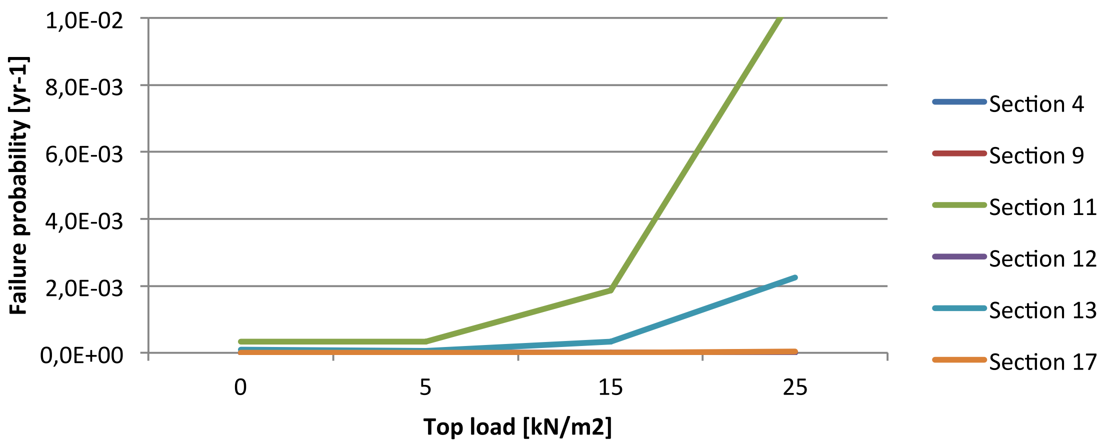
DROUGHTS

The total failure probability during droughts, which is obtained after integrating over the empirical distribution of the top loads determined in chapter 4, is given in Table 28. These are of negligible order of magnitude.

AVERAGE PHREATIC LINE

We now consider an everyday situation, with an average phreatic line (see also appendix F). The following fragility curve represents this situation. The failure probability is plotted against an increased top load on the flood defence (e.g. a traffic load). We deliberately do not show the outer water levels on the horizontal axis, which is done normally, as these do not influence the failure probability significantly.

FIGURE 50 FRAGILITY CURVE FOR ALL SECTIONS WITH AVERAGE PHREATIC LINE

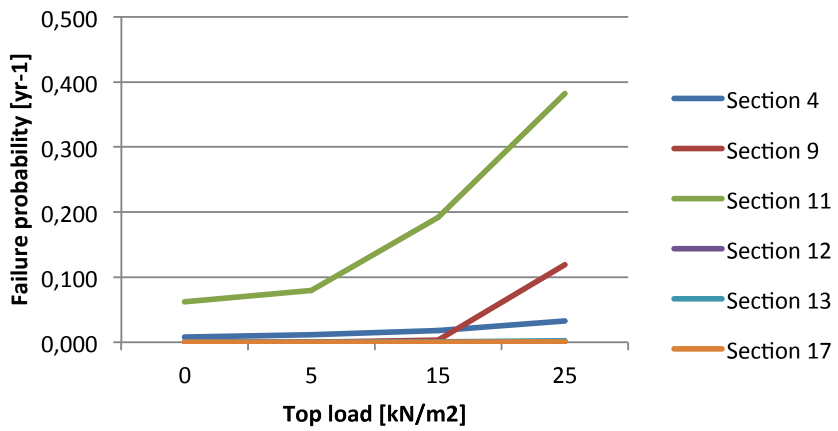


From the figure, we can conclude that failure probabilities increase significantly with increasing traffic load. Sections 11 and 13 have the highest failure probabilities, which remain below 0.01. The total failure probability, which is obtained after integrating over the empirical distribution of the top loads determined in chapter 4, is given in Table 28.

HIGH PHREATIC LINE: SATURATED

Finally, we consider saturated dikes, resulting in high phreatic lines. These are also modelled in the safety assessment. For these situations, we notice a slight difference in failure probabilities for increasing outer water levels, but the top load still has the largest influence, see Figure 50. The fragility curves for increasing water levels and high phreatic lines are shown in appendix G.

FIGURE 51 **FRAGILITY CURVE FOR ALL SECTIONS WITH HIGH PHREATIC LINE**



From the figure, we can conclude that failure probabilities increase significantly with increasing traffic load. The total failure probability, which is obtained after integrating over the empirical distribution of the traffic loads determined in chapter 4, is given in Table 28.

TABLE 28 **INSTABILITY FAILURE PROBABILITIES FOR DIFFERENT PHREATIC LINES**

Instability	Section 4	Section 9	Section 11	Section 12	Section 13	Section 17
Pf with droughts [yr-1]	< 10^{-5}	< 10^{-5}	< 10^{-5}	< 10^{-5}	< 10^{-5}	< 10^{-5}
Pf with average phreatic line [yr-1]	< 10^{-5}	< 10^{-5}	$2.8 * 10^{-3}$	< 10^{-5}	$3.8 * 10^{-4}$	< 10^{-5}
Pf with high phreatic line [yr-1]	0.033	0.007	0.230	0.001	0.001	< 10^{-5}

We conclude that the influence on the outer water level is negligible; instead, the phreatic line is much more important. The phreatic line is influenced by the outer water levels, but also by direct rainfall on the flood defence as is concluded in the correlation matrix from chapter 4. To obtain the total failure probability, we have to integrate over the probability distribution of the phreatic lines, for which we do not have data. We recommend to perform field tests to determine the probability of low, average and high phreatic lines in the dike. To obtain a first estimate of the failure probability, we provide an educated guess of the probability density:

- Drought with a probability density of 5%;
- Average with a probability density of 85%;
- Saturated with a probability density of 10%.

The resulting failure probabilities are shown in Table 29. For comparison purposes, the failure probability without traffic loads is also shown. Notice the different in failure probabilities when traffic loads are omitted. This difference can have large impacts on flood risk, which is an intervention worth discussing. The effect of excluding traffic loads is discussed in chapter 7. Furthermore, it is clear that section 11 is the weakest section, which can be expected considering the cross section: a narrow, high flood defence consisting of peat and clay.

TABLE 29 **INSTABILITY FAILURE PROBABILITIES**

Instability	Section 4	Section 9	Section 11	Section 12	Section 13	Section 17
Pf with traffic load [yr-1]	0.0033	0.0007	0.0254	0.0001	0.0004	< 10^{-5}
Pf without traffic load [yr-1]	0.0013	< 10^{-5}	0.0065	0.0001	0.0001	< 10^{-5}

6.3.4 PROVEN STRENGTH ASSESSMENT

In chapter 5, we concluded that the overflow and piping mechanism show potential for a proven strength assessment in regional flood defences. Considering these two failure mechanisms, we can conclude that a proven strength assessment for overflow is not necessary as the prior failure probabilities of overflow are of negligible order of magnitude. To apply proven strength to the instability failure mechanism more data on phreatic lines and traffic loads for the survived loads is required, which may be the subject of follow up research. In conclusion, only piping is considered in the proven strength assessment in this report.

The First Order Survival Updating method is used to update the prior failure probabilities determined for piping in paragraph 6.3.2. An explanation of the method is shortly given in chapter 5, for more information we refer to (Calle, 1999, 2005). The survived loads taken in to account are the maximum observed water levels shown in Table 20; for these events we assume the water level in the polder to be equal to the polder ground level, due to seepage. The result of the updating procedure is shown in Table 30.

TABLE 30 **PROVEN STRENGTH UPDATING FOR PIPING**

Piping updating	Section 4	Section 9	Section 11	Section 12	Section 13	Section 17
Water level survived load [m NAP]	-0.17	-0.14	-0.14	-0.14	-0.13	-0.19
Pf with reduced head [yr-1] (new sellmeijer)	0.0005	6.4 $*10^{-5}$	0.0178	0.0019	0.0.0004	0.0004
Posterior Pf [yr-1]	< e-6	< e-6	< e-6	< e-6	< e-6	< e-6

The updating procedure shows that the probability of piping reduces to below 10^{-6}, partly because the probability of water levels higher than the maximum survived water level is so small (< 2%, see appendix D) that according to this assessment the probability of piping is negligible. However, if we analyse the equations used in this method we conclude that for this specific case the method used is unrealistic, because the water level uncertainty has little influence on the failure probability (alpha values of 0,05). This results in an error in the formulas, with very low failure probabilities as a result. We therefore recommend to apply an exact method of Bayesian Updating for proven strength, which is described in (Schweckendiek, 2014). This will provide better estimates of the posterior failure probability.
Note that, even though proven strength may reduce the failure probabilities considerably, these flood defences may still be at risk for piping if the geological profile is changed. Suppose the canals are dredged out, creating cracks in the impermeable layer and exposing the aquifer to water in the canals. The effect of the resistance to intrusion is now reduced, exposing the

flood defence to piping. The prior failure probabilities now represent the actual situation, and will result in signs of piping. This was also concluded during field tests at the case study, according to water board employees. Therefore, as stated before, we recommend to include the effect of these events in the risk analysis.

6.4 STRENGTH UNCERTAINTY: PROBABILITY OF FLOODING

In the last paragraph we computed the failure probability for the governing failure mechanisms: overflow, piping and instability. The results will be summarised in this paragraph and the resulting probability of flooding for each canal and the whole system is determined. We consider the failure probability of overflow, the (prior) failure probability of piping and the failure probability of instability with traffic loads. If we combine these according to the approach explained in chapter 5, we obtain the following failure probability per section. This also represents the probability of each flood scenario, see Table 22.

TABLE 31 RESULTING FAILURE PROBABILITIES

Flood scenario	Critical section	Overflow [yr-1]	Piping [yr-1]	Instability (with traffic load) [yr-1]	Total failure probability [yr-1]
1	4	$< 10^{-5}$	0.0005	0.0033	0.0038 (1/260)
2	9	$< 10^{-5}$	$6.4 * 10^{-5}$	0.0007	0.0007 (1/1400)
3	10	$< 10^{-5}$	0.0178	0.0254	0.0428 (1/23)
4	12	$< 10^{-5}$	0.0019	0.0001	0.0020 (1/500)
5	13	$< 10^{-5}$	0.0004	0.0004	0.0008 (1/1250)
6	17	$< 10^{-5}$	0.0004	$< 10^{-5}$	0.0004 (1/2500)

We conclude that the probability of failure is governed by the probability of both piping and instability. The uncertainty of the outer water levels proved to be negligible compared to the uncertainty of the traffic load and strength of the flood defence. Therefore, each section within one canal system can be modelled independent of the next. The resulting failure probability per canal system is shown in Table 32.

TABLE 32 PROBABILITY OF FLOODING PER CANAL SYSTEM

Canal	Overflow [yr-1]	Piping [yr-1]	Instability (with traffic load) [yr-1]	Total failure probability [yr-1]
Schermer	$< e^{-5}$	0.0197	0.0262	0.0454
VRNK	$< e^{-5}$	0.0013	0.0037	0.0050

The Schermerboezem is not influenced by the VRNK boezem, as the VRNK canal drains in to the Amstelmeer canal. Moreover, we concluded before that the uncertainty in the reliability is governed by the strength, not the loads. We can therefore consider both canal systems to be independent. The probability of flooding in the polder is determined by equation 26: 0.0502 per year or 1/20 per year.

6.5 FLOOD CONSEQUENCES AND RISK

This paragraph discusses the potential consequences of flooding in the Heerhugowaard polder and the corresponding flood risk in the system. The system of flood defences surrounding the polder was decomposed in groups of sections, corresponding with flood scenarios in paragraph 6.1.

6.5.1 FLOOD CONSEQUENCES

Flood simulations were made with SOBEK. The expected flood consequences, representing both direct and indirect damage, were determined with HIS SSM and WSS. For both estimators, a flood duration of 48 hours is assumed. In addition, the following assumptions were made:

- The recovery time for roads is 48 hours;
- The recovery time for buildings is 2 * flood duration (for indirect damages);
- Flooding takes place in November.

The resulting flood damages are shown in Table 33.

TABLE 33 CONSEQUENCE ESTIMATES FOR FLOOD SCENARIOS

Flood scenario	Section	Breach location	HIS SSM	WSS
1	4	Opmeer 326	11 mln euro	15 mln euro
2	9	Oud Karspel 3006	422 mln euro	266 mln euro
3	11	Oud Karspel 6628	610 mln euro	431 mln euro
4	12	Oud Karspel 7938	628 mln euro	482 mln euro
5	13	Klohorn 1598	15.6 mln euro	93 mln euro
6	17	Berkermeer 4126	3.5 mln euro	1 mln euro

In most cases, the estimates made with WSS are lower than those of HIS SSM. This can be the result of the limited inundation depth taken in to account in the damage functions or the smaller grid taken in to account in damage estimates of WSS. For two scenarios, the damage estimates of WSS were higher than those of HIS SSM. Especially for scenario 5 a large difference is found, which can be the result of different land use used in both estimators. The major difference lies in the contribution of damage to the industry, which is 70 mln euro for WSS and only 2 mln euro for HIS SSM. There have been some local developments in the area, which may partly explain the difference between HIS SSM and WSS.

From the results cannot be concluded that one estimator provides an over- or underestimate of the flood consequences. Both estimates lie in the same order of magnitude (except for scenario 5). We assume the WSS model to determine the flood damages for regional flood defences more accurately than HIS SSM, because, in general, these floods have lower inundation depths. We do recommend changing the damage functions of buildings, to account for inundation depths larger than 0.3 meter. With larger inundation depth more damage will occur to buildings than currently estimated.

6.5.2 FLOOD RISK

Flood risk is determined by the multiplication of the annual probability of flooding with the corresponding consequences. Both these aspects have been determined in the last paragraphs. The resulting flood risk for each scenario is calculated in Table 34.

TABLE 34 OVERVIEW OF FLOOD PROBABILITY, CONSEQUENCES AND RISK PER SCENARIO

Flood scenario	Section	Canal	Probability of flooding [yr-1]	Damage [mln euro]	Flood risk [mln euro/yr]
1	4	VRNK	0.0038	15	5.75
2	9	Schermer	0.0007	266	0.20
3	11	Schermer	0.0428	431	18.5
4	12	Schermer	0.0020	482	0.95
5	13	VRNK	0.0008	93	0.07
6	17	VRNK	0.0004	1	$4.3 * 10^{-4}$

Considering the flood risk of the different scenarios, we conclude that the largest risk comes from the Schermer canal. Not only because the consequences along this area are largest (the city of Heerhugowaard lies closest to this canal system), but also because these scenarios have the highest probability of flooding.

A major advantage of considering flood risk over the safety assessment is that we can advise decision makers on risk reduction rather than compliance to safety standards. The latter only says something about the strength of the flood defence, whereas a risk assessment takes the consequences of breaches at the considered location in to account.

6.6 CONCLUDING REMARKS

In this chapter, the developed methodology is applied to a case study in the managed area of water board HHNK (Hoogheemraadschap Hollands Noorder Kwartier). The 'Heerhugowaard' polder is considered in our case study. It is surrounded by two canal systems: the Schermer-boezem and the VRNK-boezem (see Figure 37). The city of Heerhugowaard lies within the polder, on the Western side.

6.6.1 SCHEMATISATION

The water board made simulations of flooding for a number of breach locations along the flood defence system (points in Figure 40). These were used to schematise the flood defence system in six flood scenarios, each consisting of a group of dike sections (Figure 42).The weakest sections within each group are used to estimate the probability of flooding for each scenario. This method is expected to provide a reliable first estimate of the probability of flooding for each scenario.

6.6.2 LOAD UNCERTAINTY

To obtain the water level statistics, an approach is used which accounts for the regulation aspects of the water levels; it is described in paragraph 4.2.2. Failure probabilities of the drain stop level were estimated based on the observed data: the water levels in the VRNK canal have exceeded the drain stop level (-0.3 m NAP) a number of times; in contrast to the Schermer canal, where water levels have never exceeded the drain stop level (0m NAP). This information was used to generate a new empirical distribution of the water levels, see Figure 46 and Figure 47.

We should mention that changes in governance of the water levels in the period of observations may have resulted in some noise in the data. This can influence the generated empirical distribution. However, at this stage we do not include these changes in governance. We recommend investigating the possible effects of these changes on the failure probabilities.

6.6.3 STRENGTH UNCERTAINTY

For each section, the probability of the governing failure mechanisms is determined. The probability of flooding was determined by combining the probability of failure of each failure mechanisms within one section. Regarding these failure mechanisms the following is concluded:

- Overflow: The probability of overflow in this flood defence system is negligible; the retaining height of the flood defences is well above the water levels in the canals, these correspond with very low return periods (< 10^{-5} per year).
- Piping: With the current models, high probabilities of piping were computed. These were not expected, considering the absence of signs of piping along the flood defences. However, after including the resistance of intrusion of water from the canals to the aquifer more accurate failure probabilities are found;
- Instability: The probability of failure for instability largely depends on the combination of the phreatic line and top loads. Due to the absence of data on probabilities of phreatic lines, we assumed a distribution based on expert judgement. We recommend to perform field tests to determine actual distribution of the phreatic line in the defence more accurately. Traffic loads reduce the reliability of the flood defence considerably. However, several experts have stated that the current approach to for including traffic loads does not model the actual situation correct. We therefore also recommend discussing the impact of having to include traffic loads on the strength of regional flood defences.

Proven strength can potentially reduce the failure probabilities for regional flood defences, as the differences between the average and extreme water levels are very small. We concluded that, at this stage, a proven strength assessment is only effective for the piping failure mechanism. An exact method of Bayesian Updating is recommended for this purpose, because the estimates obtained with First Order Survival Updating are unrealistic.
Note that, even though proven strength may reduce the failure probability of piping considerably, these flood defences may still be at risk for piping if the geological profile is changed, for example due to dredging works in the canals. This may expose the flood defence to the maximum hydraulic head, due to intrusion of the canal water in to the aquifer below, which results in high probability of piping. Therefore, as stated before, we recommend to include these effects in follow up research.

6.6.4 FLOOD RISK

HIS SSM and WSS were used to compute the consequences of flooding for each flood scenario. The resulting consequences both lie in the same order of magnitude (except for scenario 5, due to large difference in the damage to industry). We assume the WSS model to determine the flood damages for regional flood defences more accurately than HIS SSM, because, in general, these floods have lower inundation depths. When using WSS for regional flood defences, it is recommended to change the damage functions of buildings to account for inundation depths larger than 0.3 meter. With larger inundation depth more damage will occur to buildings than currently estimated by WSS.
The flood risk of each scenario is shown in Table 34. The largest flood risk is determined by scenario 3, or section 11, which has a large probability of flooding combined with high flood consequences. A major advantage of considering flood risk over the safety assessment is that we can advise decision makers on risk reduction rather than compliance to safety standards. The latter only says something about the strength of the flood defence, whereas a risk assessment takes the consequences of breaches at the considered location in to account. This is further elaborated in the following chapter.

7

COST BENEFIT ASSESSMENT

7.1 PROBLEM APPROACH

This chapter will demonstrate how a flood risk assessment can help decision makers to prioritize interventions in the regional flood defence system based on cost benefit analyses. Currently interventions in the system are based on the assessment of regional flood defences, wherein weakest sections are prioritized over stronger sections. However, the weakest sections within a system may very well not be the sections where interventions are most cost effective. The cost of each intervention is compared with the benefits, which consist of the reduction of flood risk (i.e. the reduction of the expected damages per year). To reduce flood risk, one can aim to reduce the probability of a flood or to reduce the damages of flooding. Examples treated in this report are:

- 'Traditional reinforcements';
- Reducing the water levels in canals;
- Restricting traffic on top of the flood defences;
- Compartmentalization of canals.

Note that this analysis is mainly based on assumptions and educated guesses; a detailed study of the effectiveness of these measures is recommended in follow up research.

METHOD

The total cost of the interventions is divided in three components, which are the investments (I) at moment t=0, the present value of the operational cost during a given period of N years (OPEX) and the present value of the risk during that same period (Risk), see equation 28.

$$TC = I_{t=0} + OPEX_{t=N} + Risk_{t=N}\ [€] \qquad (28)$$

The present value of the annual operational cost (OC), denoted by $OPEX_{t=N}$, over an infinite time horizon is found with equation 29, where r represents the interest rate.

$$OPEX_{t=n} = \frac{OC}{r}\ [€] \qquad (29)$$

The annual risk is then calculated with equation 30. The present value of the risk, denoted by $Risk_{t=N}$, during a period of N years is found with equation 31. An interest rate of 5.5% is used, which is advised for Cost benefit Analysis of flood defences in the 21st century (Deltares, 2011).

$$R_{annual} = P_f * D\ [€/yr] \qquad (30)$$

$$Risk_{t=n} = \frac{R_{annual}}{r}\ [€] \qquad (31)$$

This framework is used to compare the total cost of the interventions. Cost effectiveness is obtained when the cost of the option are lower than the risk reduction (ΔRisk). Each intervention is treated in more detail in the following paragraphs. We will compare the options for a lifetime of 100 years.

7.2 COST AND BENEFIT OF EACH INTERVENTION

The following paragraph describes the cost and risk reduction of each intervention. All interventions are compared for each flood scenario in the following paragraph.

7.2.1 'DO NOTHING'

First, we will determine the expected total cost of the 'Do Nothing' option, which represents the current situation. The annual flood risk was calculated in the last chapter, a summary of the result per flood scenario is given in the following table.

TABLE 35 OVERVIEW OF FLOOD PROBABILITY, CONSEQUENCES, RISK AND TOTAL COST FOR 'DO NOTHING' SCENARIO

Flood scenario	Canal	Probability of flooding [yr-1]	Damage [mln euro]	Flood risk [mln euro/yr]	Total cost [mln euro]
1	VRNK	0.0038	15	5.75	1
2	Schermer	0.0007	266	0.20	3.4
3	Schermer	0.0428	431	18.5	340
4	Schermer	0.0020	482	0.95	18
5	VRNK	0.0008	93	0.07	1.4
6	VRNK	0.0004	1	$4.3 \cdot 10^{-4}$	0.01

The expected total cost of this option is determined by the expected damages: the flood risk. No investment or operational cost is included in the 'Do Nothing' option. The resulting total costs are shown in Table 35.

7.2.2 REINFORCEMENT

Traditional dike reinforcements are often proposed to reinforce dike sections which do not comply with the required safety standards. In this cost benefit assessment, we assume that the probability of flooding, after reinforcements, is equal to the safety standard of the considered dike section, which is a conservative assumption the flood defences are usually reinforced well above the safety standard to assure safety in the coming years. As a result, the only sections which require reinforcement are sections 4, 11 and 12; or scenarios 1, 3 and 4. These will only be considered in this chapter.

Reinforcements require a large initial investment, which depends on the required length of the reinforcement. The cost of reinforcement is assumed to be 1 mln euro per kilometre. After reinforcement, no annual operational cost is required.

TABLE 36 TOTAL EXPECTED COST OF DIKE REINFORCEMENT (ONLY FOR SCENARIOS 1,3 AND 4)

Flood scenario	Length [km]	Investment [mln euro]	Operational cost [mln euro/yr]	Flood risk [mln euro/yr]	Benefit/ Cost ratio[-]	Total cost [mln euro]
1	6	6	0	0.02	0.13	6.3
2	0	0	0	0.20	-	1
3	3	3	0	0.43	110	11
4	4	4	0	0.48	0	13
5	0	0	0	0.07	-	1.4
6	0	0	0	$4.3 * 10^{-4}$	-	0.01

We conclude that reinforcements are only cost effective for section11.

7.2.3 REDUCING THE HYDRAULIC LOAD: DRAIN STOP LEVEL

Reduction of the hydraulic load on the flood defence could potentially reduce the probability of flooding of the system. The hydraulic load can be reduced if the drain stop level is lowered, which will result in lower extreme water levels on the regional flood defences. After discussion with the water board, a reduction of the drain stop level of the Schermer canal of 20 centimetres is proposed. To make this reduction possible more drainage capacity is required in the canal, as there is less storage due to lower extreme water levels.

However, we do not expect this intervention to have considerable effect on the probability of flooding. In chapter 6, we concluded that variations of the water level between the average and extreme water levels have little effect on the probability of failure of each mechanism. Therefore, a reduction of the drain stop level with 20 centimetres will not result in a significant reduction of the probability of flooding. Instead, it will only cost money. The cost for this intervention is 20 million euro for the whole Schermer canal, we assume that 3 million can be assigned to the Heerhugowaard polder, divided over the flood scenarios. The total cost of this option is shown in the following table.

TABLE 37 TOTAL EXPECTED COST OF REDUCING HYDRAULIC LOADS ON SCHERMER CANAL

Flood scenario	Investment [mln euro]	Operational cost [mln euro/yr]	Flood risk [mln euro/yr]	Benefit/ Cost ratio [-]	Total cost [mln euro]
2	1	0	0.2	0	4.4
3	1	0	18	0	341
4	1	0	0.9	0	19

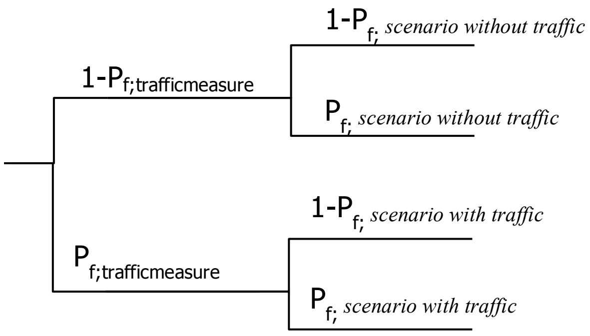
The water board may have other reasons to reduce the drain stop level in the Schermer canal. Note that it may be cheaper not to invest in the increased drainage capacity of the Schermer canal system. Instead, the same effect could be obtained if the frequency of having to stop draining the polder is increased to 1/50 per year, instead of 1/100 per year. This will result in more frequent smaller floods in the polder, often surrounding the polder drainage stations which lie in the lowest parts. More research is recommended to determine whether or not this is more cost effective than investing in more canal drainage capacity.

7.2.4 RESTRICTING TRAFFIC ON TOP OF THE FLOOD DEFENCES

Chapter 6 concluded that traffic loads reduce the reliability of the flood defence considerably. Several experts have stated that the current approach for including traffic loads does not correctly model the actual situation. Suppose we omit the traffic loads from the reliability assessment; in that case, the failure probabilities are reduced considerably. However, after discussion with several experts, we conclude that there will still be a probability that some regional flood defences are loaded by traffic.

To model this intervention, we assume that the measure 'traffic load restriction' is successful in 66% of the cases. In other words, the probability of failure of the 'traffic load restriction' measures ($P_{f;trafficmeasure}$) is 0.33 per event. The resulting probability of flooding (Pf) is computed by the summation of two conditional probabilities: the probability of flooding with traffic loads, given failure of 'traffic load restriction' and the probability of flooding without traffic loads, given successful 'traffic load restriction'. This is illustrated in Figure 52 and computed with equation 32.

FIGURE 52 METHOD TO DETERMINE THE PROBABILITY OF THE SYSTEM WHEN EXCLUDING TRAFFIC LOAD



$$P_f = P_{f;trafficmeasure} * P_{f;scenario\ with\ traffic} + (1-P_{f;trafficmeasure}) * P_{f;scenario\ without\ traffic} \quad (32)$$

To enforce such a restriction, operational costs will be made. We assumed the operational cost for traffic load restriction to be in the order of 300,000 euro per year. Furthermore, an initial investment of 1 mln euro is expected for calamity plans and organisation of the measure. Note that these costs are divided over the scenarios. We expect these estimates to be conservative.

TABLE 38 TOTAL EXPECTED COST OF RESTRICTION OF TRAFFIC LOADS

Flood scenario	Probability of flooding [yr-1]	Investment [mln euro]	Operational cost [mln euro/yr]	Flood risk [mln euro/yr]	Benefit/ Cost ratio[-]	Total cost [mln euro]
1	0.0025	0.05	0.05	0.04	0.4	1.6
2	0.0002	0.05	0.05	0.07	2.2	2.3
3	0.0304	0.05	0.05	0.01	101	240
4	0.0020	0.05	0.05	0.96	0	18
5	0.0005	0.05	0.05	0.05	0.5	1.9
6	0.0004	0.05	0.05	$4 * 10^{-4}$	0	1.0

The resulting probability of flooding, operational cost and flood risk are calculated in Table 38; the table also shows the expected total cost of this option. We conclude that restricting traffic loads are not cost effective for sections 4, 12,13 and 17, which have a benefit cost ratios below 1. Positive benefit cost ratios are only found for sections 9 and 11.

The reason why traffic loads are advised to be taken in to account is because, according to the assessment, a situation may occur where trucks with sand bags will drive over the flood defence to reinforce it at weak spots. These sand bags will be placed on top of the flood defence to reinforce it. However, it is very unpractical to move back and forth on these flood defences with these trucks. Moreover, due to negligible probability of overflowing in regional flood defences, this event is not likely to occur at all. If this were the only reason why traffic loads are advised, we recommend to re-evaluate this design choice.

7.2.5 COMPARTMENTS IN CANALS

Another option to reduce flood risk lies in compartmentalization of canals. Due to the characteristics of the canal system it may be possible to close parts of these canals after the occurrence of a breach. During the flooding at Wilnis, the canal was successfully compartmentalized, avoiding larger consequences. To determine the effect of compartmentalization of the canals on flood risk, the development of flood damage over time is analysed. The maximum flood damage occurs after 48 hours; suppose the inflow of water is stopped within 6 hours: the resulting flood damage is significantly reduced as shown in Table 39.

TABLE 39 CONSEQUENCE ESTIMATES 6 AND 48 HOURS AFTER BREACHING OF THE FLOOD DEFENCE SYSTEM

Flood scenario	WSS (48 hours)	WSS (6 hours)
1	15 mln euro	6.5 mln euro
2	266 mln euro	8.6 mln euro
3	431 mln euro	44 mln euro
4	482 mln euro	146 mln euro
5	93 mln euro	3.6 mln euro
6	1 mln euro	0 mln euro

The probability of successful compartmentalization of the canals ($P_{f;compartment}$) is assumed at 50%. The expected damages for flooding, taking compartmentalization in to account, is calculated with equation 33.

$$D_{flooding} = (1-P_{f;\ compartment}) * D_{6hours} + P_{f;compartment} * D_{48hours} \qquad (33)$$

The compartment works will need to be organised and trained annually, generating annual operational cost. We assume the initial investment for the calamity plan and the operational cost to be equal to those of a restriction of the traffic loads, which are conservative estimates. The investments, operational cost, flood damages and flood risk are calculated in Table 38; the table also shows the expected total cost of this option.

TABLE 40 TOTAL EXPECTED COST OF COMPARTMENTALIZATION

Flood scenario	Investment [mln euro]	Operational cost [mln euro/yr]	Expected flood damages [mln euro]	Flood risk [mln euro/yr]	Benefit/ Cost ratio [-]	Total cost [mln euro]
1	0.05	0.05	1.1	0.0	0.3	1.7
2	0.05	0.05	140	0.10	1.7	2.7
3	0.05	0.05	240	10	157	190
4	0.05	0.05	310	0.63	6.4	1.2
5	0.05	0.05	48	0.04	0.7	1.7
6	0.05	0.05	0.6	$2.2 * 10^{-4}$	0	1.0

We conclude that compartmentalization is not cost effective for sections 1, 13 and 17, which has a benefit cost ratios below 1. For all other sections positive benefit cost ratios are found.

Note that the compartments need to be chosen outside the influence area of the initial breach; if they are chosen inside the influence area floods may occur in adjacent polders where the outer slope of the flood defences became unstable due to the drop of the water levels. This is an important point for flood defence managers deciding upon breach closure measures or compartments in canals during a calamity.

7.3 COMPARISON OF TOTAL COST

This paragraph summarizes the results found in the preceding paragraph. The following table shows the benefit cost ratios of all interventions discussed, the green boxes show which interventions have positive cost benefit ratios.

TABLE 41 TOTAL EXPECTED COST OF COMPARTMENTALIZATION , COST EFFECTIVE MEASURES SHOWN IN GREEN

Flood scenario	Reinforcements	Reducing hydraulic loads	Restricting traffic loads	Compartments
1	0.13	-	0.4	0.3
2	-	0	2.2	1.7
3	110	0	101	157
4	0	0	0	6.4
5	-	-	0.5	0.7
6	-	-	0	0

The total cost of every option is shown, for each scenario in each canal system, in the following figures:

FIGURE 53 TOTAL COST OF INTERVENTIONS IN THE SCHERMER CANAL SYSTEM

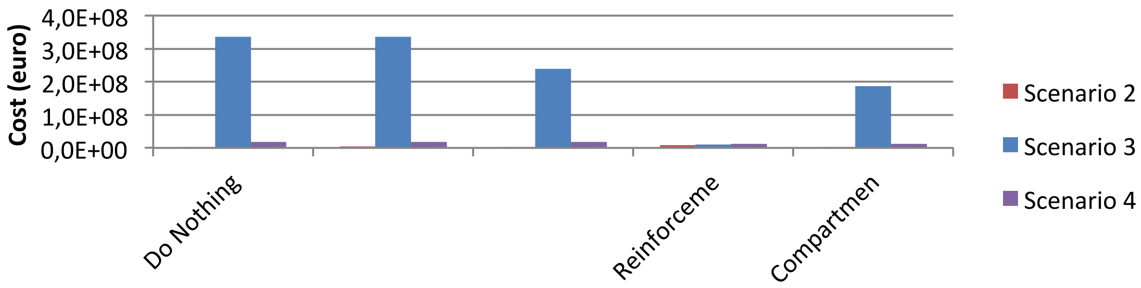
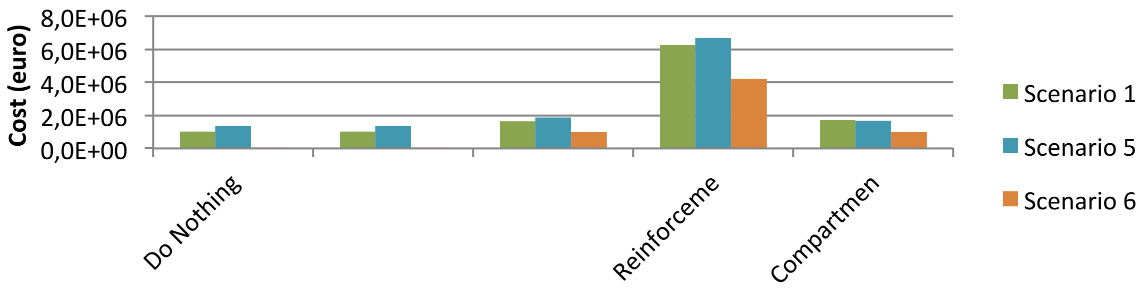


FIGURE 54 TOTAL COST OF INTERVENTIONS IN THE VRNK CANAL SYSTEM



Note that these are not actual costs made, but expected cost: a summation of the investments, operational cost and expected costs due to flooding.

7.4 CONCLUDING REMARKS

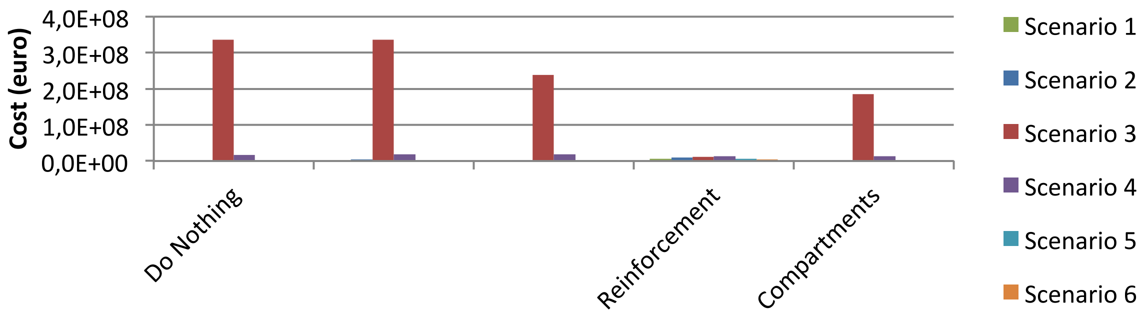
In this chapter, we demonstrated how the results of a flood risk assessment for regional flood defences can be used to make cost benefit assessment for interventions in the system. Note that the results in this assessment are presented to illustrate the method.

Currently interventions in the system are based on the assessment of regional flood defences, wherein weakest sections are prioritized over stronger sections. However, the weakest sections within a system may very well not be the sections where interventions are most cost effective. The expected total costs of several interventions aiming to reduce flood risk in a system of regional flood defences were compared. Note that the calculated total costs are based on assumptions by the research team; further research is recommended to determine the actual total costs. Based on the comparison made in the last paragraph, the following is concluded:

- Reducing the hydraulic loads on regional flood defences is not cost effective, as the influence on the flood risk of reducing the drain stop level is negligible;
- Restricting traffic loads on regional flood defences can be cost effective, if instability is the governing failure mechanism for the considered section;
- Compartmentalization of canals, to reduce the consequences after a flood, can be a cost effective intervention;
- Reinforcements prove to be the most cost effective.

The total cost of all scenarios is summarized in the following figure.

FIGURE 55 TOTAL COST PER INTERVENTION, FOR ALL SCENARIOS



We conclude that scenario 3 has the highest total cost, for this scenario reinforcement is the most cost effective compared to the other considered interventions. This section has a high failure probability for piping and instability and high consequences for flooding in case of a breach.

8

DISCUSSION AND CONCLUSIONS

This chapter discusses the results of the applied methodology and how these are influenced by the assumptions made. Furthermore, concluding remarks concerning both the methodology and case study results are discussed and recommendations for further research given.

8.1 DISCUSSION

The methodology and case study results were discussed with several experts, who provided useful insights in the validity of the results and the consequences of the assumptions made.

8.1.1 LOAD UNCERTAINTIES

Chapter 3 concluded that the governing loads for regional flood defences are the water level in the canal and the top load on the flood defences. Drought and earthquake loads were omitted. For the considered case study, we assumed that the contribution of droughts and earthquake loads to the failure probability will be negligible. However, we do recommend further developing the methodology to include a reliability assessment for both loads, as they may be dominant in other regions of the country (e.g. earthquakes in Groningen).

Considering the water levels, a method was developed to determine the combined water statistics, based on an empirical distribution of observed water levels and the probability of failure for the drain stop in the canals. This probability was estimated with the amount of independent water level peaks exceeding the 'drain stop level'. Changes in governance of water boards may lead to a different statistic, due to different regulation of the water levels in the canals. This may have resulted in some errors in the results, but it is expected that the influence of these changes is small.

No statistics were available for the traffic loads for regional flood defences. In order to compute failure probabilities, experts were asked to provide a 5^{th}, 50^{th} and 95^{th} quantile of the expected load on regional flood defences and their view on the correlation with the water levels. Based on their answers, triangular distributions were made for traffic loads on 'green' and 'grey' flood defences. Experts had different views on whether or not traffic loads have to be taken in to account, as they have a considerable influence on the reliability of the flood defence. We computed the failure probabilities with and without traffic loads. Based on the large difference in failure probabilities found, we recommend discussing the requirement of traffic loads in the safety assessment.

8.1.2 STRENGTH UNCERTAINTIES

The probability of flooding was determined for the three governing failure modes for regional flood defences: overflow, piping and instability. Based on the results the following is concluded for these mechanisms:

Overflow: The probability of overflow in this flood defence system is negligible; the retaining height of the flood defences is well above the water levels in the canals.

Piping: The hydraulic heads over the considered regional flood defences results in rather high failure probabilities. However, due to the resistance of intrusion of water from the canal to the aquifer no signs of piping were observed in the area. Therefore a reduced hydraulic head is taken in to account, which is based on field tests of the permeability of the subsoil layers. When taking the reduced hydraulic head in to account, more accurate failure probabilities are found, considering these flood defences have not failed in the last decennia.

Note that these flood defences may still be at risk for piping if the geological profile is changed, for example due to dredging works or erosion of the bottom of the canals. This may expose the flood defence to the maximum hydraulic head, due to direct contact between the water in the canal with the aquifer below, which results in high probability of piping. In follow up research, we recommend including the effect of these events. The piping probability can be computed for scenarios with and without the reduced head, and then combined.

Instability: The probability of failure for instability largely depends on the combination of the phreatic line and top loads. The influence of the outer water level for a given phreatic line is very low. We recommend performing field tests to determine the actual distribution of the phreatic line in the defence more accurately.

8.1.3 PROVEN STRENGTH

Proven strength can potentially reduce the failure probabilities for regional flood defences, as the differences between the average and extreme water levels are very small. We concluded that, at this stage, a proven strength assessment will only be effective for the piping failure mechanism. An exact method of Bayesian Updating is recommended for this purpose, because the estimates obtained with First Order Survival Updating were unrealistic.

8.1.4 FLOOD RISK

PROBABILITY OF FLOODING

The flood defence system was divided in groups of sections, each representing a flood scenario. For each scenario, the probability of flooding was computed for the weakest section within the group, which represents the failure probability of the scenario. This assumption is correct if the largest uncertainty in the method is governed by the loads. However, we concluded that the influence of the load uncertainty is low compared to the strength uncertainty. The flood risk approach used therefore may underestimate the probability of flooding, because not all sections were taken in to account.

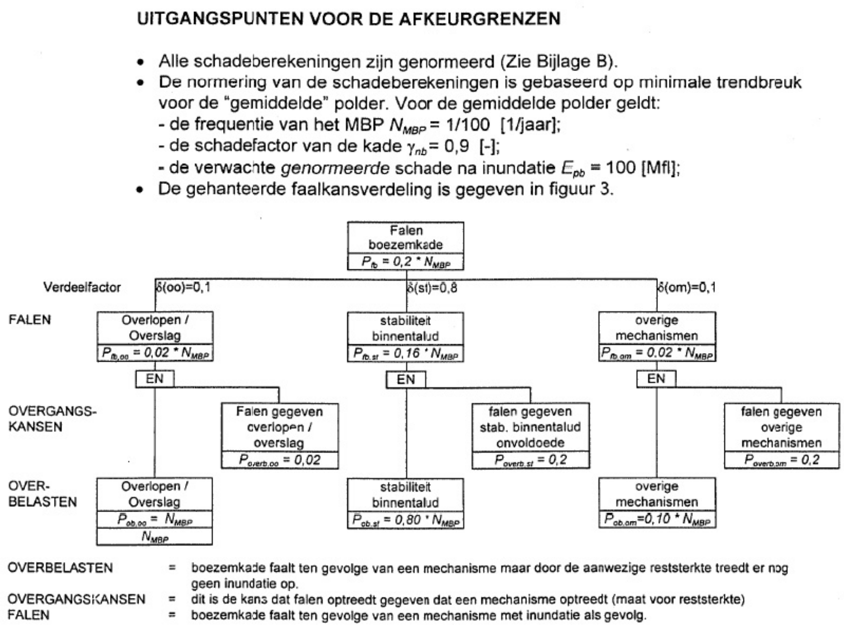
CONSEQUENCES OF FLOODING

It is assumed that floods resulting from a breach in a regional flood defence system only have inundation depths in the order of decimetres, except for the occasional deeper polders. We therefore only considered economic damages, no loss of life. HIS SSM and WSS were used to compute the consequences of flooding for each flood scenario. The resulting consequences both lie in the same order of magnitude. We assumed that the WSS model determines the flood damages for regional flood defences more accurately than HIS SSM, because, it is more accurate for low inundation depths.

FLOOD RISK

The flood risk was computed by multiplying the probability of flooding with the expected damages, see Table 35. According to the IPO safety standards, the failure probability of these flood defence system is limited to 20% of the probability the drain stop level, which is 1/100 per year (see chapter 4). The safety standard for this system is therefore required to be 1/500 per year, according to the IPO safety standards (Stowa, 2004). The corresponding failure budget is shown in Figure 56.

FIGURE 56 **DISTRIBUTION OF FAILURE PROBABILITY ACCORDING TO IPO NORMS** (IPO, 1999)



In Table 42 the failure probabilities found are compared to the results of the safety assessment for the Heerhugowaard polder.

TABLE 42 COMPARISON OF FAILURE PROBABILITY WITH SAFETY ASSESSMENT

Critical section	Overflow [yr-1]	Piping [yr-1]	Instability [yr-1]	Failure probability [yr-1]	Assessment result (2014)
4	$< 10^{-5}$	0.0005	0.0033	0.0038 (1/26)	Not Safe
9	$< 10^{-5}$	$6.4 * 10^{-5}$	0.0007	0.0007 (1/1400)	Safe
11	$< 10^{-5}$	0.0178	0.0254	0.0428 (1/23)	Not Safe
12	$< 10^{-5}$	0.0019	0.0001	0.0020 (1/500)	Safe
13	$< 10^{-5}$	0.0004	0.0004	0.0008 (1/1250)	Safe
17	$< 10^{-5}$	0.0004	$< 10^{-5}$	0.0004 (1/2500)	Safe

When we analyse the case study results, we conclude that the probabilities found for overflow are lower than the required failure budget of $0.02 * 10^{-2}$ per year. For instability a maximum failure probability of $1.6 * 10^{-3}$ per year is required, which is lower than the probabilities found for sections 4, 11 and 12 of the case study. In addition, for piping a maximum failure probability of $2 * 10^{-4}$ is required, which is lower than the probability of sections 11 and 12.

The results of a flood risk assessment of regional flood defence systems can be compared with the results of flood risk from primary flood defences. To do so, more research is required in system behaviour of several regional flood defence systems, as one primary flood defence system often surrounds several of these systems. Aspects which should be taken in to account in such a project range are the influence of several polders, the occurrence of relief within the canal system, and compartmentalization within the canals *and* within the polders.

8.1.5 COST BENEFIT ASSESSMENT

We demonstrated how the results of a flood risk assessment for regional flood defences can be used to make a cost benefit assessment for interventions in the system. The expected total costs of several interventions aiming to reduce flood risk were compared. Note that the calculated total costs are based on assumptions by the research team; further research is recommended to determine the actual total costs. Based on the comparison made in the last paragraph, the following was concluded:

- Reducing the hydraulic loads on regional flood defences is not cost effective, as the influence on the flood risk of reducing the drain stop level is negligible;
- Restricting traffic loads and/or compartmentalization of canals can be cost effective when instability is the governing failure mechanism;
- Reinforcements prove to be the most cost effective.

8.2 CONCLUDING REMARKS

We conclude that the flood risk approach can be applied to regional flood defence systems using the data used in the safety assessment of regional flood defences. The approach not only provides insight in the failure probabilities of the flood defences, but also in the corresponding consequences of flooding and therefore the flood risk. The results can be used to compare the flood risk within the system and prioritize interventions based on the expected risk reduction and cost effectiveness.

The governing failure modes for regional flood defences are piping and instability. The reliability of these systems is governed by uncertainty in traffic loads and strength of the structure. The influence of the hydraulic load uncertainty, i.e. the difference in water levels, is negligible. To obtain more accurate results we recommend investigating how the data obtained in the assessment can be used more effectively in the flood risk approach or proven strength assessments. For example, more insight in the relation between the outer water level and rainfall on the phreatic line may provide better estimates of the probability density function of the phreatic line. For these assessments, insights obtained from the water board dike supervisors can play a useful rule.

Temporary measures to increase the strength of flood defences during calamities have not been considered in this report. Considering the case study results, we conclude that these measures should focus on increasing the strength for piping and instability, as overflow is negligible. The potential effectiveness of these measures can be compared with more traditional reinforcements of the flood defences and/or consequence reducing measures such as compartmentalization of the canals. The framework determined in the previous STOWA research at the TU Delft can be used for this purpose (Lendering et al., 2013).

This report focussed on the flood risk from regional flood defences loaded by canal systems; however, these flood defences are also used to protect polders from flooding from the larger lakes and several ‘regional rivers’ (e.g. the Dommel). In these systems, the load uncertainty (i.e. water level difference) can be more influential. Moreover, it is questionable whether or not hydraulic short circuiting is present in these canals.

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APPENDICES

APPENDIX A

EXPERT ELLICITATION TRAFFIC LOADS

This appendix includes the result of ellicitation among experts of traffic loads on top of regional flood defences. The results are presented in the first section, followed by the questionnaire used.

A.1 RESULTS

TABLE 43 RESULTS EXPERT ELLICITATION TRAFFIC LOADS

Type	Average water level	Extreme water level	5th quantile kN/m2	50th quantile kN/m2	95th quantile kN/m2
Green flood defence	No correlation	No / negative correlation	0,5	2	5
Grey flood defence	No correlation	No / positive correlation	10	15	25

From the received answers can be concluded that distinction has to be made between a green and grey flood defence, meaning a flood defence without a road on top and one with a road on top. Green defences will only have small traffic loads, corresponding with small cars for maintenance. On grey defences large traffic loads can be expected, corresponding with the loads advised by the assessment. In the probability calculations for instability both situations will be compared: no traffic loads versus the loads on grey flood defences.

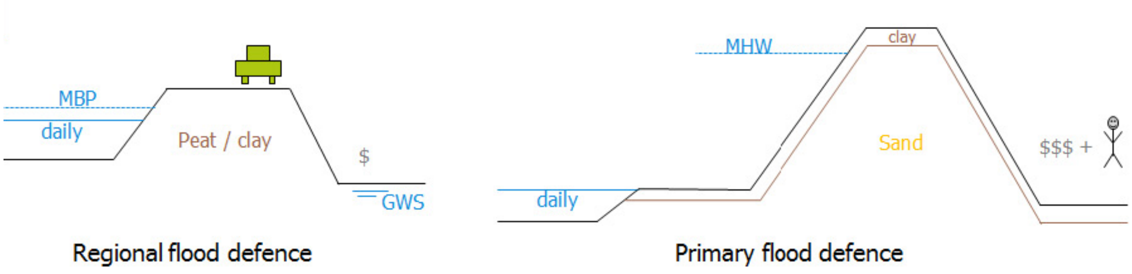
No correlations between the traffic load and water levels is expected with average water levels, the experts all agreed on this point. However, they did not agree on the correlation between the traffic load and the extreme water levels as can be seen in the table.

A.2 QUESTIONNAIRE

INTRODUCTION

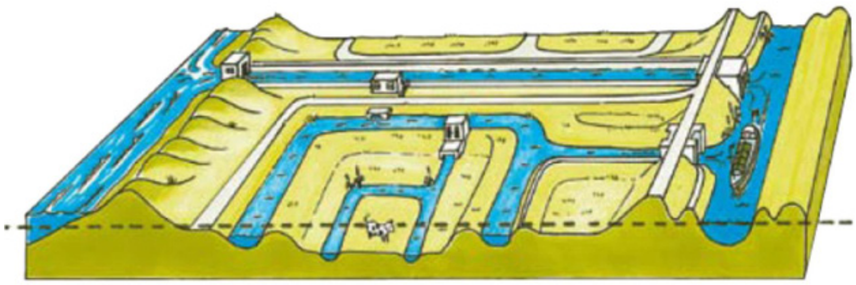
This questionnaire is concerned with the ellicitation of uncertainty distributions of traffic loads on top of regional flood defenses. A comparison of regional and primary flood defenses is made in figure 1, and explained in the following section.

FIGURE 57 COMPARISON OF REGIONAL FLOOD DEFENCES (LEFT) WITH PRIMARY FLOOD DEFENCES (RIGHT)



Regional flood defences typically have lower retaining heights (about 3 meters) than primary flood defences (about 6 meters). The dikes are often built with peat / clay material, which is present in the large polders protected by these flood defences. The probability of exceedance of water levels inside regional systems varies between 1/10 to 1/1,000 per year. Regional flood defences protect the surrounded area from 'inside water', which is excess water from the polder pumped on to the storage canals. The water levels inside the canals are determined by the inflow from the polder and the outflow on the river or sea; which are regulated systems, see figure 2.

FIGURE 58 SCHEMATIC VIEW OF POLDER CANAL SYSTEM



The resulting difference in daily average water level and extreme water level is limited to several decimetres. The protected area of a regional flood defence system is often smaller than the area protected by primary flood defences. The amount of water which can flow in a polder after a breach in a regional flood defence system is limited because the canals are closed systems. This is assumed to result in low inundation depths, low economic damage and no loss of life.

TRAFFIC LOADS

The combination of hydraulic loads (high water levels) and traffic loads is governing for the stability of the flood defence. For this failure mechanism the water levels inside the canals and rainfall determine the shape of the phreatic line inside the flood defence. The combination with traffic loads on top of these flood defences determines the effective soil pressures. In the assessment of regional flood defences traffic loads are taken in to account as a permanent vertical load on top of the flood defence (Stowa, 2007), with a magnitude of 13 kN/m^2 over a length of 10 meter and a width of 2.5 meter. This is assumed representative for one truck on top of the flood defence with a weight of 3250 kilograms (for example: a truck bringing sand bags)

The question remains if this load combination will actually occur: a positive and negative correlation with extreme water levels can be assumed:

- A positive correlation when it is assumed that during extreme water levels inside the canals trucks are used to bring sand bags to locations where the stability of the dike is critical.
- A negative correlation when it is assumed that during extreme water levels no traffic is allowed on the flood defence due to possible instability of the dike.

DATA OF INTEREST

In this questionnaire we would like to ask you to answer two questions regarding the traffic loads:

1 What is the correlation between the extreme water levels and traffic loads: positive or negative?

2 What are the 5^{th}, 50^{th} and 95^{th} quantiles of the uncertainty distribution of the magnitude of the traffic load?

The 5^{th} quantile means that you think with probability 95% that the traffic load exceeds the value you are providing, and conversely with probability 5% the traffic load will be lower than the value you are providing. Same reasoning apply for the 50^{th} and 95^{th} quantiles. An example is shown below.

5^{th} quantile: a 50^{th} quantile: b 95^{th} quantile: c

The interpretation is:

- with 95% probability the traffic load exceeds a
- with 50% probability the traffic load exceeds b
- with 5% probability the traffic load exceeds c

with $a < b < c$.

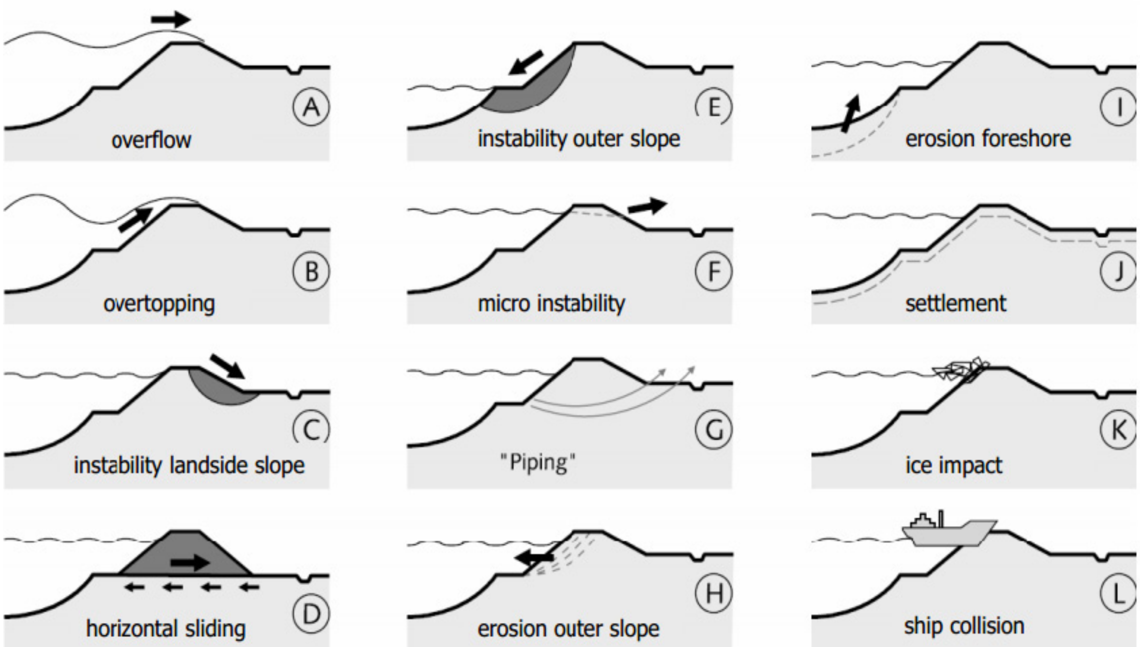
APPENDIX B

QUICK SCAN FAILURE MECHANISMS

In this appendix, a quick scan is made to determine which failure mechanisms are governing for regional flood defences. The quick scan is based, among other, on whether or not the considered failure mechanisms will directly lead to breaching of the flood defence and flooding of the protected area. The results of the last assessment of flood defences at the case study are also used to determine which failure mechanisms are governing. The following failure mechanisms have to be treated according to the assessment (Stowa, 2007):

- Overflow / overtopping (HT);
- Piping (STPI)
- Inner slope instability (STBI), including horizontal sliding;
- Outer slope instability (STBU);
- Micro instability (STMI);
- Revetments (STBK);
- Foreshore instability (STVL).

FIGURE 59 FAILURE MECHANISMS OF FLOOD DEFENCES (FLOOD DEFENCES, 2014)



B.1 OVERFLOW / OVERTOPPING (HT)

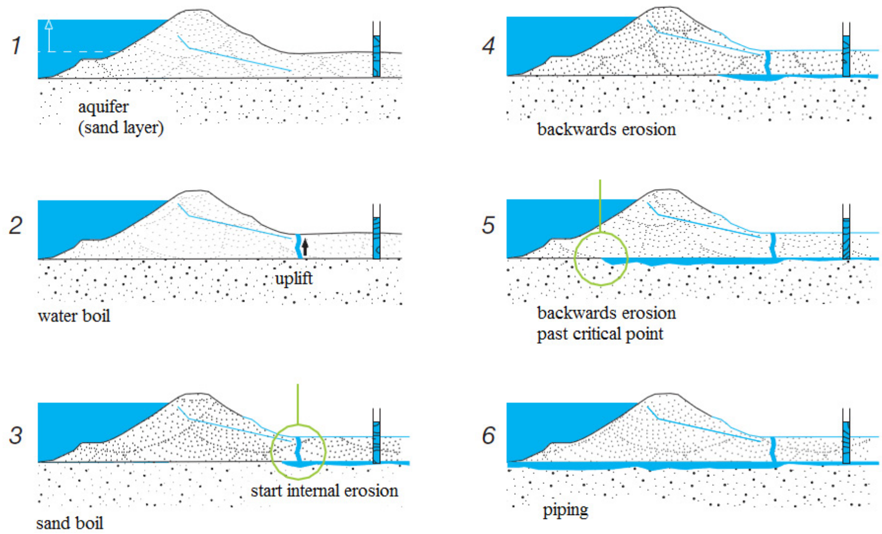
Overflow / overtopping will occur when the flood defence has insufficient retaining height. Overtopping occurs when waves overtop the dike. Wave loads on regional dikes along canals will be neglected due to the limited fetch in canal systems and the short duration of these wave loads (see also chapter 4). This report will therefore focus on overflow of the dike, which occurs when the dike has insufficient retaining height. *In the assessment of the case study several points were found with insufficient retaining height, showing that this failure mechanism cannot be ignored.*

Overflowing of the dike will lead to flooding of the polder when the storage capacity behind the dike is insufficient. Breaching of the dike may also cause flooding, which occurs when overflowing leads to erosion of the inner slope or saturation of the dike, causing instability. The maximum amount of overflow allowed is 0.1l/s/m, corresponding with the quality of the grass cover layer. If this amount is exceeded the resistance to erosion of the crest and inner slope need to be checked.

B.2 PIPING (STPI)

Piping occurs when a head difference over a flood defence causes uplift of the impermeable layer on the inland side after which backward erosion forms channels or pipes in the aquifer under the flood defence. These channels can undermine the flood defence when they become so long that they connect the inner and outer water level. The different phases of piping are explained with Figure 31.

FIGURE 60 **DEVELOPMENT OF PIPING** (VRIJLING ET AL., 2010)



The water in the canals protected by regional flood defences is not always in direct contact with the aquifer, due to the presence of impermeable layers on the bottom of the canals. This may impede the possibility of piping, because there is no connection between the water levels in the canal and the aquifer. As stated in chapter 2, hydraulic short circuiting is required for piping to be able to develop. This may occur when, due to some reason, the water in the canals can flow into the aquifer allowing pressure to build up behind the dike.

Recent research has shown that hydraulic short circuiting is likely to occur in regional flood defences. This may lead to an increase of the inner water pressure behind the flood defence and thus allow for piping to occur (Kwakman et al., 2013). The development of water pressure behind the dikes needs to be investigated to determine the hydraulic head which has to be taken in to account for piping calculations. Research at the considered case study proved that a reduced hydraulic head can be taken in to account (see chapter 6). However, piping may not be ruled out completely, even though the subsoil does not favour its occurrence. Therefore it is considered to be governing in the reliability assessment.

B.3 INNER SLOPE INSTABILITY (STBI)

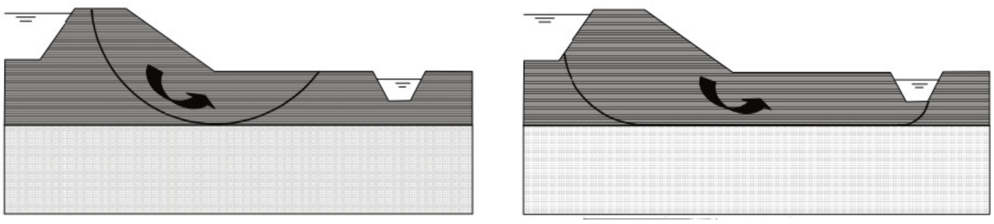
Inner slope instability occurs when major soil masses slide of the inner slope of the dike. Distinction is made between two situations: sliding of the inner slope along circular planes or sliding of the inner slope along straight planes. The stability of the dike is defined by the shear resistance of the soil masses.

Sliding of the inner slope can be the result of an increase of the phreatic line in the dike, due to infiltration of water. The infiltration of water increases the weight of the dike and decreases the effective soil pressures. Sliding along straight planes can occur when the shear resistance of the cover layers behind the dike is reduced due to the upward water pressure under these layers. These upward water pressures may result in uplifting of the cover layer, reducing the effective soil pressure to zero.

HORIZONTAL SLIDING

Another form of macro instability is horizontal sliding, which can occur when the weight of the flood defence decreases. The reduced weight may lead to insufficient shear capacity in the subsoil, which leads to sliding of the flood defence. This failure mechanism can be governing in peat dikes, where the soil material loses weight in dry periods (Keizer, 2008). The dike breach at Wilnis is one example of horizontal sliding due to droughts, see Figure 18.

FIGURE 61 **INNER SLOPE INSTABILITY: CIRCULAR PLANE (LEFT) AND STRAIGHT PLANE (RIGHT)** (STOWA, 2007)



In the assessment of regional flood defences, the inner slope stability is checked for a both high water loading and droughts. The governing loads consist of a combination of extreme water levels and rainfall, which result in an increase of the phreatic line in the dike, and traffic loads on top of the dike. In most cases, this failure mechanism is considered governing for regional flood defences. As instability may directly lead to breaching it will be included as one of the governing failure mechanisms in the reliability assessment in this report.

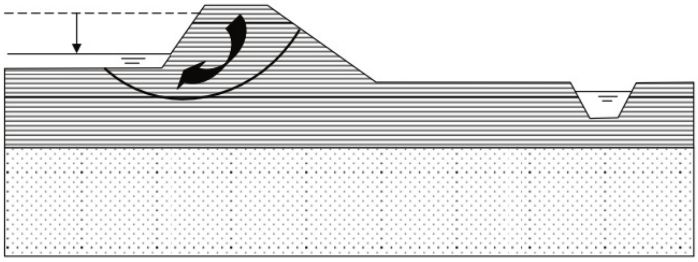
B.4 OUTER SLOPE INSTABILITY (STBU)

Outer slope instability occurs when large soil masses slide of the outer slope of the flood defence. Whether or not this leads to flooding depends on the magnitude of the sliding and the possibility of water overflowing the reduced crest of the flood defence. The guidelines for the assessment of regional flood defences state that not all dike sections need to be checked for outer slope instability, because not all defences are subject to loads which can lead to outer slope sliding (Stowa, 2007).

This mechanism can have several causes, see Figure 62:

- Extreme low water levels in the canals due to natural causes or human interventions;
- A drop of the water level in the canal due to a calamity (i.e. breaching) elsewhere;
- Deepening of the canal due to dredging or erosion;
- Large top loads, for example from traffic loads.

FIGURE 62 **OUTER SLOPE SLIDING ALONG A CIRCULAR PLANE** (STOWA, 2007)

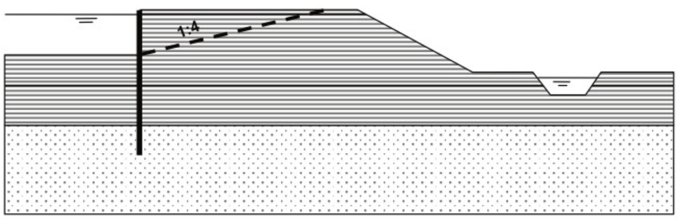


Outer slope sliding is often the result of low water levels in the canals. Lower water levels can be the result of a breach in the regional flood defence system. Suppose this occurred, leading to sliding of the outer slope of the flood defence. If the reduced crest height is still higher than the water levels in the canal no additional flood will occur. However, if the breach is closed or compartments are made in the canal, the water levels will increase back to the original level. Now the reduced crest height can be overflown leading to another flood. The probability of flooding of the considered section will increase significantly after the initial breach, even when this section is not overflown due to the reduced crest height.

RETAINING WALLS

Often soil retaining structures are present in the outer slope of a regional flood defence, as shown in Figure 63. Failure of such retaining walls in earthen structures will lead to outer slope sliding of the flood defence. In the assessment of regional flood defences it is assumed that failure of the retaining wall will not lead to flooding provided that an outer slope of 1:4 is present behind the retaining wall. If this requirement is not met, the retaining wall needs to comply with standards of retaining walls in safety class 2, according to (de Gijt & Broeken, 2013).

FIGURE 63 **RETAINING WALL IN A REGIONAL FLOOD DEFENCE (STOWA, 2007)**



Breaches due to outer slope instability are considered to be a second order effect of breaches which had already occurred. Moreover, the area in the canal where the water levels drop significantly often lies within the boundaries of a single flood scenario. A second breach within these boundaries will not lead to a significant increase of the flood damage; the only increase of damage is the result of the damage to the flood defence. Therefore, the flood risk of that polder will not be significantly higher when taking outer slope instability in to account. Based on these arguments the contribution of outer slope instability to the risk of flooding of a single polder is assumed negligible.

B.5 MICRO INSTABILITY (STMI)

Micro instability occurs when sand parts are washed out of the inner slope of the dike due to seepage, which damages the inner slope. This mechanism can potentially lead to local instability (sliding) in flood defences which contain a permeable core; in flood defences with an impermeable core this failure mechanism does not play a role. It is assumed that after local sliding of the inner slope due to micro instability the flood defence will have sufficient strength remaining that no breach will occur. Therefore this failure mechanism is not considered to be governing in the reliability assessment. This assumption is also supported by the fact that it is also not taken in to account in the reliability assessment of primary flood defences. Furthermore, in the assessment of the case study the regional flood defences all comply with the required safety standards.

B.6 REVETMENTS (STBK)

The outer slope of regional flood defences is often protected by grass or other forms of vegetation; on dike sections subject to relatively large wave loads stone revetments are used. Damages to the revetments only play a role at sections where these wave loads play a significant role: on lakes with large wind fetches or in canals where a lot of shipping is present. These load situations are of short duration: large wind waves are the result of storms which have an average duration of several hours, while shipping waves have much shorter durations (order of minutes). Damages to revetments will not lead to breaches in the regional flood defence, as the duration of loading is short. Water boards are expected to be able to repair any small damages to revetments before breaches will develop.

FORESHORE STABILITY

Foreshore instability plays a negligible role in regional flood defences, because the canals surrounded by regional flood defences are often of limited width and depth. Deep gullies or large excavations which can lead to sliding of the foreshore are absent. In the assessment of regional flood defences the check often only consists of a verification of the absence of deep gullies. It will not be treated further in this report.

B.7 PIPES INSIDE FLOOD DEFENCES

Failure of pipes inside regional flood defences may lead to breaching, if these failures result in instability of the structure. In the current report, this failure mechanism will not be taken in to account, but it is recommended to be investigated further. Pipes may fail during normal conditions or extreme conditions. The difference in water levels is so low that no significant difference in failure probability of pipes between both events is expected. Conditional probabilities will have to be taken in to account, as breaching will only occur if these failures lead to instability of the flood defence.

APPENDIX C

WATER LEVEL OBSERVATIONS

WOGMEER

FIGURE 64 WATER LEVEL OBSERVATIONS AT WOGMEER

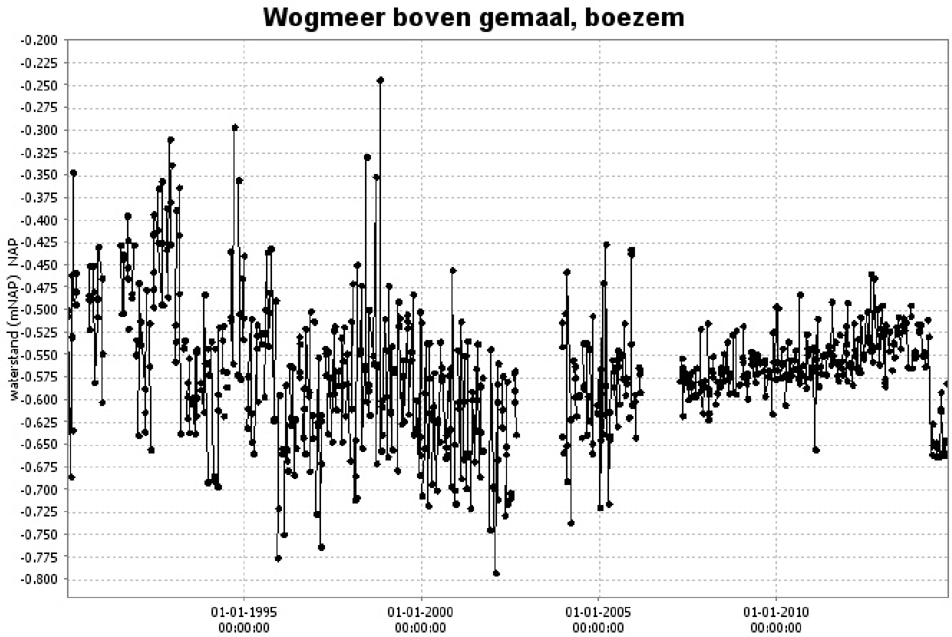
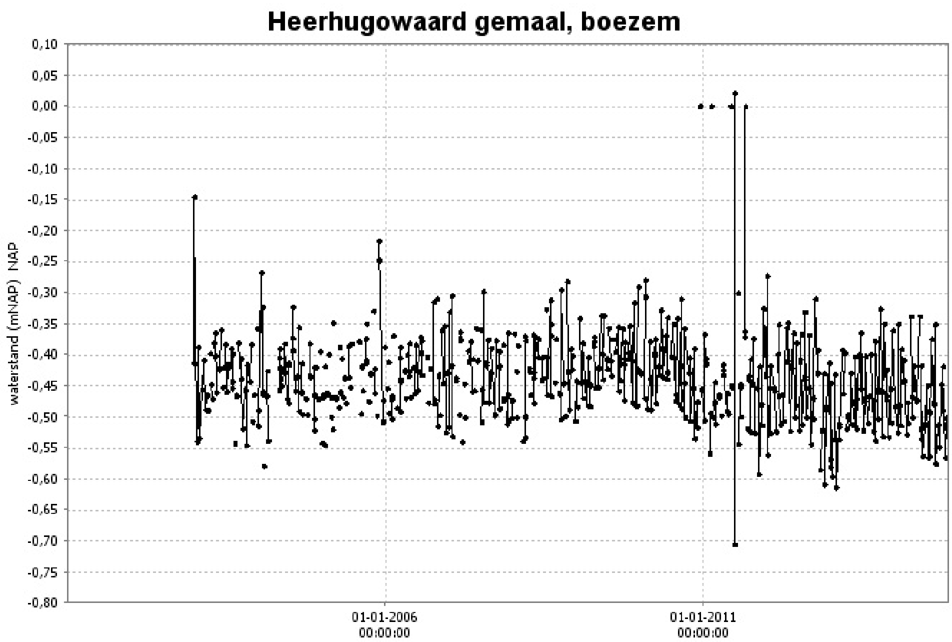
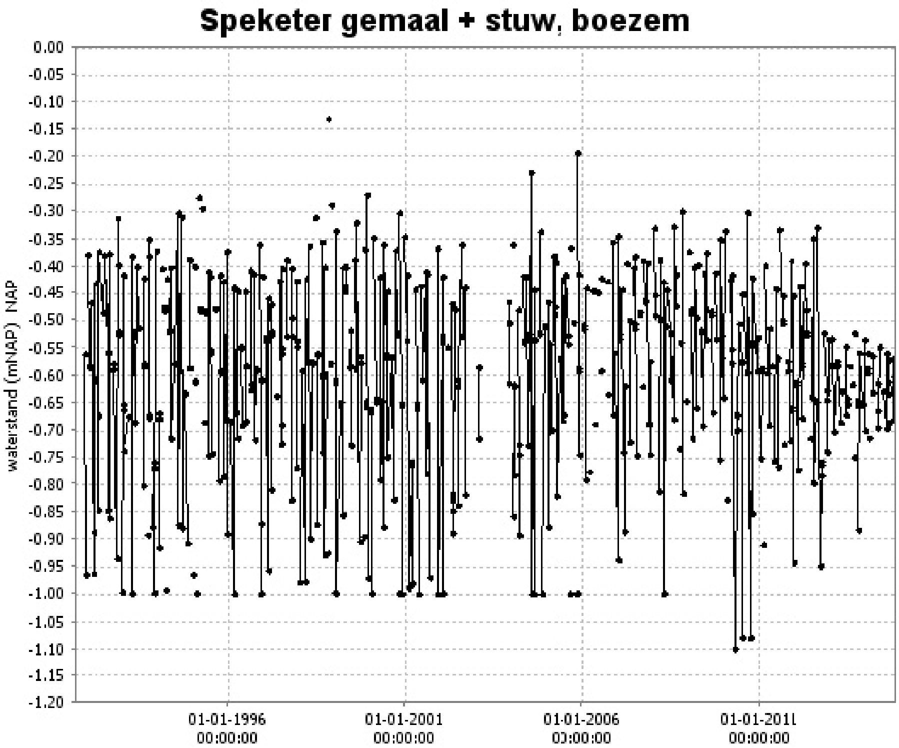


FIGURE 65 WATER LEVEL OBSERVATIONS AT HEERHUGOWAARD



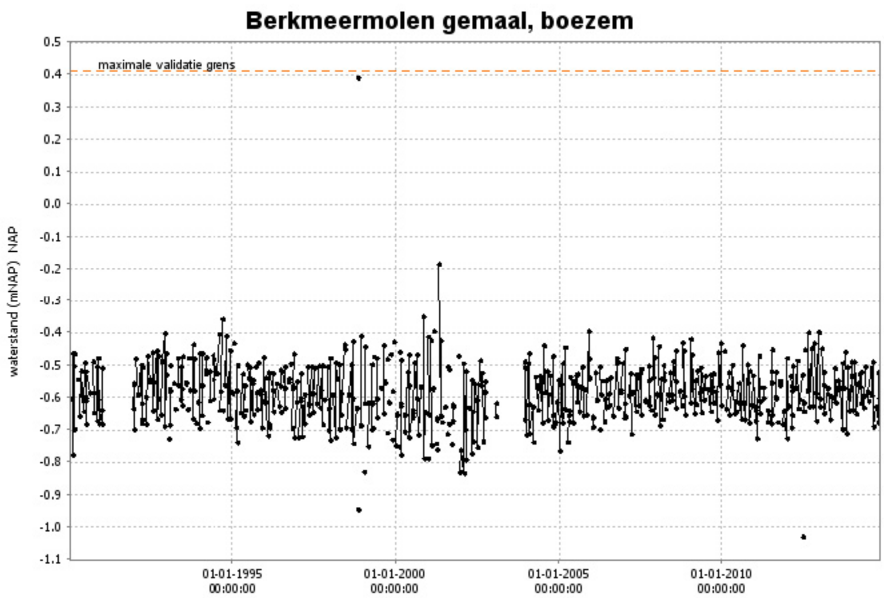
SPEKETER

FIGURE 66 WATER LEVEL OBSERVATIONS AT SPEKETER



BERKMEER

FIGURE 67 WATER LEVEL OBSERVATIONS AT BERKMEER



APPENDIX D

PROBABILITY TABLE OF RESULTING WATER LEVEL STATISTICS

WOGMEER

CDF	Water level
0.000045	-0.511779
0.006738	-0.478973
0.035674	-0.459613
0.082085	-0.445801
0.135335	-0.435043
0.188876	-0.426224
0.239651	-0.418749
0.286505	-0.412258
0.329193	-0.406521
0.367879	-0.401380
0.606531	-0.367340
0.716531	-0.347252
0.778801	-0.332920
0.818731	-0.321758
0.846482	-0.312608
0.866878	-0.304850
0.882497	-0.298115
0.894839	-0.292163
0.904837	-0.286828
0.951229	-0.251508
0.967216	-0.230665
0.975310	-0.215793
0.980199	-0.204211
0.983471	-0.194717
0.985816	-0.186668
0.987578	-0.179680
0.988950	-0.173503
0.990050	-0.167968
0.995012	-0.131319
0.996672	-0.109692
0.997503	-0.094261
0.998002	-0.082244
0.998335	-0.072392
0.998572	-0.064040
0.998751	-0.056789
0.998890	-0.050380
0.999000	-0.044637

0.999500	-0.006610
0.999667	0.015831
0.999750	0.031842
0.999800	0.044312
0.999833	0.054534
0.999857	0.063200
0.999875	0.070724
0.999889	0.077374
0.999900	0.083333

HEERHUGOWAARD

CDF	Water level
0.000045	-0.388155
0.006738	-0.365113
0.035674	-0.351144
0.082085	-0.341007
0.135335	-0.333013
0.188876	-0.326394
0.239651	-0.320736
0.286505	-0.315789
0.329193	-0.311389
0.367879	-0.307425
0.606531	-0.280656
0.716531	-0.264428
0.778801	-0.252652
0.818731	-0.243364
0.846482	-0.235675
0.866878	-0.229102
0.882497	-0.223355
0.894839	-0.218244
0.904837	-0.213638
0.951229	-0.182541
0.967216	-0.163688
0.975310	-0.150007
0.980199	-0.139218
0.983471	-0.130285
0.985816	-0.122649
0.987578	-0.115972
0.988950	-0.110035
0.990050	-0.104685
0.995012	-0.068557
0.996672	-0.046656
0.997503	-0.030763
0.998002	-0.018228
0.998335	-0.007851
0.998572	0.000000
0.998751	0.000000
0.998890	0.000000
0.999000	0.000000
0.999500	0.000000
0.999667	0.000000
0.999750	0.000000
0.999800	0.000000
0.999833	0.000000
0.999857	0.000000
0.999875	0.000000
0.999889	0.000000
0.999900	0.000000

SPEKETER

CDF	Water level
0.000045	-0.438607
0.006738	-0.399966
0.035674	-0.378480
0.082085	-0.363715
0.135335	-0.352530
0.188876	-0.343559
0.239651	-0.336092
0.286505	-0.329708
0.329193	-0.324143
0.367879	-0.319215
0.606531	-0.295673
0.716531	-0.289428
0.778801	-0.285136
0.818731	-0.281885
0.846482	-0.279277
0.866878	-0.277107
0.882497	-0.275251
0.894839	-0.273633
0.904837	-0.272201
0.951229	-0.263116
0.967216	-0.258065
0.975310	-0.254594
0.980199	-0.251964
0.983471	-0.249855
0.985816	-0.248099
0.987578	-0.246599
0.988950	-0.245290
0.990050	-0.244132
0.995012	-0.236784
0.996672	-0.232699
0.997503	-0.229891
0.998002	-0.227764
0.998335	-0.226059
0.998572	-0.224639
0.998751	-0.223425
0.998890	-0.222366
0.999000	-0.221429
0.999500	-0.215487
0.999667	-0.212183
0.999750	-0.209912
0.999800	-0.208192
0.999833	-0.206812
0.999857	-0.205664
0.999875	-0.204682
0.999889	-0.203826
0.999900	-0.203068

BERKMEER

CDF	Water level
0.000045	-0.513101
0.006738	-0.482333
0.035674	-0.464667
0.082085	-0.452279
0.135335	-0.442753
0.188876	-0.435023
0.239651	-0.428524
0.286505	-0.422922
0.329193	-0.418001
0.367879	-0.413616
0.606531	-0.385155
0.716531	-0.368812
0.778801	-0.357353
0.818731	-0.348540
0.846482	-0.341389
0.866878	-0.335377
0.882497	-0.330195
0.894839	-0.325643
0.904837	-0.321587
0.951229	-0.299746
0.967216	-0.298936
0.975310	-0.298367
0.980199	-0.297930
0.983471	-0.297576
0.985816	-0.297278
0.987578	-0.297021
0.988950	-0.296795
0.990050	-0.296594
0.995012	-0.295288
0.996672	-0.294539
0.997503	-0.294013
0.998002	-0.293609
0.998335	-0.293281
0.998572	-0.293005
0.998751	-0.292768
0.998890	-0.292559
0.999000	-0.292373
0.999500	-0.291165
0.999667	-0.290472
0.999750	-0.289985
0.999800	-0.289611
0.999833	-0.289308
0.999857	-0.289053
0.999875	-0.288833
0.999889	-0.288640
0.999900	-0.288468

APPENDIX E

PIPING DATA

The following table shows the data used for piping reliability calculations. The explanation of each variable, including the distributions used can be found in chapter 5.

TABLE 44 **VARIABLES FOR PIPING CALCULATION**

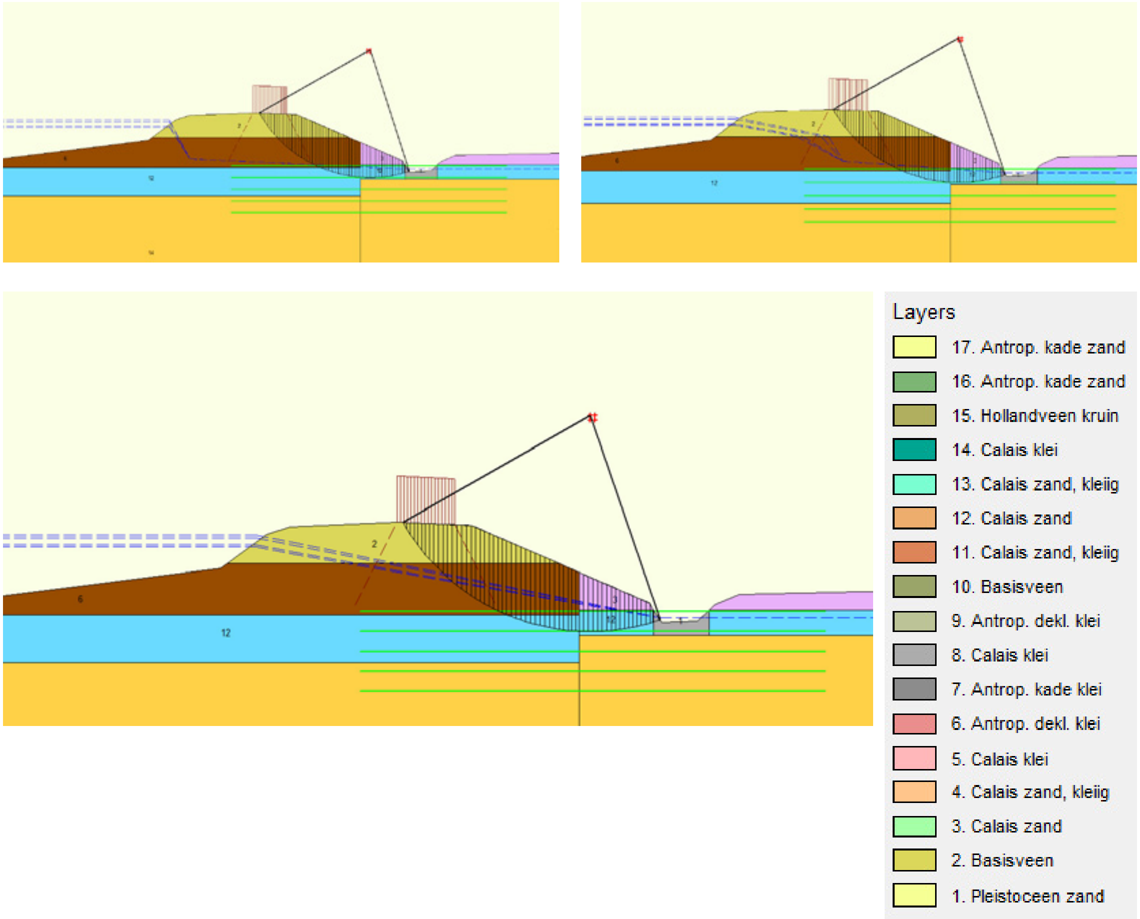
Variable	Standard deviation	Mean 4	Mean 9	Mean 11	Mean 12	Mean 13	Mean 17
H_i	CV = 0.1	-3.9	-2.6	-2.7	-2.85	-3.1	-2.9
H_{ir}	CV = 0.22	1.0	2.7	2.7	2.7	1.0	1.0
L	CV = 0.1	22	37.5	17.75	39	25	22
D_0	CV = 0.1	0.3	0.3	0.01	0.01	0.01	0.01
D_1	CV = 0.1	1	15	15	15	4	5
g	-	9.81	9.81	9.81	9.81	9.81	9.81
$\mathbf{m_0}$	CV = 0.1	1	1	1	1	1	1
$\mathbf{m_b}$	CV = 0.12	1	1	1	1	1	1
$\mathbf{y_s}$	-	26.5	26.5	26.5	26.5	26.5	26.5
γ_{nat}	CV = 0.05	17	17	17	17	17	17
γ_w	-	10	10	10	10	10	10
k	CV = 1	0.12 e-4	0.93 e-4	0.93 e-4	1.74 e-4	0.12 e-4	0.12 e-4
$\mathbf{d_{70}}$	CV = 0.15	110 e-6	250 e-6	262 e-6	250 e-6	100 e-6	100 e-6
$\mathbf{d_{70m}}$	-	2.08e-4	2.08e-4	2.08e-4	2.08e-4	2.08e-4	2.08e-4
θ_0	-	37	37	37	37	37	37
η	-	0.25	0.25	0.25	0.25	0.25	0.25
ν	-	$1.33*10^{-6}$	$1.33*10^{-6}$	$1.33*10^{-6}$	$1.33*10^{-6}$	$1.33*10^{-6}$	$1.33*10^{-6}$

APPENDIX F

INSTABILITY FILES

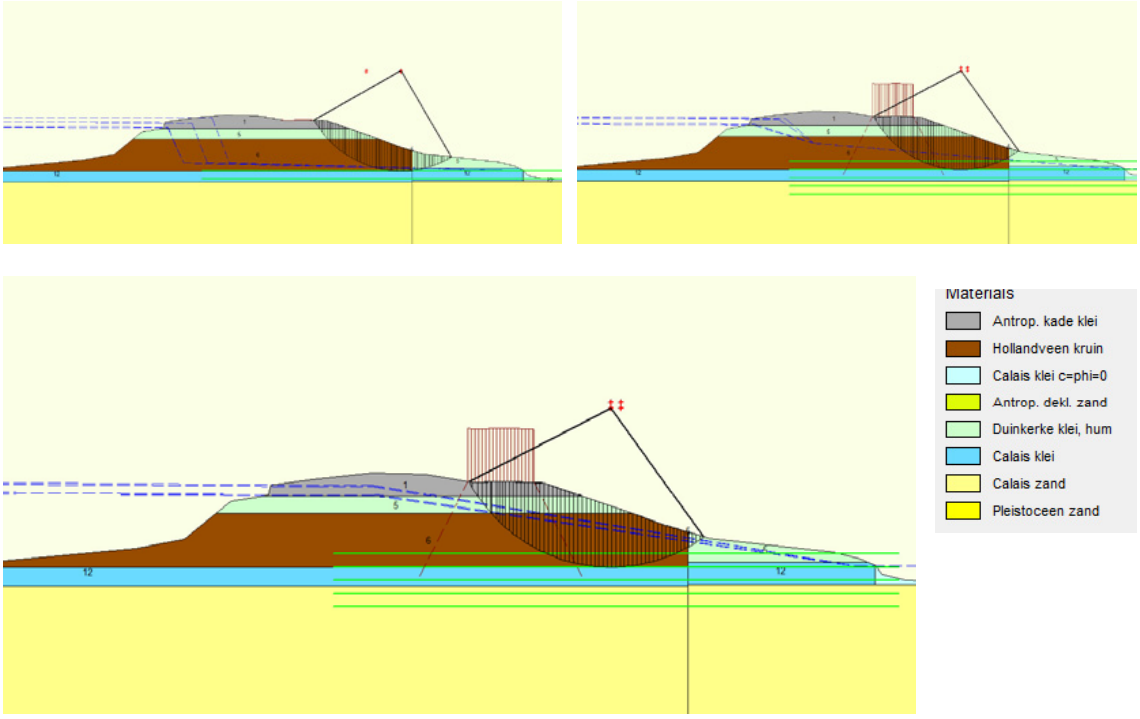
SECTION 4

FIGURE 68 **CROSS SECTION FOR STABILITY ASSESSMENTS FOR LOW (TOP LEFT), AVERAGE (TOP RIGHT) AND HIGH PHREATIC LINES (BOTTOM)**



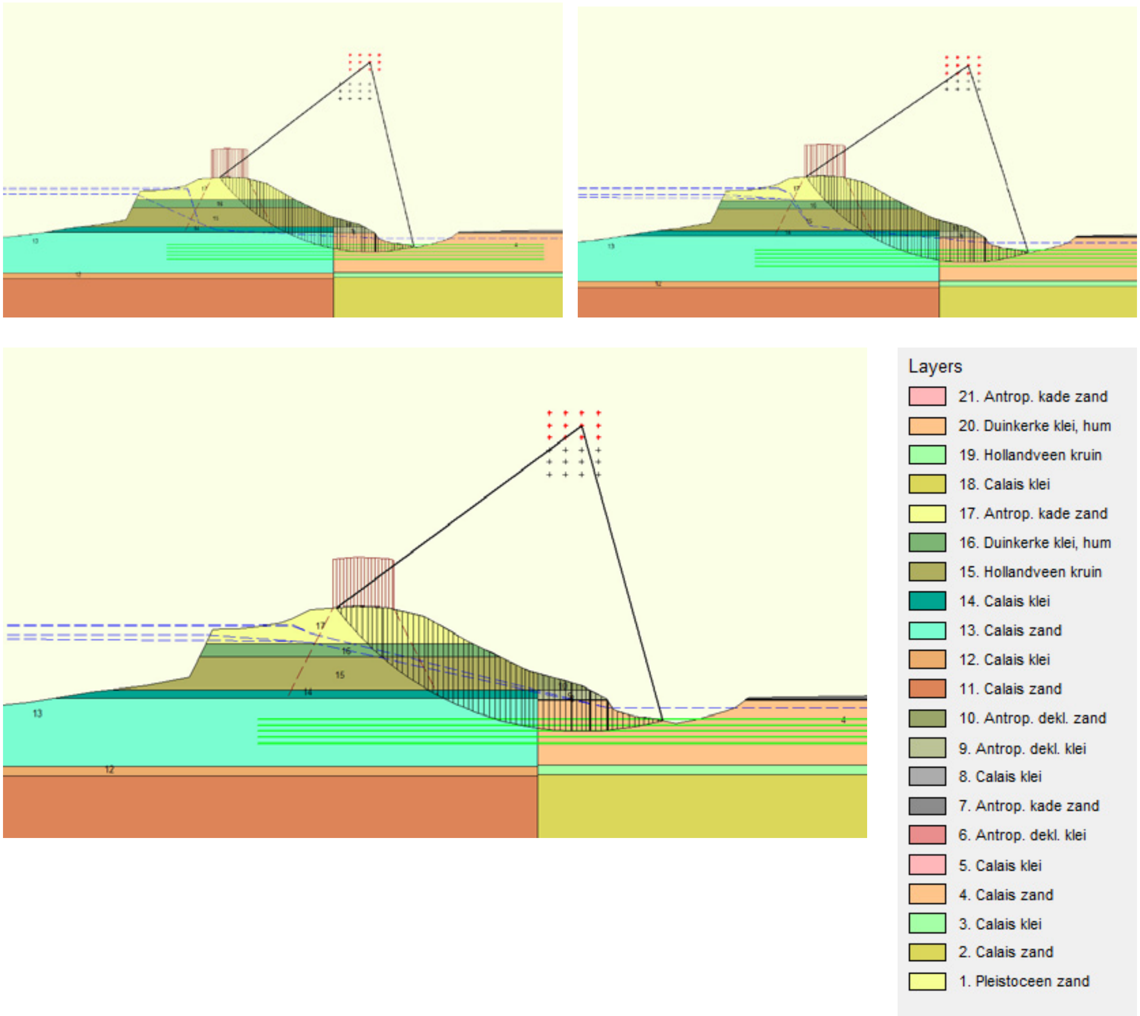
SECTION 9

FIGURE 69 CROSS SECTION FOR STABILITY ASSESSMENTS FOR LOW (TOP LEFT), AVERAGE (TOP RIGHT) AND HIGH PHREATIC LINES (BOTTOM)



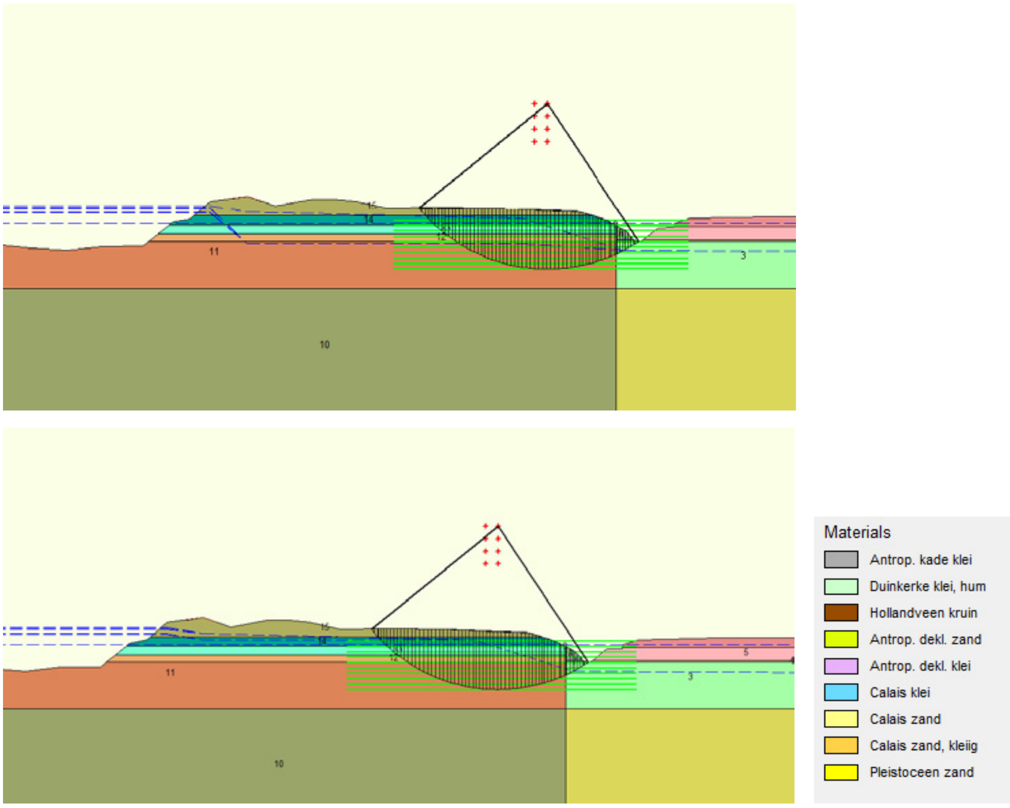
SECTION 11

FIGURE 70 **CROSS SECTION FOR STABILITY ASSESSMENTS FOR LOW (TOP LEFT), AVERAGE (TOP RIGHT) AND HIGH PHREATIC LINES (BOTTOM)**



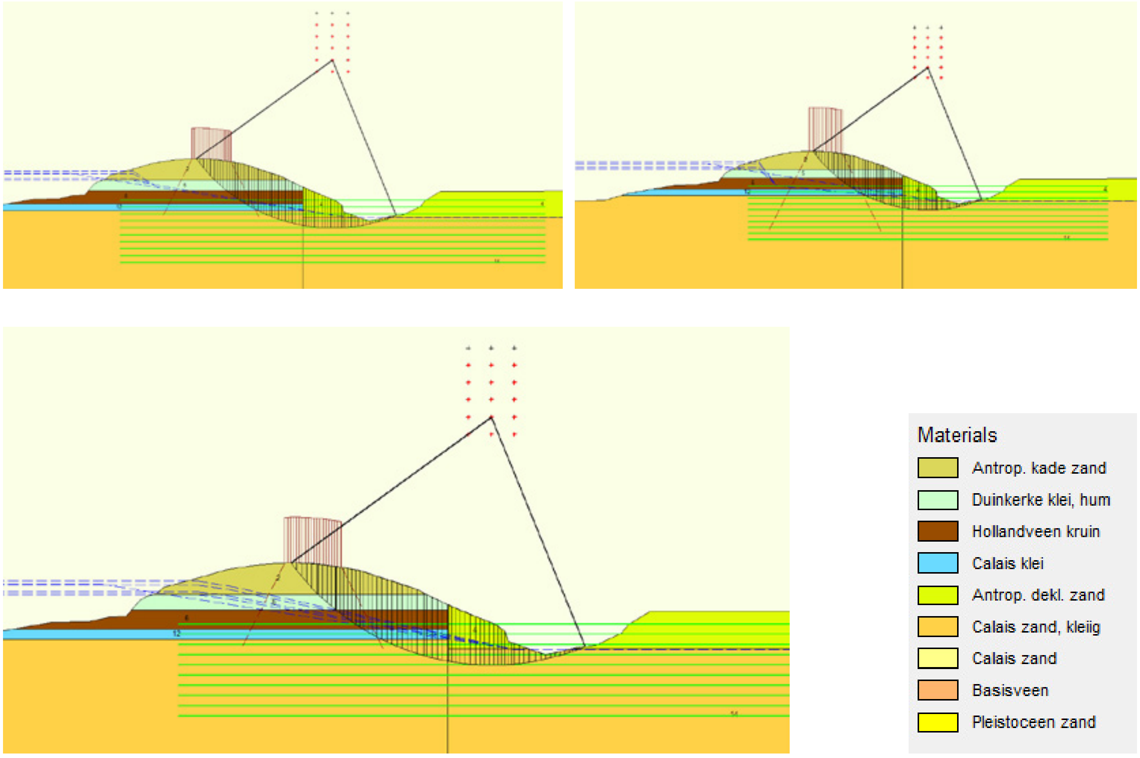
SECTION 12

FIGURE 71 CROSS SECTION FOR STABILITY ASSESSMENTS FOR LOW, AVERAGE (TOP) AND HIGH PHREATIC LINES (BOTTOM)



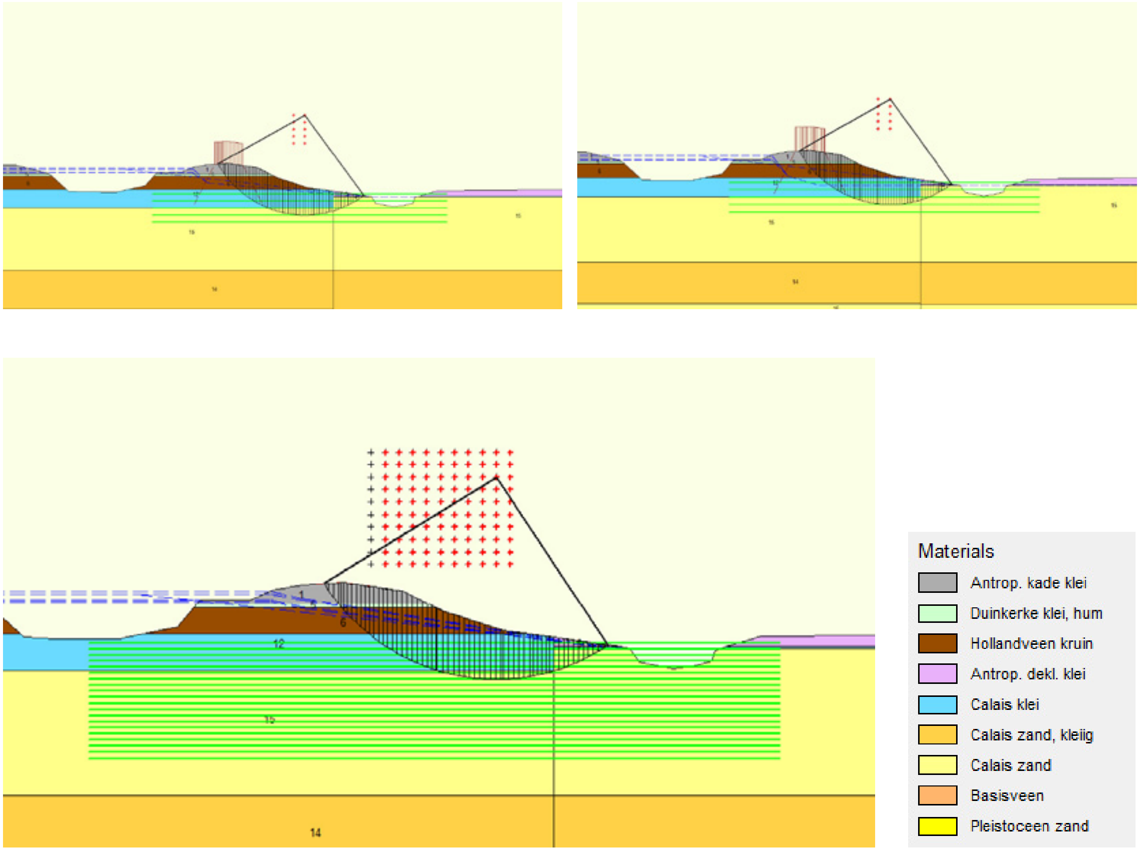
SECTION 13

FIGURE 72 CROSS SECTION FOR STABILITY ASSESSMENTS FOR LOW (TOP LEFT), AVERAGE (TOP RIGHT) AND HIGH PHREATIC LINES (BOTTOM)



SECTION 17

FIGURE 73 CROSS SECTION FOR STABILITY ASSESSMENTS FOR LOW (TOP LEFT), AVERAGE (TOP RIGHT) AND HIGH PHREATIC LINES (BOTTOM)



APPENDIX G

FRAGILITY CURVES FOR HIGH PHREATIC LINES

FIGURE 74 FRAGILITY CURVE FOR DIFFERENT TRAFFIC LOADS WITH HIGH PHREATIC LINE, SECTION 4

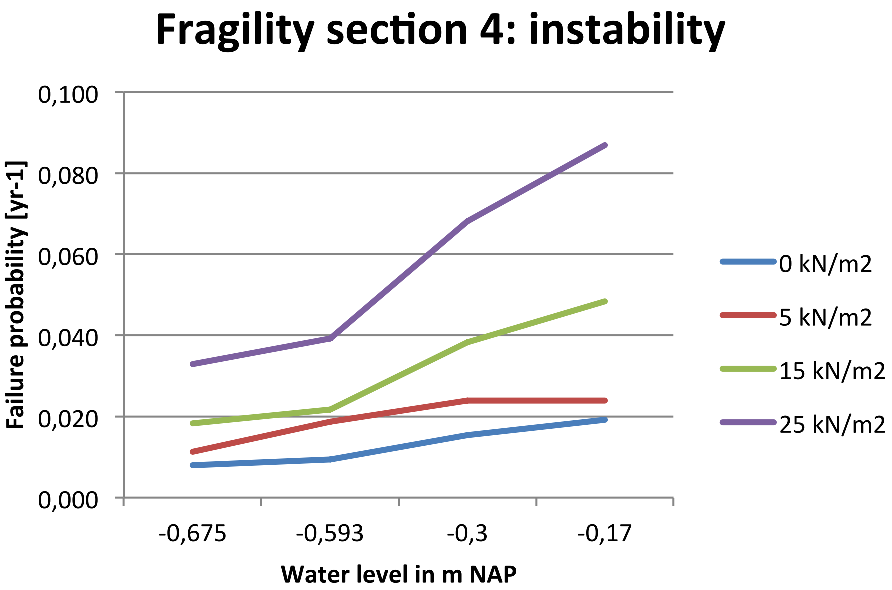


FIGURE 75 FRAGILITY CURVE FOR DIFFERENT TRAFFIC LOADS WITH HIGH PHREATIC LINE, SECTION 9

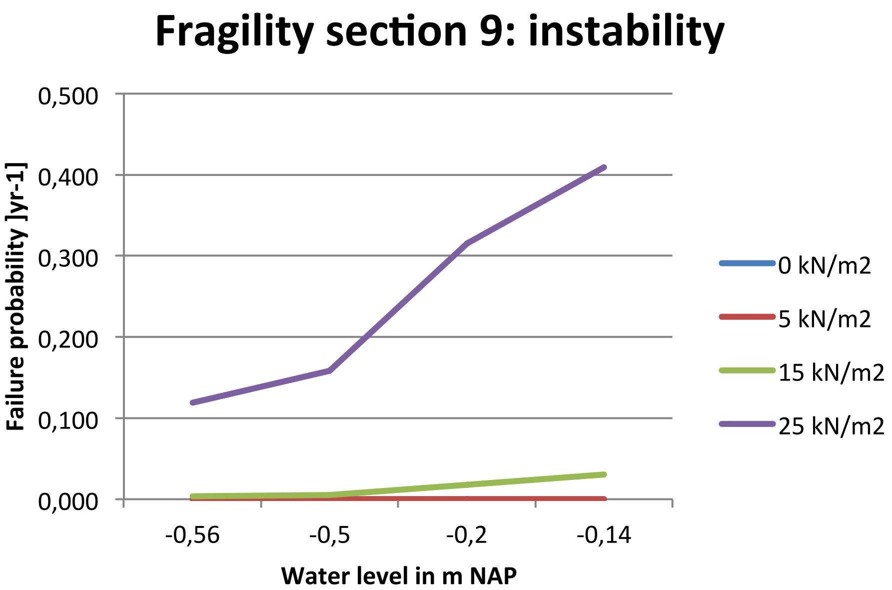


FIGURE 76 FRAGILITY CURVE FOR DIFFERENT TRAFFIC LOADS WITH HIGH PHREATIC LINE, SECTION 11

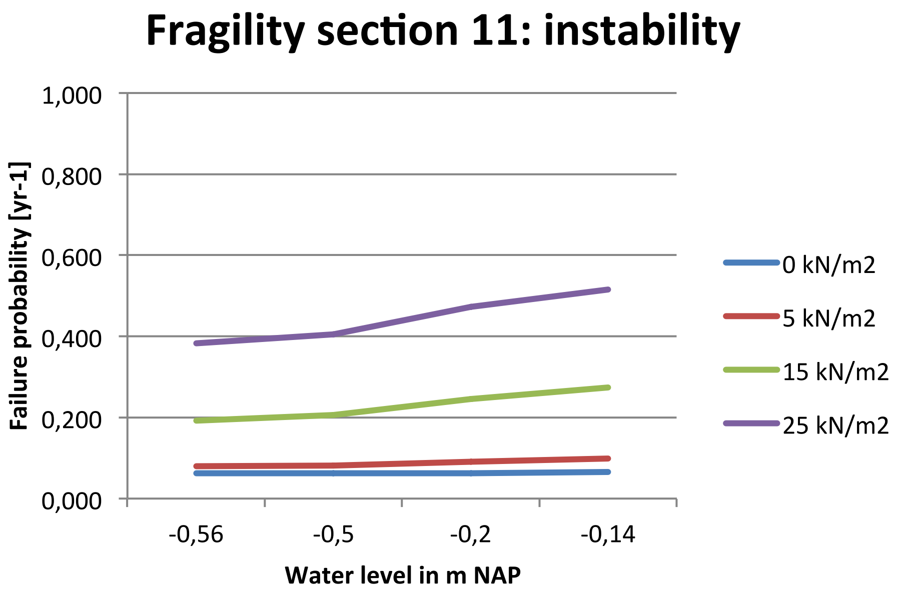


FIGURE 77 FRAGILITY CURVE FOR DIFFERENT TRAFFIC LOADS WITH HIGH PHREATIC LINE, SECTION 12

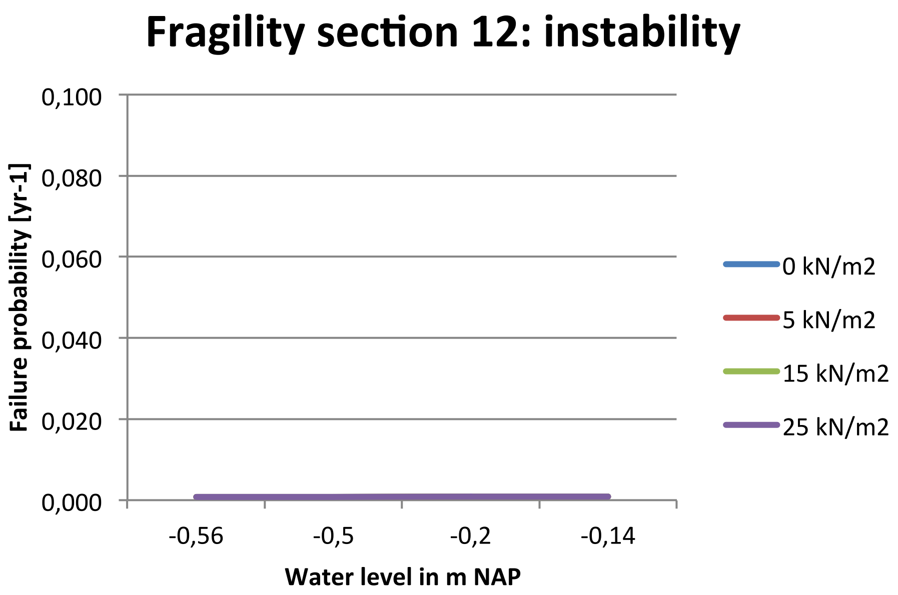


FIGURE 78 FRAGILITY CURVE FOR DIFFERENT TRAFFIC LOADS WITH HIGH PHREATIC LINE, SECTION 13

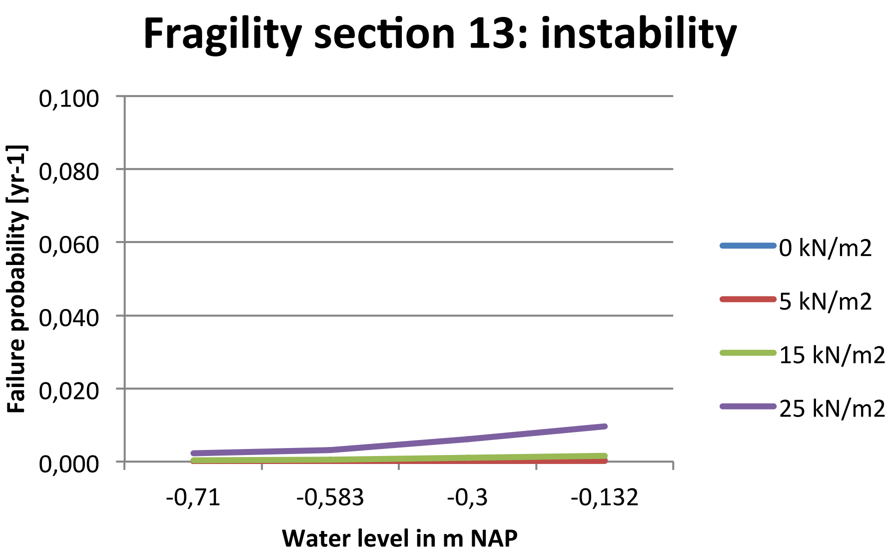
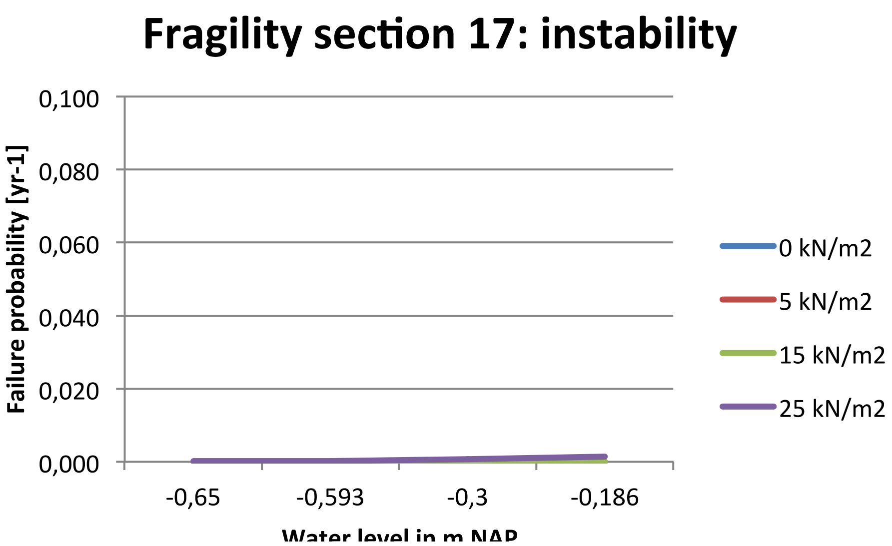


FIGURE 79 FRAGILITY CURVE FOR DIFFERENT TRAFFIC LOADS WITH HIGH PHREATIC LINE, SECTION 4



APPENDIX H

TOTAL COST OF ALL INTERVENTIONS FOR EACH SECTION

FIGURE 80 TOTAL COST OF INTERVENTIONS IN SECTION 4

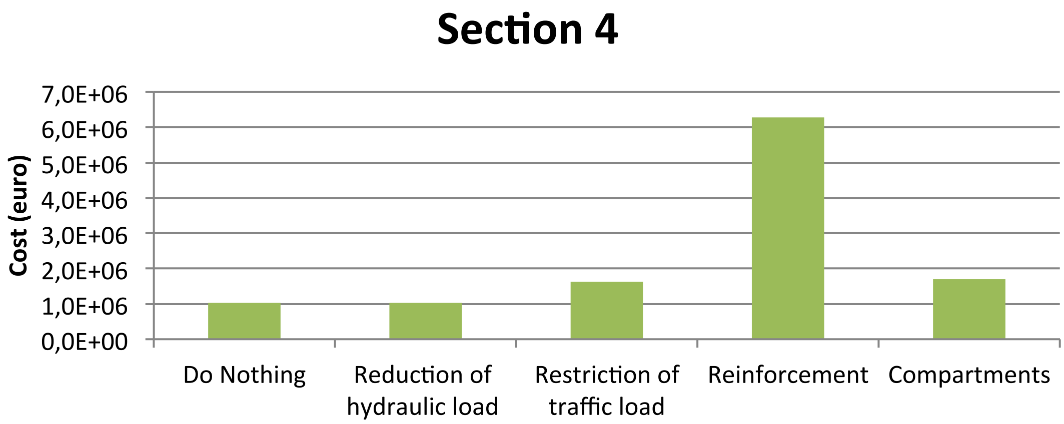


FIGURE 81 TOTAL COST OF INTERVENTIONS IN SECTION 9

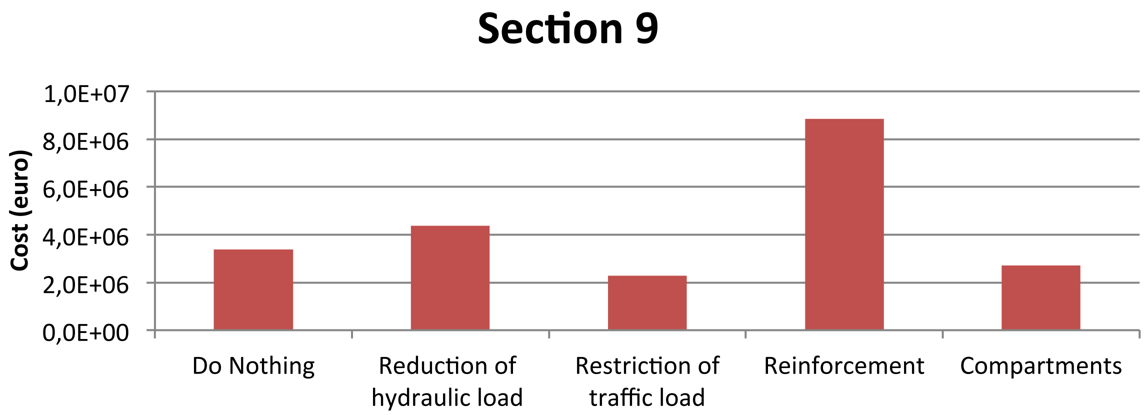


FIGURE 82 TOTAL COST OF INTERVENTIONS IN SECTION 11

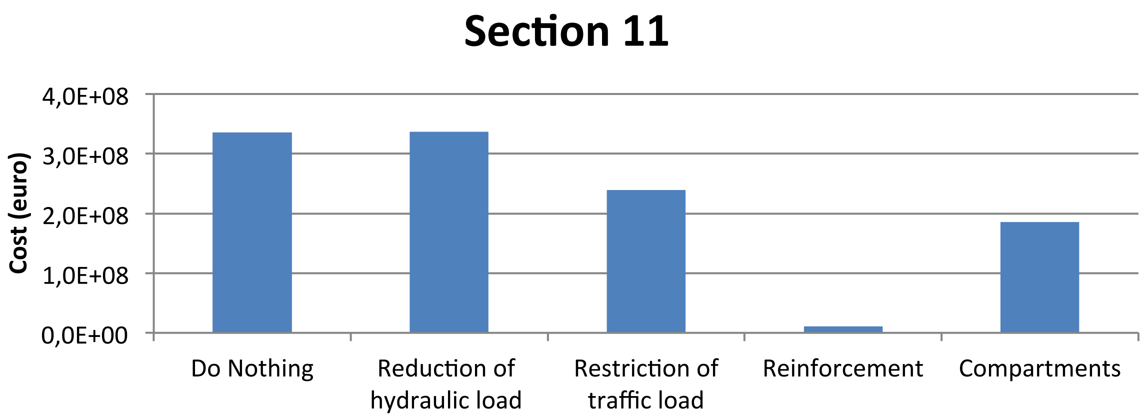


FIGURE 83 TOTAL COST OF INTERVENTIONS IN SECTION 12

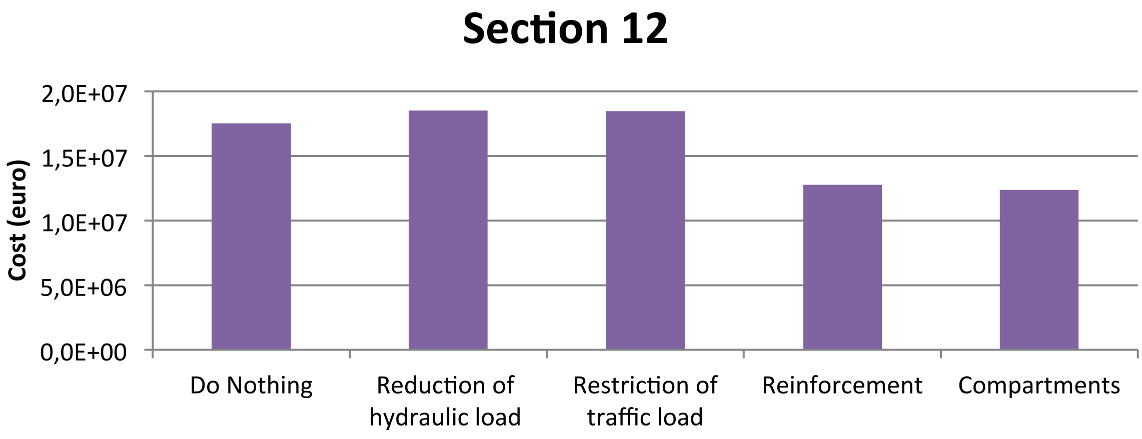


FIGURE 84 TOTAL COST OF INTERVENTIONS IN SECTION 13

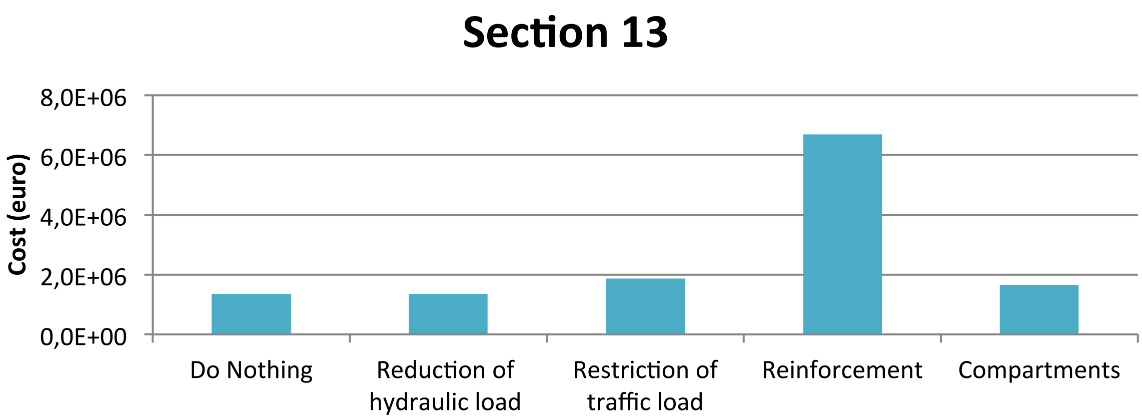


FIGURE 85 TOTAL COST OF INTERVENTIONS IN SECTION 17