N. Huijsman

Analysis of piping at hydraulic structures through an event tree approach

Cover photo: Picture of the Markkanaal chamber lock (Marksluis) at Oosterhout (Noord-Brabant), the Netherlands, own photograph.

Analysis of piping at hydraulic structures through an event tree approach

By

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Preface

This document is the thesis report with the name 'Analysis of piping at hydraulic structures through an event tree approach'. The report describes how the failure mechanism piping at hydraulic structures can be assessed by making use of an event tree. This thesis concludes my time as a student at the Delft University of Technology. With that I finish my master program Civil Engineering where I specialized myself into two fields: Hydraulic Engineering and Watermanagement.

During the research I found out how much knowledge there is to assess primary flood defense on their offered protection against a potential flood. Also, I found out how much is not known yet. It was difficult sometimes to deal with these unknowns. However, in the end these unknowns led to interesting conversations with experts in the field that helped me getting more insight in how to proceed in designing an assessment method for piping at hydraulic structures. The input that I obtained from these conversations is very valuable for this thesis. A special thanks to Rob Delhez. His view and knowledge on potential structural failure modes as consequence of piping led to valuable findings that are presented in this report. I would also like to thank Jorg Willems for sharing his view on the current assessment method of hydraulic structures.

Also, the feedback that I obtained from my committee members helped me a lot during my research. Without their guidance and input this thesis report would not have reached the level that is has today. Therefore, I would like to thank Matthijs Kok for being the chairman of the committee. Matthijs helped a lot in finding the scope of the interesting topic of this master thesis. Also, his feedback during the research period helped in keeping on track. I would like to say thank you to Martine Rutten, Robert Lanzafame and Arno Willems as well. Their valuable feedback during several meetings made sure to stay focused at every detail while still zooming out to the total report. Thank you Martine for your dedicated support each week, it helped me to stay focused.

This thesis has been carried out in association with Iv-Infra. I would like to thank Iv-Infra for providing the facilities that supported me during my research. The expertise of Iv-Infra helped me getting insight in several important details that were necessary to conduct this research. I would also like to thank my colleagues at Iv-Infra for their support and enthusiasm.

Finally, I want to thank my family and friends for their support. Bram, especially without your help and support I would not have come this far.

I hope you enjoy reading the report that lies before you.

N. Huijsman
Vlaardingen, October 2020

Summary

The Netherlands is a low-lying country that has been threatened by water for a long time. To protect areas that are susceptible to potential flooding from seas, rivers and lakes, flood defenses have been constructed throughout history. To guarantee that the offered protection by these dikes, dunes and hydraulic structures keeps meeting the requirements, systematic assessments of primary flood defenses take place regarding a set of failure mechanisms. These failure modes include macro-stability, micro-stability, revetment failure, piping, et cetera. The different failure modes can be assessed with different methods. The available methods from most accurate to least accurate are:

- Level III: full numerical probabilistic assessment;
- Level II: probabilistic assessment with approximations;
- Level I: semi-probabilistic assessment with characteristic values for load and strength.

Since 2017, these assessments have been performed according to the 'Wettelijke Beoordelingsinstrumentarium', or in short WBI 2017 (in English: 'Statutory Assessment tool'). The assessment result is a safety judgment category. In total there are 7 possible categories, I_V to VII_V (not to be confused with the levels of probabilistic methods), where I_V is the upper safety category and VII_V is the lower category. What category is obtained depends on the difference between the requirement of the failure probability by law, also known as the norm, and the actual failure probability of the assessed dike trajectory. To determine this failure probability each applicable failure mechanism needs to be assessed after which the assessment results of the different failure modes are assembled. However, the exact probability of failure is not always calculated. This could lead to conservative assessment results. One of the failure mechanisms that is not assessed in a probabilistic manner is piping at hydraulic structures. The objective of this thesis is to find a method that can be used to obtain a failure probability for the failure mode piping at hydraulic structures.

The current assessment strategy for piping at hydraulic structures is based on a comparison between the critical difference in hydraulic head and the occurring difference in hydraulic head at both sides of the structure. The conclusion of this assessment is whether the critical hydraulic head difference is exceeded or not. This leads to two possible assessment outcomes, II_V (satisfies the norm) and V_V (does not satisfy the norm). The outcome of the current assessment method is therefore binary without a probabilistic approach.

To develop a probabilistic approach, first applicable models to describe piping were analyzed by means of a literature study. When the seepage path only contains horizontal elements, the models Bligh and Lane are suitable. Only when a horizontal seepage path does not change direction in the horizontal plane, the Sellmeijer model can solely be applied in addition. For hydraulic structures this is almost never the case due to the presence of horizontal seepage screens at the connection between the structure and its surroundings. If also significant vertical elements are involved in the seepage path, the Lane and Heave model can be applied.

Currently, flood defense assessment methods are mainly based on the application of fault trees. This is one method that can be used for a system analysis. Another possible method is the use of event trees. The purpose of both methods is to clarify and relate all needed sub-events before failure due to piping happens. This clarification can have a qualitative as well as a quantitative purpose. To model the piping failure mode the event tree method has been chosen. This has two main reasons: a single event tree can describe multiple consequences and an event tree is not limited to Boolean operators. For example, for piping this means that one event tree can be used to model all potential types of structural failure as a consequence of piping.

In total three event tree variants were constructed that describe the piping failure mechanism at hydraulic structures. Variant 1 contains nine sub-events and a precondition. These nine sub-events are sorted into a precondition and three main phases: A) the geotechnical part, B) detection and intervention and C) remaining strength of the structure. The precondition marks the initiating event, which is the exceedance of the critical head difference. Because of the level of detail of the first variant and hence data requirement, it solely serves qualitative and communicative purposes. The other two variants have quantification as goal.

The difference between the latter two variants is the schematization of the geotechnical part of the failure mode. In variant 2 this phase is divided in the sub-events heave and backward erosion while in variant 3 a distinction between these two sub-events is not made. It is recommended to start an assessment with event tree variant 3. When the assessment result is category I_V, the assessment procedure is stopped. If this is not the case the assessment is continued with event tree variant 2.

To test the event tree method including a probabilistic approach, a case study was conducted. The case study involves the Marksluis chamber lock located in dike trajectory 34-1 at Oosterhout in Noord-Brabant, the Netherlands.

The geotechnical part of the failure mode can be quantified with existing models. Probabilistic calculations with both level III (in this thesis Monte Carlo analysis) and level II (in this thesis FORM analysis) analyses are possible to obtain a probability of occurrence for these sub-events. However, not all sub-events displayed in the event trees can be quantified with a model. Phase C) the remaining strength of the structure, consisting of sub-events structural failure and breach formation, is quantified by using expert knowledge. During an expert interview, the characteristics (how it is constructed, what materials it consists of, the presence of seepage screens) of the Marksluis were discussed. Given this information, the order of magnitude of the upper limit of the conditional probabilities of the occurrence of structural failure and breach formation have been determined. From the expert knowledge session it can be concluded that the upper limit of these values is 0.1.

For the Marksluis the outcome of the assessment procedure can be concluded after application of event tree variant 3. The difference between the requirement derived from the norm and the actual probability of failure due to piping differ more than a factor 30. This leads to assessment category I_V, which is the highest obtainable safety category.

Whether the assessment procedure needs to be continued depends amongst others on the resistance against piping. A characteristic for the Marklsuis is that the length of the structure compared to the width is long. The presence of vertical seepage screens makes the resistance even larger. In contrast, structures with a relatively small length to width ratio (e.g. coupures) are more susceptible to piping.

To know if continuation of the assessment procedure is needed it is important to be able to quantify the probability of piping in an accurate manner. To improve the accuracy of the quantification of the probability of piping, monitoring wells and hydrostatic pressure sensors can be installed at hydraulic structures. Hence, more data on groundwater flow near these structures can be obtained.

Besides applying an event tree approach for quantification during an assessment it is also possible to implement it in the design phase. Application of an event tree approach during the design phase urges the engineers to think systematically about what potential pathways of a flood defense could be. This makes it possible to detect the weakest links in the design in a stage before the actual structure has been built, preventing unnecessary changes to be made after construction.

By using an event tree approach the probability of failure of different types of hydraulic structures due to piping can be quantified. This quantification is based on both models and expert knowledge. Currently, only two assessment categories can be assigned (II_V and V_V). When adjusting the current assessment procedure for this failure mechanism in the WBI 2017 to an event tree based approach the full range of possible assessment outcomes can be assigned. This adjustment is useful for failure modes that cannot be assessed in a probabilistic manner yet. By implementing an event tree approach it becomes easier to think of what sub-events must be assessed and how this can be done in a probabilistic manner. Therefore, using event tree approaches in the WBI 2017 can be a useful addition to the already used fault tree approach for other failure modes of dikes, dunes and hydraulic structures.

Contents

List of symbols

Symbol Latin	Unit	Description
C_B	$[-]$	Creep factor in Bligh's model
C_{LANE}	$[-]$	Creep factor in Lane's model
D	$[m]$	Thickness of aquifer layer
d	$[m]$	Thickness of the blanket layer
d_{70}	$[m]$	70%-fractile of the grain size distribution
$d_{70.m}$	$[m]$	Reference value for the 70%-fractile
$F_{exc,H}(h)$	$[-]$	Survival (or reliability) function of the water level h
$F_{geometry}$	$[-]$	Geometry factor
$F_H(h)$	$[-]$	Cumulative distribution function of water level h
$F_{resistance}$	$[-]$	Resistance factor
F_{scale}	$[-]$	Scale factor
$f_S(x)$	$[-]$	Sampling distribution
$f_X(x)$	$[-]$	Probability density function of variable X
g	$[m/s^2]$	Gravitational constant
H	$[m]$	Water level difference over the hydraulic structure
H_c	$[m]$	Critical water level difference over the hydraulic structure
h	$[m\ + NAP]$	Upstream water level
h_{bi}	$[m\ + NAP]$	Downstream water level
h_{dec}	$[m]$	Decimate height
h_{exit}	$[m]$	Phreatic level at exit point
i	$[m]$	The occurring gradient at the exit point
$i_{c,h}$	$[-]$	The critical heave gradient
k	$[m/s]$	Specific conductivity
L	$[m]$	Present seepage length
L_B	$[m]$	The present seepage length according to Bligh
$L_{C,B}$	$[m]$	The critical seepage length according to Bligh
$L_{C,L}$	$[m]$	The critical seepage length according to Lane
L_h	$[m]$	Horizontal seepage path
$L_{h,B}$	$[m]$	Horizontal seepage path to Bligh
L_L	$[m]$	The present seepage length according to Lane
L_{screen}	$[m]$	Length of seepage path, downstream of seepage screen
L_v	$[m]$	Vertical seepage path
m_c	$[-]$	Model parameter for Bligh and Lane
m_L	$[-]$	Model parameter of the present seepage length
m_p	$[-]$	Model parameter for the piping model of Sellmeijer
N	$[-]$	Number of simulations
$N(\mu;\sigma)$	$[-]$	Normal distribution function with mean μ and standard deviation σ
N_f	$[-]$	Number of simulations that result in failure
N_{PKW}	$[-]$	Length-effect factor for piping at hydraulic structures
n_{kw}	$[-]$	The number of hydraulic structures for which piping is a relevant failure mechanism in the applicable dike trajectory
$P(X)$	$[-]$	Probability of occurrence of event X
$P_{breach\|SF\|piping}$	$[-]$	Probability of occurrence of breach formation given structural failure and piping have occurred

$\widehat{P_f}$	$[-]$	Estimator of the failure probability
$P_{f,kw\|piping}$	$[-]$	Conditional failure probability given piping has occurred
$P_{f,PKW}$	$[-]$	Probability of failure due to the failure mode piping at hydraulic structures
$P_{f,reach}$	$[1/year]$	Failure probability per dike reach (or hydraulic structure)
$P_{f,req,PKW}$	$[1/year]$	The requirement for the annual failure probability of the mechanism piping at a hydraulic structure for a dike reach
$P_{f,trajectory}$	$[1/year]$	The requirement for the annual failure probability for the entire dike trajectory
P_{piping}	$[-]$	Probability of occurrence of piping
$P_{SF\|piping}$	$[-]$	Probability of occurrence of structural failure given piping has occurred
$P_{req,llv}$	$[1/year]$	Lower limit value of the dike trajectory
$P_{req,llv,reach}$	$[1/year]$	Failure probability requirement per dike reach following from the lower limit value
$P_{req,wlv}$	$[1/year]$	Warning limit value of the dike trajectory
$P_{req,wlv,reach}$	$[1/year]$	Failure probability requirement per dike reach following from the warning limit value
R	$[applicable\ unit]$	(Combined) value of the resistance variable(s)
R_d	$[applicable\ unit]$	Design value of the resistance
r_c	$[-]$	Reduction factor
S	$[applicable\ unit]$	(Combined) value of the load variable(s)
S_d	$[applicable\ unit]$	Design value of the load
u	$[m]$	Location parameter of the Gumbel distribution
u	$[-]$	Standard normal distributed variable
Z	$[applicable\ unit]$	Value describing whether failure occurs or not

Symbol Greek	**Unit**	**Description**
α	$[-]$	Scale parameter of the Gumbel distribution
α_i	$[-]$	Importance factor of variable i
$\beta_{kw\|piping}$	$[-]$	Reliability index according to the conditional probability of failure of the hydraulic structure given piping has occurred
$\gamma_{sub.particles}$	$[kN/m^3]$	Volumetric weight of submerged sand particles
γ_{water}	$[kN/m^3]$	Volumetric weight of water
η	$[-]$	Coefficient of White
θ	$[°]$	Bedding angle
κ	$[m^2]$	Intrinsic permeability
μ	$[applicable\ unit]$	Mean of the normal distribution
ν_{water}	$[m^2/s]$	Kinematic viscosity of water
σ	$[applicable\ unit]$	Standard deviation of the normal distribution
φ_{screen}	$[m\ +NAP]$	Hydraulic head at the bottom side of the seepage screen
ω_{pkw}	$[-]$	Factor of maximum contribution to the annual failure probability for the entire dike trajectory by the mechanism piping at a hydraulic structure

1. Introduction

This chapter introduces the research that is described in this thesis. First an overview is given on what flood defense approaches have been used throughout history to assess the level of safety of primary flood defenses. Secondly, the approach that is used nowadays is described. This leads to the problem statement that will be dealt with in this thesis. Thirdly, the importance of the problem is highlighted in this chapter. Then, the research goal and the method on how to meet the goal are described. The chapter ends with an elaboration on the structure of the report.

1.1 Flood defense approaches through history

Water: either too much at some instances or too little at other times. For a low-lying country as the Netherlands it is of utmost importance to manage the water well to prevent flooding. In history it has proven to be an ongoing battle between water and mankind. For mankind to be on the winning hand flood defenses must be up to standard and improvements must be made continuously so the battle is no lost game.

Monitoring the primary flood defenses is key in ensuring the safety level as it is expected. Primary flood defenses are structures that protect the land against a potential flood due to water coming from the seas (North Sea and Wadden Sea), the rivers (Rhine, Muese and Hollandsche IJssel), tidal inlets (Eastern Scheldt and Western Scheldt) and lakes (IJsselmeer, Volkerak-Zoommeer, Grevelingenmeer and the Veluwerandmeren) (Atlas Leefomgeving, 2017; Helpdesk Water, n.d.-c). Throughout history multiple approaches have been used when it comes to assessing the safety of the dikes, dams, sluices, storm surge barriers and other structures that are part of the primary flood defense system of the Netherlands. According to (Voorendt & Molenaar, 2019) the following approaches can be distinguished:

- Deterministic approach;
- Exceedance probability approach;
- Failure probability approach.

Deterministic approach
In the deterministic approach no statistics were involved in the design of flood defenses. The retainable height was determined by the water levels that occurred during the last flood event. To account for uncertainties in this water level, a freeboard (usually an extra height between 0.5 to 1.0 meters) was added to the previous flood event water levels (Voorendt & Molenaar, 2019). The assessment strategy that is fully based on a deterministic approach was used until the year 1960.

Exceedance probability approach
From 1960 a change in the design approach of flood defenses took place. Instead of reacting to the last occurring flood an approach based on statistics was used. To know what height a flood defense should have it is important to know the level of safety that must be guaranteed. This was determined by the allowable risk. As shown in formula [1] risk can be defined as the probability of occurrence of a certain event (in this case a flood) multiplied by the consequences of this event (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017).

$$Risk = Probability * Consequences \qquad [1]$$

To find the allowable risk three different types of risk were used. According to (Vrijling, Van Hengel, & Houben, 1995; Voorendt & Molenaar, 2019) the different types of risk are:

- Individual risk: a citizen living in a flood prone area that would not survive a flood event;
- Societal risk: a group of citizens living in a flood prone area that would not survive a flood event;
- Economical risk: a large difference in investment costs for flood defenses and the obtained reduction in risk.

The three types of risk were compared in monetary terms. The largest risk obtained did result in the safety level. Different areas in the Netherlands have different risks. Therefore, the country was divided in so-called dike ring areas. For each dike ring area, the applicable risk was determined. Through an estimate of the consequences of a potential flood, the probability of this event could be traced back by rewriting formula [1] $\left(Probability = \frac{Risk}{Consequences}\right)$. The obtained annual exceedance probability can be written in terms of a return period by using formula [2].

$$Return\ period\ = \frac{1}{Annual\ exceedance\ probability} \qquad [2]$$

This return period can be coupled to the occurring water levels. To find these water levels one has to extrapolate water level measurements by means of a regression analysis. For example, for dike ring areas 13, 14 and 19 the water levels corresponding to an event with a return period of 10,000 years were needed. Existing water records are not long enough to contain values that belong to such an event. Therefore, the regression analysis is needed. An overview of the annual exceedance probabilities for the different dike ring areas is shown in Figure 1.

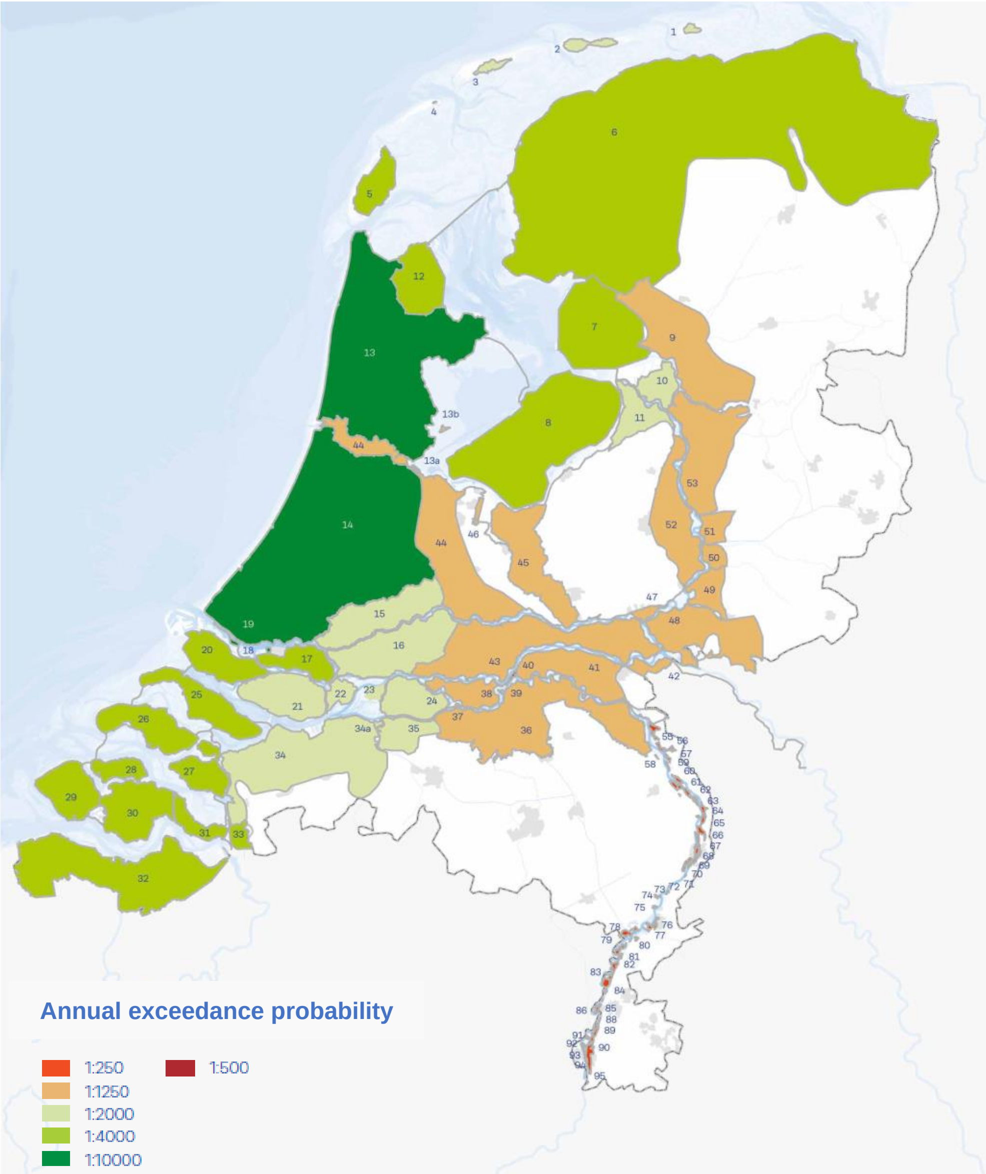

Figure 1 Annual exceedance probabilities for different dike ring areas. Note that these dike ring areas are not used anymore since 2017. Now, dike trajectories are used as shown in Figure 3 (source: (Kok, Jongejan, Nieuwjaar, & Tánczos, 2017))

In the exceedance probability approach the main design variable is the water level. Therefore, failure mechanisms of flood defenses directly related to the exceedance of an occurring water level, like overflow and wave overtopping, are taken into account by this approach (Kenniscentrum InfoMil, n.d.). However, failure of flood defenses can also have a geotechnical nature. Examples of geotechnical failure modes are macro-instability and piping.

During the determination of the probability of failure it is important to understand that failure of flood defenses is not only dependent on the occurrence of a certain hydraulic load in the area, but also on the strength of the flood defense itself. Especially for dikes, the resistance against geotechnical failure modes is important to include in the risk analysis (Vergouwe, 2014). To incorporate the strength of the flood defenses a new approach is used in the Netherlands. This approach is called the failure probability approach and is used since 2017.

Failure probability approach
A flood defense system can fail through various failure mechanisms. Whether failure occurs does depend on the occurring load and the resistance of the flood defense against this load. In Figure 2 an example is given of a fault tree for a dike reach. It shows that not only overtopping is a relevant failure mechanism, but also macro-stability, piping, erosion of the outer slope and other failure mechanism have a contribution to the probability of failure for that certain dike reach. The applicable failure mechanisms depend on the characteristics of the dike reach. By using a fault tree the relevant failure mechanisms are considered in a systematic manner.

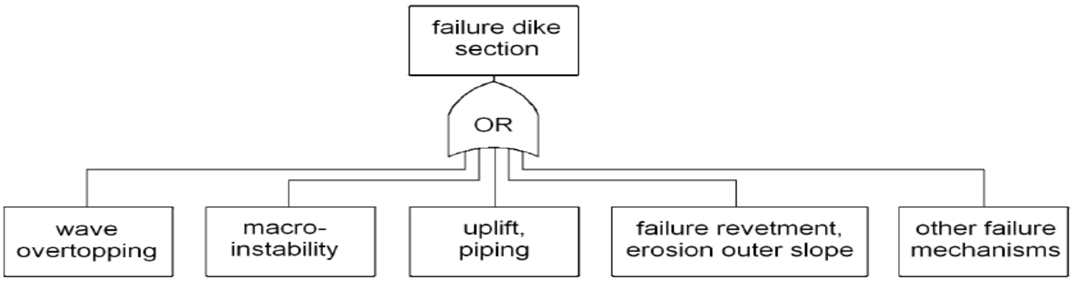

Figure 2 Fault tree for a dike section including the relevant failure mechanisms (source: (Voorendt & Molenaar, 2019))

Since 2017, the change to a probability of failure instead of an exceedance probability was not the only change. As of 2017, the dike ring areas are divided into dike trajectories. Due to the relative large size of some dike ring areas the consequences of a potential flood may be very different depending on where a dike ring could breach (Rijksoverheid, n.d.). By dividing the dike ring areas in dike trajectories each dike trajectory is characterized by its own threats and consequences of a potential breach (Kok, Jongejan, Nieuwjaar, & Tánczos, 2017). As a result, it becomes more clear what parts of the previous dike ring area need a higher protection level. The dike trajectories that have been defined and used since 2017 can be found in Figure 3.

Figure 3 Dike trajectories in the Netherlands. Each colored line represents a different trajectory (source: (Kok, et al., 2017))

1.2 Comparison exceedance probability and failure probability approaches

Since 2017, each of the defined dike trajectories as shown in Figure 3 needs to be assessed according to the failure probability approach. The largest difference compared to the previous exceedance probability approach is the degree of probabilistic assessment.

In probabilistic calculations three different levels can be distinguished. These levels are called reliability methods. According to (Jonkman, et al., 2017) the following reliability methods exist:

- Level III methods: with these methods a full probabilistic calculation is carried out. This is done by models considering the joint distribution functions of the variables of interest. By using this method, the failure probability can be calculated in an exact way;
- Level II methods: level II is also a full probabilistic calculation method. It differs from level III in the sense that the distribution types for the variables of interest are resembled by normal distributions. This is done by transforming the original distribution types to normal distributions in the design point values of the variables of interest. The design point is characterized by the largest probability density of the intersection of the joint probability density distribution and the limit state function (Jonkman, et al., 2017);
- Level I methods: these methods are also called semi-probabilistic methods. Not the full distribution of the involved variables is considered anymore. However, in a semi-probabilistic calculation one compares characteristic values for the load and the resistance. This is often the 95th percentile value for the load variables and 5th percentile value for the resistance variables.

The previous exceedance probability approach can be characterized as a semi-probabilistic calculation method and hence is a level I method. It can be summarized as comparing the characteristic value for the load to the characteristic value for the resistance (VNK2 Projectbureau, 2012). The probability of failure approach is about finding the actual probability that the load is larger than the resistance (VNK2 Projectbureau, 2012). This can be done through a level II or level III analysis. The difference between the two methods is shown in Figure 4. For the level I analysis only the design value for the load (S_d) and the design value for the resistance (R_d) are compared. For a level II or level III analysis the overlapping area of the two probability density curves should be calculated. This area equals the probability of failure. Since for a full probabilistic assessment a failure probability is calculated the distance to the requirement can be determined in an exact manner. This is not the case for a semi-probabilistic assessment where only a comparison between design load and resistance can be made. The advantage of the full probabilistic approach is that it is more cost efficient than using a semi-probabilistic approach (Slomp, Knoeff, Bizzarri, Bottema, & De Vries, 2016; VNK2 Projectbureau, 2012). This is because the requirement of flood protection can be reached in a more precise manner since the exact distance to that requirement is known.

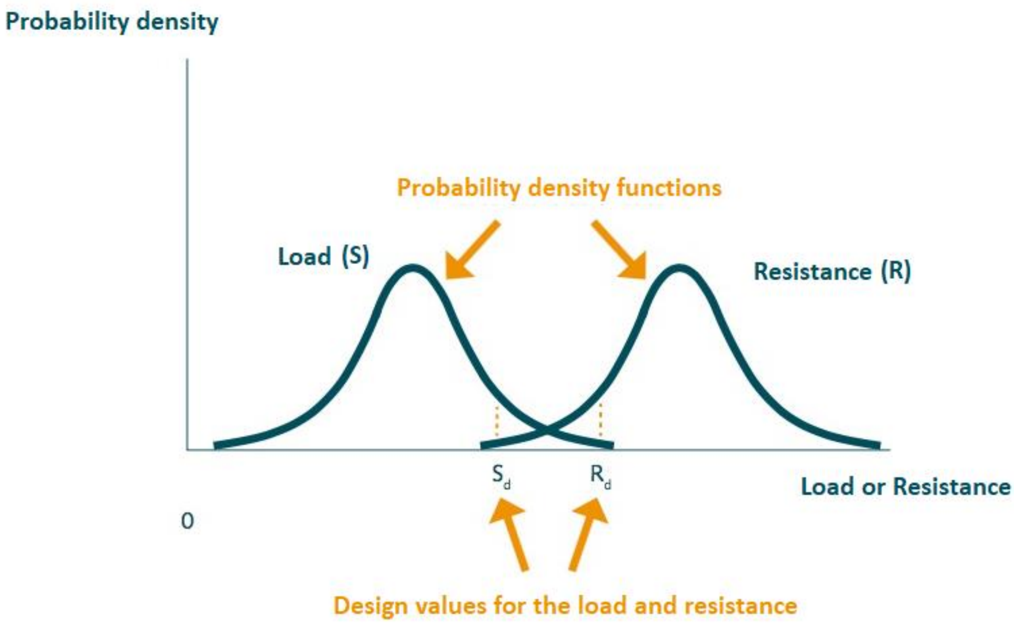

Figure 4 Probability density values for load and resistance variables and corresponding design values (based on: (VNK2 Projectbureau, 2012))

1.3 Problem statement

The goal of the assessment is to compare the probability of flooding of a dike trajectory to the applicable norm. Both values are expressed as a probability of flooding per year (De Waal, 2016). To find the actual failure probability the trajectory can be divided into several dike reaches. Each dike reach has its own characteristics and therefore its own applicable failure mechanisms. This is shown in Figure 5. A schematization of the division of a dike ring into dike trajectories and dike reaches is shown in Figure 6.

How this assessment should be performed is laid out in the 'Wettelijke BeoordelingsInstrumentarium' (WBI) (in English[1]: 'Statutory Assessment tool'). In this tool it is written which assessment methods are available for the different layers (dike trajectories and dike reaches) and the different failure mechanisms. In literature it is stated that especially hydraulic structures are assessed in a conservative way nowadays (Van der Vlist, Barneveld, Breedeveld, Van Doorn, & Luyten, 2019). This is supported by an interview held with Jorg Willems, Matthijs Kok and Arno Willems (Willems, Kok, & Willems, 2020). From this interview it was concluded that not all failure modes are assessed in a probabilistic manner. This often yields in a conservative outcome as assessment result.

[1] This thesis contains references to literature that is available in Dutch only. Therefore, a glossary of terms is included at the tail of the report in Appendix F. This glossary contains an overview of the translations of key terms that are of importance for this thesis.

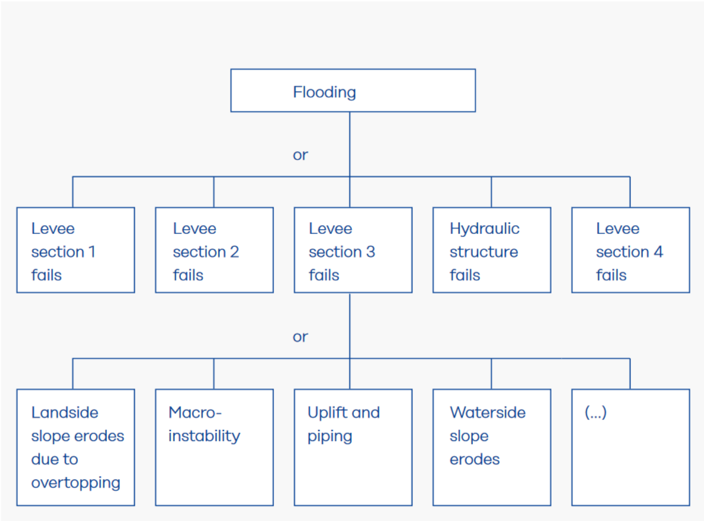

Figure 5 The probability of flooding of a dike trajectory is determined by finding the failure probabilities of the different dike reaches. Each dike reach has its own failure probability that is determined by the applicable failure mechanisms and their corresponding failure probabilities (source: (Kok, et al., 2017))

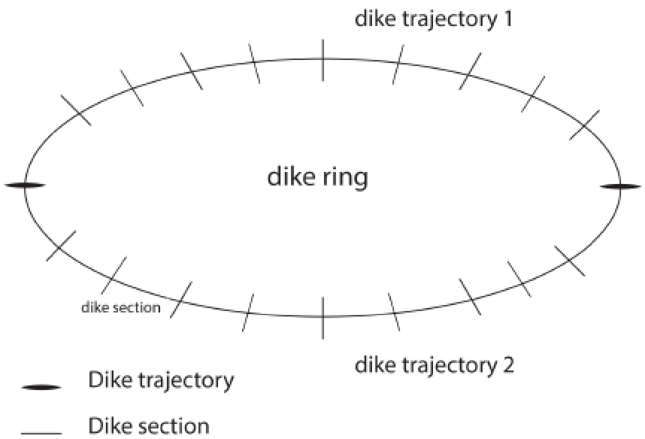

Figure 6 Difference between a dike ring area, dike trajectory and dike reach (also called dike section) (source: (Jonkman, Jorissen, Schweckendiek, & Van den Bos, 2018)

While focusing on hydraulic structures it becomes apparent that a (semi-)probabilistic analysis is not possible for the failure mechanism piping at hydraulic structures according to the WBI 2017 (Rijkswaterstaat, 2017c; Rijkswaterstaat, 2019). Piping occurs when water is able to flow underneath or around a hydraulic structure. This is caused by a difference in water pressure at both ends of the hydraulic structure. Figure 7 shows a schematization of the occurrence of piping at a hydraulic structure. The involved sub-mechanisms that are displayed in Figure 7 are explained in section 3.1 of this thesis. Due to the lack of a (semi-)probabilistic approach the piping at hydraulic structures failure mechanism is still assessed according to calculation rules from the ‘Wettelijk Toetsinstrumentarium 2011’ (WTI2011) (Rijkswaterstaat, 2019). Furthermore, the assessment of time dependent failure mechanisms (such as piping) can be improved according to (De Waal, 2016).

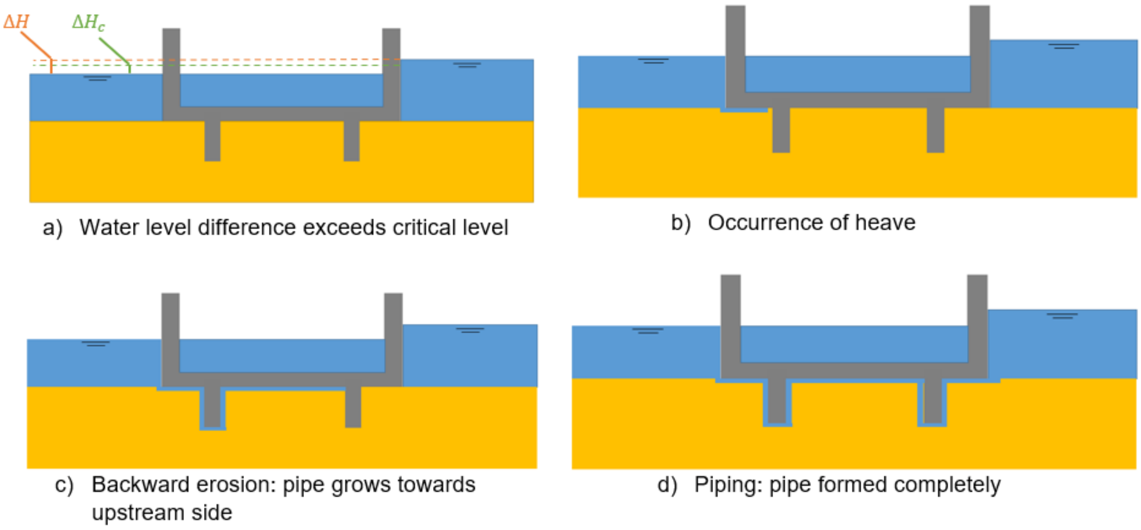

Figure 7 Occurrence of piping at a hydraulic structure. The current assessment method is a comparison between the critical head difference (ΔH_c) and the occurring head difference over the hydraulic structure (ΔH)

According to the failure probability approach all failure mechanisms must be assessed by a method that results in a failure probability. However, since there is no (semi-)probabilistic assessment method available for piping at hydraulic structures the assessment of this failure mechanism does not have a failure probability as result. The current assessment method is based on a comparison between the critical head difference and the occurring head difference at the hydraulic structure (see Figure 7a). These values correspond to the return period belonging to the norm of the involved dike trajectory. If the occurring head difference is larger than the critical head difference ($\Delta H > \Delta H_c$) then the assessment outcome is that the structure does not satisfy the norm. If the occurring head difference is smaller than the critical one the structure does satisfy the norm. However, to obtain an accurate failure probability of the hydraulic structure all relevant failure mechanisms and their respective failure probabilities should be included, also the failure mode piping at hydraulic structures.

1.4 Research goal

The objective of this research is to explore the possibilities of including a full probabilistic calculation method (either level II or level III) for the failure mechanism piping at hydraulic structures in the current assessment procedure. This goal has been translated into the following research question:

'How can the current WBI 2017 assessment method of the failure mechanism piping at hydraulic structures be altered to obtain a failure probability as the assessment result?'

To be able to answer the posed research question a literature study will be conducted. This literature study has two different focusses. The first focus is on the current WBI 2017 assessment procedure and dives into the methodology of how primary flood defense systems are currently assessed. The second focus is on the failure mechanism piping at hydraulic structures. In this part of the literature study further research is done on how the failure mechanism works and what information lacks to come to the probability of failure due to piping. The literature study is followed by a description of the applied method in this thesis. Then, a case study on the chamber lock of the Markkanaal near Oosterhout, the Netherlands is performed where the possibilities of a full probabilistic approach will be explored.

1.5 Structure of the report

This report is structured as follows. In chapter 2, the current assessment strategy according to the WBI 2017 is treated. An analysis of this assessment tool that is currently in use is performed in this chapter. It concludes with several critical notes regarding the methodology of the assessment strategy for hydraulic structures.

Chapter 3 contains the literature study on the failure mechanism piping at hydraulic structures. It elaborates on the physical processes that are of importance for this failure mechanism. Furthermore, the fault tree for this failure mechanism and the applicable models of each sub-mechanism are analyzed in this chapter. It also treats the available tests in the current WBI 2017 framework for the failure mode piping at hydraulic structures.

The methodology on how the possibilities for a probabilistic assessment method are explored is given in chapter 4. It compares the more conventional method of a fault tree analysis with the more innovative method of an event tree approach. This comparison is done by discussing negative and positive aspects of both methods. After providing the conclusion that an event tree approach fits best, several event tree variants are designed in section 4.4. Section 4.5 concludes chapter 4 with methods of quantification that are used in this thesis to perform the probabilistic assessment

Chapter 5 contains the case study that is performed in this thesis. It gives background information of the area and it contains an analysis of the relevant data for the probabilistic assessment. The subject of the case study is the Marksluis chamber lock (in Dutch: 'schutsluis') located in Oosterhout, Noord-Brabant, the Netherlands.

Chapter 6 treats the results of the probabilistic assessment of the Marksluis. It starts with a derivation of both the warning limit value and the lower limit value (together the norm) for the occurrence of piping at a hydraulic structure. It concludes with a comparison of the calculated failure probability and the derived requirements.

In chapter 7 a discussion is presented. This chapter treats both positive and negative aspects of the applied method to calculate the probability of failure of the chamber lock due to piping in the case study. Furthermore, additional critical notes that were found during the literature study on the WBI 2017 are presented in this chapter.

Chapter 8 contains the conclusions that are drawn after the analysis of the results. It also answers the main research question as posed in section 1.4. Recommendations for further research are given as well in chapter 8.

To summarize, this thesis can be divided into four different phases (phase I to phase IV). Each phase contains a certain number of chapters and answers relevant questions for this research. An overview of the different phases, the belonging chapters and relevant questions is presented in Figure 8.

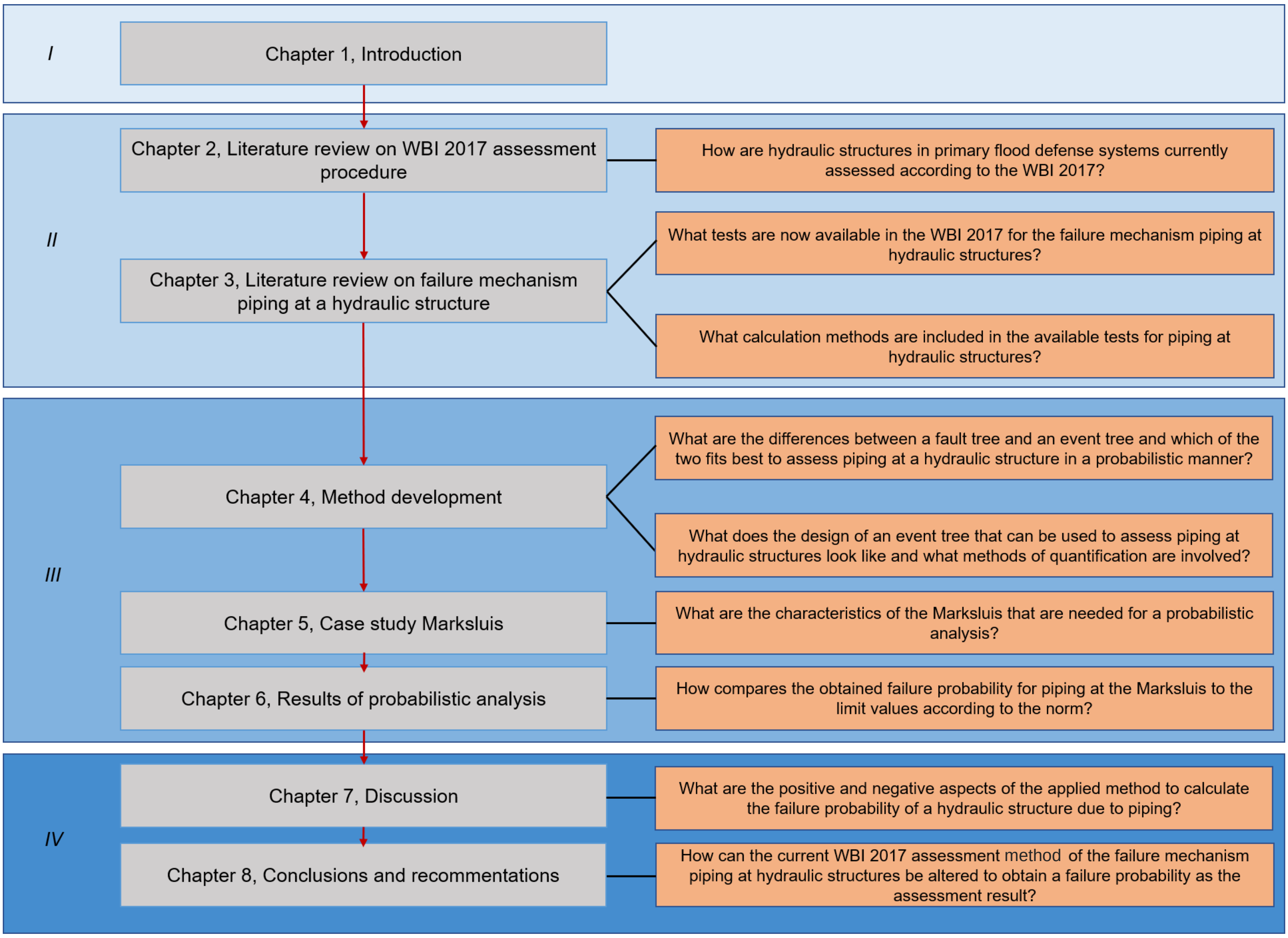

Figure 8 Overview of the structure of the report. On the left the phase of the report is shown. In the grey boxes the chapters are given and in the red boxes relevant questions that are answered in the belonging chapters are shown

2.Assessment method according to WBI 2017

For this research it is important to have a good understanding of the methods that are applied nowadays for the assessment of primary flood defenses. Therefore, this chapter treats the set-up and application of the WBI 2017 framework. This chapter has the following composition:

- The main steps of the assessment (section 2.1);
- The execution of the assessment (section 2.2);
- Reporting after the assessment (section 2.3);
- Available assessment methods for hydraulic structures (section 2.4).

Section 2.4 ends with an analysis on the available assessment methods for primary flood defenses that are hydraulic structures. This section highlights where the assessment procedure can be improved.

Furthermore, during the study of the WBI 2017 framework a number of critical questions and challenges are formulated. These critical notes are summarized in the discussion, treated in chapter 7.

The information in this chapter is based on two documents: 'Regeling veiligheid primaire waterkeringen 2017, bijlage I Procedure' and 'Regeling veiligheid primaire waterkeringen 2017, bijlage III Sterkte en veiligheid' (Rijkswaterstaat, 2017b; Rijkswaterstaat, 2017c).

2.1 The prescribed procedure to assess flood defenses

According to the WBI 2017 it is prescribed how flood defenses must be assessed. Within this assessment three main phases can be distinguished:

1. Preparation
2. Execution
3. Reporting

Each of these activities can have multiple sub-steps. Explanation on the three main phases is given below.

1. Preparation

The main task during the preparation phase is to obtain all information that is needed to carry out the assessment of the involved primary flood defenses. After all relevant information is collected, a strategy on how to assess the flood defenses must be thought of. The basis of this strategy is the collected information.

2. Execution

The second phase is focused on the execution of the assessment of the primary flood defenses. This phase can be divided into three sub-steps:

- The general filter;
- The assessment procedure;
- The composition of the safety judgment.

2.1 The general filter

If the general filter is applicable to the entire dike trajectory of interest, a direct safety judgment of this trajectory can be composed. However, if it only applies to a certain reach of the involved dike trajectory (in Dutch: 'dijkvak') a general filter cannot be applied. In such a case a custom test must be carried out.

In other words, the general filter is the fastest way to draw a conclusion of the safety of a dike trajectory. However, this is only possible if the criteria for conducting an assessment through the general filter are met. More information about when it is allowed to use the general filter is given in section 2.2.

2.2 The assessment procedure

If a dike trajectory cannot be assessed according to the general filter as described in the previous sub-section, a different approach must be taken. This procedure can be divided into four different possible sequential tests. These tests are getting more detailed and less conservative for each next test. From the least detailed and exact to most detailed and exact, the following tests are possible:

- A simple test;
- A detailed test per dike reach;
- A detailed test per dike trajectory;
- A custom test.

The custom test is the most advanced test possibility. This test will be used when the simple test or detailed test per dike reach or trajectory are not applicable. It is also a possibility to conduct a custom test when the manager of the involved flood defenses finds the results of the other three possible tests not trustworthy. Section 2.2 gives more insight in the execution of the four test possibilities.

2.3 The composition of the safety judgment

When the assessment is completed the manager of the flood defenses must come up with a safety judgment. This judgment is expressed in a category. The category tells to what extent the involved flood defenses do or do not satisfy the applicable safety norm. The different categories are treated in section 2.2 by Table 1.

3. Reporting

When the assessment is completed, the manager must report the results to the minister of infrastructure and water management. This is done through the 'Inspectie Leefomgeving en Transport' (ILT). Section 2.3 explains the elements that must be present in the report.

2.2 Execution of the assessment

During the assessment it is important to have clear understanding of what the different limit values involve that are described by the norm. There are two possible values prescribed by the norm to use during the assessment:

- The warning limit value (in Dutch: 'signalerinswaarde');
- The lower limit value (in Dutch: 'ondergrens').

The warning limit value indicates that maintenance on the involved flood defenses is needed in the near future. The actual amount of time is sufficient to make sure the maintenance activities for the involved structure or dike can be completed before the lower limit value will be reached. The lower limit value is the value that the resistance of the structure or dike needs to meet. If it does not meet this value, it does not satisfy the safety the minimal safety level anymore.

From the previous section it can be concluded that there are multiple assessment options available that in the end lead to a safety judgment of a primary flood defense system. A main division can be made between the general filter (which will be named layer 1) and the assessment procedure (layer 2). When the general filter cannot be applied to assess the safety of the entire dike trajectory of interest, one should proceed to the assessment procedure. Within the assessment procedure different tests are available to draw a conclusion on the safety of a flood defense system. In Figure 9 a schematic overview of all possibilities is shown that result in a flood safety judgment (layer 3).

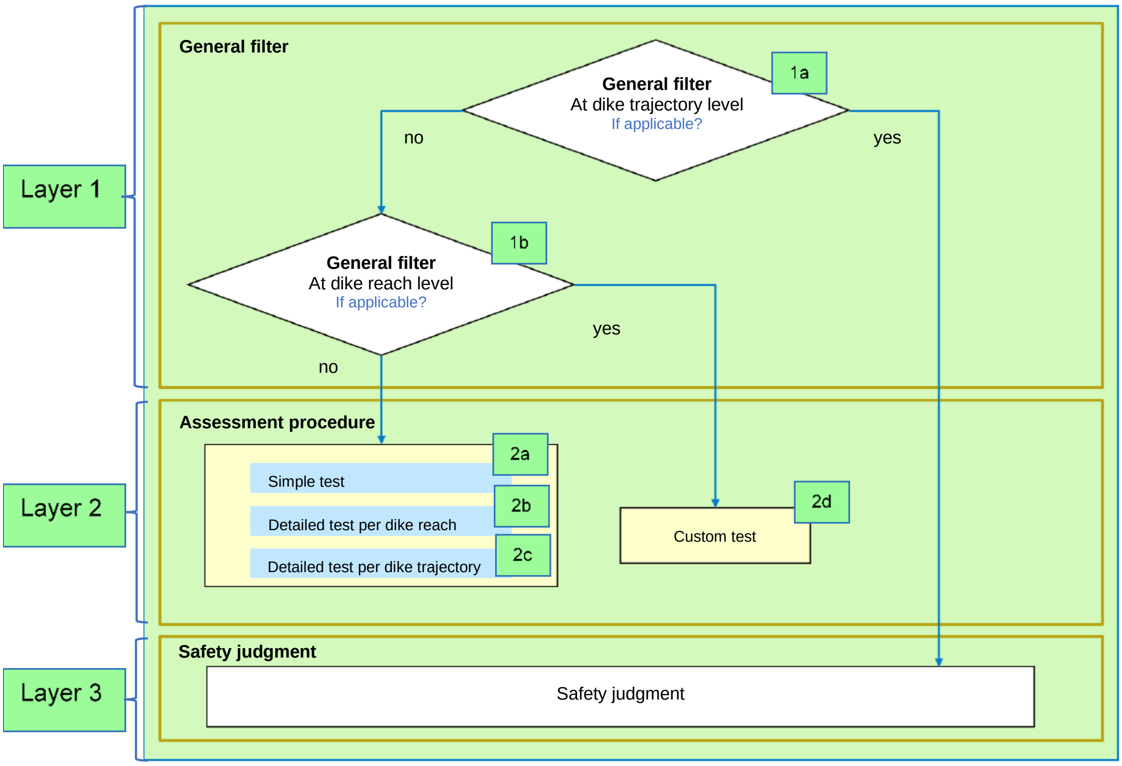

Figure 9 Schematic overview of the general filter and assessment procedure (based on: (Rijkswaterstaat, 2017b))

The different three layers that are shown in Figure 9 involve different assessment strategies. These strategies are explained per layer in the remaining part of this section.

Layer 1: General filter
The general filter can be applied in two ways:

- To an entire dike trajectory;
- To a dike reach.

Layer 1a: General filter applied to an entire dike trajectory
The general filter applied to an entire dike trajectory is only applicable when the probability of flooding is well above or well below the warning limit value. If this condition is met, the results of the project ‘Veiligheid Nederland in Kaart’ (VNK) in combination with expert judgment can be used to obtain a safety judgment.

The obtained results of the VNK project are used to make a selection of dike trajectories that are expected to have a probability of flooding that suffices the condition of applying the general filter. This has resulted in:

- 14 trajectories that have a probability of flooding that is well above the warning limit value (factor 90 or higher);
- 9 trajectories were found with a probability of flooding that is well below the warning limit value (at least a factor 100).

If the dike trajectory of interest is one of these 21 trajectories, the general filter can be applied to the entire trajectory. The general filter for an entire dike trajectory is also applicable when the new assessment method of the WBI 2017 and changes to the primary flood defense system do result in only a negligible change in probability of flooding. When the general filter can be applied to an entire dike trajectory, a direct safety judgment for the entire trajectory can be made.

Layer 1b: General filter applied to a dike reach
Another general filter is the filter applied to a dike reach. This filter can be used when:

- The application of the generic tests of the assessment procedure (layer 2) does not result in a reliable result, or;
- A custom test gives a comparable result as to when the guidelines of the WBI 2017 are followed, however with less needed effort.

When one of these conditions is met the general filter for a dike reach can be used. This holds that a custom test can be carried out.

Layer 2: Assessment procedure
To be able to start the assessment procedure, data of the involved dike trajectory is needed as input. With this input and a model, the probability of failure belonging to different failure mechanisms can be calculated. However, to do so one needs to make schematizations. This starts at a conservative level. To get to a more accurate result more data may be needed.

To map the different failure mechanisms and their belonging probabilities of failure, schematization guidelines can be used. When schematizing it is important to work with the same reference. Therefore, it is decided to make the calculations with the resistance and load at the end of the current assessment period (31st of December 2022).

Layer 2a: Assessment procedure through a simple test
There are three possible tests available when the general filter is not applicable. The first one is the simple test. As the name suggests, this test is the least advanced of the three possible tests. Through characteristics and dimensions of flood defenses it is decided per dike reach whether a certain failure mechanism should be considered. Only if the probability of failure is negligible the simple test is sufficient and no further tests are needed. If not, one should proceed with the detailed test per dike reach.

For some failure mechanisms a simple test is not available. In those cases one must start with a detailed test per dike reach immediately.

Layer 2b: Assessment procedure through a detailed test per dike reach
The next available test after the simple test is the detailed test per dike reach. To conduct this test the limit values according to the norm for the entire involved dike trajectory must be translated to a minimum resistance that the dike reach must provide (expressed as a probability of flooding as well). Such a dike reach could fail in multiple ways, therefore a minimum resistance for each failure mechanism is needed. Then, the assessment can be performed either in a probabilistic or semi-probabilistic manner. The assessment itself can be done multiple times, especially when refinement is desired.

For each failure mechanism, it must be checked if a mathematical model can be used. If this is not the case, one should proceed with follow-up steps.

The outcome of this test is a conclusion whether the probability of flooding per dike reach per failure mechanism is sufficiently low. This is expressed with categories. Each category stands for how the obtained probability of flooding relates to the requirement of the probability of flooding stated by law (Waterwet, 2020). Based on these results follow-up steps are taken.

After the completion of the detailed test per dike reach per failure mechanism, four potential follow-up steps are possible:

- Proceed with a detailed test per dike trajectory;
- Refinement of the schematization;
- Proceed with a custom test;
- End the assessment.

The first three options can be taken when the results obtained so far are not considered reliable enough. The option to end the assessment is only possible when:

- It can be explained that further tests do not lead to a change of the obtained results;
- The assessment provides all information to write the report;
- Maintenance oriented solutions are economically more attractive compared to further testing.

An exemption for the first criterium is possible when the manager of the involved flood defenses can show that further testing is not resulting in more valuable information in this assessment round. This only holds for assessment results that are not part of category D (the worst category). Explanation of the various safety judgment categories is provided in the section about layer 3: Safety judgment.

Layer 2c: Assessment procedure through a detailed test per dike trajectory
Another available test is the detailed test for an entire dike trajectory. This test will always follow either a full or semi-probabilistic approach. This test approach is only available for the assessment of a limited amount of failure mechanisms (see Table 3). The outcome of this test can show in what way the flood defenses contribute most to the total probability of flooding of the dike trajectory. This information can then be used to show where an improvement in assessment result can potentially be obtained (by making use of more accurate input data).

The result of a detailed test per dike trajectory is comparable to the result of the detailed test per dike reach. It is expressed in safety categories too.

Layer 2d: Assessment procedure through a custom test
As mentioned earlier the detailed test per dike reach and detailed test per dike trajectory are tests that can be applied to numerous cases because of the use of general models and failure mechanisms. However, it can occur that these general models are not good enough to provide acceptable results. In such cases a custom test can be carried out. With a custom test the following information can be included in the assessment:

- Information on the location;
- More in depth analyses;
- The knowledge and experience of the manager of the involved flood defenses.

The execution of a custom test may be either deterministic or probabilistic per dike reach or per dike trajectory. An important side note of performing a custom test is that the manager of the involved flood defenses is responsible for ensuring the quality of the performed assessment.

Layer 3: Safety judgment
When the assessment is completed a safety judgment must be given. This can be done by combining the assessment results of the different dike reaches and their corresponding different failure mechanisms. After combining these results, the entire trajectory can be placed into a category. This process is called assembling (Lam, Diermanse, & Knoeff, 2016). The obtained category describes the safety judgment in reference to both the warning limit value and the lower limit value. The following categories exist:

- A+: Probability of flooding is much smaller than the warning limit value;
- A: Probability of flooding is smaller than the warning limit value;
- B: Probability of flooding is larger than the warning limit value, but smaller than the lower limit value;
- C: Probability of flooding is larger than the lower limit value;
- D: Probability of flooding is much larger than the lower limit value.

Figure 10 summarizes the needed steps and possible tests needed to come to a safety judgment for a dike trajectory.

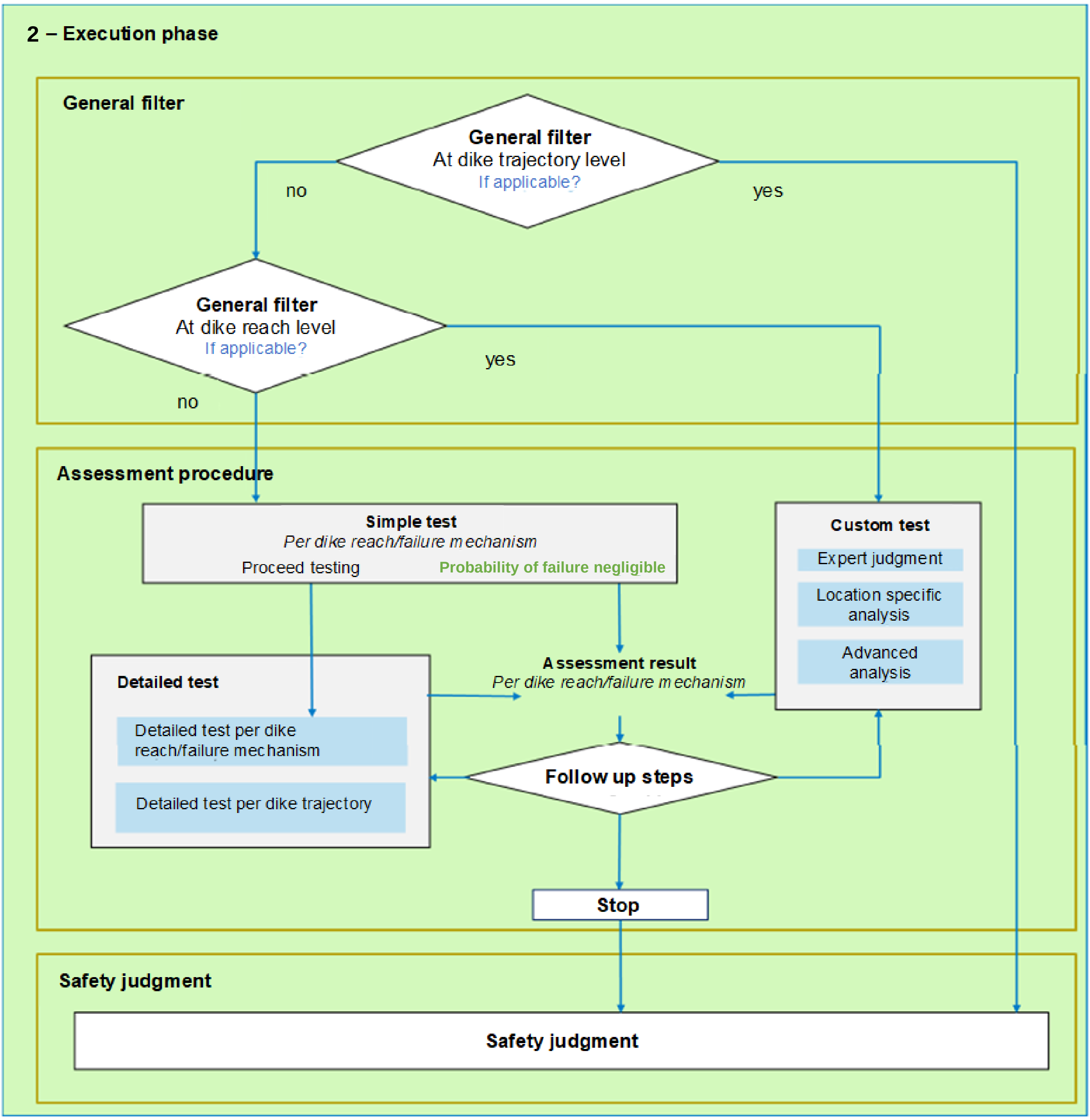

Figure 10 Overview of possible tests and steps needed to obtain a safety judgment for a dike trajectory (based on: (Rijkswaterstaat, 2017b))

Before any safety judgment for the full dike trajectory can be given the assessment results of each dike reach must be known. Per dike reach the requirement of the failure probability is spread over the different applicable failure mechanisms. In this way an assessment result of each failure mechanism can be obtained. Table 1 shows the different possible assessment categories for a certain failure mechanism applicable to a certain dike reach. The safety judgment is obtained by comparing the obtained failure probability from the assessment to the warning limit value and the lower limit value.

Table 1 Categories for the safety judgment per dike reach per failure mechanism (source: (Rijkswaterstaat, 2017c), translation own courtesy)

Cat.	Safety judgment per dike reach per failure mechanism	Boundaries of category
		$P_{f,reach}$ Failure probability per dike reach (or hydraulic structure) $[1/year]$ $P_{req,wlv}$ Warning limit value of the dike trajectory $[1/year]$ $P_{req,llv}$ Lower limit value of the dike trajectory $[1/year]$ $P_{req,llv,reach}$ Failure probability requirement per dike reach (or hydraulic structure) following the lower limit value $[1/year]$ $P_{req,wlv,reach}$ Failure probability requirement per dike reach (or hydraulic structure) following the warning limit value $[1/year]$
I_v	More than meets the warning limit value	$P_{f,reach} < \frac{1}{30} * P_{req,wlv,reach}$
II_v	Satisfies the warning limit value	$\frac{1}{30} * P_{req,wlv,reach} < P_{f,reach} < P_{req,wlv,reach}$
III_v	Satisfies the lower limit value and possibly satisfies the warning limit value	$P_{req,wlv,reach} < P_{f,reach} < P_{req,llv,reach}$
IV_v	Possibly satisfies the lower limit value or the warning limit value	$P_{req,llv,reach} < P_{f,reach} < P_{req,llv}$
V_v	Does not satisfy the lower limit value	$P_{req,llv} < P_{f,reach} < 30 * P_{req,llv}$
VI_v	Less than satisfies the lower limit value	$P_{f,reach} > 30 * P_{req,llv}$
VII_v	No judgment available yet	N/A

After conducting the assessment for a certain failure mode for all dike reaches of a dike trajectory, the obtained assessment results can be combined to an assessment result of the dike trajectory for that certain failure mechanism (Lam, et al., 2016; Rijkswaterstaat, 2017c). This must be done for all applicable failure modes. The final step is to combine the obtained assessment results of all applicable failure modes of the dike trajectory to a safety judgment for the full trajectory (Lam, et al., 2016; Rijkswaterstaat, 2017c). When this is done, conclusions can be drawn whether the full dike trajectory satisfies the warning limit value and/or the lower limit value.

Next to the obtained assessment result for the full trajectory it can also be interesting to have an assessment result for each dike reach. By doing this, an overview can be obtained of where the weakest links of the dike trajectory are located. Because assessment of different failure modes can result in different definitions of dike reaches a new definition of dike reaches is needed (Lam, et al., 2016). When this is done, the obtained assessment results of the different failure modes for a certain dike reach can be combined to form a safety judgment for the that dike reach.

2.3 Reporting after the assessment

After completion of an assessment a report must be written. The report must be made available through the 'waterveiligheidsportaal'. At least the following information has to be present in the report:

- **The safety judgment**: a safety judgment must be given in the report. This is done with the five possible categories as named in section 2.2 of this thesis;
- **An elucidation of the safety judgment**: an explanation on how the safety judgment is obtained is part of the report as well. Through the elucidation it must become clear which failure mechanisms and dike reaches have the largest contribution to the probability of flooding of the trajectory. Also, the influence of maintenance on the safety judgment must be named. Furthermore, it must be stated whether special assessments apply on the trajectory and how they contribute to the assessment result. Finally, dike reaches that are not considered during the assessment must be named. This only holds for reaches that are already being improved;
- **A summation of the needed measures**: the report must name potential measures that are highly likely to be needed to make sure the trajectory meets the norm (again). Also, it should be stated how the obtained assessment results (both for approved flood defenses and rejected flood defenses) influence all aspects of the management of the flood defenses;

- **Supplementary information if applicable**: this only holds for trajectories with extra safety requirements. It should then give information whether these extra requirements are met.

For every assessment a general rule applies. This rule holds that each assessment needs to be reliable and each assessment step needs to be reproducible. To make sure this applies, the manager of the involved flood defenses needs to report all choices that are made during the assessment. Furthermore, reasoning behind these choices must be clear as well.

2.4 Available calculation methods for the assessment of hydraulic structures

When a dike trajectory is assessed according to the WBI 2017 it is important to first identify what elements the involved dike trajectory contains. A division between the following main types of flood defenses can be made:

- Dikes and dams;
- Dunes;
- Hydraulic structures

Each element has its own set of potential failure mechanisms. During an assessment all these relevant failure modes have to be taken into account. It depends on the type of flood defense whether a simple test, detailed test per dike reach, detailed test per dike trajectory or a combination of the previously named tests can be performed. In Table 3 an overview is presented of the availability of the different test for each failure mechanism. If a test is available an 'X' is shown in the table. An empty spot means that a test lacks for the involved failure mechanism.

Table 3 is separated into three main sections. These sections correspond with the three main types of flood defenses. Each section shows failure modes that can be relevant. For simplicity, the failure modes can be abbreviated according to the column 'Code'. The next column displays a group number. The assigned group number is an indication for what type of calculation method is available for that certain failure mechanism. An overview of the calculation method available per group number is shown in Table 2.

Table 2 Group number that applies to failure mode to be assessed and its corresponding calculation method (based on: (Rijkswaterstaat, 2017c; Van Bree, 2015; Lam, Diermanse, & Knoeff, 2016))

Group	Calculation method
1	The detailed test per dike reach is performed by a full probabilistic calculation method.
2	The detailed test per dike reach is performed by a semi-probabilistic calculation method. The assessment result is a safety factor. Through a relationship between the safety factor and failure probability the distance from the norm, hence the assessment category can be determined.
3	The detailed test per dike reach is performed by a semi-probabilistic calculation method. Again, the assessment result is a safety factor. However, for failure modes in this group there is no relationship available between the safety factor and failure probability. Therefore, extra calculations with different hydraulic loads (corresponding to the possible assessment categories) must be performed to obtain an assessment result.
4	The detailed test per dike reach cannot be performed by either a full or semi-probabilistic calculation method. Assessment of failure modes within this group is most of the time not done by a comparison between a calculated failure probability and the requirement for the failure probability of the involved dike reach.

As explained in section 1.3, this research focuses on the assessment of hydraulic structures. Therefore, the third section of Table 3 is most relevant. From this section it becomes clear that for almost all failure mechanisms a simple test is available to perform the assessment. The simple test is absent for the failure mechanism strength and stability for pointed structures. This means that for the assessment of this failure mode one should directly start with the detailed test per dike reach. As can be observed from Table 3 the detailed test per dike reach is available for all failure mechanisms related to hydraulic structures. As explained in section 2.2, if the result obtained by a detailed test per dike reach is not considered reliable enough a follow-up action could be conducting a detailed test per dike trajectory. From Table 3 it becomes apparent that this follow-up action can be done for almost all potential failure modes for a hydraulic structure. However, it does lack for the failure mode piping at hydraulic structures.

The absence of a detailed test per dike trajectory for the failure mechanism piping at hydraulic structures means that there is no full probabilistic assessment method available. In fact, according to the schematization guideline for the failure mechanism piping at hydraulic structures, a semi-probabilistic assessment method is lacking as well (Rijkswaterstaat, 2019). This automatically means that the outcome of the detailed test per dike reach for piping at hydraulic structures is not a failure probability. The outcome is a comparison between the critical hydraulic gradient and the occurring hydraulic gradient. By definition, such an outcome is less accurate than an exact failure probability as assessment result. More information on the current assessment procedure for this failure mode is presented in chapter 3 of this thesis.

It is desired to obtain a failure probability as the outcome of the assessment. In this way the obtained failure probabilities of the different relevant failure modes for a certain dike reach can be combined. Once the failure probabilities of all dike reaches are known a final assembly can take place to obtain the probability of failure for the full dike trajectory. This value can then be compared with the norm and a conclusion can be drawn on whether the dike trajectory fulfills the requirement regarding flood safety.

Because of the absence of the failure probability as outcome of the assessment of the failure mode piping at hydraulic structures, the procedure as described above cannot be followed accurately once piping at hydraulic structures is a relevant failure mechanism. This causes a large gap between the assessment results of the different failure mechanisms for hydraulic structures (three times group 1 versus two times group 4). This stresses the relevance of research on how it is possible to obtain a failure probability as assessment outcome for this failure mechanism.

To understand what is needed to have a failure probability as outcome of the assessment for piping at hydraulic structures, chapter 3 gives more information on the physical processes behind piping. Furthermore, it explains the current assessment procedure for this failure mechanism to stress what is exactly lacking in terms of a probabilistic assessment.

Table 3 Available assessment tests for different failure mechanisms applicable to dikes and dams, dunes and hydraulic structures. ST = Simple Test, DT = Detailed test, X = prescription on test is available (based on: (Rijkswaterstaat, 2017c; Rijkswaterstaat, 2019))

Failure mechanism	Code	Group[2]	ST	DT per reach	DT per trajectory	Schematization guidelines
Dikes and dams						
Macro stability inward	STBI	2	X	X	X	Macro stability
Macro stability outward	STBU	4	X	X		Macro stability
Piping	STPH	2	X	X	X	Piping
Micro stability	STMI	4	X	X		Micro stability
(coverings)						
Wave impact on asphalt covering	AGK	3	X	X		Asphalt covering
Water overpressure on asphalt covering	AWO	4	X			Asphalt covering
Grass covering erosion outward embankment	GEBU	3	X	X		Grass covering
Grass covering slip off outward embankment	GABU	4	X	X		Grass covering
Grass covering erosion top and inward embankment	GEKB	1		X	X	Grass covering and height
Grass covering slip off inward embankment	GABI	4		X		Grass covering and height
Stability pavement	ZST	3		X		Stability pavement
Dunes						
Dune erosion	DA	3		X	X	Dune erosion
Hydraulic structures						
Height hydraulic structure	HTKW	1	X	X	X	Height hydraulic structure
Closing reliability hydraulic structure	BSKW	1	X	X	X	Closing reliability hydraulic structure
Piping at hydraulic structure	PKW	4[3]	X	X		Piping at hydraulic structure
Strength and stability pointed structures	STKWp	1		X	X	Strength and stability hydraulic structure, pointed structure
Strength and stability longitudinal structures	STKWl	4	X			

[2] An explanation of what each group number means is given in Table 2.
[3] Please note that in 'Bijlage III Sterkte en veiligheid' (Rijkswaterstaat, 2017c) the failure mode piping at hydraulic structures is classified as a group 2 failure mechanism. This is incorrect since no (semi-)probabilistic test method is available for this failure mode. Therefore, it must be classified as a group 4 failure mode as is done accordingly by (Rijkswaterstaat, 2019).

3.Piping at hydraulic structures

As concluded in chapter 2, a probabilistic analysis for the failure mechanism piping at a hydraulic structure is lacking. To know what is needed for a probabilistic analysis it is important to understand the physical processes that drive piping. This chapter starts with an explanation of these processes (section 3.1). It also discusses potential strategies to mitigate piping once it is detected. Section 3.2 relates to the assessment strategy discussed in chapter 2 and elaborates on how the piping failure mechanism is assessed nowadays according to the WBI 2017. Section 3.3 describes how the full failure mechanism can be divided into multiple sub-mechanisms that eventually can lead to piping. Each sub-mechanism is treated along with its applicable model(s).

3.1 Physical processes behind piping

This section introduces the piping failure mechanism at hydraulic structures. Section 3.1.1 distinguishes the different sub-mechanisms that in the end lead to failure due to piping. To prevent failure of a hydraulic structure as a result of piping certain emergency measures can be taken once piping becomes apparent. Different emergency strategies are discussed in section 3.1.2.

3.1.1 The sub-mechanisms that involve piping

Piping is the phenomenon when groundwater is able to flow around or under a dam, dike or hydraulic structure. As the name suggests, this mechanism is characterized by the formation of a pipe as a consequence of the erosion process. The pipe, which is an undesired phenomenon, enables the water to flow underneath or alongside the hydraulic structure (the focus of this thesis is piping at hydraulic structures, therefore the processes are mainly focused on hydraulic structures). If not managed adequately the pipe is able to grow to the point where all supporting soil underneath and around the structure has eroded, eventually causing the hydraulic structure to fail structurally. The likelihood for piping to occur depends on the soil type. In this section the different physical processes that involve the growth of a pipe are described. Along with this description an overview of the different phases is illustrated in Figure 11.

First, the conditions for a pipe to form must be present. This means that an aquifer (soil layer with a high level of permeability) must be present right next to and/or underneath the hydraulic structure. Also, a head difference (difference in water levels) over the hydraulic structure needs to be present. When this head difference exceeds a certain critical head difference the conditions for piping to commence are met. The value for the critical head difference is dependent on the resistance against groundwater flow at the structure. Also, heterogeneities in the soil and zones of the subsoils that contain disturbances have an influence on the critical head difference (Achmus & Mansour, 2006). For hydraulic structures constructed directly on top of an impermeable layer (e.g. clay) piping is less of an issue since water then first needs to burst through that layer. This is called uplift.

When the conditions for a pipe to form are present the first sign of a pipe starting to form is that a groundwater flow is noticeable. This flow goes from the side with the higher water level towards the lower water level side. This water flow (both flow velocity and discharge) increases linearly with an increase in water level difference ('t Hart, 2018). At first no erosion occurs. However, if this groundwater flow continues to increase and exceeds the resistance of the sand particles it does initiate erosion.

As soon as erosion starts it will initiate the sub-mechanism heave (see Figure 11a). Here, sand between the bottom side of the seepage screen at the downstream side of the hydraulic structure and the exit point of the groundwater flow starts to liquify. The groundwater flow is now able to transport particles of sand in this liquified stage towards the exit point, thereby forming a well (see Figure 11b). At the exit point the sand particles are dropped right next to the well. In the sand layer the pipe now starts to form. By continued erosion the pipe grows upstream, which is called backward erosion (see Figure 11c). This continues until an equilibrium is reached according to Sellmeijer's theory. Then, the flow gradient decreases, thereby slowing down or even stopping the erosion process. However, experiments at the IJkdijk have shown that once a well has formed no equilibrium can be set anymore ('t Hart, 2018). In that way the piping process can be described as a self-enhancing process where erosion of the soil will continue once it has started. The results of the IJkdijk experiments therefore argue against Sellmeijer's theory.

At locations with a difference in water level over the hydraulic structure exceeding the critical water level difference, the pipe is able to continue to grow: erosion continues until the point where an open connection between either side is reached. At this stage the pipe is fully developed, and piping has become a fact (see Figure 11d).

After complete formation of the pipe there is no resistance against the flow anymore. This loss of resistance causes a pressure wave that widens the already existing pipe (Van Bree, 2015). More and more sand gets transported towards the exit point, which is displayed in Figure 11e.

It is also possible that multiple pipes form. At this point the structure is still structurally intact since it is still supported by its surroundings. The pipes can however interconnect and if erosion still continues they can grow into one large pipe. This accelerates the erosion process.

At the instance when the structure is not adequately supported anymore it can cause the structure to shift, tilt or collapse. With a collapse of the hydraulic structure (see Figure 11f) it is clear that it has lost its water retaining function. It can then be said that piping has caused the structure to fail leading to a breach in the dike trajectory (see Figure 11g). When the structure tilts or shifts due to severe piping but does not collapse it can be the case that the structure is still able to retain (most of) the water (except for the water running through the pipe) even though piping has occurred at the involved hydraulic structure.

To research piping in reality, (Achmus & Mansour, 2006) have set up model tests in a flume. Figure 11h shows the result of these tests where a widened seepage path can be observed.

It is desired to prevent piping where possible. Once it becomes evident that the failure mechanism piping is ongoing emergency measures can be taken to limit the impact. Section 3.1.2 elaborates on four potential emergency measures that can be taken.

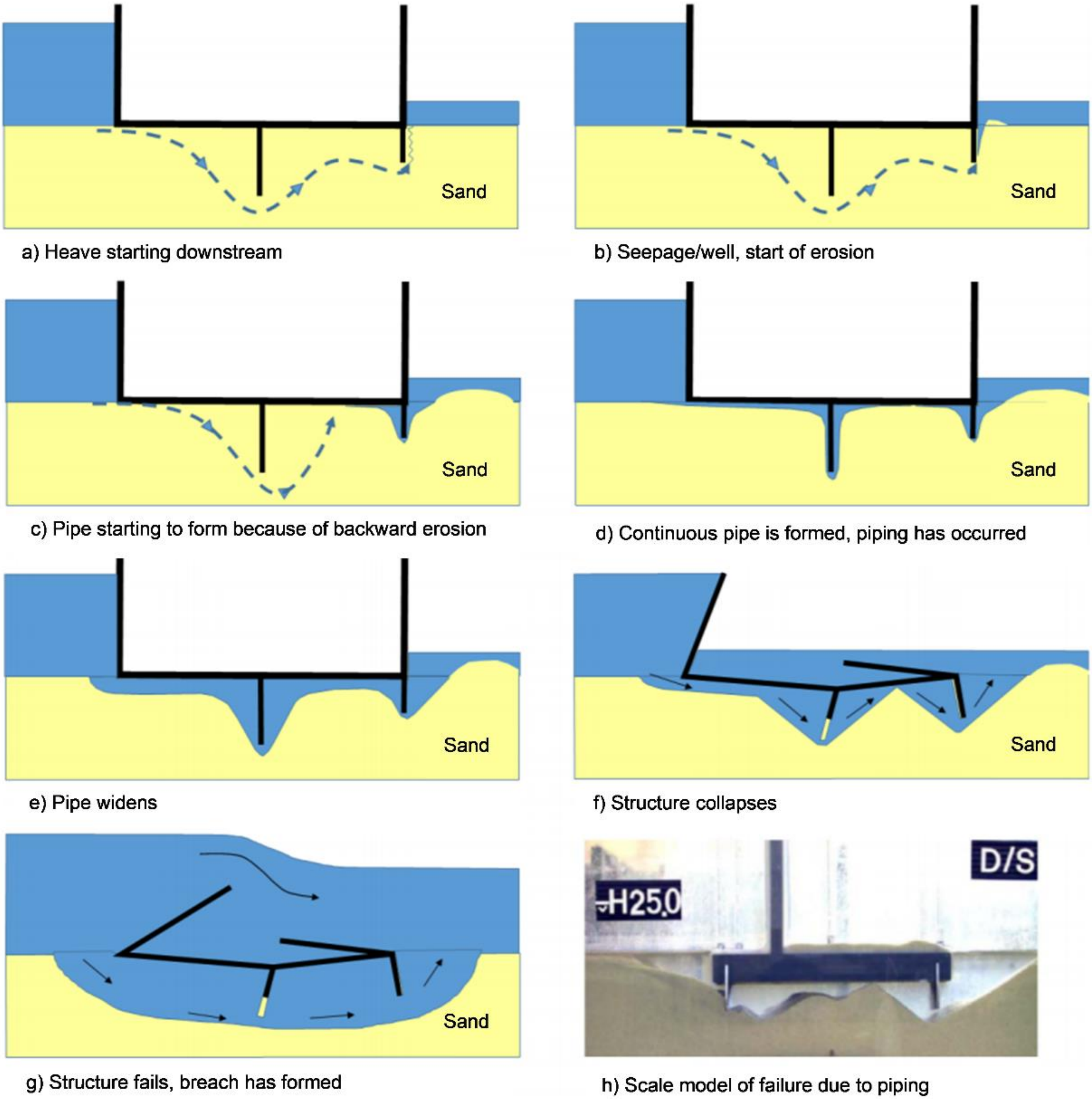

Figure 11 Different phases during the formation of piping at a hydraulic structure with seepage screens. Picture h shows a scale model proving the principle that is explained in the drawn figures (sources: (Achmus & Mansour, 2006; 't Hart, 2018))

3.1.2 Emergency measures

In Figure 11 the various phases of the formation of piping are presented. When the failure mechanism piping is ongoing, one wishes to have an early detection of the forming pipe. According to the result of the IJkdijk experiments (see section 3.1.1) once piping has started it continues until so much sand has eroded that the hydraulic structures has no foundation anymore. Therefore, emergency measures can be needed to prevent a flooding due to piping. In this section an overview of four commonly used emergency measures is given. These four emergency measures are: 1) placing relief wells, 2) constructing sandbag rings around the water boil, 3) placing parallel dams and 4) increasing the water level in ditches or canals on the low side of the levee (Jonkman, Jorissen, Schweckendiek, & Van den Bos, 2018). Below, a description of each emergency measure follows.

- Placing relief wells

One can release the pressure on the pipe by placing a relief well. The relief well is placed vertically into the ground. It passes through the impermeable layer (only applicable if an impermeable layer is present) and is inserted into the aquifer layer. Since the pressure in the relief well is lower than the pressure in the aquifer layer the water goes directly into the relief well. This releases the pressure, thereby lowering the piezometric head, that otherwise would potentially have caused a pipe to form and to grow.

Relief wells are only effective when they are placed in between the structure and the location where otherwise the sand well would have formed (or already started to show the first signs). In other words, relief wells are effective in the zone where uplift and/or heave can occur.

A point of attention for relief wells is that they must be designed such that no transport of material can take place. Otherwise erosion can take place even easier due to the presence of the manmade well.

- Constructing sandbag rings around the sand boil

In the event when a full pipe has already formed a sand boil can be detected at the exit location of the pipe on the side with the lower water level. Here the water leaves the soil. The sandbags function as an extra dike ring in itself where the water level can rise. By constructing (a) sandbag ring(s) around the water boil(s) one increases the counter pressure on the pipe on the lower water level side. A consequence of this measure is a decrease of ΔH thereby limiting the pipe growth (in transverse direction). As soon as an equilibrium state is achieved (the head difference is now smaller than the critical water level difference) the internal erosion process is halted.

- Placing parallel dams

When multiple wells have formed and form a larger well sandbag rings might not be effective anymore. Instead, parallel dams can be constructed. These dams are constructed parallel to the main structure. They function in a similar way as the sandbag rings. Their goal is to reduce ΔH below the critical head difference.

The disadvantage of these dams is that a lot of preparation is needed before the dams can be installed. This time needed for preparation is often longer than what the emergency situation allows (Jonkman, et al., 2018).

Furthermore, since the construction of the parallel dams must take place well before the emergency arises (due to the construction time) a lot of attention needs to go into the determination of the location where the dam is constructed. One needs to make sure that the dam is neither constructed too close nor too far away from the main structure. If the parallel dam is too close sand boils can appear at the land side of the parallel dam; if the parallel dam is constructed too far away the sand boils will form as they previously did due to an insufficient water level rise.

- Increasing the water level in the ditches/canals on the low side

A sand boil forms on the lower side of the levee when a pipe is forming. This sand boil can start to form on land or in ditches or canals. If the latter is the case the construction of sandbag rings is not handy since the sand boil is situated in a waterway. Instead, by raising the water level in the ditch/canal the water pressure exerted by the water in the canal counters the uplift pressure of the groundwater. This results in a lower head difference over the structure. Consequently, the erosion process will slow down or even stop. A limiting factor of this measure is the maximum allowable increase of the water level in the canal/ditch. When the maximum allowed increase is reached, the measure could be combined with the placement of a parallel dam which would allow a larger increase of the canal/ditch water level.

Now that the physical processes behind piping at a hydraulic structure are described, the next section (section 3.2) discusses how this failure mechanism is assessed according to the WBI 2017.

3.2 Assessment of piping at hydraulic structures according to the WBI 2017

As mentioned in chapter 2, multiple tests are available to assess flood defenses on the provided level of safety. For piping at a hydraulic structure the following tests are available (Rijkswaterstaat, 2017c):

- Simple test;
- Detailed test per dike reach;
- Custom test.

The more detailed a test gets, the more information and data is needed. This leads to a more accurate assessment result. It is common to start with a test that is broad and simple (the simple test). If no assessment result can be obtained from that test or if the result is that further testing is needed, one proceeds to the more exact and detailed test (first detailed test per dike reach, then custom test). In Figure 12 an overview is given of how the different test possibilities for piping at a hydraulic structure are related to each other.

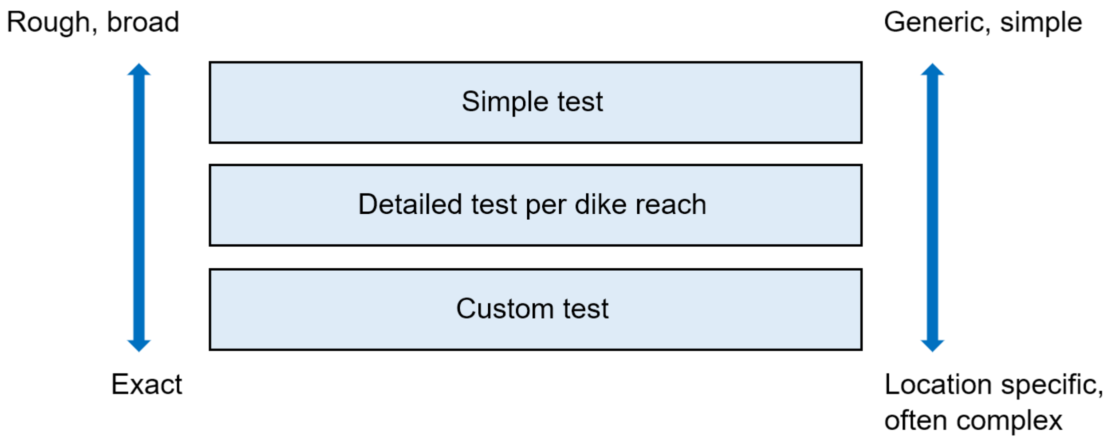

Figure 12 Available tests for assessment of piping at a hydraulic structure. The relation of the corresponding level of detail and exactness of the different tests is given by the blue arrows (based on: (Helpdesk Water, n.d.-a))

One may notice that the detailed test per dike trajectory[4] is missing. Due to the absence of both a full and a semi-probabilistic assessment method for piping at a hydraulic structure, the detailed test per dike trajectory cannot be carried out for this failure mechanism. This section treats the three available tests for piping at a hydraulic structure.

3.2.1 Simple test

For the simple test, two questions must be answered. The answers given conclude whether piping at a hydraulic structure is a relevant failure mechanism for the involved hydraulic structure. Figure 13 gives an overview of the questions to be asked for the simple test for the failure mode piping at hydraulic structures.

Section 2.1 of the schematization guideline for the piping at hydraulic structures failure mechanism (Rijkswaterstaat, 2019) can be consulted to answer these questions. When the outcome is that the assessment has to be continued, one proceeds with the detailed test per dike reach.

[4] As discussed in section 2.1 in general there are four test possibilities: the simple test, the detailed test per dike reach, the detailed test per dike trajectory and the custom test.

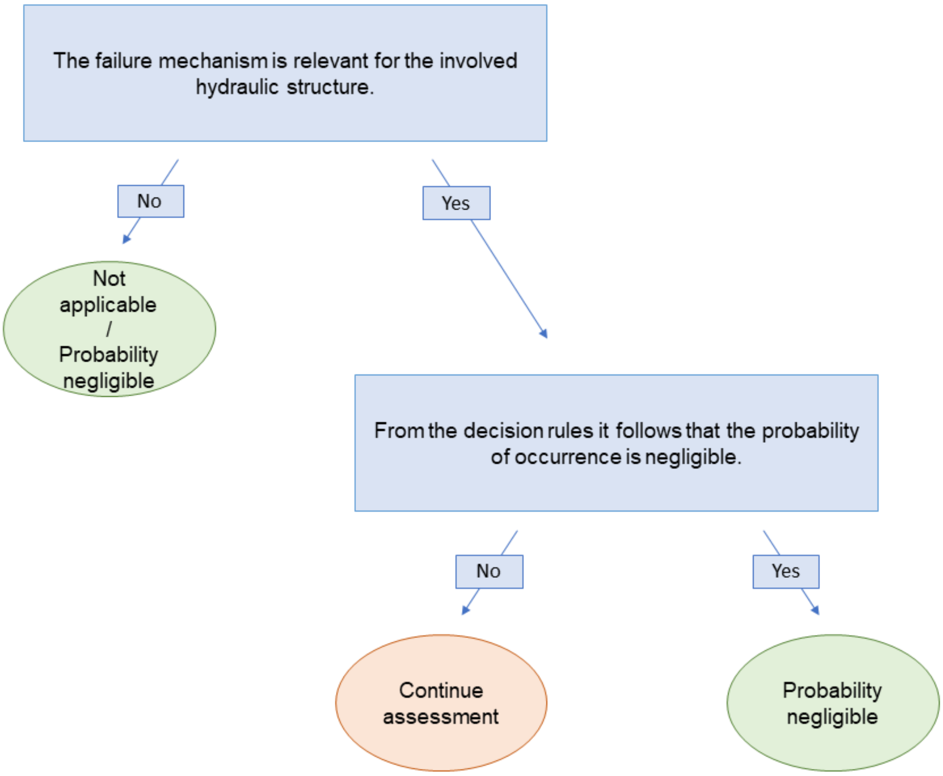

Figure 13 Simple test PKW (based on: (Rijkswaterstaat, 2019))

3.2.2 Detailed test per dike reach

When the simple test concludes that further assessment is needed the detailed test per dike reach is conducted. This test has an iterative character (Rijkswaterstaat, 2019).

Before the assessment can be conducted data needs to be collected. First, ground samples must be available. If these are not available soil research is needed. Amongst ground samples also other relevant data is gathered. This can for instance be data on bed protection that might be applied at the hydraulic structure. Data on the geometry of the hydraulic structure that is going to be assessed and data concerning the hydraulic load on the structure are needed too.

After collection of all data the schematization needs to take place. In this way the structure can be assessed on piping with the obtained data. Finally, an analysis is performed on the obtained assessment result. If it turns out that refinement of the result is desired, data that is needed to allow this refinement must be collected. Then, a new schematization is thought of after which a new assessment takes place. This explains the iterative character.

The steps that are taken in the detailed test for piping at hydraulic structures according to (Rijkswaterstaat, 2019) are visualized in Figure 14. The blue box shown in Figure 14 contains the assessment of a hydraulic structure on piping. For the detailed test per dike reach a three-dimensional approach is used where the seepage paths around both sides of the structure and under the structure are analyzed. The orientation (horizontal/vertical) of the seepage paths determines which model is best applicable for the analysis. The available models to assess piping are discussed in section 3.3. (Rijkswaterstaat, 2019) has designed a flow chart to decide which steps to take and what models to use. The flow chart is displayed in Figure 38 in Appendix A.

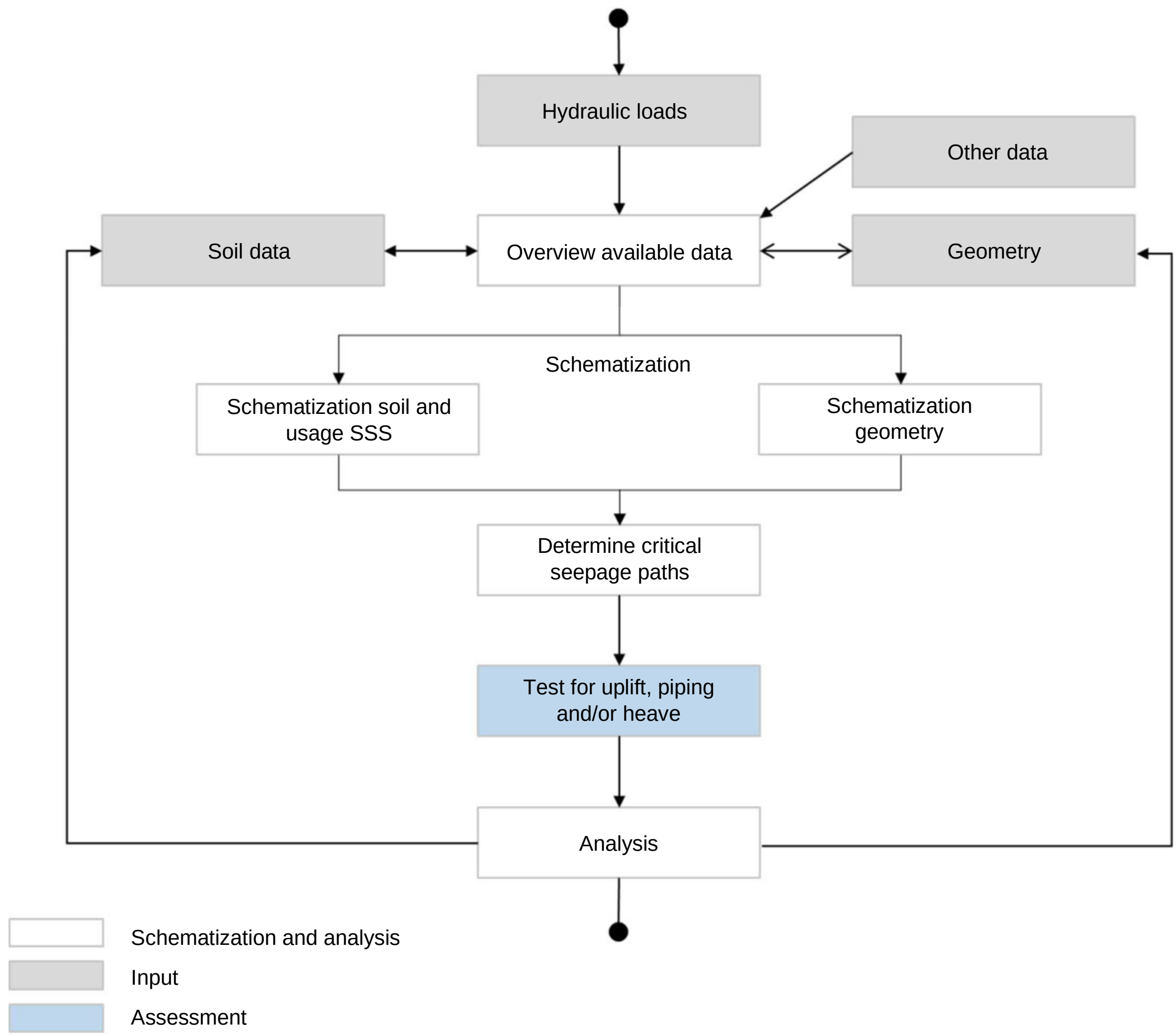

Figure 14 Schematic overview of steps to take for the assessment[5] of piping at hydraulic structures, SSS stands for Stochastic Schematization of the Soil (source: (Rijkswaterstaat, 2019), translation own courtesy)

Since piping at hydraulic structures is a group 4 failure mode (see section 2.4) and not a group 2 there is no (semi-)probabilistic analysis available from the WBI 2017 for the failure mechanism piping at a hydraulic structure. Consequently, the failure probability is not calculated hence only safety judgments[6] II_V and V_V are possible outcomes of the detailed test per dike reach for piping at a hydraulic structure.

3.2.3 Custom test

Another method to assess a hydraulic structure is by using the custom test. The custom test is only used when the detailed test per dike reach resulted in a score of V_V (Rijkswaterstaat, 2019). This means that the structure does not meet the safety norm. The custom test will further analyze the failure mechanism piping at hydraulic structures. The custom test is the assessment method with the highest level of detail and it is the most location specific test (see Figure 12).

The custom test can be performed by using several methods (Rijkswaterstaat, 2019):

- Installation of groundwater monitoring well;
- Advanced groundwater flow modelling;
- The principle of ‘proven strength’;
- Probabilistic analysis for heave or piping.

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[5] Note that the assessment (indicated in the blue box of Figure 14) can contain tests on all relevant sub-mechanisms of piping (see Figure 38 in Appendix A). However, if for example no aquitard is present at the hydraulic structure that needs to be assessed, then the assessment on uplift is not applicable.

[6] See section 2.2 for an explanation of safety judgments II_V and V_V.

Each of these four potential methods to give substance to the custom test are described in short in the following paragraphs. For a more extended explanation one could consult section 2.3 of the schematization guideline for piping at hydraulic structures (Rijkswaterstaat, 2019).

Groundwater monitoring well
Near hydraulic structures there can be large uncertainties regarding the hydraulic head. By using groundwater monitoring wells this uncertainty can be eliminated. Most of the time seepage paths at hydraulic structures have vertical components due to the presence of seepage screen under the structures. By installation of a monitoring well the hydraulic head at the location of the downstream located seepage screen can be measured. Data obtained from these measurements can be used as input for the heave model (Rijkswaterstaat, 2019). This model will be explained in section 3.3.1.

Advanced groundwater flow modelling
For heave and piping it can be of value to investigate the groundwater flow by using advanced flow models. Models such as non-stationary groundwater flow models and three-dimensional or quasi three-dimensional groundwater flow models can be used for this (Rijkswaterstaat, 2019). The output of these models is an overview of the hydraulic heads which can be used for the assessment on heave and/or piping.

The principle of 'proven strength'
It may be the case that the critical difference in hydraulic head over the hydraulic structure has already been reached in the past. When this has happened an assessment according to the principle of proven strength can be used. When applying this method on the failure mechanism piping at hydraulic structures expert knowledge is required (Rijkswaterstaat, 2019).

Probabilistic analysis for heave or piping[7]
With 'Ringtoets', the software that is used to assess failure mechanisms according to the WBI 2017, it is not possible to assess piping in a probabilistic manner. However, with the predecessor of this software, 'PC-Ring', it is possible to assess piping in a probabilistic manner. This is only directly possible for the models Lane and Bligh (these models are explained in respectively section 3.3.1 and section 3.3.2). Both current and previous software cannot be used to perform a probabilistic analysis when the heave model is applied (Rijkswaterstaat, 2019).

3.3 Overview of available piping models

As explained in section 1.1 of the introduction, flood defenses have many different potential failure mechanisms. Each of these mechanisms can be decomposed into multiple sub-events that in the end lead to one top-event. This top-event is the same for all failure mechanisms: failure of the flood defense.

This also holds for the failure mechanism piping at a hydraulic structure. After correct decomposition of the full failure mechanism each sub-event can be assessed with the appropriate models. The assessment of each sub-event can then be combined to find the assessment result for the top-event.

One method that can be used to provide an overview of the sub-events and the links to the top-event, is a fault tree. The fault tree shows the relation between the sub-events and how they lead to the top-event. In Figure 15 the fault tree for the failure mode piping at a hydraulic structure is shown.

[7] Note the difference between a general applicable probabilistic assessment method and the probabilistic analysis described here. For the failure mechanism piping at a hydraulic structure there is no probabilistic method developed for assessment according to the WBI 2017 (Rijkswaterstaat, 2019). The probabilistic analysis described here is one of multiple possible ways to give substance to the custom test. The goal of this thesis is to find and describe a general applicable method that can be used to calculate the probability of failure due to piping at a hydraulic structure.

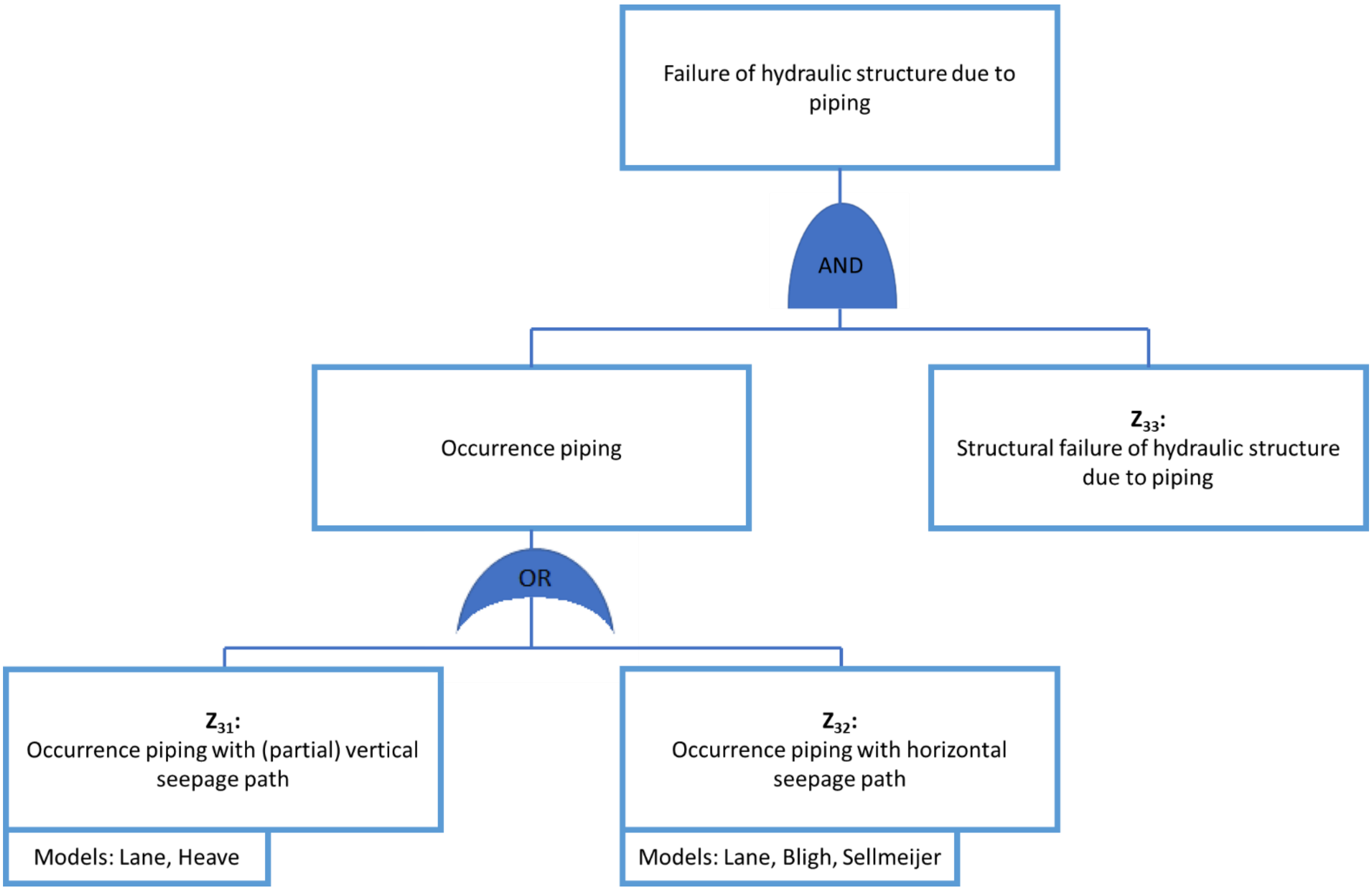

Figure 15 Fault tree describing piping at a hydraulic structure (source: (Rijkswaterstaat, 2017c), translation own courtesy)

From the top-event shown in the fault tree of Figure 15 the tree splits into two different sub-events: occurrence of piping and structural failure of the hydraulic structure due to piping. These sub-events are connected to the top-event by means of an 'AND' gate. This means that both sub-events need to happen before the top-event happens. The sub-event occurrence of piping then splits further into two sub-events. One of them describes piping with a (partial) vertical seepage path while the other is characterized by a horizontal seepage path. These two bottom events of the fault tree are connected to the sub-event occurrence of piping by an 'OR' gate. An 'OR' gate is used when only one of the (in this case) two sub-events needs to happen. It is important to note that piping can occur in two different ways:

- It can occur under the structure;
- It can occur next to the structure.

These two options are represented by the bottom events (Z_{31} & Z_{32}) of the fault tree presented in Figure 15. In the end the fault tree distinguishes three sub-events:

1. Sub-event Z_{31}: The occurrence of piping in a (partially) vertical manner under and/or next to the structure.
2. Sub-event Z_{32}: The occurrence of piping in a fully horizontal manner under and/or next to the structure.
3. Sub-event Z_{33}: The collapse of the structure due to the occurrence of piping under and/or next to the structure.

Each sub-event is marked by a Z and two numbers. This is done to distinguish the different limit state functions (or Z-functions). Section 3.3.1 explains the concept of a Z-function. The three different sub-events and their corresponding applicable models are discussed in the remaining part of section 3.3.

3.3.1 Sub-event Z_{31} piping in a (partial) vertical manner: Lane and Heave model

Each sub-event has its own limit state function. Such a limit state function describes when the sub-event occurs. The most general shape of a limit state function is given in formula [3] (Jonkman, et al., 2017):

$$Z = R - S \qquad [3]$$

Where:

Z	$[applicable\ unit]$	Value describing whether failure occurs or not
R	$[applicable\ unit]$	(Combined) value of the resistance variable(s)
S	$[applicable\ unit]$	(Combined) value of the load variable(s)

When $Z < 0$, it means that the load exceeds the resistance, hence the sub-event occurs (which in this case is the occurrence of piping in a (partially) vertical manner. It can also happen that $Z > 0$. Then, the resistance is larger than the load, hence the sub-event does not occur. The last option is $Z = 0$. In that case the resistance is equal to the load. This is the critical case since occurrence of the sub-event does almost take place in this situation.

For the sub-event Z_{31} the following models are available (Van Bree, 2015):

- Lane's model;
- Heave model.

Each model has its own variables that are used in the limit state function. For both models the limit state functions are described.

Lane's model

For Lane's model the limit state function can be described as equation [4] (Van Bree, 2015):

$$Z_{31} = L_L - L_{c,L} \qquad [4]$$

Where:

L_L	$[m]$	The present seepage length according to Lane
$L_{C,L}$	$[m]$	The critical seepage length according to Lane

In this limit state function the resistance is described by the present seepage length. This length can be described by formula [5] (Van Bree, 2015):

$$L_L = m_L \left(L_v + \frac{L_h}{3} \right) \qquad [5]$$

Where:

m_L	$[-]$	Model parameter of the present seepage length
L_v	$[m]$	Vertical seepage path
L_h	$[m]$	Horizontal seepage path in Lane's model

In Lane's model there is a difference between the resistance offered by a vertical and a horizontal seepage path. This difference is a factor 3, indicating that a vertical seepage path offers a resistance that is three times as large compared to the offered resistance by a horizontal seepage path (Lane, 1935).

Each variable shown in formula [5] can be described by a parametric distribution. Table 4 shows the various distribution types of the variables that describe the resistance in Lane's model. The table also shows the values of the various involved parameters of the distribution types.

Table 4 Distribution types of resistance variables of Lane's model (Van Bree, 2015)

Variable	Distribution type	Distribution parameters	
		μ	σ
m_L	Lognormal	2.2	0.27
L_v	Normal	Nom[8]	0.1 m
L_h	Normal	Nom	0.1 m

The load can be described by a set of variables and parameters too. This can be by making use of formula [6] (Van Bree, 2015):

$$L_{c,L} = C_{LANE} * m_c * (h - h_{bi}) \qquad [6]$$

Where:

C_{LANE}	$[-]$	Creep factor belonging to Lane's model
m_c	$[-]$	Model parameter for Bligh and Lane
h	$[m + NAP]$	Upstream water level
h_{bi}	$[m + NAP]$	Downstream water level

Formula [6] contains a so-called creep factor, C_{LANE}. This variable is a measure for the resistance of the soil particles against erosion by groundwater flow. The value for the resistance depends on the soil type. Table 5 shows the value for the creep factor for different types of soil. Also, the grain diameter belonging to the soil type is presented in Table 5.

Table 5 Creep factors belonging to Lane's model (Rijkswaterstaat, 2019)

Soil type	**Grain diameter** $[\mu m]$ conform NEN 5104 (version September 1989)	C_{LANE}
Extremely fine sand or silt	< 105	8.5
Very fine sand	105 – 150	
Very fine sand (mica)		7
Moderately fine sand (quartz)	150 – 210	7
Moderately coarse sand	210 – 300	6
Very/extremely coarse sand	300 – 2000	5
Fine gravel	2000 – 5600	4
Moderately coarse gravel	5600 – 16000	3.5
Very coarse gravel	> 16000	3

Besides the creep factor, the other variables that describe the load in Lane's model are given in Table 6. The model parameter m_c accounts for the uncertainty of Lane's model. The main driver of the load is the head difference over the hydraulic structure. This is expressed as the difference in upstream and downstream water level at the location of the hydraulic structure.

[8] Nom is the abbreviation of nominal value. For the seepage paths this is the obtained value if the geometry of the hydraulic structure and the seepage screens is used. However, in reality the length of the seepage path may differ from this nominal value. Therefore, it is assumed that the seepage paths are normally distributed with a standard deviation of 0.1 m (Van Bree, 2015).

Table 6 Distribution types of load variables of Lane's model (Van Bree, 2015)

Variable	Distribution type	Distribution parameters	
		μ	σ
C_{LANE}	Normal	Nom	0.1 * Nom
m_c	Normal	1	0.1
h	Normal[9]	Nom	0.1 m
h_{bi}	Normal	Nom	0.1 m

Heave model

For the Heave model the limit state function is described as formula [7] (Van Bree, 2015):

$$Z_{31} = i_{c,h} - i \qquad [7]$$

Where:

$i_{c,h}$	$[-]$	The critical heave gradient
i	$[m]$	The occurring gradient at the exit point

In this relation the resistance is characterized by the critical heave gradient. The critical heave gradient can be described by a lognormal distribution with a mean value (μ) of 0.7 and a standard deviation (σ) of 0.1 (Jonkman, et al., 2018; Van Bree, 2015).

A description for the load i is given by relation [8] (Van Bree, 2015):

$$i = \frac{(\varphi_{screen} - h_{exit})}{L_{screen}} \qquad [8]$$

Where:

φ_{screen}	$[m + NAP]$	Hydraulic head at the bottom side of the downstream seepage screen
h_{exit}	$[m]$	Phreatic level at exit point
L_{screen}	$[m]$	Length of seepage path, downstream of seepage screen

As can be deducted from formula [8] the heave gradient is described as the difference in hydraulic head at the bottom side of the downstream seepage screen and the phreatic level at the exit point of the groundwater flow over the length of the seepage path downstream of the seepage screen. The applicable parametric distributions of each of these variables are given in Table 7.

Table 7 Distribution types of load variables of the heave model (Van Bree, 2015)

Variable	Distribution type	Distribution parameters	
		μ	σ
φ_{screen}	Deterministic	-	-
h_{exit}	Normal	Nom	Nom * 0.1 m
L_{screen}	Normal	Nom	0.1 m

Application of Lane's model and heave model

Both Lane's model and the heave model can be applied in cases where assessment of piping needs to be done when the seepage path involves vertical segments. However, it is common practice to start the assessment with Lane's model to obtain a first rough result. Especially for the assessment of piping at

[9] Note that the upstream water level distribution is presented here as the arbitrary point in time (APT) distribution. However, in a probabilistic analysis it is important to know the extreme value (EV) distribution for this load variable. Section 5.3.1 elaborates on the applicable EV distribution of the case study of this thesis.

hydraulic structures that have vertical seepage screens Lane's model is advised to be used (Van Bree, 2015). If it turns out that the load and resistance variables are close to each other in value (Z is close to zero) then it is advised to continue the assessment with the Heave model (Van Bree, 2015).

An important difference between the two models is that Lane's model is used for the assessment of the full geotechnical part of the piping failure mechanism. This means that no separate assessment for heave is done when using Lane's model. But in the assessment result the sub-mechanism heave is included. When using the heave model the sub-mechanism heave is assessed separately from the full pipe development (piping) (see section 3.1.1 for the difference between the sub-mechanisms). For the pipe development part Sellmeijer's model is recommended to be applied (Van Bree, 2015). This model is explained in section 3.3.2.

3.3.2 Sub-event Z_{32} piping in a horizontal manner: Lane, Bligh and Sellmeijer model

In total three models can be applied for piping with a horizontal seepage path (Van Bree, 2015):

- Bligh's model;
- Lane's model;
- Sellmeijer's model.

Just as done in section 3.3.1 this section explains the models by their corresponding limit state functions. It starts with an explanation on Bligh's model after which an elaboration on Sellmeijer's model is given. For Lane's model, please refer to section 3.3.1.

Bligh's model

According to (Van Bree, 2015) Bligh's model can be described by formula [9]:

$$Z_{32} = L_B - L_{c,B} \qquad [9]$$

Where:

L_B	$[m]$	The present seepage length according to Bligh
$L_{C,B}$	$[m]$	The critical seepage length according to Bligh

This model also distinguishes a load and a resistance part. The part of Bligh's model that describes the resistance is the present seepage path length. The present seepage path length according to Bligh's model is given by formula [10]:

$$L_B = m_L * L_{h,B} \qquad [10]$$

Where:

$L_{h,B}$	$[m]$	Horizontal seepage path length in Bligh's model

Formula [10] is comparable to formula [5], however Bligh's model does not make a difference between the contribution of vertical and horizontal seepage paths to the offered resistance against piping. The model parameter of the present seepage path in Bligh's model is the same as the one used in Lane's model.

In Table 8 an overview is given of the parametric distribution types of the involved variables that describe the resistance against piping according to Bligh's model.

Table 8 Distribution types of resistance variables of Bligh's model (Van Bree, 2015)

Variable	Distribution type	Distribution parameters	
		μ	σ
m_L	Lognormal	2.2	0.27
$L_{h,B}$	Normal	Nom	0.1 m

A description of the load according to Bligh's model is given by formula [11]:

$$L_{c,B} = C_B * m_c * (h - h_{bi}) \qquad [11]$$

Where:

C_B	$[-]$	Creep factor belonging to Bligh's model

Again, the load part contains a creep factor. The values for the creep factor for Bligh's model do differ from the creep factor values according to Lane's model. Table 9 describes the applicable creep factors for different soil types belonging to Bligh's model. The grain diameters indicating the soil type are also given in Table 9.

Table 9 Creep factor belonging to Bligh's model

Soil type	**Grain diameter $[\mu m]$** conform NEN 5104 (version September 1989)	C_B
Extremely fine sand or silt	< 105	
Very fine sand	105 – 150	18
Very fine sand (mica)		18
Moderately fine sand (quartz)	150 – 210	15
Moderately coarse sand	210 – 300	
Very/extremely coarse sand	300 – 2000	12
Fine gravel	2000 – 5600	9
Moderately coarse gravel	5600 – 16000	
Very coarse gravel	> 16000	4

The other variables that describe the load in Bligh's model are described in Table 10. The model parameter m_c accounts for the uncertainty of Bligh's model. This is the same model parameter as is used in Lane's model (see formula [6]). Also, in Bligh's model the main driver of the load is the head difference over the hydraulic structure. This is expressed as the difference in upstream and downstream water level at the location of the hydraulic structure.

Table 10 Distribution types of load variables of Bligh's model (Van Bree, 2015)

Variable	**Distribution type**	**Distribution parameters**	
		μ	σ
C_B	Normal	Nom	0.1 * Nom
m_c	Normal	1	0.1
h	Normal	Nom	0.1 m
h_{bi}	Normal	Nom	0.1 m

Sellmeijer's model
Another model that can be used to describe the occurrence of piping in a horizontal manner is Sellmeijer's model. The limit state function of Sellmeijer's model is given by equation [12] (Van Bree, 2015):

$$Z_{32} = m_p H_c - H \qquad [12]$$

Where:

m_p	$[-]$	Model parameter for the piping model of Sellmeijer
H_c	$[m]$	Critical water level difference over the hydraulic structure
H	$[m]$	The present water level difference over the hydraulic structure

The difference between Sellmeijer's model and Lane's and Bligh's models is the expression of the limit state function in water level differences instead of seepage lengths.

The resistance part of the limit state function of Sellmeijer's model is given as the model parameter m_p multiplied by the critical water level difference H_c. This can also be expressed as the multiplication of a resistance factor, a scale factor, a geometry factor and the present seepage length. This is shown by equation [13] (Van Bree, 2015). The description of these three involved factors is given by equations [14], [15] and [16] (Van Bree, 2015):

$$m_p H_c = \left(F_{resistance} * F_{scale} * F_{geometry}\right) * L \qquad [13]$$

$$F_{resistance} = \eta \frac{\gamma_{sub.particles}}{\gamma_{water}} * \tan \theta_{sellmeijer.revised} \qquad [14]$$

$$F_{scale} = \frac{d_{70.m}}{\sqrt[3]{\kappa L}} \left(\frac{d_{70}}{d_{70.m}}\right)^{0.4} \text{ where } \kappa = \frac{\nu_{water}}{g} k \qquad [15]$$

$$F_{geometry} = 0.91 \left(\frac{D}{L}\right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8} - 1} + 0.04} \qquad [16]$$

Where:

$F_{resistance}$	$[-]$	Resistance factor
F_{scale}	$[-]$	Scale factor
$F_{geometry}$	$[-]$	Geometry factor
L	$[m]$	Present seepage length
η	$[-]$	Coefficient of White
$\gamma_{sub.particles}$	$[kN/m^3]$	Volumetric weight of submerged sand particles
γ_{water}	$[kN/m^3]$	Volumetric weight of water
θ	[°]	Bedding angle
$d_{70.m}$	$[m]$	Reference value for the 70%-fractile
d_{70}	$[m]$	70%-fractile of the grain size distribution
κ	$[m^2]$	Intrinsic permeability
ν_{water}	$[m^2/s]$	Kinematic viscosity of water
g	$[m/s^2]$	Gravitational constant
k	$[m/s]$	Specific conductivity
D	$[m]$	Thickness of aquifer layer

Parameters in Sellmeijer's model with a fixed deterministic value are listed in Table 11.

Table 11 Parameters in Sellmeijer's model with a fixed deterministic value

Variable	Deterministic value
η	0.25
$\gamma_{sub.particals}$	$16.5\ kN/m^3$
γ_{water}	$9.81\ kN/m^3$
θ	$37°$
ν_{water}	$1.33 * 10^{-6}\ m^2/s$
g	$9.81\ m/s^2$

As shown in formula [12] the load part of Sellmeijer's model is the occurring water level difference over the hydraulic structure. This can be written according to (Van Bree, 2015) as equation [17]:

$$H = h - h_{bi} - r_c d \qquad [17]$$

Where:

r_c	$[-]$	Reduction factor
d	$[m]$	Thickness of the blanket layer

Lane's model
Lane's model can be applied in the case of the occurrence of piping in a horizontal manner too. The description of Lane's model can be found in section 3.3.1.

Application of Bligh's model, Lane's model and Sellmeijer's model
According to (Förster, Van den Ham, Calle, & Kruse, 2012) either Lane's or Sellmeijer's model can be applied in the case of piping with horizontal seepage paths. However, this is based on research on piping at dikes and not at hydraulic structures. Due to the difference in groundwater flow also Bligh's model can be applied when assessing a hydraulic structure on piping (Van Bree, 2015).

Only when the seepage path is growing in one direction Sellmeijer's model can be applied. This almost never happens at hydraulic structures. Hydraulic structures almost always have seepage screens at the sides of the structure to prevent piping next to the structure. This makes a horizontal seepage path at a hydraulic structure quite different than one at a dike. Due to the change in direction Sellmeijer's model cannot be applied. Because of the lack of an improved model[10], Bligh's model is therefore still used to assess piping with a horizontal seepage path at hydraulic structures. Only when the seepage screens at the sides of the structure are not considered Sellmeijer's model can be applied to assess piping in a horizontal manner at hydraulic structures. Another application of Sellmeijer's model is when heave and backward erosion are modelled separately.

3.3.3 Sub-event Z_{33} structural failure given piping
When piping has occurred it does not always imply that the hydraulic structures involved fails. Therefore, a third sub-event is shown in the fault tree of Figure 15. This sub-event represents structural failure of the hydraulic structure given piping has occurred. It does therefore describe a conditional sub-mechanism. The limit state function given by formula [18] describes the conditional failure (Van Bree, 2015):

$$Z_{33} = \beta_{kw|piping} - u \qquad [18]$$

Where:

$\beta_{kw\|piping}$	$[-]$	Reliability index according to the conditional probability of failure of the hydraulic structure given piping has occurred
u	$[-]$	Standard normal distributed variable

The reliability index can be determined as the inverse of the normal distribution at the value of the conditional failure probability of the hydraulic structure given piping ($P_{f,kw|piping}$). The model described by equation [18] can only be applied when $P_{f,kw|piping}$ is known beforehand. This is practically never the case, since the conditional failure of a hydraulic structure given piping depends on the remaining strength of the structure. According to (Van Bree, 2015) the remaining strength of the structure is dependent on the following factors:

- Type of soil underneath the hydraulic structure;
- Type of foundation of the hydraulic structure;
- Presence of seepage screens;
- Geometry of the hydraulic structure.

Due to the unpredictability of how the structure behaves after piping has occurred, it is difficult to say in what manner the structure will fail. Hence, it is difficult to find a way to attach a conditional probability to this sub-mechanism. Accordingly, in practice the remaining strength of a hydraulic structure against piping is not considered in the assessment (Van Bree, 2015; Casteleijn, Delhez, Van Bree, & Jongejan, 2018). Consequently, the conditional probability of structural failure given piping is set to 1 ($P(Z_{33} < 0) = 1$) (Casteleijn, et al., 2018).

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[10] This does not state that Bligh's model is not safe. Seepage screens at the sides of hydraulic structures have been designed by using Bligh's model. Until today there are no failures known of hydraulic structures due to piping in horizontal direction (Van Bree, 2015).

4. Method development

In this chapter the principles of both an event tree and a fault tree are discussed. Both methods can be of value for performing a system analysis. The two method might seem comparable, however there are some key differences between the two methods. The chapter starts with an explanation of the purpose of event trees and fault trees in section 4.1. Then, a more detailed analysis of the characteristics of both trees is discussed in section 4.2. This section also highlights the difference in approach of both methods. Section 4.3 explains what method is chosen to apply in this thesis and why this method has been chosen. Section 4.4 concludes this chapter with the design of three different event tree variants that can be used to analyze the piping at hydraulic structures failure mode.

4.1 Purpose of event trees and fault trees: system analysis

Before diving into the theory of event trees and fault trees, this section discusses what the actual purpose of these modeling methods are. Both trees can be used for the analysis of a system. The term system here can be explained as an element that has an input and a certain output (Jonkman, et al., 2017). The element representing the system can often be divided into multiple components and sub-systems. The components can consist of physical processes. But it is also possible to include components that represent organizational elements that have a human nature. In this way the influence of human action can be included in the definition of the system, which is often important to account for (Stoelsness, Gudmestad, & Bea, 2001). By including the relations between the sub-elements the total system can be represented. The input and output of the sub-systems determine what the output of the system itself is. Figure 16 shows how a system, elements and sub-systems relate to each other.

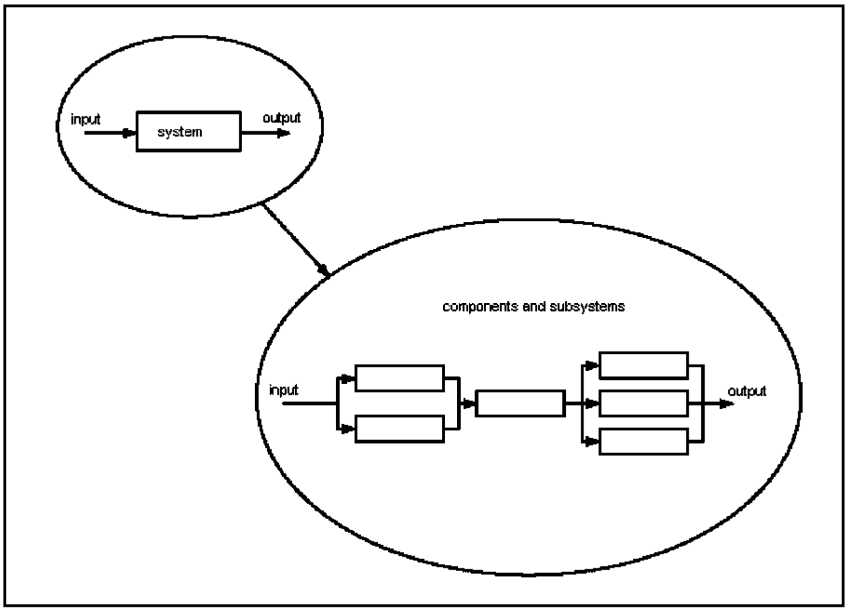

Figure 16 Definition of system, components and sub-systems (source: (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2017))

The purpose of a system analysis is to determine the probability of failure of the involved system. To do so, it is important to know the right relationships between the components that define the system. By finding the probabilities of occurrence of these components the probability of failure of the full system can be deducted.

In this thesis the system that is analyzed can be translated to the failure mechanism piping at hydraulic structures. The input of this system is a certain initiating event marking the start of piping. The elements that make up the system can be seen as sub-mechanism or sub-events. Whether these sub-events happen or not does determine the output of the full system: occurrence of failure of the hydraulic structure due to piping.

Methods that can be used to model a system include fault trees and event trees. Both trees are meant to systematically schematize a system by providing a pictorial representation of the system (Bedford & Cooke, 2001). Section 4.2 gives are more in-depth explanation of the characteristics of both methods.

4.2 Characteristics of fault trees and event trees

This section explains the logics behind fault trees (section 4.2.1) and event trees (section 4.2.2). It concludes with an analysis of the differences and similarities between the two trees (section 4.2.3).

4.2.1 Fault trees

Fault trees describe the sequence of events that lead to an unwanted or hazardous event. After definition of the event describing failure called the top-event an overview of the possible causes is given in the fault tree. The top-event can therefore be seen as an accumulation of intermediate events leading up to the hazardous event.

To find the combination of events that can lead to the top-event a fault tree makes use of Boolean operators (Bedford & Cooke, 2001). These operators include AND, OR and NOT. In the pictorial representation the two main operators a fault tree shows are AND-gates and OR-gates. AND-gates are gates where all components connected to that gate need to happen before the gate can be passed in the fault tree. Therefore, a sub-system that includes an AND-gate can be called a parallel system. OR-gates are used when only one of the components that is connected to that gate needs to occur in order to pass that gate in the fault tree. Sub-systems including an OR-gate are therefore called serial systems.

Figure 15 in section 3.3 shows an example of a fault tree that describes the failure mechanism piping at hydraulic structures. The fault tree has as top-event failure of the structure due to piping. This happens when piping occurs and structural failure due to piping happens. These two sub-events are therefore connected through an AND-gate with the top-event. The occurrence of piping can be by development of a seepage path with both vertical and horizontal elements or by development of a seepage path with solely horizontal elements. Just one of these two variants need to develop to let piping occur. Therefore, these two sub-events are connected through an OR-gate with the occurrence of piping sub-event.

Even though the fault tree has a Boolean logic it is only a visual model. To quantitatively analyze the probability of occurrence per component reliable data is needed. Since this might be rather specific data it can be the case that the needed data is not available. In those cases expert judgment can be conducted to overcome the problem of data shortage.

In conclusion, a fault tree uses backward logic to describe a combination of events that lead to the unwanted top-event (Bedford & Cooke, 2001). This means that from the top-event all possible causes are derived. An important assumption here is that these causes are binary (the cause either happens or does not happen) (Kurowicka, 2020).

4.2.2 Event trees

Where the fault tree starts with the definition of the top-event which described the system's failure, an event tree starts with the definition of an initiating event. Subsequently, an event tree describes the consequences that can follow from that initiating incident. It contains a pictorial overview of all possible consequences of the occurrence of the initiating event.

An event tree consists of multiple components. Figure 17 shows an example of an event tree to distinguish the different components. As mentioned above, an event tree starts with a described initiating event. This event can lead to a successive event (also called a sub-event). This is marked by a node in Figure 17. The possible outcomes of a sub-event are indicated by branches. It can be the case that the outcome is whether the sub-event does happen. In that case there are only two branches originating from a node. But it can also be the case that a node contains connections to multiple branches (Te Nijenhuis, et al., 2020). In this way different scenarios resulting from the occurrence of a sub-event can be indicated. The right-hand side of Figure 17 shows the leaves. These leaves represent the different potential outcomes.

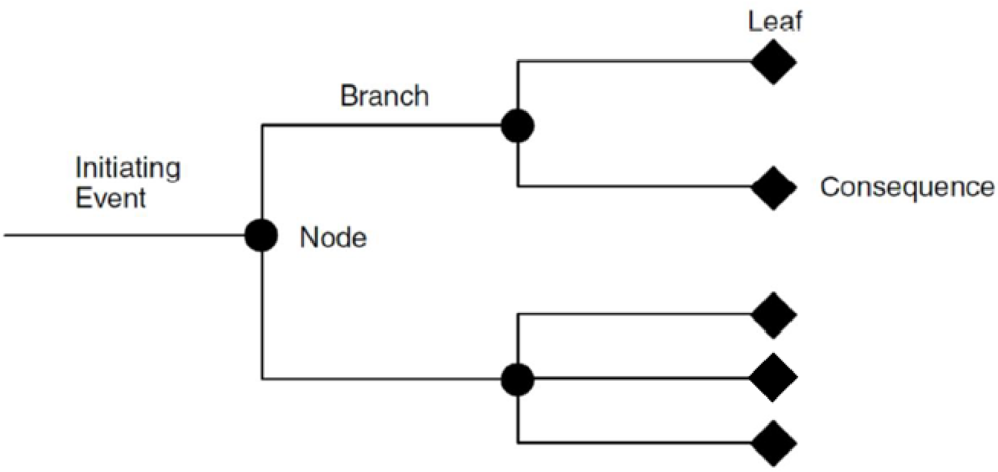

Figure 17 Definition of different components of an event tree (based on: (Beacher & Christian, 2003))

A combination of nodes and branches that leads to failure is called a failure path (Te Nijenhuis, et al., 2020). In this thesis the failure path would be that combination of sub-events that would lead to the loss of the water retaining function of a flood defense system. To calculate the probability of the occurrence of a certain pair of events, equation [19] can be used (Jonkman, et al., 2017):

$$P(A \cap B) = P(A) * P(B|A) \qquad [19]$$

Where:

$P(A \cap B)$	$[-]$	Probability of occurrence of event A and B
$P(A)$	$[-]$	Probability of occurrence of event A
$P(B\|A)$	$[-]$	Conditional probability of occurrence of event B given event A

In the case of independence between events A and B, the conditional probability of the occurrence of event B given event A can be simplified to $P(B|A) = P(B)$. Therefore, when independence holds, equation [19] can be rewritten into equation [20]:

$$P(A \cap B) = P(A) * P(B) \qquad [20]$$

In conclusion, an event tree uses forward logic (Bedford & Cooke, 2001); from the initiating event all possible consequences are derived. These consequences do not have to be binary as shown in Figure 17. Attention must be paid to the lay-out of the event tree. In the end it is meant to provide a clear overview of the different consequences.

4.2.3 Relation between fault trees and event trees: bow tie diagram

As discussed in sections 4.2.1 and 4.2.2 a fault tree describes all factors that can potentially cause a hazardous situation whilst an event tree describes how an initiating event can evolve into multiple consequences if not managed properly. The relation between a fault tree and an event tree can be shown by using a so-called bow tie diagram (Kurowicka, 2020). Figure 18 shows an example of a bow tie diagram.

The red dot in the middle of Figure 18 shows the hazardous event. On the left-hand side of the red dot the potential causes are stated. Using backward logic these causes can be derived. Hence, the left-hand side represents a fault tree. It expresses a sequence of faults and events that in the end lead to the top-event. The red dot is therefore the top-event for the left-hand side of the bow tie diagram. The right-hand side of the hazardous event represented by the red dot in Figure 18 shows the possible consequences that could originate from the hazardous event. Using forward logic these consequences can be derived. This means that the right-hand side of a bow tie model can be marked as an event tree. The red dot is therefore the initiating even for the right-hand side of a bow tie diagram.

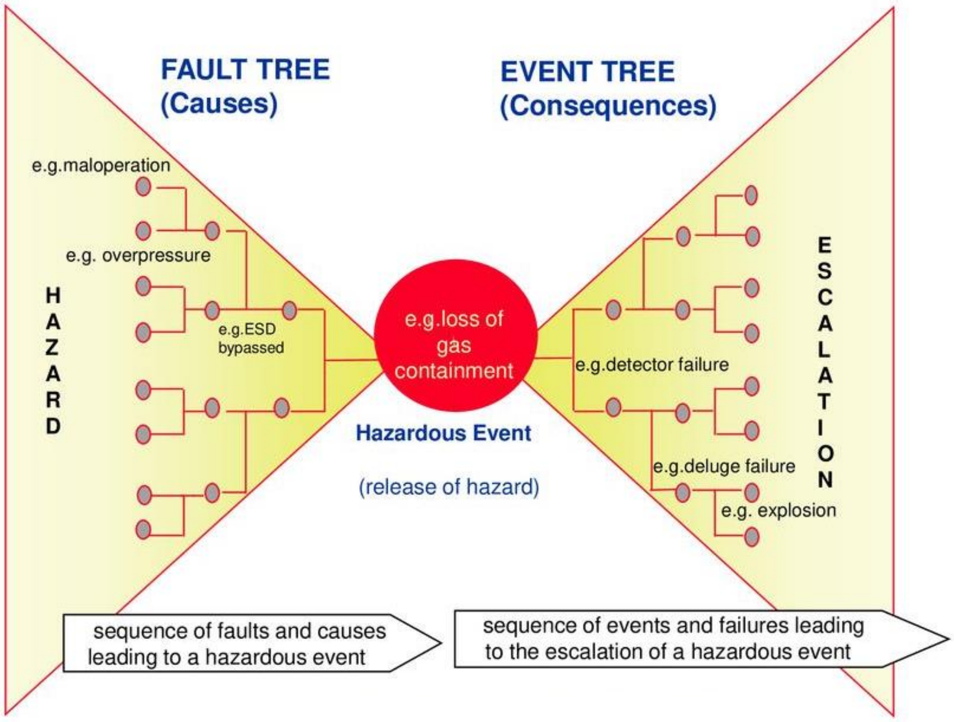

Figure 18 Example of a bow tie model showing the relation between a fault tree and an event tree (source: (Shell Exploration & Production, 2016))

4.2.4 Similarities and differences between fault trees and event trees

One of the similarities between event trees and fault trees is that both trees are methods that can be used to perform quantitative as well as qualitative system analyses. When conducting a quantitative analysis both methods can be used to calculate the probability of failure of the system under consideration. Both methods can consider what can go wrong in a system either expectedly and unexpectedly (Kurowicka, 2020).

An important difference between the two trees is that only a fault tree uses Boolean operators. This involves that sub-events that are part of the fault tree can either happen or not happen. An event tree does not have to follow Boolean logic. It can have multiple branches (in this case more than two) originating from a node indicating that multiple outcomes are possible. Therefore, an event tree can be used to define multiple scenarios leading to different consequences (Jonkman, et al., 2017). This cannot be done with a single fault tree; multiple fault trees are needed to model multiple consequences since a fault contains only one top-event.

Another difference between a fault tree and an event tree is the definition of the sub-events. In an event tree these sub-events (nodes) must be defined in such a way that the different consequences do exclude each other and that they together cover all possible outcomes (Te Nijenhuis, et al., 2020). In technical terms this is respectively called mutually exclusive and collectively exhaustive. This involves that the summation of the probabilities describing the occurrence of the possible outcomes must be equal to 1. The potential outcomes of the sub-events depicted by a fault tree do not have to add up to 1. This is because not all potential outcomes have to be considered in a fault tree approach (Te Nijenhuis, et al., 2020).

4.3 Chosen method

It is important to acknowledge that both the event tree method and the fault tree method can be used to quantify the probability of failure. The probability of failure in this thesis is defined as the probability that a hydraulic structure fails due to piping. However, a fault tree can only contain one top-event, meaning that only one consequence can be examined. The advantage of the event tree method is that it can describe multiple consequences originating from one initiating event. Furthermore, an event tree approach can help in getting acquainted with location specific aspects that might play an important role in the failure process (Te Nijenhuis, et al., 2020).

Given the variability in potential outcomes of the various sub-events that the failure mechanism piping at a hydraulic structure contain it is hard to use solely Boolean operators. Therefore, an event tree can help in obtaining an overview of all possible outcomes of the different sub-events. One important sidenote is that it is important to realize that the event tree can get too extensive. One should therefore be aware that the event tree covers all relevant outcomes but is still quantifiable. Hence data availability is also important.

The fact that a single event tree can cover multiple consequences, hence providing a full overview of the failure mechanism dealt with in this thesis is considered to be valuable. According to (Te Nijenhuis, et al., 2020) when determining the probability of flooding, inclusion of all relevant scenarios can sometimes lead to a reduction of the probability of flooding with a factor 1,000. Furthermore, the event tree method supports to get more insight in the sequence of sub-events that need to happen before failure happens. The method also helps to visualize the system accurately and stimulates communication about the covered system.

This makes the event tree method the most attractive to use in this thesis. Therefore, an event tree will be used to visualize the involved sub-events in the piping failure mechanism. This chapter ends (section 4.4) with the construction and discussion of multiple event tree variants that can be used to describe piping at hydraulic structures.

4.4 Event tree design

As explained in section 4.3 the chosen method to analyze the probability of failure due to piping for hydraulic structures is by means of an event tree. According to (Kanning, et al., 2019) it is important to first have an overview of how the system under consideration can fail. This is also called the narrative. When this is clear, the narrative can be divided into multiple sub-mechanisms (these are represented by nodes in the event tree) that lead to the undesired event: failure due to piping. The undesired event happens when a specific combination of sub-events occurs. Whether this combination takes place depends on the outcome of each sub-mechanism. Most of the time the outcome is that the sub-event either did occur or did not occur[11]. It is therefore important to analyze the different nodes that make up the event tree. In the end the probability of failure can be determined by multiplying the probabilities of occurrence of the different sub-events along the failure path under consideration (Te Nijenhuis, et al., 2020).

In this thesis the method proposed by (Kanning, et al., 2019) to set up an event tree will be followed. In this section the first three steps are elaborated:

1. Description of potential ways of failure (the narrative);
2. Get the correct sequence of events that result in failure (the failure path);
3. Elaboration of the sub-events including their potential outcomes (the nodes of the event tree).

The quantitative analysis which contains the elaboration of the probabilities of the sub-events and in the end the probability of failure (occurrence of the undesired event) is done in chapter 6.

4.4.1 The narrative

Since this thesis focuses on the failure mode piping at hydraulic structures, the undesired event in this case is failure due to piping. As explained in section 4.2 the event tree uses forward logic. Therefore, it is most convenient to start the narrative with the initiating event. After explanation of the initiating event, the sub-events that contribute to the failure mode piping at hydraulic structures are introduced. This section concludes with the undesired event. The narrative is based on a combination of Figure 11 and Figure 19[12].

[11] In event tree logics this is indicated by two or more branches originating from a node. If only two branches are displayed, one of these branches represents occurrence of the sub-event and contains the text 'YES' while the other branch shows 'NO', indicating the sub-event did not happen.

[12] Please note that Figure 19 displays the failure mode piping at a dike. The same sub-events apply to piping at a hydraulic structure, however the structural collapse (sub-event 'f') is dependent on the material properties and remaining strength that apply to the hydraulic structure rather than the ones that apply to a dike.

Initiating event
Before any soil erosion due to piping can happen, there must be a load acting on the soil particles. This load is an excessive pressure that can be expressed as a head difference. When the critical head difference is exceeded, the next sub-event of the piping failure mechanism will be initiated. This next sub-event may be either uplift or heave, depending on the soil type below the hydraulic structure (see the paragraph 'Uplift, seepage and heave' for an explanation). Therefore, it can be concluded that piping always starts with the exceedance of a critical head difference that is exceeded. The event tree starts with a node that represents exceedance of the critical head difference. From there the tree splits into two branches, one that corresponds to the exceedance of the critical head difference (the branch containing the text 'YES') and the other displaying the text 'NO' indicating exceedance does not occur. The latter branch directly results in a green leaf, meaning failure due to the failure mechanism piping at the hydraulic structure does not occur.

Uplift, seepage and heave
When analyzing the failure mode piping, it is important to know the characteristics of the soil. This determines if and how piping could start. First of all, it must be clear if a confining layer (aquitard), such as a clay layer is present. If this is the case, piping starts with the sub-event uplift. This is indicated in Figure 19 as sub-event 'a'. Uplift occurs when the pressure of the groundwater on the inner side of the flood defense is larger than the weight of the clay layer (Jonkman, et al., 2018). Most hydraulic structures have a shallow foundation which means that they are directly built on a sand layer. For this reason the sub-event uplift is less likely to play a role at piping at hydraulic structures compared to piping at a dike. Therefore, it is assumed that uplift is not a relevant sub-event for piping at hydraulic structures in general.

This also applies for seepage, sub-event 'b' in Figure 19. Seepage is the consecutive event of uplift, where groundwater is leaving the soil at the location where uplift took place.

This makes the sub-mechanism heave (see section 3.1.1 for an explanation of heave), shown as sub-event 'c' in Figure 19, the second occurring sub-event that in the end can lead to a failure of the hydraulic structure due to piping. From there the tree splits into two branches, one that corresponds to the occurrence of heave (the branch containing the text 'YES') and the other displaying the text 'NO' indicating heave does not occur.

Mechanical condition
Over time, when the critical gradient keeps being reached or exceeded by the groundwater flow, the erosion will continue. However, a condition for this progressive erosion process is that a stable roof must be present to prevent the pipe from collapsing (U.S. Department of the Interior Bureau of Reclamation & U.S. Army Corps of Engineers, 2019; Jonkman, et al., 2018). A collapse can either be prevented by the presence of a layer of cohesive soil above the aquifer (a clay layer) or by the bottom of a hydraulic structure. Of course, the structure must be intact to provide a stable roof. This stable roof condition is also called the mechanical condition.

Hydraulic condition
The progressive erosion only continues when the hydraulic condition is met. When the critical gradient of the flow (for which progression of the pipe takes place) is equal to or exceeded by the occurring gradient of the groundwater flow, erosion continues. This process is shown as sub-event 'd' in Figure 19. In this sub-mechanism the time dependency becomes relevant (Förster, et al., 2012). The critical gradient could for example be exceeded for a given time. However, when full pipe development has not occurred yet and the upstream water level dropped again the hydraulic condition is not met anymore hence erosion stops.

Piping
When both the mechanical and hydraulic condition are met for a time long enough, erosion continues until the pipe reaches the upstream side of the hydraulic structure. From that moment, the water on the upstream boundary is connected to the water on the downstream side. The connection that the pipe makes between the two water bodies marks the stage where piping is officially occurring (Van Bree, 2015). This is shown in Figure 19 as sub-event 'e'. Section 3.1.1 gives a more in-depth explanation about this sub-mechanism.

Widening of the pipe
After the connection between the upstream and downstream water bodies is made (as described under 'piping') the final resistance to the flow underneath the structure by soil particles is gone. A pressure wave can now travel from the upstream side to the downstream side ('t Hart, 2018). This causes an acceleration of the erosion of the soil, thereby causing the pipe to grow in lateral direction.

Detection and intervention
When the pipe is fully developed and widening of the pipe has started more and more sand starts to leave the soil. At the downstream side of the hydraulic structure a sand boil is now forming. This is a clear sign that the failure mechanism piping is at a far stage. Such a sand boil can be detected by a manager of the hydraulic structure. When detected an emergency measure can be applied (examples of emergency measures are presented in section 3.1.2). This is the final opportunity to prevent structural failure of the hydraulic structure. The detection and intervention involve a sub-event that has a human nature. As mentioned in section 4.1 it is important to include human and organizational factors besides physical processes in the definition of the system that is represented by the event tree to have a realistic representation of the full failure mode.

Structural failure
The structural failure of the hydraulic structure is characterized as the sub-event where (parts of) the structure collapse(s) or where the structure tilts or shears. However, it is difficult to exactly predict how the structure will fail. This is partially due to the lack of experiments into the structural failure behavior caused by piping (Van Bree, 2015). Also, the transient nature of the groundwater flow through the pipe makes it hard to predict how the structure will eventually fail. It can for example also fail in a consecutive manner where different parts of the structure fail successively.

Next to direct structural failure of the structure, it can also happen that the dike connected to the structure fails and the hydraulic structure itself remains intact. This scenario is also covered by structural failure.

An important characteristic that involves the determination whether structural failure will happen is the remaining strength of the structure. The remaining strength is influenced by several variables. According to ('t Hart, 2018) the most important variables that are involved are:

- The type of soil underneath the structure;
- The type of foundation of the structure;
- The presence of seepage screens;
- The geometry of the hydraulic structure.

An important remark for most hydraulic structures is that they can consists separate sections (this is for example common practice for sluices). The type of structural failure where parts of the structure fail consecutively is therefore plausible.

Breach formation
As was already named under structural failure, when detection and intervention did not succeed, the hydraulic structure may fail structurally in different manners. How the structure fails depends on how the erosion progresses in both time and space. It was concluded that a structural failure where parts of the structure fail consecutively could be plausible. However, also other types of failure where tilting or shearing of the structure occurs could happen.

All these different types of failures have different consequences. They can either result in a breach of the dike trajectory or not. For example, according to the definition of structural failure the hydraulic structure has failed when it tilts as a consequence of piping. However, it can tilt in such a way that it still retains the water at the upstream boundary. When this happens, there is no breach in the dike trajectory, however the structure still has failed in a structural way (it does not fulfill the stability requirement anymore).

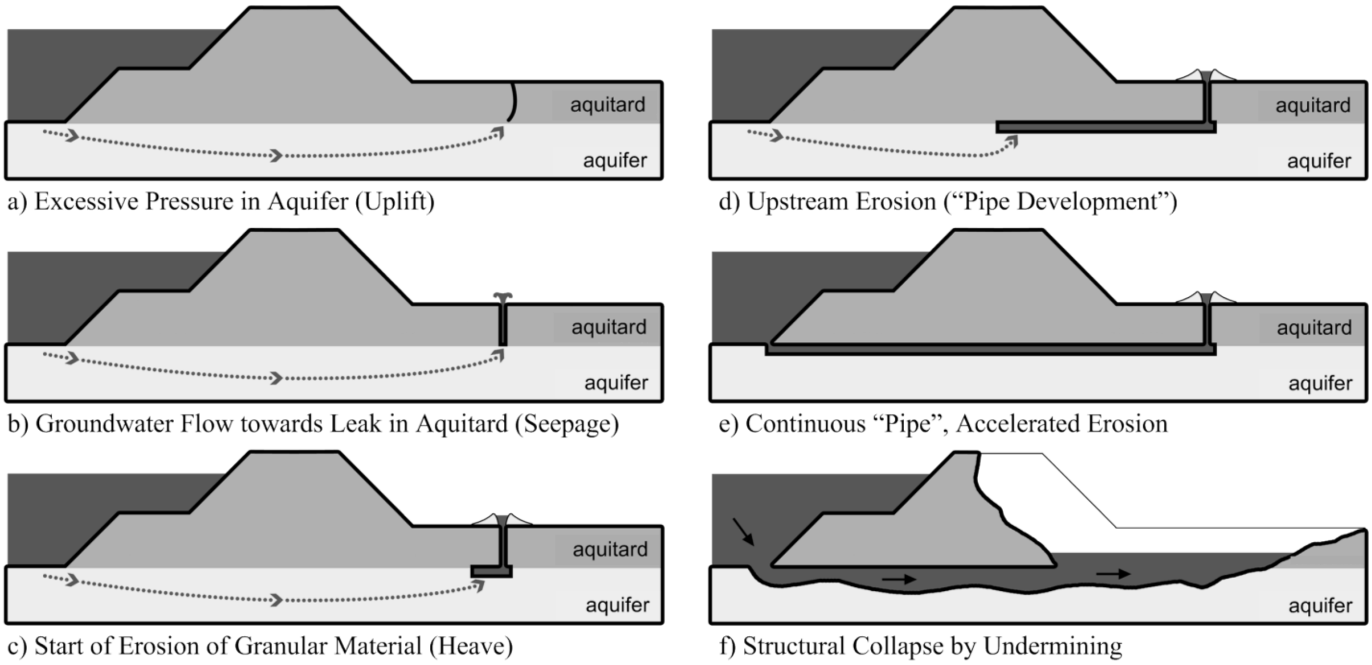

Figure 19 The piping process divided into sub-events (source: (Schweckendiek, Vrouwenvelder, & Calle, 2014))

4.4.2 The failure path

In section 4.4.1 all sub-events that are needed to occur to let the sluice fail due to piping are explained. The sequence in which these sub-events need to occur corresponds to the order of how the narrative is written. The sequence is partially based on (U.S. Department of the Interior Bureau of Reclamation & U.S. Army Corps of Engineers, 2019; Te Nijenhuis, et al., 2020) and is as follows:

1. Initiating event: Exceedance of critical head difference
2. Heave
3. Fulfillment of the mechanical condition
4. Fulfillment of the hydraulic condition
5. Piping
6. Widening of the pipe
7. Detection and intervention
8. Structural failure
9. Breach formation

In total the narrative contains nine sub-events that need to occur before the structure fails. These nine sub-events can be categorized into three main phases and a precondition to make the failure process more distinct. The precondition is the exceedance of the critical head difference. If this does not happen, piping will not occur. The following main phases comprise the defined sub-events to describe piping:

A) **Geotechnical part**: This is the first main phase. It contains all sub-events that range from the initiating event to the erosion of the soil after complete formation of the pipe.

B) **Emergency response**: This comprises the second main phase. The emergency response phase is about the detection of the piping failure mechanism and the application of any emergency measures (the intervention).

C) **Remaining strength of structure**: This covers the third and final main phase. When the emergency response is not successful, the structure will fail in the end. The type of failure determines whether a breach in the dike trajectory is formed or not. These two sub-events can be categorized as the remaining strength of the structure.

An overview of the sub-events corresponding to each main phase is shown in Table 12. The order in which the sub-events are presented in Table 12 is also the chronological order in which the sub-events take place in reality. Each phase has its own color to clearly show which sub-events belong to that phase.

Table 12 The sub-events involving piping at a hydraulic structure categorized into three main phases: A) Geotechnical part, B) Emergency response and C) Remaining strength of structure

Sub-event	**Precondition**
Critical head difference exceeded	Precondition
Sub-event	**Main phase**
Heave	A) Geotechnical part
Mechanical condition	
Hydraulic condition	
Piping	
Widening of the pipe	
Detection and intervention	B) Emergency response
Structural failure	C) Remaining strength of structure
Breach	

4.4.3 Event tree and its corresponding nodes

In this section the event trees for piping at a hydraulic structure are presented. In total three variants are designed. Each variant is elucidated below.

Variant 1: Qualitative event tree for piping at hydraulic structures

Each sub-event of the failure mechanism is represented by a node in the event tree. The failure path as discussed in section 4.4.2 contains all important sub-events and hence all nodes of the event tree. Figure 20 shows the event tree for the failure mechanism piping at a hydraulic structure. All sub-events corresponding to the ones of Table 12 are presented in the bar above the tree in Figure 20. Each sub-event has a node indicated by the dashed lines in the event tree. As can be observed, at each node a bifurcation is drawn. This bifurcation clarifies the possible outcomes of the sub-events. There are two options:

- YES: meaning the sub-event does occur;
- NO: meaning the sub-event does not occur.

In total there are nine possible outcomes for the event tree shown in Figure 20. Eight of these possible outcomes are marked as a green leaf. They all indicate that the dike trajectory does not have a breach at the location of the hydraulic structure. In other words, no breach formation takes place when ending at a green leaf of the event tree. Only one leaf (the upper one) is colored red. This one corresponds to the formation of a breach in the dike trajectory at the location of the hydraulic structure.

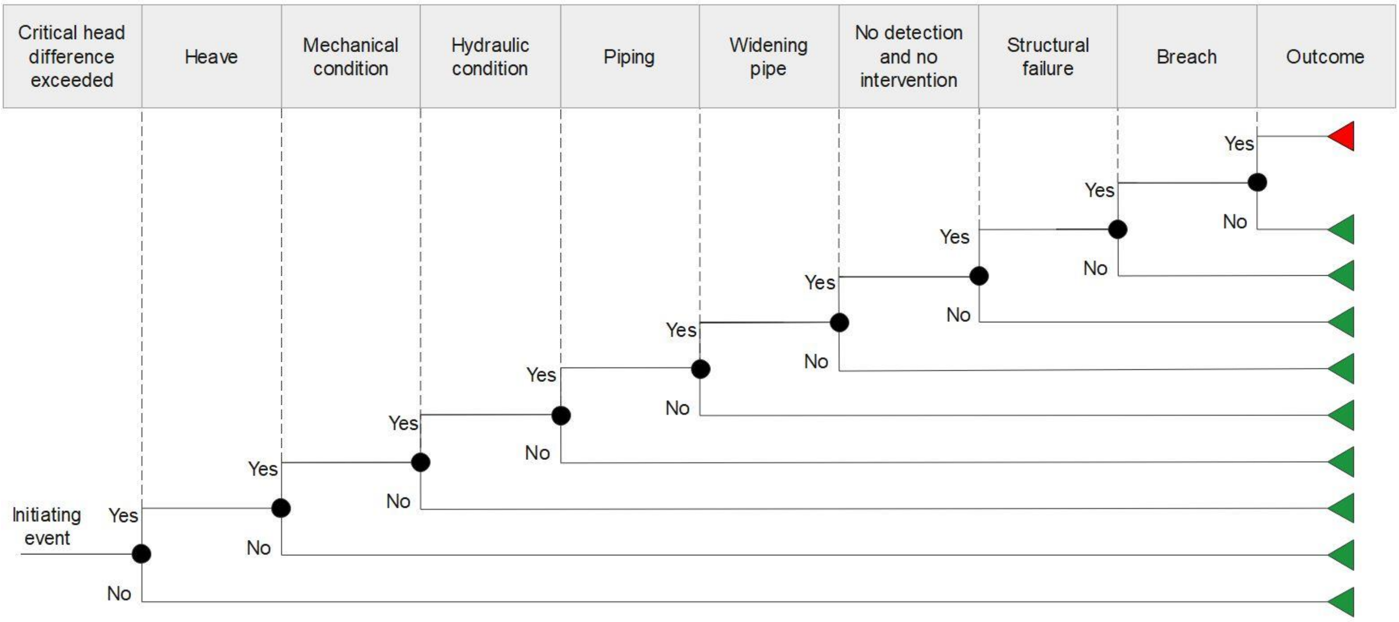

Figure 20 Event tree of the piping failure mechanism at a hydraulic structure

According to (Te Nijenhuis, et al., 2020) the failure path is that sequence of events that results in an unwanted event (in this case a breach in the dike trajectory at the location of the hydraulic structure). As can be concluded from section 4.4.2 the failure path is the combination of all upper branches. This is made clear in Figure 21 as the combination of red branches ending at the red leaf.

Note that in addition to the event tree shown in Figure 20, the event tree drawn in Figure 21 also contains the three main phases and the precondition of the piping failure mechanism. To make a clear distinction between the three phases and the precondition the same colors have been used as the ones separating the phases in Table 12.

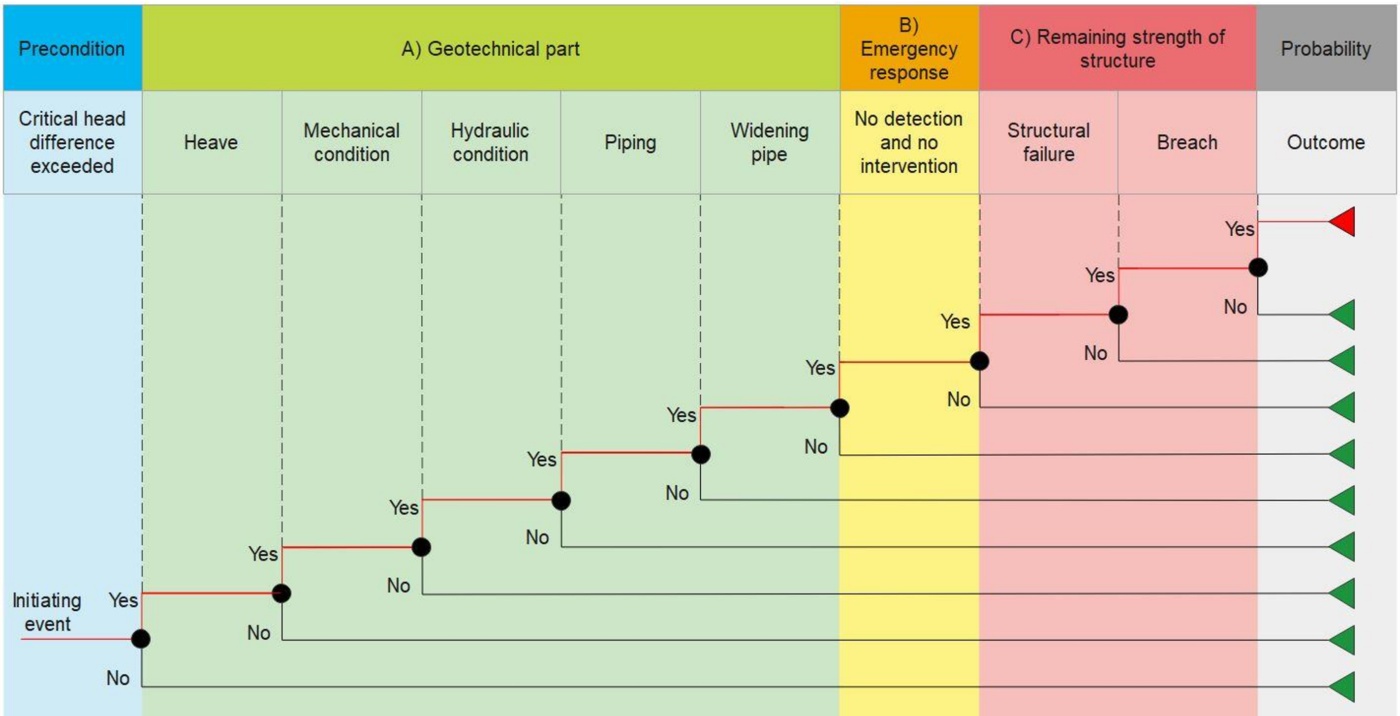

Figure 21 Event tree variant 1: extensive event tree of the piping failure mechanism at a hydraulic structure including the three main phases, precondition and the failure path which is indicated by the combination of red branches

Level of detail vs. accuracy of event tree
According to (Te Nijenhuis, et al., 2020) it is more practical to combine multiple nodes into one node when a model is available that covers the processes of the combined nodes. This brings down the complexity of the model and the involved calculations while still providing an accurate representation of the piping failure mechanism. Furthermore, in general it is often hard to obtain the distribution functions of the involved variables for the models that represent the nodes in an event tree approach (Ferdous, Khan, Sadiq, Amyotte, & Veitch, 2011).

One can conclude that there is a trade-off in the level of detail and the obtainable accuracy of the result of an event tree approach. For example, the event tree shown in Figure 21 contains nine nodes, which is quite extensive. It is hard to obtain data for each node. For some nodes there are even no models and/or data available. This will be treated in more detail in chapter 6.

Variant 2: Quantitative event tree for piping at hydraulic structures, based on the Heave model and Sellmeijer's model
Due to the trade-off between level of detail and accuracy, event tree variant 1 of Figure 21 is adjusted to a more compact event tree. This second event tree variant is shown in Figure 22. Here the first phase comprises two nodes. A distinction has been made in the two sub-mechanisms heave and backward erosion. This is done because there are two models available that describe these sub-events. Heave can be described by the Heave model (see section 3.3.1) while backward erosion can be described by Sellmeijer's model (see section 3.3.2) (Van Bree, 2015).

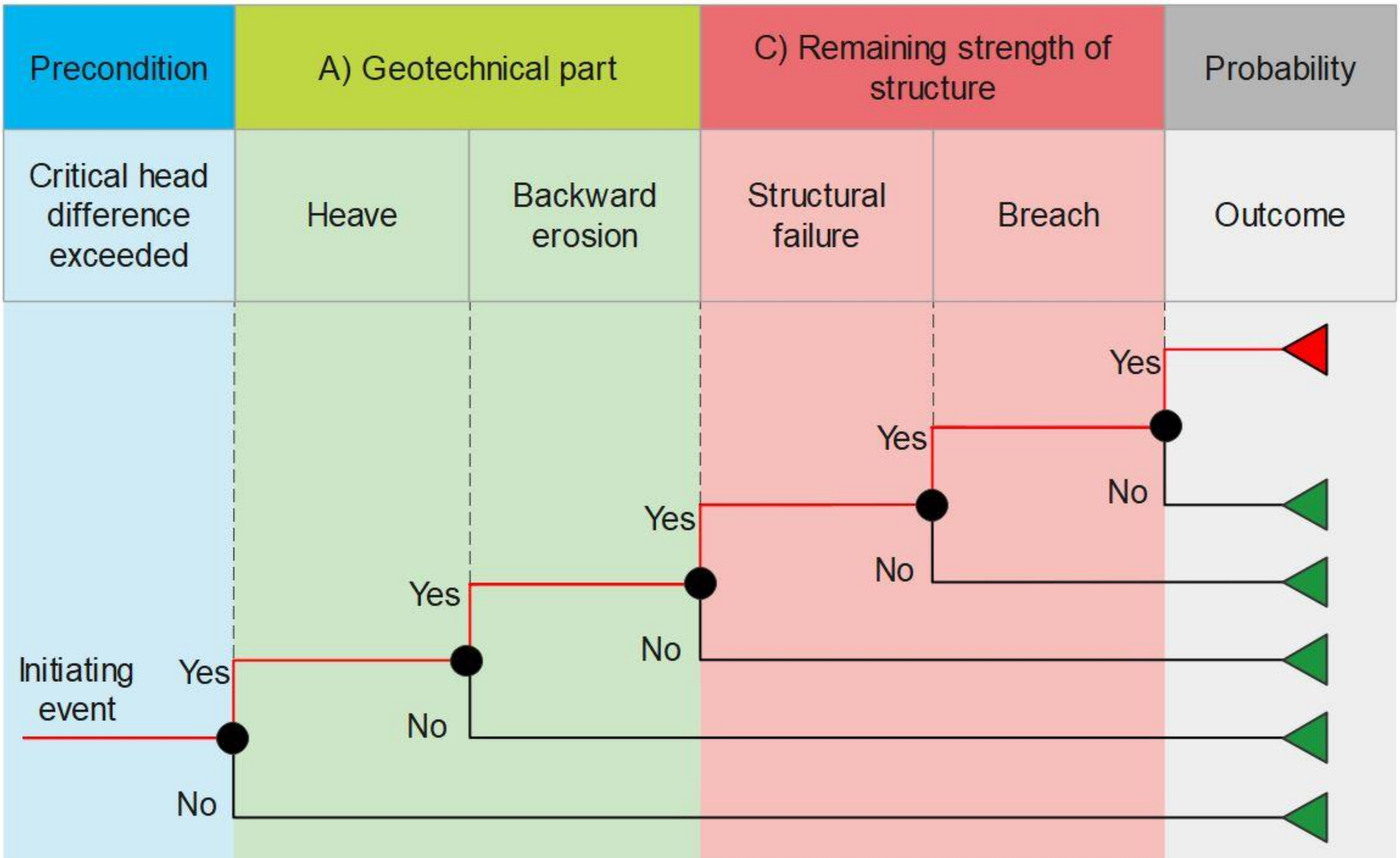

Figure 22 Event tree variant 2: concise event tree of the piping failure mechanism at a hydraulic structure. This event tree makes use of the Heave model and Sellmeijer's model for phase A

Variant 3: Quantitative event tree for piping at a hydraulic structure, based on Lane's model
As described in section 3.3.1 a first test on heave can also be carried out by Lane's model. As stated in section 3.3.1 Lane's model can be applied to assess the full geotechnical part of the piping failure mechanism. Hence, a third event tree is created that combines the sub-mechanisms heave and backward erosion into one sub-event, piping. This third variant is shown in Figure 23.

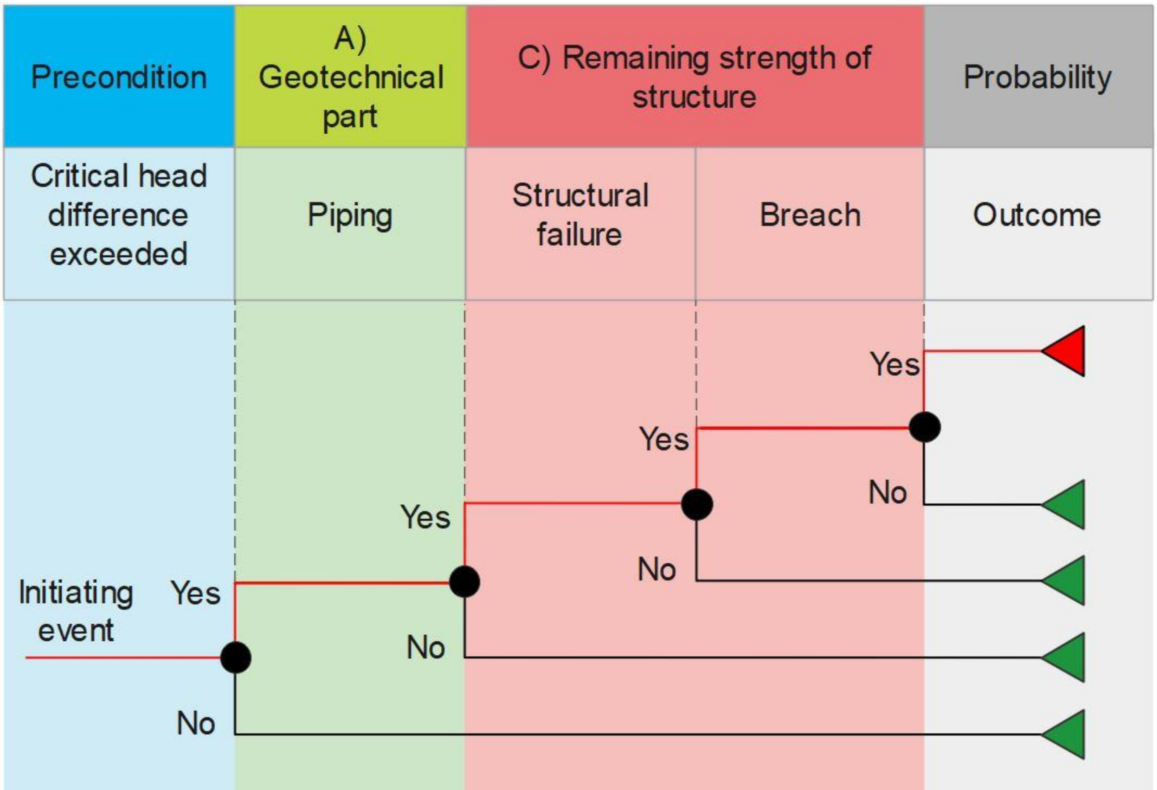

Figure 23 Event tree variant 3: concise event tree of the piping failure mechanism at a hydraulic structure. This event tree makes use of Lane's model for phase A

Since it is recommended to carry out a first test on piping by making use of Lane's model (Van Bree, 2015) it is practical to start modelling the piping failure mechanism at a hydraulic structure with event tree variant 3. The probability of piping can then be calculated by a probabilistic analysis with Lane's model. If this obtained probability is far below the warning limit value $\left(P_{piping} < \left(\frac{1}{30} * P_{f,req,wlv}\right)\right)$, then the structure can be considered safe. However, if the probability of piping is close to the warning limit value it is recommended to switch to event tree variant 2. With this event tree a more detailed calculation is carried out regarding the piping process. The only downside is that more data is needed to calculate both the probabilities on the occurrence of heave and backward erosion.

Table 13 contains a concluding overview of the characteristics of both models on event tree variant 2 and 3. A minus indicates a negative aspect of the model while a plus indicates a positive aspect of the model.

Table 13 Characteristics of event tree variant 2 and 3, a minus indicates a negative aspect of the model while a plus indicates a positive aspect of the model

	Event tree variant	
Model characteristics	Variant 2	Variant 3
Amount of data needed	-	+
Accuracy	+	-

When a hydraulic structure is assessed by using the event tree method, it is most practical to start with event tree variant 3. As shown in Table 13 variant 3 scores positive on the characteristic of amount of data needed when compared to event tree variant 2. Event tree variant 3 needs less data as input since only data on the seepage paths (hence data on the geometry of the hydraulic structure), soil characteristics and head difference over the structure are needed. As described above, when the probability of occurrence of piping is close to the warning limit value it is advised to continue the assessment with event tree variant 2. This event tree is more accurate but also needs more data. The output of the calculation with event tree variant 2 can then be compared to the warning limit value and the lower limit value corresponding to the norm of the dike trajectory in which the hydraulic structure is located.

A flow chart of the assessment procedure described in this section is shown in Figure 24.

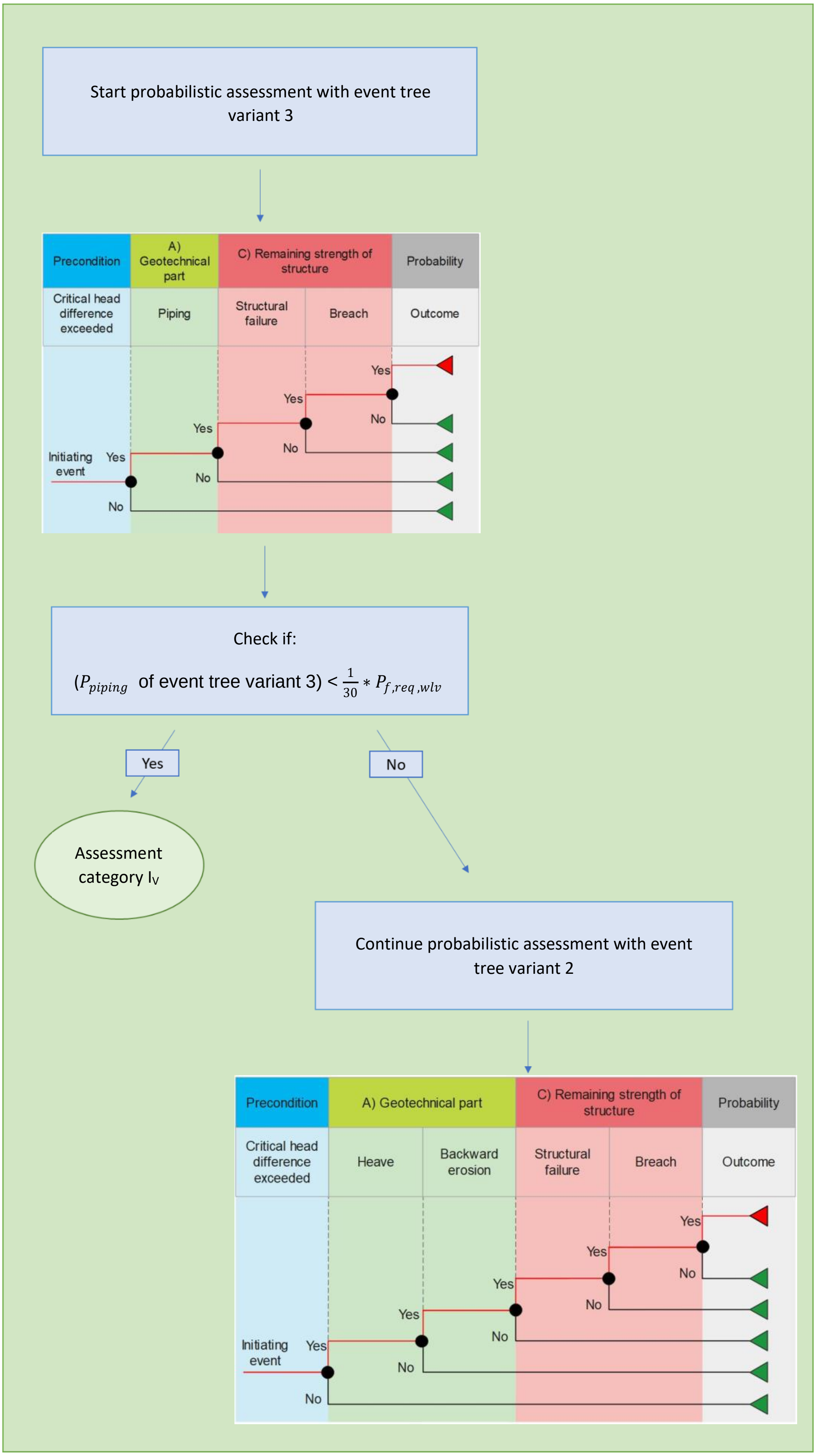

Figure 24 Flow chart of probabilistic assessment procedure

4.5 Methods of quantification

By using the flow chart shown in Figure 24 the probability of failure due to piping can be quantified. Therefore, the probabilities of occurrence of the different involved sub-events have to be calculated. For quantification of the probabilities of the different sub-events two main methods are used in this thesis. these two methods are:

- Probabilistic methods;
- Combination of expert knowledge and literature.

4.5.1 Probabilistic methods

The first method is the use of probabilistic methods. In this thesis two different probabilistic methods are used; First Order Reliability Method (FORM) analysis and Monte Carlo analysis. The main difference between these two methods is that a FORM analysis has a relatively small computation time, whereas a Monte Carlo analysis needs a larger computation time. On the other hand, a Monte Carlo analysis leads to more accurate results since no approximations are involved in the calculation.

The FORM analysis is based on a linearization of the limit state function in the design point. The design point is known as point with the highest probability density, so the point that marks the highest likelihood of failure. To find the design point iterations are performed in a FORM analysis. Next to the failure probability, the FORM analysis also provides so-called importance factors (α-factors) as outcome of the calculation. These factors indicate to what extent (in terms of percentages) each variable contributes to the uncertainty of the calculation result.

A Crude Monte Carlo analysis is a full probabilistic analysis where the probability of failure is calculated by means of simulations. For each involved stochastic variable of the limit state function a value is drawn from a uniform distribution (so a value between 0 and 1). Then, this value of the uniform distribution is used to calculate the value that will be used for the simulation. This is done by calculating the inverse of the cumulative distribution function of the considered variable with the drawn random value of the uniform distribution as argument. This is also done for each involved variable. The obtained values for the different stochastic variables are then used as input for the limit state function. If the outcome is larger than zero, no failure occurred. If the outcome is smaller or equal to zero a failure is simulated. This process is repeated for a given number of simulations, which is 100,000 times in this thesis. The probability of failure is calculated by equation [21]:

$$\widehat{P_f} = \frac{N_f}{N} \qquad [21]$$

Where:

$\widehat{P_f}$	[-]	Estimator of probability of failure
N_f	[-]	Number of simulations that result in failure (Z≤0)
N	[-]	Number of simulations

It can be the case that a Crude Monte Carlo analysis does not result in a failure probability due to too little simulations that result in failure. This could be solved by application of a Monte Carlo importance sampling method. Such a method focusses on obtaining more simulations that result in failure (N_f in equation [21]) by making use of a sampling function. This function has its maximum located in the failure domain (Z≤0). After performing the simulations the ratio between the sampling function and the original function is used to calculate the failure probability (see equation [22]) (Jonkman, et al., 2017).

$$\widehat{P_f} = \frac{N_f}{N} * \frac{f_X(x)}{f_S(x)} \qquad [22]$$

Where:

$f_X(x)$	$[-]$	Original probability density function of variable X
$f_S(x)$	$[-]$	Sampling function

FORM, Crude Monte Carlo and Monte Carlo with importance sampling analyses will be applied when models and hence limit state functions are available. This holds for the sub-events involved in main phase A) geotechnical part of the event trees.

4.5.2 Combination of expert knowledge and literature

It is not always the case that a probabilistic method can be used to quantify a probability. Especially when no limit state functions or models are available, a different method of quantification needs to be applied. For main phase C) remaining strength of the structure no models are available. As mentioned in section 3.3.3, in practice this problem is overcome by not taking into account the remaining strength of the structure. This could lead to conservative assessment results.

To still be able to quantify the remaining strength of the structure (i.e. the probability of occurrence of structural failure and breach formation) a combination of expert knowledge and literature will be used in this thesis. During an expert session the characteristics of the structure that is going to be assessed as well as the characteristics of the load variables are discussed. From this discussion the likelihood of the probability of occurrence of structural failure and breach formation are estimated. Also, literature will be used to take stock of what is already known about the likelihood of the occurrence of these two sub-events.

4.5.3 Methods to be applied

In Table 14 it is summarized which nodes of event tree variants 2 and 3 involve which method of quantification.

Table 14 Methods of quantification applied in this thesis for the quantification of the probability of occurrence of the involved sub-events

Node / sub-event[13]	Model(s) available	Method of quantification
Event tree variant 3		
Piping	Yes	Probabilistic methods (FORM & Monte Carlo)
Structural failure	No	Expert knowledge & literature
Breach formation	No	Expert knowledge & literature
Event tree variant 2		
Heave	Yes	Probabilistic methods (FORM & Monte Carlo)
Backward erosion	Yes	Probabilistic methods (FORM & Monte Carlo)
Structural failure	No	Expert knowledge & literature
Breach formation	No	Expert knowledge & literature

To compare the quantified failure probability, the warning limit value and lower limit value for piping at a hydraulic structure need to be deduced. This is done through the failure probability budget and the length-effect.

[13] Please note that the node exceedance of critical head difference is not included in Table 14. This is because this node represents a precondition; it either has value 1 or 0. When it has value 1 the assessment continues with the consecutive nodes of the event tree. If not, the assessment is concluded.

5.Case study Marksluis

To be able to test an event tree approach for the probabilistic assessment of the failure probability of a hydraulic structure due to piping a case study is introduced in this chapter. The case study involves the Marksluis. The Marksluis is part of the primary flood defense system of dike trajectory 34-1. This chapter starts with a general introduction of the Marksluis (section 5.1). In this section background information on the Marksluis is presented and an overview of the water system surrounding the Marksluis is given. Section 5.2 treats the specifications of the Marksluis. These specifications are important for the probabilistic assessment on piping. Section 5.3 gives more insight in the relevant data of this case study. An explanation on a possible seepage path that can develop during the piping failure mechanism is treated in section 5.4.

5.1 General introduction of the Marksluis

For this thesis a case study is performed on the Marksluis. The Marksluis is situated near the town of Oosterhout, Noord-Brabant, the Netherlands. Figure 25 shows the location of the sluice. The structure is managed by Rijkswaterstaat within the area of Waterschap Brabantse Delta and it is part of dike trajectory 34-1.

The reason why this chamber lock has been chosen as case study is because from the simple test (see Figure 13 and section 3.2.1) it does not follow that the failure probability is negligible (Den Adel, 2019). Therefore, the assessment must be continued by means of a detailed test.

For dike trajectory 34-1 the lower limit is determined to be 1/300 $[1/year]$ and the warning limit value is 1/1000 $[1/year]$ according to (Waterwet, 2020). Despite being situated in close proximity to dike trajectory 35-2 it is stated in (Den Adel, 2019) that a potential failure of the Marksluis will not cause a flood in dike trajectory 35-2. Therefore, only the safety norms of dike trajectory 34-1 apply to the Marksluis.

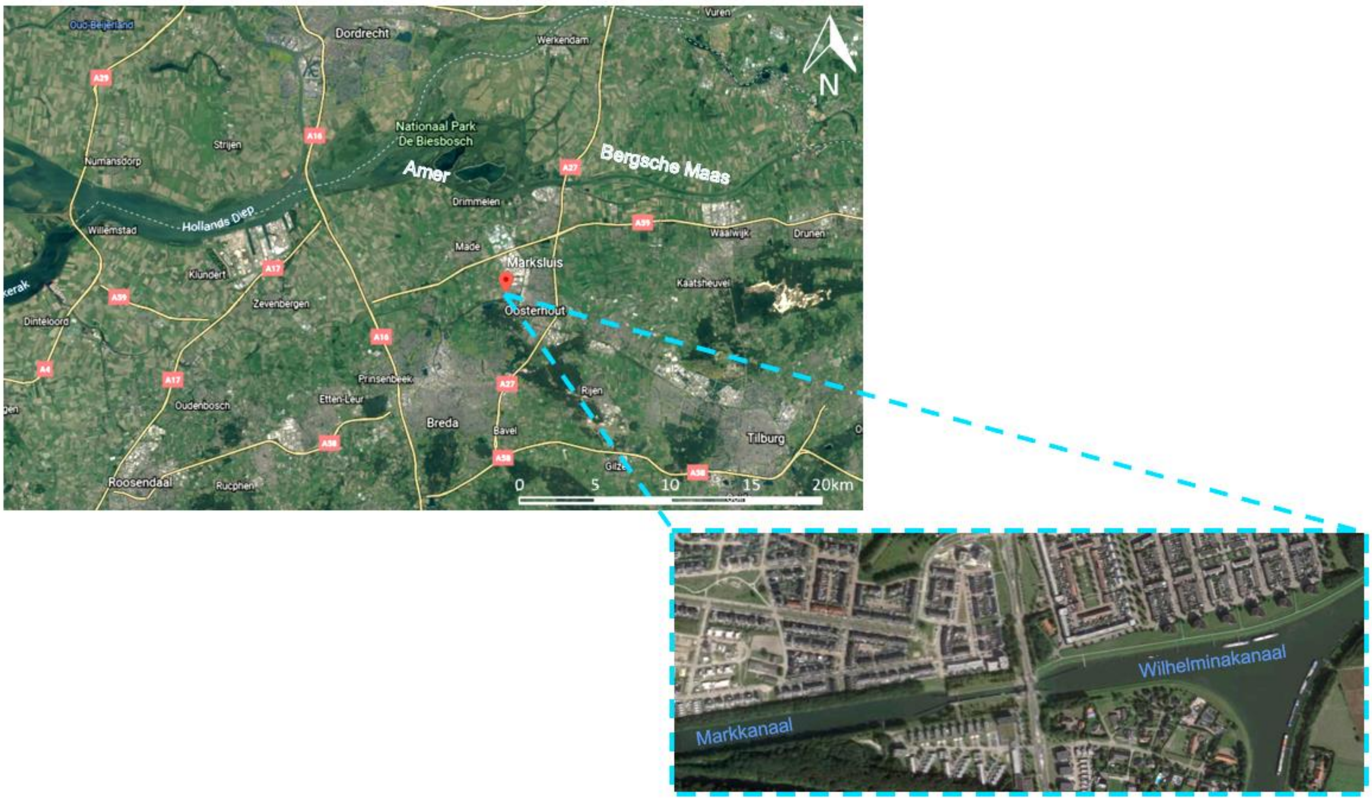

Figure 25 Location of the Marksluis including close-up, showing the Marksluis, the upstream Wilhelminakanaal (east side of the sluice) and the downstream Markkanaal (west side of the sluice) (source: (Google Maps, 2020))

The Marksluis is a chamber lock situated at the intersection of the Wilhelminaknaal and the Markkanaal. The Wilhelminakanaal connects to the Amer, which is connected to the Bergsche Maas. The discharge of the Bergsche Maas is therefore of influence of the water level in the Wilhelminakanaal. The average depth of the Wilhelminakanaal is 2.3 meters (Binnenvaart in Beeld, 2013b). The Markkanaal is connected to the Volkerak. The connection can be closed in times of high water by means of a guard lock. The average depth of the Markkanaal is 3.8 meters (Rijkswaterstaat, n.d.-a). A full overview of the surrounding water system of the Marksluis is shown by Figure 39 in Appendix B.

The sluice complex was constructed in 1968 (Den Adel, 2019). Today, it still operates in the same configuration as it was originally designed and constructed with no adjustments to the structure (besides regular maintenance). However, adjustments were made to the surroundings of the structure. The Markkanaal was widened in 1976 (Binnenvaart in Beeld, 2013a; Rijkswaterstaat, n.d.-b).

The Marksluis is part of the primary flood defense system. Besides managing the water level in both channels the chamber lock is also used for ships to navigate (commercial shipping class CEMT IV) (Den Adel, 2019).

5.2 Specifications of the Marksluis

The Marksluis consists of a concrete lock chamber and two concrete lock heads. The lock has a shallow foundation (Den Adel, 2019). The complex is equipped with two sets of mitre gates per lock head. The gates are oriented in opposing directions such that a head difference in both directions can be retained. The mitre gate set at the east end of the sluice is shown in Figure 26.

Figure 26 Set of mitre gates at the east side of the Marksluis (own photograph)

The length of the sluice complex is 157.9 meters (Rijkswaterstaat Directie sluizen en stuwen, 1968). The total available length for lockage of ships is 120 meters. The lock chamber is divided into seven concrete sections. Each of these sections has a length of 14.3 meters (Rijkswaterstaat Directie sluizen en stuwen, 1968). In Figure 26 the different sections can be observed when having a look at the retaining wall of the sluice. At the west end of the sluice the concrete structure is extended by 17 meters. This extension is needed due to presence of a bridge that allows traffic to cross the Wilhelminakanaal. A view from the Wilhelminakanaal to the Marksluis is shown in Figure 27.

Figure 27 East side of the Marksluis. The bridge allows road traffic to cross the Wilhelminakanaal (own photograph)

At both the eastern and the western lock head there are seepage screens installed. The seepage screens reach a depth of NAP-10.3 m and are made of steel (Rijkswaterstaat Directie sluizen en stuwen, 1968). Furthermore, at both ends of the complex bed and bank protection is applied. The bed protection consists of two parts of concrete blocks over a length of 10 meters (first part, only installed on the east side) and 4 meters (second part, both on the east and west side) and a sheet pile to NAP-7.30 m at the intersection between the two parts. Underneath the concrete blocks and the concrete piles is a 30 centimeters thick layer of gravel.

To summarize the characteristics of the Marksluis Table 15 gives an overview of specifics such as the dimensions and levels of the different elements of the Marksluis are given.

Table 15 Specifications of the Marksluis (sources: (Den Adel, 2019; Rijkswaterstaat Directie sluizen en stuwen, 1968))

Subject	**Specifics**
Length of sluice complex	157.9 m
Extension of concrete construction on east side	17 m
Length of the lock chamber	120 m
Height of wall of the lock chamber	NAP+4.30 m
Threshold level of sluice (bottom)	NAP-3.80 m
Inner width of lock chamber	14 m
Length of concrete section	14.3 m
Number of concrete sections	7
Thickness of floor of lock chamber	1.5 m
Thickness of walls of lock chamber	1.5 m (near the floor) to 0.7 m (near the top)
Number of doors	4 sets of doors (2 doors per set, 2 sets per lock head)
Width of door	7.82 m
Bottom and top of the door	NAP-3.80 m to NAP+4.30 m
Number of seepage screens	2 (one underneath each lock head (west and east))
Bottom level of seepage screens	NAP-10.3 m
Type of foundation	Shallow

5.3 Relevant data

To be able to assess the sluice on the failure mechanism piping relevant data is needed. The type of data needed to perform an analysis of piping at a hydraulic structure can be distinguished into multiple categories. The three main type categories of needed data are:

- Hydraulic loads;
- Data regarding the soil;
- Geometry of the sluice.

5.3.1 Hydraulic loads

When taking stock of load variables regarding the piping failure mechanism, only hydraulic loads are of importance (Rijkswaterstaat, 2019). Since piping is driven by the water level difference over the structure, data availability regarding water levels on both sides of the structure is important. Not only these water levels, but also the groundwater flow is important (Rijkswaterstaat, 2019). Given the flow, the change of water level in time can be considered. This captures the time dependence that is relevant for the lower parts of the rivers in the Netherlands (Rijkswaterstaat, 2019).

Water level of the Wilhelminakanaal

As shown in Figure 25 the upstream side of the Marksluis is the Wilhelminakanaal. This channel is connected to the Amer. This connection makes the water level of the Wilhelminakanaal dependent on the tide. In Figure 25 it can be observed that the Amer is connected to the Bergsche Maas. This connection has as consequence that the water level in the Wilhelminakanaal is also dependent on the discharge of the Bergsche Maas.

Since a probabilistic calculation needs the distribution types of the involved variables as input also the distribution of the water level of the Wilhelminakanaal is needed. To capture the extreme water levels that could occur at the location of the Marksluis a distribution type is needed that can describe the uncertainty of the annual maximum water level. This can be accomplished by a Gumbel distribution (Onen & Bagatur, 2017).

The cumulative distribution function (CDF) for a Gumbel distribution has two parameters: α and u. The function is shown by formula [23] (Jonkman, et al., 2017):

$$F_H(h) = \exp\left(-\,exp\left(-\alpha(h-u)\right)\right) \qquad [23]$$

Where:

α	$[m^{-1}]$	Scale parameter for the Gumbel distribution function
u	$[m]$	Location parameter for the Gumbel distribution function
$F_H(h)$	$[-]$	Cumulative distribution function of water level h
h	$[m]$	Water level

To find the Gumbel distribution describing the annual maximum water level at the Marksluis, the location and scale parameters must be determined. The procedure of determination of these parameters for both the maximum water level distribution at the warning limit value and the lower limit value is described in Appendix D.

Water level of the Markkanaal

The water at downstream side of the Marksluis is called the Markkanaal. The water levels in this channel are regulated. According to the ‘Waterakkoord Volkerak-Zoommeer’ one aims for a natural fluctuation of the water level between NAP-0.10 m and NAP+0.15 m (Waterschap Brabantse Delta, Waterschap Hollandse Delta, Waterschap Scheldestromen, & Rijkswaterstaat, 2016). In Dutch, this decision is called ‘peilbesluit’. Due to the regulated water levels no extreme value analysis has to be performed as was done for the water levels on the Wilhelminakanaal. Instead, a time series is obtained of the water levels directly downstream of the Marksluis from (Rijkswaterstaat, 2020). This series has a length of almost six years. It starts on March 30th 2014 and ends on January 1st 2020. It contains measurements of the water level every ten minutes. A graph showing the variation in water level is shown in Figure 28.

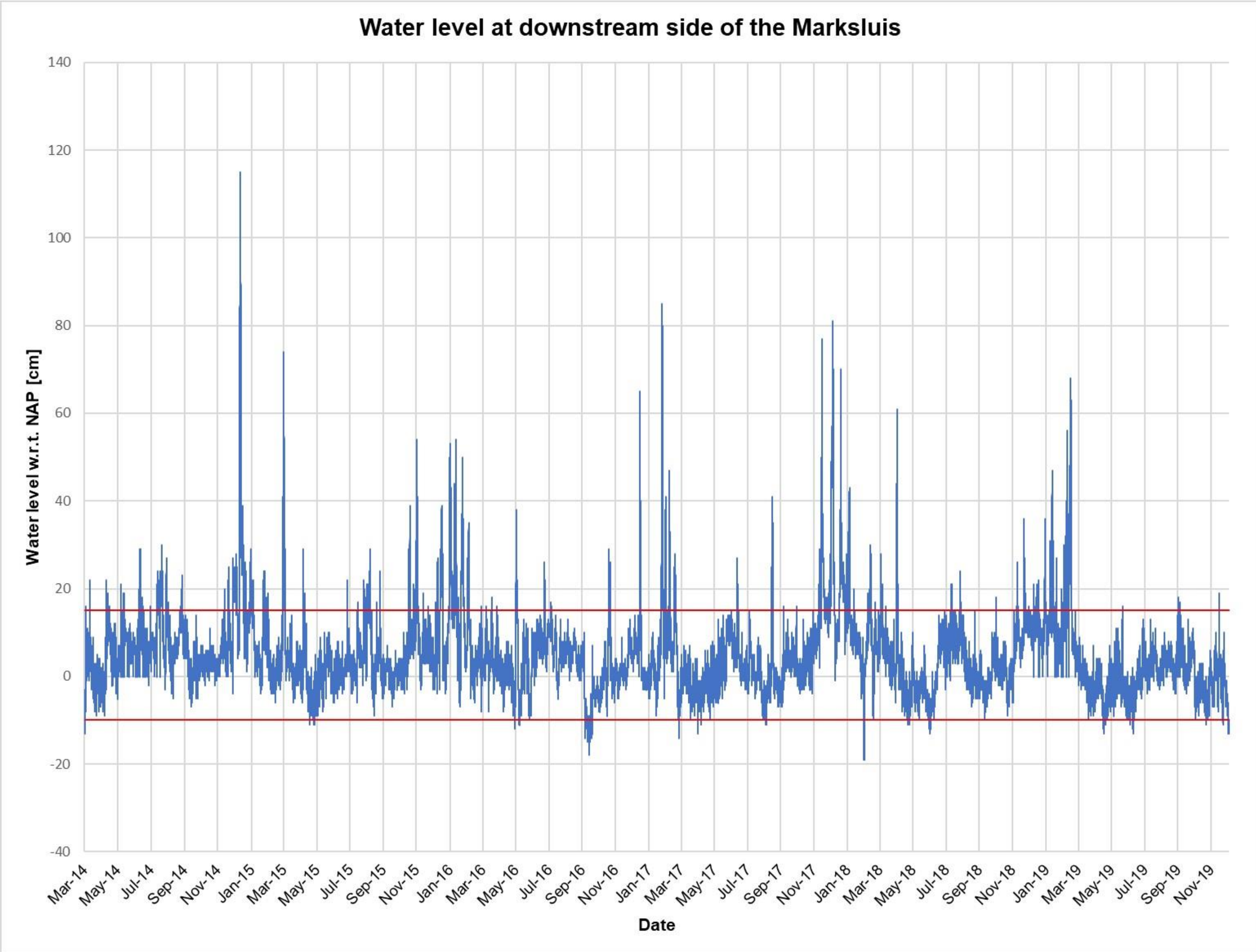

Figure 28 Water level variation on the Markkanaal directly downstream of the Marksluis. The blue line indicates the measured water level. The data is obtained from (Rijkswaterstaat, 2020). The red lines indicate the upper and lower bound of respectively NAP+0.15 m and NAP-0.10 m according to the 'peilbesluit' (Waterschap Brabantse Delta, Waterschap Hollandse Delta, Waterschap Scheldestromen, & Rijkswaterstaat, 2016)

Due to the regulation of the water level on the Markkanaal it is assumed that the water level behaves according to a normal distribution ($N(\mu, \sigma)$). After calculation of the mean water level and the standard deviation the distribution for the water level on the Markkanaal is known. The mean is calculated to be NAP+4.9 cm and the standard deviation 9.7 cm. Hence, the water level of the Markkanaal can be described by $N(0.049\ m; 0.097\ m)$.

5.3.2 Data regarding the soil

The soil at the Marksluis mainly consists of sandy material (Den Adel, 2019). This is observable in the result of a cone penetration test performed by the DINOloket at the location of the Marksluis (DINOloket, 1967). The result of the test is shown in Figure 29. The grain size diameter[14] varies with depth, ranging from silt to coarse sand.

[14] For the applicable ranges of grain size diameters for the different types of soil refer to Table 5 and Table 9.

Figure 29 Result of cone penetration test at location of the Marksluis at a surface elevation of NAP+3.78 m shown on the left. The test was taken at the spot of the green dot shown on the right (source: (DINOloket, 1967))

5.3.3 Geometry of the sluice

The following data on the geometry of the sluice is needed to perform an analysis of piping:

- Dimensions of the sluice;
- Information on the seepage screens;
- The type of foundation.

This data can be obtained from the specifications of the Marksluis that are listed in Table 15.

5.4 Schematization of piping at the Marksluis

Before any calculation on the failure mechanism piping can be performed, it is important to have an adequate schematization of the potential seepage paths. In this section an analysis is done on the potential location of critical seepage paths (section 5.4.1). The characteristics of the soil are discussed in section 5.4.2. Finally, the critical seepage path length is determined in section 5.4.3.

5.4.1 Location of critical seepage paths

To schematize piping at a hydraulic structure different steps have to be taken. The first step is to think of where the most critical seepage paths can occur. This will always be determined by both the geometry of the hydraulic structure and the soil characteristics (Rijkswaterstaat, 2019).

When critical seepage paths are identified it is important to distinguish two types of seepage paths (Förster, et al., 2012):

- Seepage paths that contain solely horizontal segments (in Dutch: ‘achterloopsheid’);
- Seepage paths that contain both horizontal and vertical segments (in Dutch: ‘onderloopsheid’).

By definition, the horizontal seepage path length underneath the structure equals the length of the structure including the length of any bed protection applied, given that the connection between the structure and the bed protection is adequate.

In this thesis only a seepage path that includes vertical segments is assessed. By the assessment of seepage paths that contain both horizontal and vertical paths the idea behind determining a seepage path is explained. This procedure could then be repeated for a seepage path that solely contains horizontal segments.

5.4.2 Characteristics of the soil

To identify the soil profile at the location of the hydraulic structure it is important to have a result of a cone penetration test available at the exact location of the structure (Rijkswaterstaat, 2019). During construction of the structure excavation works might have changed the soil profile compared to the original profile. Usage of results of cone penetration tests close to the structure – but not directly at the structure – might therefore result in an incorrect soil profile. The obtained result of the cone penetration test shown in Figure 29 dates from 1967 (DINOloket, 1967). This means that the cone penetration test was performed before the construction of the Marksluis.

As can be observed from the results of the cone penetration test of Figure 29 the soil mostly consists of moderately fine sand with little variation to very fine sand and coarse sand. Therefore, the present soil type is modelled completely as moderately fine sand. According to Table 5 and Table 6 of section 3.3 this results in a creep factor that follows a normal distribution with a mean value of 7 and a standard deviation of 0.7, which can be written as $N(7;\ 0.7)$.

5.4.3 Determination of critical seepage path

The critical seepage path consists of both horizontal and vertical segments. In the determination of the critical seepage path it is assumed that the connection between the seepage screens and the bottom of the sluice is of good quality. Both seepage screens have a length of 4.3 meters (Den Adel, 2019). Since the seepage path is forced around the seepage screen the length of the screen needs to be considered twice. So, the vertical seepage path at each seepage screen is 8.6 meters leading to a total vertical seepage path of 17.2 meters.

The horizontal segment of the seepage path consists of two sections. The first section is covered by the length of the sluice complex, which involves the length of the lock chamber and the length of both lock heads. The length of the sluice complex is 157.9 meters (Rijkswaterstaat Directie sluizen en stuwen, 1968). The extension of the concrete structure on the east side (side of the Wilhelminakanaal) must be included in the horizontal segment of the seepage path. This extension has a length of 17.0 meters (Den Adel, 2019). This makes the horizontal segment of the seepage path in total 174.9 meters long. The full critical seepage path at the Marksluis is drawn in Figure 30.

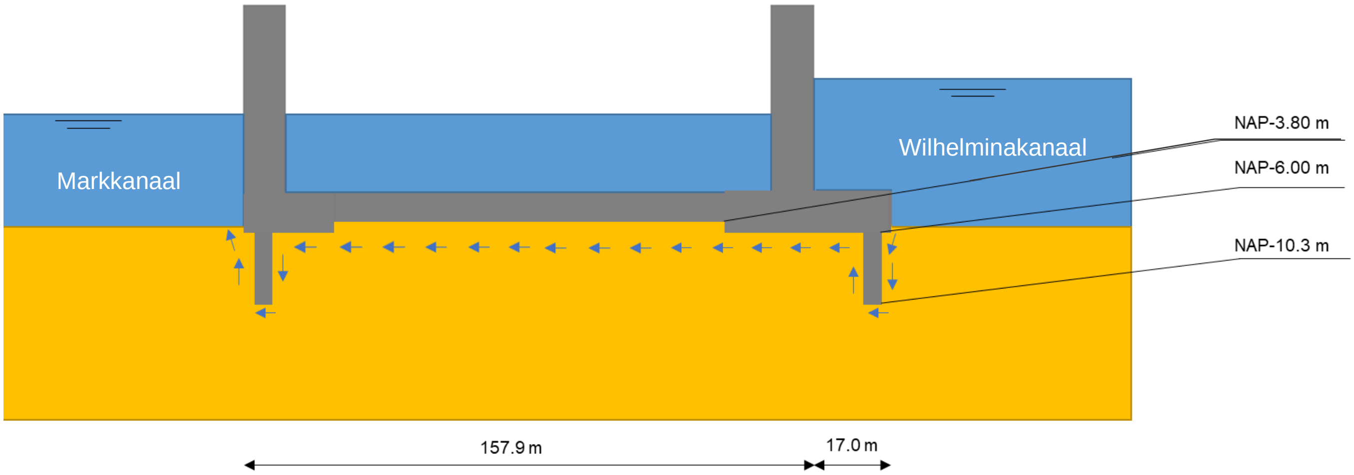

Figure 30 Schematization of the critical seepage path at the Marksluis

6. Probabilistic analysis results

This chapter treats the probabilistic analysis of the failure mechanism piping at the hydraulic structure (the Marksluis) introduced in the case study. To be able to compare the obtained failure probability from the probabilistic analysis the requirement for the piping failure mechanism must be derived. How this is done is explained in section 6.1. From then, the actual probability of failure due to piping at the Marksluis is calculated. This is done according to the flow chart presented in Figure 24 in section 4.4.3. Therefore, first the probability on the geotechnical part of piping is calculated in section 6.3. The consecutive node of the event tree is the node describing structural failure. The probability belonging to this sub-event is discussed in section 6.4. Section 6.5 continues the assessment with the analysis of the node describing breach formation. This chapter concludes with a calculation on the failure probability due to piping at the Marksluis by the obtained (conditional) probabilities and compares it to the derived requirement in section 6.6.

6.1 Failure probability according to the norm

Before any comparison between a calculated failure probability (flooding due to piping) and the allowable failure probability for this failure mechanism according to the norm can be made the allowable failure probability has to be derived. This can be accomplished by using the failure probability budget (in Dutch: 'faalkansbegroting') and the length effect (Kok, et al., 2017).

6.1.1 Failure probability budget

Each dike trajectory has its own safety standard with a corresponding lower limit value and a warning limit value. These two values combined form the norm. Since multiple failure mechanisms can cause the trajectory to fail the failure probability budget can be used to derive the norm for a specific failure mechanism.

For this thesis the failure mechanism of interest is piping at a hydraulic structure. According to (Rijkswaterstaat, 2017a; Kok, et al., 2017) the maximum contribution of piping at a hydraulic structure to the failure probability of the full dike trajectory is 2%. When multiplying the maximum contribution with the norm of the dike trajectory the requirement for the failure probability of piping of the dike trajectory can be deducted.

6.1.2 Length-effect

After deducting the requirement for piping the failure probability requirement for this failure mechanism must be distributed over the different dike reaches of the dike trajectory. To account for this, the length-effect can be used (Kok, et al., 2017).

According to (Casteleijn, et al., 2018) the length-effect factor for piping at hydraulic structures can be calculated with formula [24][15]:

$$N_{PKW} = \min\,(n_{kw}; 10) \qquad [24]$$

Where:

N_{PKW}	[–]	The length-effect factor for piping at hydraulic structures
n_{kw}	[–]	The number of hydraulic structures for which piping is a relevant failure mechanism in the applicable dike trajectory

[15] The WBI 2017 does not describe a relation that could be used to find the length-effect factor for piping at hydraulic structures. Therefore, the relation from 'Handreiking ontwerpen met overstromingskansen (OI2014v4)' (Rijkswaterstaat, 2017a) is used to calculate the length-effect factor. For further details please refer to Table 2 of (Casteleijn, et al., 2018).

To find the number of hydraulic structures for which piping is a relevant failure mechanism, the results of the project 'Veiligheid in Kaart 2' (VNK2)[16] have been used. In this project a screening is performed to find the hydraulic structures for which piping is relevant. This screening consists of three parts:

- Rough screening;
- Screening by using assessment results;
- Screening by using parameters.

A detailed explanation on these screening methods is provided by (Van Lammeren, 2011). From these three screening methods it followed that in total six hydraulic structures have piping as a relevant failure mechanism for the entire dike ring area 34. Since the assessment according to the WBI 2017 focusses on dike trajectories instead of dike ring areas it is important to find which of these six hydraulic structures are located in dike trajectory 34-1. After comparison between the present hydraulic structures in dike ring area 34 and the corresponding locations in dike trajectory 34-1 and 34-2 it can be concluded which structures need to be taken into account to determine the length effect. Figure 31 shows dike ring area 34. The red dotted line marks the division of the dike ring area into dike trajectory 34-1 and 34-2. All hydraulic structures to be considered are therefore located on the right-hand side of the red dotted line.

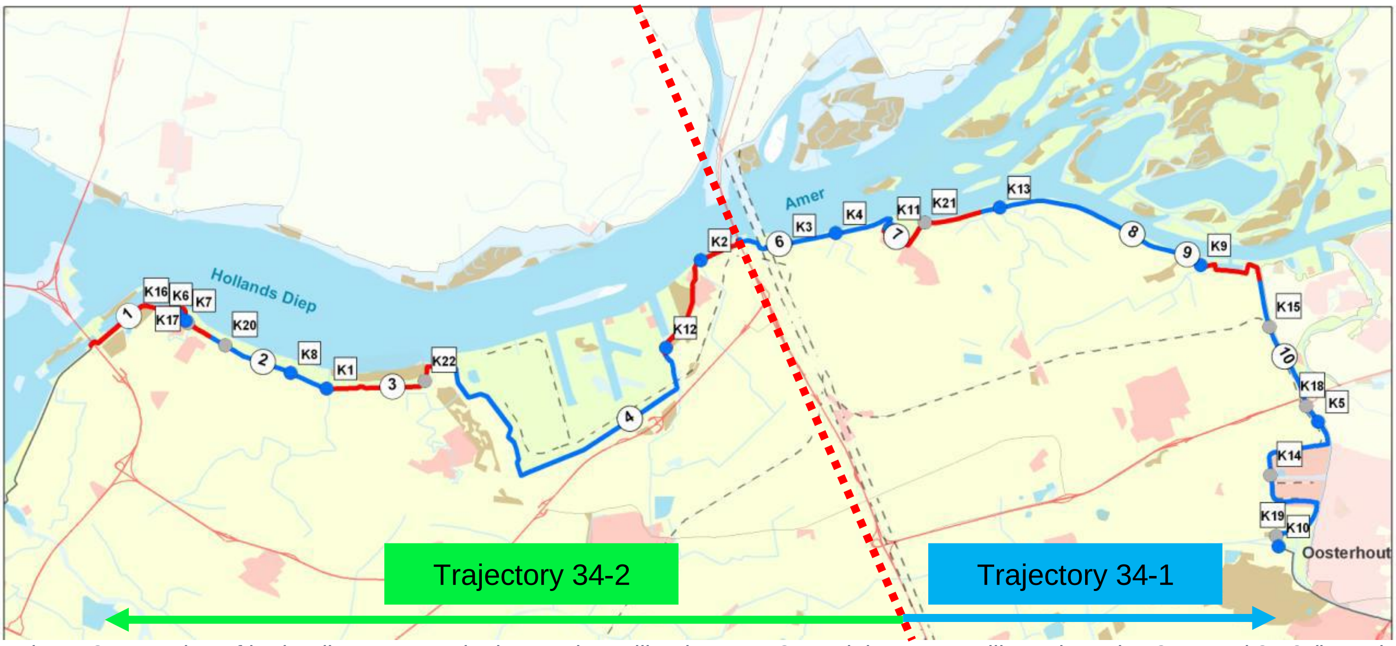

Figure 31 Location of hydraulic structures in the previous dike ring area 34 and the current dike trajectories 34-1 and 34-2 (based on: (Van Lammeren, 2011)). The red dotted line indicates the boundary between current dike trajectories. On the right-hand side of the boundary, dike trajectory 34-1 is located. Dike trajectory 34-2 is located on the left-hand side of the boundary.

The result of the screening in the VNK2 project is four hydraulic structures that have piping as a relevant failure mechanism that are located in the current 34-1 dike trajectory. An overview of these structures is given in Table 16.

Table 16 Hydraulic structures located in dike trajectory 34-1 that have piping as a relevant failure mechanism according to (Van Lammeren, 2011)

Name of hydraulic structure	Type of hydraulic structure	Number in Figure 31
Schuddebeurs	Pumping station	K4
Amersluis	Inlet and discharge sluice	K13
Herengracht	Inlet structure	K9
Houtse Steeg	Inlet structure	K14

[16] In the VNK2 project dike ring areas were assessed on flood risk. In the current assessment procedure according to the WBI 2017 dike trajectories are assessed. For the difference between a dike ring area and a dike trajectory please refer to Figure 6.

In addition, the Marksluis (introduced in chapter 5) can be added to this list. Even though VNK2 did not flag this structure, WBI 2017 did flag it. From the simple test of WBI 2017 it followed that assessment for the Marksluis on the piping failure mechanism must be continued (Den Adel, 2019). This makes it a relevant failure mechanism. This results in a value of 5 for n_{kw}. Since this is smaller than 10, the value for N_{PKW} equals the value of n_{kw} as can be concluded from formula [24].

6.1.3 Requirement of the failure probability

In respectively section 6.1.1 and section 6.1.2 the contribution factor and the length-effect for piping at hydraulic structures have been discussed. Since these values are known, the requirement for the failure probability can be derived. To do this, formula [25] (Casteleijn, et al., 2018; Jonkman, et al., 2018) can be used:

$$P_{f,req,PKW} = \frac{P_{f,trajectory} * \omega_{PKW}}{N_{PKW}} \qquad [25]$$

Where:

$P_{f,req,PKW}$	$[1/year]$	The requirement for the annual failure probability of the mechanism piping at a hydraulic structure for a dike reach
$P_{f,trajectory}$	$[1/year]$	The requirement for the annual failure probability for the entire dike trajectory[17]
ω_{PKW}	$[-]$	Factor of maximum contribution to the annual failure probability for the entire dike trajectory by the mechanism piping at a hydraulic structure

The requirement for the annual failure probability for dike trajectory 34-1 can be divided into a warning limit value and a lower limit value. According to (Waterwet, 2020) the annual warning limit value is 1/1,000 and the annual lower limit value is 1/300. The $P_{f,req,PKW}$ column of Table 17 shows the result of application of relation [25], which is the requirement of the failure probability for piping at the Marksluis.

Table 17 Limit values according to the norm of the 'Waterwet' for dike trajectory 34-1 (Waterwet, 2020) and the deducted requirement for the annual failure probability of piping at a hydraulic structure for a dike reach

	Requirement for the annual failure probability	
Limit values	$P_{f,trajectory}$	$P_{f,req,PKW}$
Warning limit value ($P_{req,wlv}$)	1/1,000	$4.0 * 10^{-6}$
Lower limit value ($P_{req,llv}$)	1/300	$1.3 * 10^{-5}$

6.2 Node describing critical head exceedance

As mentioned in section 4.4.2 the exceedance of the critical head difference is seen as a precondition in this thesis. This is done because the probability of the occurrence of certain water levels in both the Wilhelminakanaal and the Markkanaal and hence the head difference over the structure is incorporated in the consecutive node describing piping. This node uses Lane's model to calculate the probability of occurrence of piping in which the probability density functions of the water levels of both canals are included.

The precondition can take two values; 0 and 1. If the critical head difference is exceeded it takes value 1 which means that the consecutive nodes need to be analyzed. If it takes value 0 no further assessment is needed since piping will not occur in that situation.

[17] This requirement is directly related to the warning limit value and the lower limit value according to the norm for the involved dike trajectory. These values are provided in the 'Waterwet' (translates literally to Water law) (Waterwet, 2020).

6.3 Node describing piping

Now the requirement for the failure probability due to piping is known, the actual failure probability must be calculated to be able to make a comparison between the two values and conclude in which safety judgment category the Marksluis belongs.

To calculate the actual probability of occurrence of piping at the Marksluis, the procedure described at section 4.4.3 is followed[18]. Therefore, the analysis starts by making use of event tree variant 3 (see Figure 23).

The first node of event tree variant 3 is piping. Since the schematization of the potential seepage path contains vertical elements Lane's model (see section 3.3.1) is used. From the schematization described at section 5.4 all relevant variables and their belonging distributions can be determined. An overview of the distributions that are used as input for Lane's model are shown in Table 18.

Table 18 Input values for distribution types for stochastic variables of Lane's model applied to the Marksluis case

Resistance variable	**Distribution type**	**Distribution parameters**	
		$\boldsymbol{\mu}$	$\boldsymbol{\sigma}$
m_L	Lognormal	2.2	0.3
L_v	Normal	17.2 m	0.1 m
L_h	Normal	174.9 m	0.1 m
Load variable	**Distribution type**	**Normal: $\boldsymbol{\mu}$, Gumbel: $\boldsymbol{\alpha}$**	**Normal: $\boldsymbol{\sigma}$, Gumbel: $\boldsymbol{u}$**
C_{LANE}	Normal	μ = 7.0	σ = 0.7
m_c	Normal	μ = 1	σ = 0.1
h	Gumbel	α = 7.6 m^{-1}	u = 2.4 m
h_{bi}	Normal	μ = 0.05 m	σ = 0.1 m

The limit state function described by formulas [4], [5] and [6] (see section 3.3.1) is used to determine the probability of occurrence of piping at the Marksluis. As stated in section 4.5 the probabilistic assessment is done with two methods: FORM, which is a level II analysis, Crude Monte Carlo and Monte Carlo with importance sampling, which are both level III analyses.

The results of the FORM, Crude Monte Carlo and Monte Carlo with importance sampling analyses are shown in Table 19.

Table 19 Results of the probabilistic analysis on piping at the Marksluis

Reliability method	$\boldsymbol{P_{piping}}$
FORM	$1.3 * 10^{-26}$
Crude Monte Carlo	≈ 0
Monte Carlo with importance sampling	$2.3 * 10^{-27}$

As can be observed from Table 19 the calculated probabilities of occurrence of piping are extremely low. According to the Crude Monte Carlo reliability method the probability is even zero. This is a consequence of the fact that no design point can be found for an analysis with 100,000 simulations. This could be solved by application of a Monte Carlo importance sampling method. The obtained result is still an extremely small probability as can be seen in Table 19. The obtained results are in line with the results of the performed detailed test by (Den Adel, 2019).

Next to a probability, the FORM analysis also results in an overview of so-called influence factors. These factors show the relative contribution of a random variable to the uncertainty of the outcome of the limit state function. These influence factors (also called α-factors) relate according to formula [26] (Steenbergen, 2018):

$$\alpha_1^2 + \alpha_2^2 + \ldots + \alpha_n^2 = 1 \qquad [26]$$

[18] A summarized overview of this procedure is shown in Figure 24.

Where:

α_i $[-]$ The influence factor of stochastic variable i

The squared values of the influence factors for the random variables involved in the probabilistic assessment of piping at the Marksluis are shown in Table 20. The larger the value of the influence factor, the larger the contribution to the uncertainty of the calculation of the probability. To mark the difference in contribution to the uncertainty color markings are used in Table 20. A green color indicates a relatively low contribution to uncertainty whereas a red color marks a relatively large contribution. A yellow color means that the contribution is neither relatively small nor relatively large.

From Table 20 it can be concluded that the upstream water level (so the water level of the Wilhelminakanaal) has the largest contribution to the uncertainty of the calculation of the probability. Also, the model parameter of the present seepage path length has a large share in the uncertainty contribution. Both horizontal and vertical seepage path lengths as well as the downstream water level (water level of the Markkanaal) have a negligible contribution to the uncertainty.

Table 20 Squared influence factors for random variables in Lane's model applied to the Marksluis. The color in the column on the right indicates the extent of contribution to the uncertainty of the outcome of the limit state function. Red indicates a relatively high contribution, yellow indicates a medium contribution and green indicates a relatively low contribution

Variable	Symbol	Influence factor (α_i^2)	
Model parameter of the present seepage path length	m_L	0.30	
Vertical seepage path	L_v	$3.4 * 10^{-5}$	
Horizontal seepage path in Lane's model	L_h	$3.8 * 10^{-6}$	
Creep factor belonging to Lane's model	C_{LANE}	0.10	
Model parameter for Bligh and Lane	m_c	0.10	
Upstream water level	h	0.50	
Downstream water level	h_{bi}	$4.5 * 10^{-3}$	+
Summation of squared influence factors	$\sum \alpha_i^2$	1.0	

By performing the calculation of the probability of occurrence of piping the first node of event tree variant 3 is treated. This also concludes phase A (geotechnical part) of the event tree. Section 6.3 continues the assessment by analysis of structural failure.

6.4 Node describing structural failure

The sub-mechanism following piping is structural failure. This node marks the start of main phase C (remaining strength of the structure). There is no model describing a limit state function for structural failure of a hydraulic structure given piping has occurred (yet). Therefore, in practice the conditional probability of the occurrence of structural failure is set to 1 (Casteleijn, et al., 2018).

What makes this sub-mechanism complex is the various ways of how a hydraulic structure might fail when piping has occurred. Options for the type of structural failure can be:

- Tilting;
- Shearing;
- Consecutive failure of parts of the structure;
- Failure at the connection of the hydraulic structure to the surrounding dike.

It is hard to tell which of these types of structural failure occurs given the temporal and spatial dependence on the erosion pattern. A qualitative analysis of these different types of structural failure is given in section 7.2.

Since a quantitative analysis by using a model is not (yet) possible for this sub-mechanism expert knowledge is collected on the case of the Marksluis. This approach follows the advice of (Diermanse, 2016; Goldreich, Mahajan, & Phinney, 1999) when data is lacking. The expert knowledge is used to refine the upper limit for the value of the conditional probability of occurrence of structural failure at the Marksluis. With this approach the goal is to find an order of magnitude of the conditional probability.

To get expert knowledge on this conditional sub-mechanism Rob Delhez has been interviewed (Delhez, 2020). During the session technical drawings of the Marksluis have been studied (Rijkswaterstaat Directie sluizen en stuwen, 1968) together with the technical details of the Marksluis as provided in chapter 5.

Given the dimensions and the characteristics of the Marksluis it is very conservative to equate the conditional probability of structural failure to 1.0 as prescribed (Delhez, 2020). Its dimensions including the seepage screen are actually providing a lot of resistance against the occurring head difference at the exceedance frequencies corresponding to the limit values. This is also in line with the results of the executed detailed test according to the WBI 2017 for piping at the Marksluis by (Den Adel, 2019).

From these reasons it can be concluded that the likelihood that the Marksluis could fail in a structural sense as a consequence of piping is small. Following (Goldreich, et al., 1999), the order of magnitude of the upper limit value for the conditional probability of occurrence of structural failure can be decreased by a factor 10. Therefore, the conditional probability that is used for this node is set to $P_{SF|piping} = 0.1$ (Delhez, 2020).

6.5 Node describing breach formation

In the hypothetical case that a hydraulic structure would fail structurally as a consequence of piping it does not immediately mean that a breach is formed in the involved dike trajectory. First of all, whether a breach is formed depends on the type of structural failure that happened. For example, a hydraulic structure might have failed in structural sense, but it could still be able to retain water. This example is shown in Figure 42 in Appendix E.

To quantify the conditional probability of breach formation as a consequence of piping and structural failure, the approach of (Te Nijenhuis, et al., 2020; Goldreich, et al., 1999) is followed. In (Te Nijenhuis, et al., 2020) the conditional probability of breach formation is set to be 0.1. This chosen value indicates that the breach is not automatically formed when the structure would fail in structural sense. It also implies that emergency measures that could be applied must also be ineffective in order to let breach formation, leading to flooding, happen (Te Nijenhuis, et al., 2020).

This approach is supported by the interview with Rob Delhez. If in the hypothetical case the structure would fail the likelihood that parts of the structure could be taken by the existing current is very small given the dimensions and characteristics of the Marksluis (Delhez, 2020). Therefore, the assumption that the conditional probability of breach formation can be set to $P_{Breach|SF|piping} = 0.1$ as upper limit is used in this thesis.

6.6 Conclusion

Since all nodes have been quantified it is possible to determine the probability of failure due to piping at the Marksluis. Therefore, the probabilities at each node have been determined by making use of the multiplicative rule. This implicitly assumes that the different branches of a node are mutually exclusive and collectively exhaustive (the different branches exclude their outcomes and cover all possible outcomes together) (Te Nijenhuis, et al., 2020). Also, independence between the events is assumed. This makes it possible to make use of equation [20] as discussed in section 4.2.2. The resulting probabilities are shown in Table 21.

Table 21 (Conditional) probabilities for event tree variant 3 describing piping at the Marksluis. A probability with a bar indicates the opposite event. For example, $\overline{P_{piping}}$ indicates the probability that piping does NOT occur

Node	Probability	Value
Piping	P_{piping}	1.3 * 10^{-26}
	$\overline{P_{piping}}$	≈ 1.0
Structural failure	$P_{SF\|piping}$	0.1
	$\overline{P_{SF\|piping}}$	0.9
Breach	$P_{breach\|SF\|piping}$	0.1
	$\overline{P_{breach\|SF\|piping}}$	0.9

To find the probability of all leaves of the event tree, the (conditional) probabilities of each involved node of the failure path leading to that leaf must be multiplicated[19] (Te Nijenhuis, et al., 2020). In Figure 32 the result is shown in the event tree. The calculated probabilities according to the FORM analysis are shown at each node and leaf.

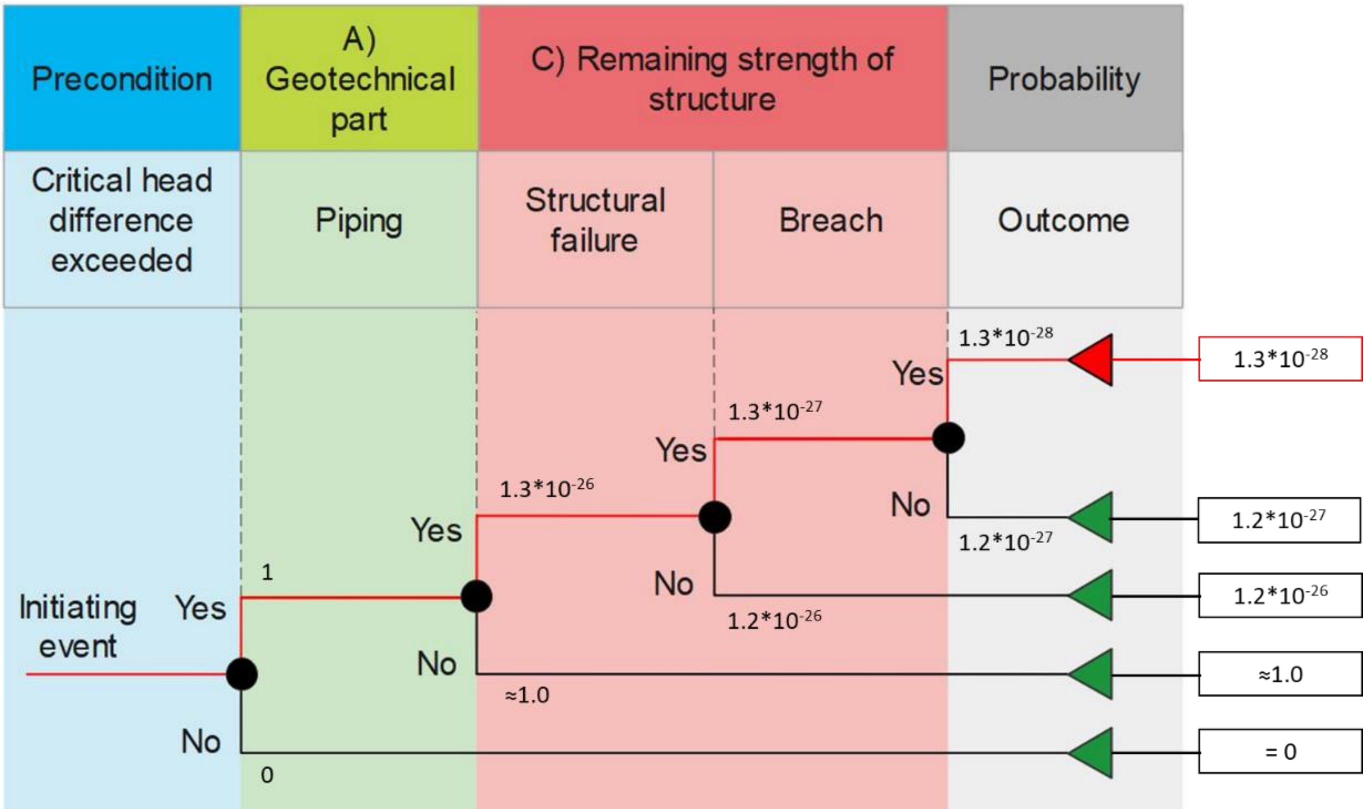

Figure 32 Event tree variant 3 including calculated probabilities with FORM

From Figure 32 it can be concluded that the probability of failure due to piping at the Marksluis, $P_{f,PKW} = 1.3 * 10^{-28} \approx 0$. To find a safety judgment for the Marksluis for the failure mechanism piping the calculated failure probability is compared to the warning limit value. To do so, an overview of the different probabilities is provided in Table 22.

[19] This is under the assumption of independence between the nodes.

Table 22 Overview of probabilities

Probability	Symbol	Value
Probability of failure due to piping at the Marksluis	$P_{f,PKW}$	$1.3 * 10^{-28}$
Warning limit value for piping at the Marksluis	$P_{req,wlv}$	$4.0 * 10^{-6}$
1/30 of warning limit value for piping at the Marksluis	$\frac{1}{30} * P_{req,wlv}$	$1.3 * 10^{-7}$

As can be concluded from Table 22 $P_{f,PKW} < \frac{1}{30} * P_{f,req,wlv}$. According to the flow chart shown in Figure 24 this means that the safety judgment category is I_V. With this, no further assessment is needed since the obtained category is the safest one.

7. Discussion

In this chapter several findings are discussed that were discovered to be not straightforward during the execution of this research. Consecutively, the following items will be discussed: event tree set-up (section 7.1), possible types of structural failure as a consequence of piping (section 7.2), the influence of the geometry and specifications of hydraulic structures on the likelihood of piping (section 7.3), the need of both data and expert knowledge (section 7.4) and remarks about the current WBI 2017 framework (section 7.5).

7.1 Event tree set-up

In this thesis three different event tree variants have been constructed that describe the failure mechanism piping at a hydraulic structure. The first variant contains nine sub-events that describe the failure path. Due to the extended size of this fault tree a lot of data is needed before one can use it for a quantitative analysis. Therefore, two smaller event trees have been constructed that can be used to quantify the probability of failure. This shows that the construction of an event tree is an iterative process. The process depends on the availability of data that is needed to calculate the probability of occurrence of the unwanted event.

Due to the limited amount of available data and models describing the full failure mechanism, the event tree that is used is rather concise. At every node two branches are connected. For some of the sub-events more branches could have been defined. For example, the node describing backward erosion could have had four branches indicating the percentage of eroded soil (e.g. 25%, 50%, 75% and 100%). Also, the number of branches originating from the node structural failure could have been higher. Each branch could then represent one of the possible types of structural failure (see also section 7.2). However, with an increasing number of possible outcomes the complexity of calculations increases further, too.

It is therefore important to find an optimum in the degree of representation of reality[20] by the created event tree and the involved complexity of the calculations. The results of the probabilistic calculation performed in chapter 6 showed a large difference between the limit values for piping at a hydraulic structure in dike trajectory 34-1 and the obtained failure probability for piping at the Marksluis. It should be reminded that the case study used in this thesis is for illustrative purposes. It shows how an event tree method can be applied to a real hydraulic structure and what the process of schematization of both the structure and the event tree involves.

7.2 Structural failure due to piping

As discussed in section 6.4 structural failure as a consequence of piping can take place in multiple variants. Unfortunately, no models are available yet describing structural failure due to piping. Given the complexity of this sub-event it is difficult to summarize it in a model. However, this section elaborates on the conditional structural failure of a hydraulic structure in a qualitative manner.

In principle, a structure fails in structural sense when it does not fulfill any of the following conditions (Molenaar & Voorendt, 2019):

- Stability condition;
- Strength condition;
- Stiffness condition.

As named in section 6.4, the following types of structural failure can occur once piping has occurred:

- Tilting;
- Shearing;
- Consecutive failure of parts of the structure;
- Failure at the connection of the hydraulic structure to the surrounding dike.

[20] One should realize that a perfect representation of reality is not possible since a model is always a simplification of reality.

Each of these structural failure types are qualitatively analyzed. This is done by the help of sketches indicating the forces that are of importance for the different types of structural failure.

7.2.1 Tilting

As a consequence of erosion of soil due to piping it can occur that the foundation of the involved hydraulic structure is affected. This holds for hydraulic structures that have a shallow foundation (in Dutch: 'fundering op staal'). As mentioned in section 3.1.1, when the pipe formation is completed, the resistance against groundwater flow drops dramatically. This leads to widening of the pipe. When the pipe becomes wide enough, the stability criterion cannot be met anymore. The structure will tilt leading to a structural failure. From then on, the structure cannot fulfill its function to allow vessels to navigate through the structure anymore. However, it does not mean that it has lost its water retaining function. A tilted structure could still retain water, provided the retaining height of the structure is still high enough.

To analyze whether tilting could occur potential seepage paths must be thought of. Especially, the widening process is important since this determines how the structure could tilt. As shown in Figure 33, by using the principle of moments an overview can be made of the torque caused by the forces acting on the structure. If there is no balance (i.e. the resulting torque is not equal to zero) the structure will tilt (see picture on the right of Figure 33).

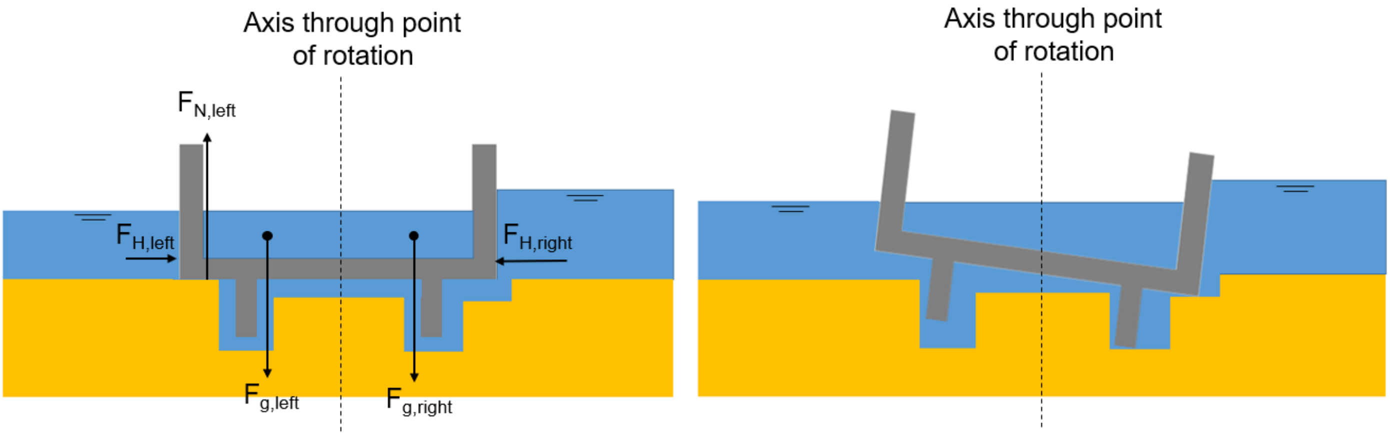

Figure 33 On the left a hydraulic structure where widening of the pipe has occurred is shown. Considering the principle of moments one can conclude that the resulting torque in clockwise direction is larger than the resulting torque in counterclockwise direction. This leads to a tilted hydraulic structure with a clockwise orientation. This is shown on the right

7.2.2 Shearing

When erosion continues over time, more and more sand gets transported. If this process continues long enough the structure could shear instead of tilt. So, the same theory applies as discussed in section 7.2.1. However, during tilting the structure still has a connection to the soil (see Figure 33). This connection prevents it from shearing. Therefore, shearing can take place after tilting has occurred when erosion of the soil continued (see Figure 34). In Figure 34 it is shown that during shearing the structure could break if the strength is exceeded by the load acting on the structure.

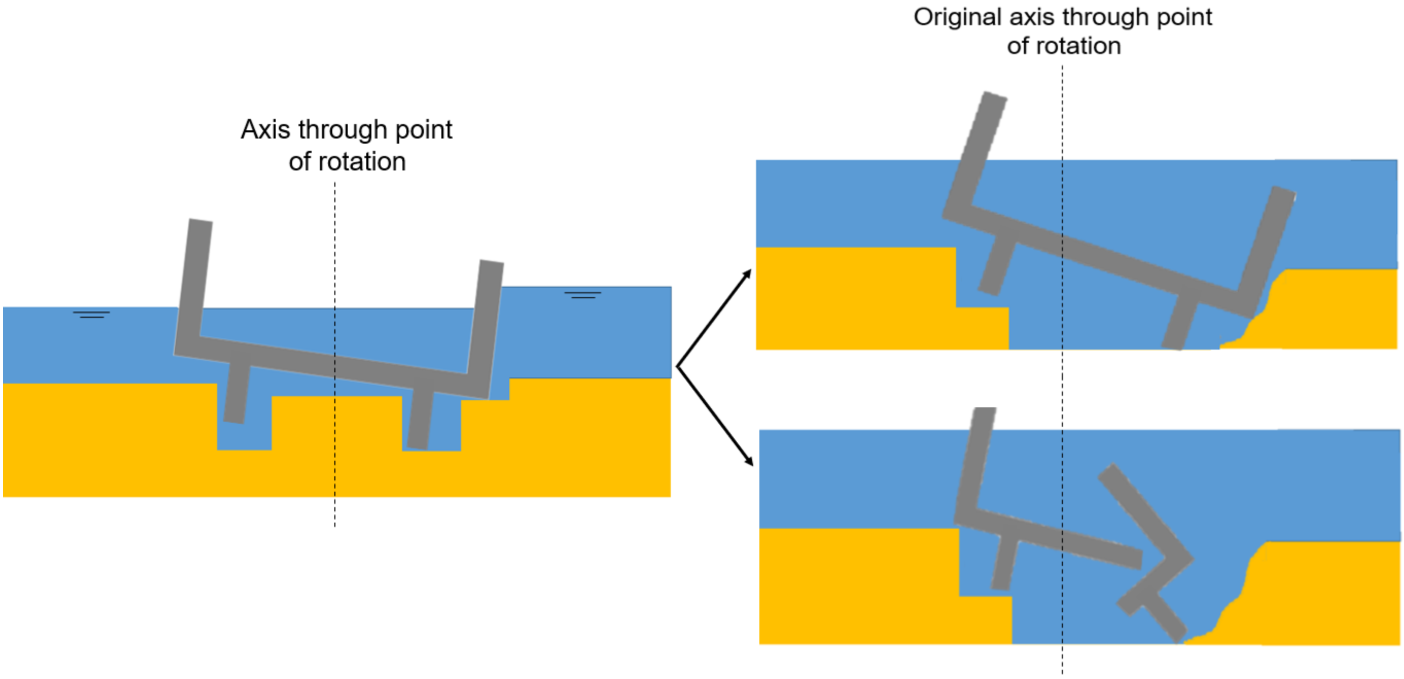

Figure 34 Shearing of a hydraulic structure as a consequence of piping. On the left a tilted structure is shown. Due to ongoing erosion the structure can shear when connection with the soil is lost. This is shown in the top right picture. If the forces acting on the structure are larger than the strength, it could break. This is shown in the bottom right picture

7.2.3 Consecutive failure of parts of the structure

In practice, most hydraulic structures consist of segments. This is also the case for the Marksluis (the structure is divided in seven concrete sections). The connection between the segments are joints. Figure 35 shows a hydraulic structure that contains 7 concrete sections. The black lines indicate the presence of a joint. Again, piping starts with heave as is shown in Figure 35b. Backward erosion follows heave. When the pipe reaches a joint that offers too less resistance against the water pressure the joint can fail. This is shown in Figure 35c. Failure at a joint breaks the connection of the concrete section of the structure. This could lead to loss of the strength condition of the structure, leading to consecutive failure of different parts of the structure as shown in Figure 35d.

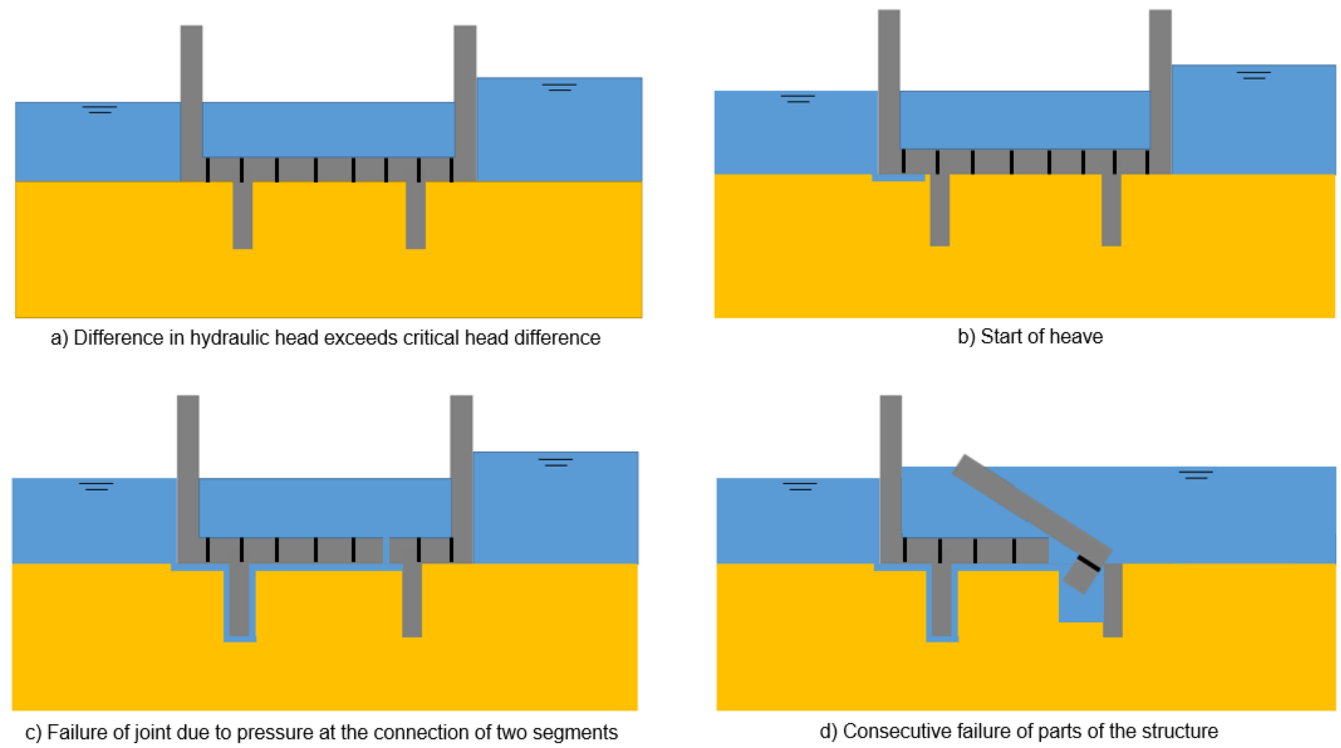

Figure 35 Consecutive failure of parts of the structure. Sub-figure a shows the joints (black lines) between the different segments. Sub-figure b indicates the start of erosion; heave. In sub-figure c failure at one of the joints is shown; the structure has lost connection now. Sub-figure d shows a possible consequence of consecutive failure of parts of the structure

7.2.4 Failure at the connection of the structure

Where the previous three types of structural failure focused on the structure itself, it is also possible that structural failure occurs at the connection of the structure to the surrounding dike. Especially for over dimensioned structures this might be a relevant type of structural failure (Delhez, 2020). The larger the needed seepage path length, the larger the resistance against piping is. Relatively long hydraulic structures do have a large horizontal seepage length. In addition, the length of the vertical seepage screens needs to be added twice to this horizontal seepage length if the analysis is focused on piping with vertical segments. For these cases it is more likely that erosion starts next to the structure instead of underneath the structure. The difference between these two types of erosion is shown in Figure 36.

When piping next to the structure starts, it might cause failure in the end at the connection between the hydraulic structure and the surrounding dike. When the erosion continues long enough (hence when the hydraulic condition is met long enough to let the pipe make a connection between the upstream and downstream side of the structure) a continuous flow of water could in the end exist next to the structure. It then fails to fulfil its water retaining function. Furthermore, erosion at the interface of the structure and the dike might lead to the loss of stability in lateral direction.

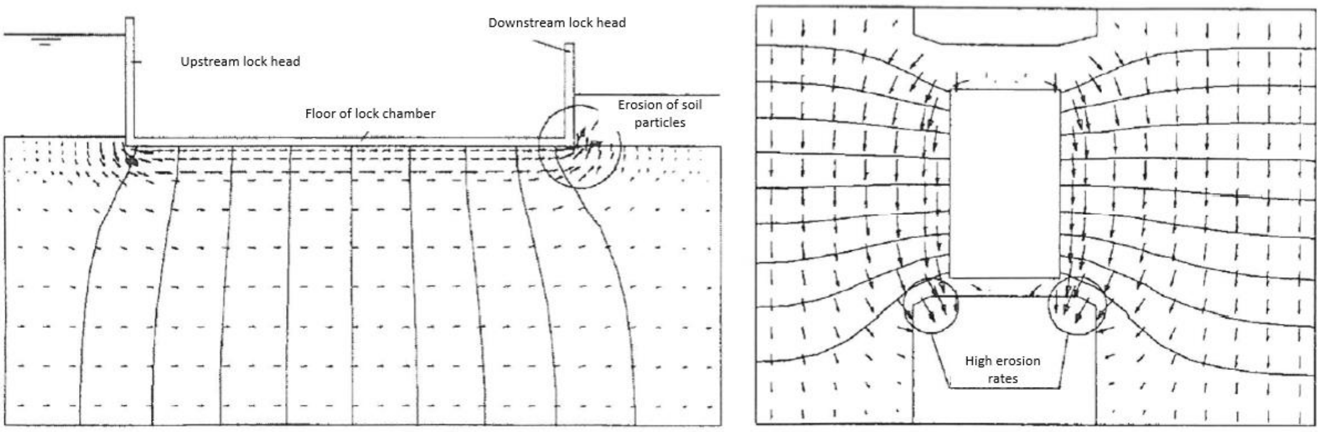

Figure 36 On the left erosion underneath a hydraulic structure is shown ('onderloopsheid'), on the right erosion next to a hydraulic structure is shown ('achterloopsheid') (source: (Van Bree, 2015))

Figure 37 shows a possible seepage path next to a hydraulic structure with a horizontal seepage screen. The variables that are important in such a case are the following:

- Hydraulic load;
- Soil characteristics of surrounding dike;
- Condition of connection horizontal seepage screen.

As mentioned before the difference in hydraulic head over the structure needs to exceed the critical value long enough before piping occurs. The more the occurring head difference exceeds the critical head difference, the faster the pipe develops. The soil of which the dike consists determines partially the resistance against erosion. The smaller the hydraulic conductivity, the larger the resistance. Finally, the condition of the connection between the hydraulic structure and the horizontal seepage screen is of importance. When this connection is in a bad condition the erosion could bypass the seepage screen. This would result in a smaller seepage length leading to a smaller resistance against piping next to the structure.

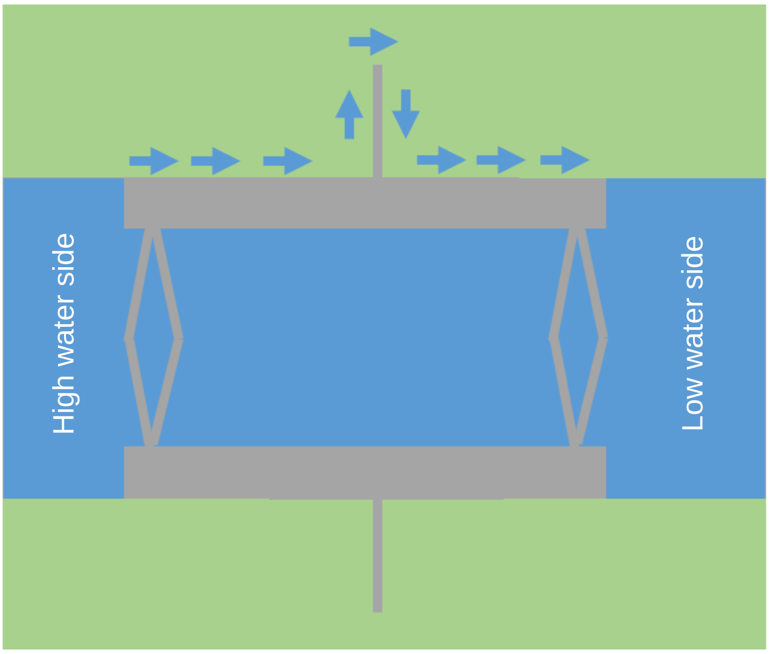

Figure 37 Possible seepage path next to a hydraulic structure at the connection with a surrounding dike given a good connection of the horizontal seepage screen to the hydraulic structure

7.3 Influence of geometry and specifications of hydraulic structures

Whether piping is likely to occur depends on multiple variables. In this thesis it was found that the geometry of the hydraulic structure plays a large role in the piping process. The case study contains a relatively long structure (large length to width ratio). This makes the resistance against piping high. In other words, soil erosion along a large seepage length[21] is needed before piping occurs. Also, the occurring head difference at the exceedance frequencies of the limit values is not within the range of the critical head difference. Therefore, the conclusion that the structure is safe against piping (category I_V) could have been drawn after the calculation of the node describing the geotechnical part of the event tree.

This does however not imply that the nodes describing the remaining strength of the structure (structural failure and breach formation) are not important. When considering other types of hydraulic structures that have a smaller length to width ratio (e.g. coupures, or smaller chamber locks used for recreational sails) the probability for the occurrence of piping may be more in the range of the limit values. Then, the calculation of the probability on the occurrence of structural failure and breach formation are important (Delhez, 2020). Inclusion of the potential outcomes of these sub-events may have a large impact on the assessment result.

The main question is whether it becomes useful to perform a full probabilistic assessment following the full event tree approach. This question can be answered checking a certain condition that is linked to the length to width ratio of the hydraulic structure that is being assessed. This condition can be called the critical length to width ratio ($(L{:}W)_{critical}$)[22]. If the actual length to width ratio is smaller than or equal to the critical value the assessment needs to be continued after calculation of the probability of occurrence of piping (phase A of the event tree). If the actual length to width ratio is larger than the critical value the calculation can be concluded after calculation of the probability of occurrence of piping. This is shown in the conditional statement below.

$$Assessment\ of\ piping\ at\ hydraulic\ structures \begin{cases} Continue\ after\ piping\ node\ if\ L{:}W \leq (L{:}W)_{critical} \\ Conclude\ after\ piping\ node\ if\ L{:}W > (L{:}W)_{critical} \end{cases}$$

Determining such a critical length to width ratio is out of the scope of this thesis. However, it may be useful to determine this value in future research when more primary flood defenses have been assessed. By 2023 all primary flood defenses need to be assessed, so all assessment results are available by then. This makes it possible to have access to all assessed primary flood defenses, resulting in more available data to determine the critical length to width ratio. With the data the critical value of the length to width ratio can be approached by narrowing down an upper and lower limit. These upper and lower limit values can be derived from available case studies.

7.4 Data versus expert knowledge

Two important topics for this thesis are data availability and expert knowledge. During the calculation of the failure probability of the Marksluis due to piping it became clear that not all sub-events could be represented by models. Especially, the part of the event tree describing the remaining strength of the structure after piping has occurred lacks applicable models and data. Therefore, expert knowledge was collected to still be able to provide an estimation of the probabilities of the sub-events describing this remaining strength.

It may be questioned whether expert knowledge is as valuable as models leading to a value for a probability. What should be remembered is that models are always a simplified representation of reality. By definition they do not perfectly resemble reality. For the study performed in this thesis where models are lacking expert knowledge is valuable. Especially when multiple experts can be involved a structured expert judgment can be performed leading to a description of the failure probability, too.

An option to perform structured expert judgment is to use the classical model of R. Cooke. This model works with a performance-based weighting (Bedford & Cooke, 2001). This means that the weight of the

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21 The seepage length consists of the length of the structure as well as the length of the seepage screen(s) if present.

22 Please note that other involved variables (soil composition and hydraulic loads) also have an influence on the probability of the occurrence of piping. A critical length to width ratio therefore only holds for comparable soil composition and hydraulic loading conditions.

answers to questions provided by the experts is weighted according to their performance. Their performance is a combination of the accuracy of their answers (called the calibration score) and the distribution of their given answers (called the information score) (Bedford & Cooke, 2001). To determine these scores questions must be composed to which the analyst knows the answers. The questions are so-called seed questions. The experts answer the seed questions by providing what they think the appropriate 5^{th}, 50^{th} and 95^{th} percentiles are (Cooke & Goossens, 2008). The more narrow-banded their answered distribution is given the true value of the seed variable is located within the answered distribution, the higher their performance score. A higher performance score leads to a larger weight to the given answers by that expert on the follow-up questions. These follow-up questions involve the values of variables that are not known yet (e.g. due to lack of data).

The classical model could be applied to overcome the problem of data unavailability. However, the other problem of model unavailability cannot be fixed by application of structured expert judgment. It is therefore important to first know what variables are involved in a description of the remaining strength of a hydraulic structure given piping has occurred. Then, these variables could be quantified by application of the classical model.

For now, a combination of models and expert knowledge can be used to determine the failure probability due to piping as described in this thesis. Section 8.2.4 elaborates on this by recommending the expansion of knowledge of the failure mode piping at hydraulic structures.

7.5 The current WBI 2017 framework

This thesis has focused on the assessment of the failure mechanism piping at hydraulic structures. Therefore, the assessment procedure according to the WBI 2017 has been studied. During this study some remarks were placed at the proposed assessment procedure. Specifically for the failure mechanism piping at a hydraulic structure a remark is the uncertainty in available calculation methods. According to (Rijkswaterstaat, 2017b) this failure mode can be assessed in a semi-probabilistic way. However, it turned out that this is not the case and that the actual assessment procedure does not involve a semi-probabilistic calculation (Rijkswaterstaat, 2019).

Next to this comment also other more general remarks about the assessment procedure described by the WBI 2017 are posted. These remarks are either questions about the written approach and assessment strategy or potential challenges during an assessment of a primary flood defense system. The notes are presented in Table 23. For each note the corresponding section of the original WBI 2017 Bijlage I Procedure (Rijkswaterstaat, 2017b) is named.

Table 23 Critical notes for the assessment procedure prescribed by WBI 2017

Section of 'Bijlage I Procedure' document (Rijkswaterstaat, 2017b)	**Critical note**	**Raised question**
1.2	Sometimes the manager of the involved flood defense system may decide whether the assessment is based upon the warning limit value of the lower limit value. From then on the chosen value is named the norm. This can have an influence on the obtained assessment result and it may not be clear what is meant with the norm.	Should it be prescribed when the warning limit value or the lower limit value must be used as the norm?
2.3.2	When performing a custom test, no specific guidelines are present on how to perform that test. Only suggestions for in depth analyses are given. In this way, large differences between different assessments of the same dike reach might arise.	Is there a way to have a more uniform method of assessment when using the custom test?
3.2.1	In the current assessment round (2017-2022) it is an option to include projected maintenance in the coming years as if it is completed. However, in this way the scenario that maintenance cannot be finished by 2022 is not considered.	Should the uncertainty that projected maintenance work cannot be completed before the assessment round ends (December 2022) be included in the assessment?
3.2.2	The simple test consists of questions that must be answered to decide whether a more detailed test is needed. These questions differ per failure mode.	Are the questions covering every aspect to be able to decide to conclude the assessment with a simple test?
3.2.3	The detailed test per dike reach uses an iterative approach. In this way a more accurate assessment result can be obtained after multiple iterations.	What level of detail is sufficient to stop the iteration process?
3.2.7	A custom test can consist of many different assessment strategies. One of these strategies may be a deterministic approach.	Is a deterministic approach as custom test sufficient to obtain a reliable result?
3.3	The boundaries between different categories of assessment results (both in terms of dike reaches and dike trajectories) are sometimes stated with a factor 1/30.	What is the reasoning behind this factor of 1/30?
3.3	For the assessment results of failure mechanisms of dike reaches an extra category is introduced. This category may be used when no assessment result has been found yet.	What advice is given when no assessment result can be found (yet)?
Overall	There is an influence of the knowledge and experience of the manager involved in the assessment procedure.	Can this influence lead to subjective assessment results?

8. Conclusions and recommendations

This chapter treats the conclusions and recommendations that can be drawn from the research described in this thesis. In section 8.1 the conclusions are presented. It provides an answer to the main research question of this thesis. It also explains how the different posed sub-questions contribute to this answer. Section 8.2 discusses recommendations for further research.

8.1 Conclusion

The goal of this thesis is to find an assessment method that results in a failure probability of the failure mechanism piping at hydraulic structures by using an event tree approach. This failure mode cannot be assessed in a probabilistic manner yet. Due to the lack of a (semi-)probabilistic approach this failure mechanism is still assessed according to calculation rules from the 'Wettelijk Toetsinstrumentarium 2011' (WTI2011). For that reason, the current assessment method might result in conservative results. Furthermore, the assessment of time dependent failure mechanisms (such as piping) can be improved.

To obtain a failure probability for the failure mechanism piping at hydraulic structures, the main research question posed in the introduction of this thesis was as follows:

'How can the current WBI 2017 assessment method of the failure mechanism piping at hydraulic structures be altered to obtain a failure probability as the assessment result?'

To get to know why the current assessment strategy does not yield a failure probability as outcome of the analysis the assessment procedure of the WBI 2017 has been studied. From the study on the WBI 2017 assessment procedure it is concluded that the failure mechanism piping at hydraulic structures is a group 4 failure mode. This means that the calculation method does not result in a failure probability for that failure mechanism. This is because no parametric distributions of variables are included in the assessment, hence a probabilistic analysis is not possible. The current assessment method according to the WBI 2017 is based on the water level corresponding to the exceedance frequency of the applicable norm for the dike trajectory in which the hydraulic structure is located. This results in only two possible outcomes as safety judgment categories, namely II_V (satisfies the norm) or V_V (does not satisfy the norm).

To know what distributions of variables are needed in order to perform a probabilistic analysis the failure mode piping at hydraulic structures is studied in detail. From this study the applicable geotechnical models have been found to be Lane, Bligh, Sellmeijer and the heave model.

To find a systematic approach to describe piping at a hydraulic structure, a comparison between two applicable methods was made. The first method uses a fault tree approach whereas the second method makes use of an event tree approach. These two methods are distinguished by the logic they use. An event tree uses forward logic whereas a fault tree uses backward logic. The event tree method has been used in this thesis, because a single event tree can cover multiple consequences hence providing a full overview of the failure mechanism. Furthermore, the event tree method supports to get more insight in the sequence of sub-events that need to happen before failure due to piping happens.

To be able to construct an event tree that describes piping at a hydraulic structure a case study has been defined. The case study contains a chamber lock named the Marksluis, which is a primary flood defense located in dike trajectory 34-1. After schematization of this chamber lock three event tree variants were constructed. The first variant provides a qualitative overview of the sub-mechanism that involves piping at the chamber lock. This variant is the most extensive event tree. Due to the size of the event tree it is rather complex to quantify all involved probabilities. Hence, the other two variants are more concise while still providing a realistic representation of the failure mechanism. These two (smaller-sized event tree) variants are meant for quantitative analyses. The two smaller variants focus on heave and backward erosion, respectively. A flow chart states in what order these two event tree variants can be used and with what outcome of the probabilistic analysis the assessment can be concluded.

According to the flow chart event tree variant 3 can be used to start the assessment. This event tree variant distinguished two main parts describing the failure mechanism. The first part involves the geotechnical part covering the erosion of the soil due to piping. It contains one node that represents the occurrence of piping. After performing the probabilistic calculations of the first node an extremely low probability ($P_{piping} = 1.3 * 10^{-26}$) for the occurrence piping was found. The probabilistic methods used for these calculations are the First Order Reliability Method (FORM) and the Monte Carlo method. Both methods resulted in the same order of magnitude for the probability of the occurrence of piping.

The second part of the event tree, which covers the remaining strength of the hydraulic structure contains two nodes. The first node involves the structural failure of the hydraulic structure given piping has occurred. The second node described the formation of a breach in the dike trajectory once the structure has failed. Models to calculate the probabilities of occurrence for these sub-events are non-existent. With the help of expert knowledge and literature the probabilities for these nodes have been quantified. In this case study they do not have a big influence given the extremely small probability of occurrence of piping. However, these conditional probabilities have a larger influence when the probability describing the occurrence of piping is in the order of the requirement set by the norm.

The method applied in this thesis, where an event tree is used to describe the piping failure mechanism has shown that it is possible to obtain a failure probability as assessment result. Therefore, it is also possible to assign each of the available safety judgment categories to the assessed hydraulic structure for the failure mechanism piping instead of just two categories. Another advantage of the application of an event tree approach is that it becomes easier to think of what sub-events must be assessed. By implementing an event tree approach in the statutory assessment tool it becomes possible to assess piping at hydraulic structures in a probabilistic manner. This is also possible for other failure modes that cannot be assessed in a probabilistic manner yet.

8.2 Recommendations

This section contains recommendations that are based on the discussion (chapter 7) and conclusion (section 8.1). In total four recommendations are provided. Some of these recommendations have a practical nature, others are recommendations for further research.

8.2.1 Possible improvements of WBI 2017

This thesis shows that an event tree approach can be used to quantify the probability of failure of hydraulic structures due to piping. In the current WBI 2017 assessment method this failure probability cannot be quantified. Therefore, only two assessment categories (II_V and V_V) are possible as assessment outcome. When an event tree based approach is added to the assessment procedure as detailed test it becomes possible to quantify the failure probability of hydraulic structures due to piping, allowing the assessor to assign each of the available assessment categories (I_V to VII_V). For this reason it is recommended to add the probabilistic assessment procedure for the assessment of piping at hydraulic structures as described by the flow chart shown in Figure 24. This flow chart can be added to the detailed test possibilities for piping at hydraulic structures of a subsequent version of the WBI.

Another possible improvement is the inclusion of a criterium based on the length to width ratio of the hydraulic structure that needs to be assessed. As mentioned in section 7.3 such a criterium can be used to determine whether the assessment on the remaining strength of the structure needs to be done after quantification of the probability of occurrence of piping. Adding this criterium to the a next version of the WBI becomes valuable once the critical length to width ratios are determined (see section 7.3).

The assessment method of the failure mechanism piping at hydraulic structures is not the only assessment method that can be improved. Also, the assessment of other failure modes that is not done in a probabilistic manner yet can be improved by implementing an event tree based approach as detailed test.

8.2.2 Increasing the number of measurements at hydraulic structures

As described in section 7.4, during this thesis it was discovered that data that could be used to assess the involved probabilities was often non-existent. By increasing the number of measurements at hydraulic structures this problem can be minimized. For example, monitoring wells can be installed to obtain data on the behavior of the groundwater (Delhez, 2020). With these data time series analyses can be performed.

This makes it easier to directly use the heave model, describing the initiating event of piping at hydraulic structures.

Another example is the installation of hydrostatic pressure sensors. By installing these sensors during the construction phase of hydraulic structures they can later be used to obtain data on the piezometric head underneath the structure. This can be an early warning for potential structural failure due to piping.

8.2.3 Application of an event tree approach during design

By using an event tree approach during the design of a hydraulic structure all possible failure paths can be mapped. Application of an event tree approach during the design phase urges the designers to think systematically about what potential pathways of a flood defense could be. This pictorial overview can be used to discuss what the weakest link(s) in the design is (are). In this way it can be decided to apply extra safety measures to prevent unnecessary changes to the design when the structure has already been built.

8.2.4 Expanding knowledge on the remaining strength of hydraulic structures

As mentioned in section 7.2 a model describing the structural failure of a hydraulic structure as a consequence of piping is non-existing. One could argue that it is not needed to do research on such a model following the low probability calculated for this case study. Furthermore, if the probability for the occurrence of piping might be in the range of the limit value derived from the norm one could easily decide to apply larger dimensions for the seepage screens to overcome this problem already in the design phase. This might be more economical beneficial compared to conducting a research on the conditional structural failure.

However, it is still interesting to research this conditional structural failure. By doing this research knowledge on how a hydraulic structure might fail once piping has occurred can be expanded. Furthermore, by conducting experiments relations between involved variables that cause the conditional structural failure might be found leading to a model that could be used to assess this sub-event in the future.

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Appendices

A. Flow chart detailed test per dike reach

The schematization of the detailed test is given in Figure 38. In this flow chart questions about the orientation of seepage paths can be answered. The given answer thereby determines which model to use for the assessment. For more information regarding the detailed test please refer to section 3.2.2.

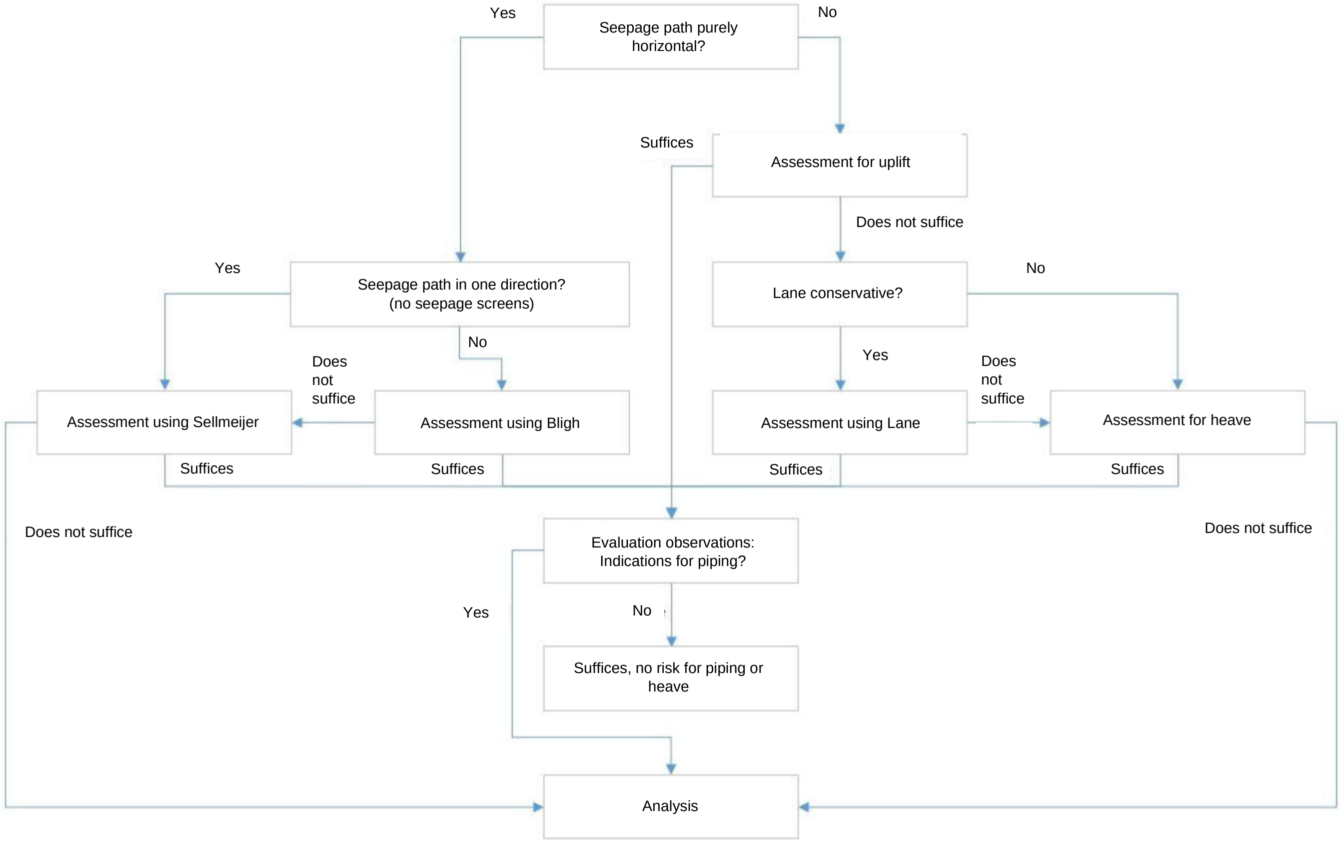

Figure 38 Flow chart explaining the detailed test per dike reach for piping at a hydraulic structure (source: (Rijkswaterstaat, 2019), translation own courtesy)

B. Overview of water systems around the Marksluis

Figure 39 shows an overview of the surrounding water system of the Marksluis. As explained in section 5.1 on the west side of the Marksluis, the Wilhelminakanaal is connected to the Amer, which is connected to the Bergsche Maas. On the east side of the sluice the Markkanaal is located. The Markkanaal is connected to the Volkerak by the Mark and the Dintel. The location of the Marksluis is highlighted by the red circle in Figure 39.

Figure 39 Water system around the Marksluis (source: (Google Maps, 2020))

C. Parameter determination of Gumbel distribution

As explained in section 5.3.1 the scale parameter and location parameter need to be calculated to find the Gumbel distribution describing the annual maximum water level at the Marksluis. This can be done with formulas [27] and [28] (Jonkman, et al., 2017):

$$F_{exc,H}(h) = 1 - \exp\left(- exp\left(-\alpha(h-u)\right)\right) \qquad [27]$$

$$\frac{F_{exc,H}(h)}{10} = 1 - \exp\left(- exp\left(-\alpha((h+h_{dec})-u)\right)\right) \qquad [28]$$

Where:

$F_{exc,H}$	$[-]$	The survival (or reliability) function of the water level h
h_{dec}	$[m]$	The decimate height

This set of equations can be solved with respect to the scale parameter and the location parameter. The scale parameter is determined first. From the set of equations [27] and [28] formula [29] can be derived.

$$\alpha = \frac{z\left(F_{exc,H}\right) - z\left(\frac{F_{exc,H}}{10}\right)}{h_{dec}} \qquad [29]$$

$$z\left(F_{exc,H}\right) = \ln\left(- ln\left(1 - F_{exc,H}\right)\right) \qquad [30]$$

The decimate height is defined as the increase in water level that reduces the exceedance probability by a factor 10 (Bouw, 2008). To determine the value of the scale parameter, the exceedance probabilities of both the warning limit value and the lower limit value for dike trajectory 34-1 according to (Waterwet, 2020) are needed. These are presented in section 5.1. The decimate height can be determined by making use of the map presented in Figure 41 in Appendix D. At the location of the Wilhelminakanaal the map of Figure 41 indicates that the decimate height is in the range of 0.21 m – 0.40 m. For the sake of simplicity an average value of 0.305 m is used as the decimate height. Table 24 shows the calculated value for the scale parameters of the Gumbel distribution.

Table 24 Calculation of the scale parameters of the Gumbel distribution

	Variables					
Limit values	F_{exc} $[years^{-1}]$	$F_{exc}/10$ $[years^{-1}]$	$z(F_{exc})$ $[-]$	$z(F_{exc})/10$ $[-]$	h_{dec} $[m]$	α $[m^{-1}]$
Warning limit value	1/1,000	1/10,000	-6.9073	-9.2103	0.305	7.551
Lower limit value	1/300	1/3,000	-5.7021	-8.0062	0.305	7.554

The second parameter of the Gumbel distribution is the location parameter. This parameter can be determined by making use of the set of equations [27] and [28] again. From this set of equations formula [31] can be derived:

$$u = MHW + \frac{1}{\alpha} z(F_{exc}) \qquad [31]$$

Where:

MHW[23]	$[m + NAP]$	Water level corresponding to the exceedance probability (F_{exc})

[23] MHW is an abbreviation for ‘Maatgevend HoogWater’ in Dutch. It literally translates to normative high water in English.

To get the design water levels that correspond to the exceedance probabilities it is helpful to construct an exceedance graph (in Dutch: 'frequentielijn') (Diermanse, 2016). This graph shows the relation between the value of the water level and the exceedance frequency. The software package 'Hydra-NL' contains a database with which the water levels and their corresponding exceedance probabilities can be calculated (Helpdesk Water, n.d.-b). The software can also construct an exceedance graph for the water levels at primary flood defenses.

Within the Hydra-NL software package multiple databases can be used. The database containing data on the water levels at the Marksluis is 'WBI2017_Benedenrijn_34_1_V04'. Within the database multiple points are defined where the water levels and corresponding exceedance frequencies can be calculated. The point called '034-01_0004_9_AM_km0250' is closest to the Marksluis (located 60 meters upstream of the sluice). Therefore, this point has been chosen to perform the analysis on. The output of the Hydra-NL calculation is the exceedance graph as shown in Figure 40.

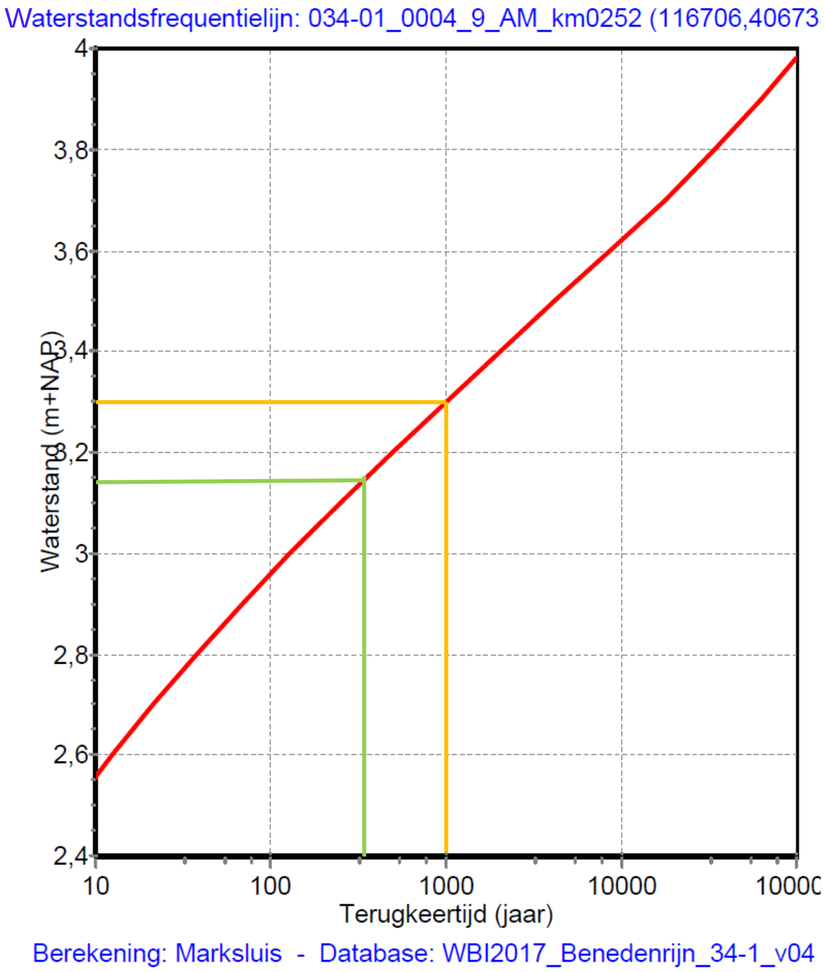

Figure 40 Exceedance graph for the water level at the upstream side of the Marksluis (Wilhelminakanaal side). The horizontal axis shows the return period in years and the vertical axis shows the corresponding water level in m +NAP. The green line indicates the water level corresponding to the lower limit value of 1/300 years. The yellow line indicates the water level corresponding to the warning limit value of 1/1,000 years

The water levels corresponding to the exceedance frequencies of the limit values can be deducted from the exceedance graph of Figure 40. The values for the water levels corresponding to the exceedance frequencies are presented in Table 25.

Table 25 Normative high water levels corresponding to the exceedance frequencies that describe the norm of dike trajectory 34-1

	Exceedance frequencies and water levels	
Limit values	$F_{exc}\ [years^{-1}]$	$MHW\ [m + NAP]$
Warning limit value	1/1,000	3.299
Lower limit value	1/300	3.127

With the normative high water levels now known, the location parameter, u, can be determined. Table 26 shows the calculated values for the location parameters of the Gumbel distribution.

Table 26 Calculation of the location parameters of the Gumbel distribution

	Variables					
Limit values	$MHW\ [m\ +NAP]$	$F_{exc}\ [years^{-1}]$	$z(F_{exc})\ [-]$	$\alpha\ [m^{-1}]$	$1/\alpha\ [m]$	$u\ [m]$
Warning limit value	3.299	1/1,000	-6.9073	7.551	0.132	2.387
Lower limit value	3.127	1/300	-5.7021	7.554	0.132	2.374

With this calculation the Gumbel distribution $(Gumbel(\alpha; u))$ of the normative high water levels on the upstream side of the Marksluis is known for both the warning limit value and the lower limit value. For the warning limit value the distribution is $Gumbel(7.551; 2.387)$ and for the lower limit value the normative high water levels are distributed according $Gumbel(7.554; 2.374)$.

D. Map of decimate heights across the Netherlands

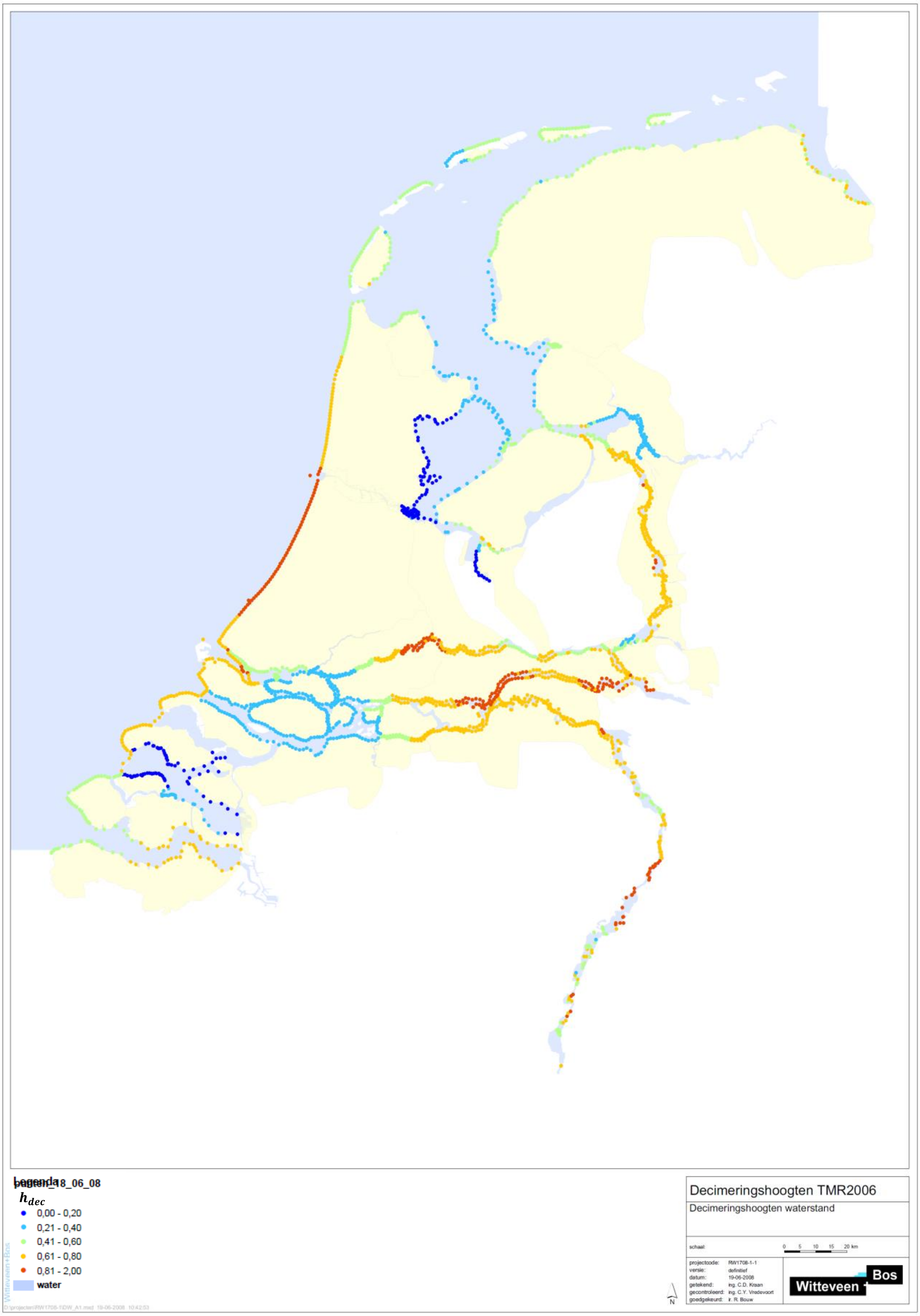

Figure 41 Decimate heights (h_{dec}) across the Netherlands for water levels (Kraan, 2008)

E. Graphical representation of tilting

Figure 42 shows a graphical representation of structural failure of a hydraulic structure. The structure has tilted as a consequence of piping. Figure 42e shows that the foundation of the structure is gone due to the erosion process. It can therefore tilt in clockwise direction, leading to structural failure. However, from Figure 42f it can be observed that no breach has formed in the involved dike trajectory. This example therefore shows that structural failure does not automatically mean that a breach is formed.

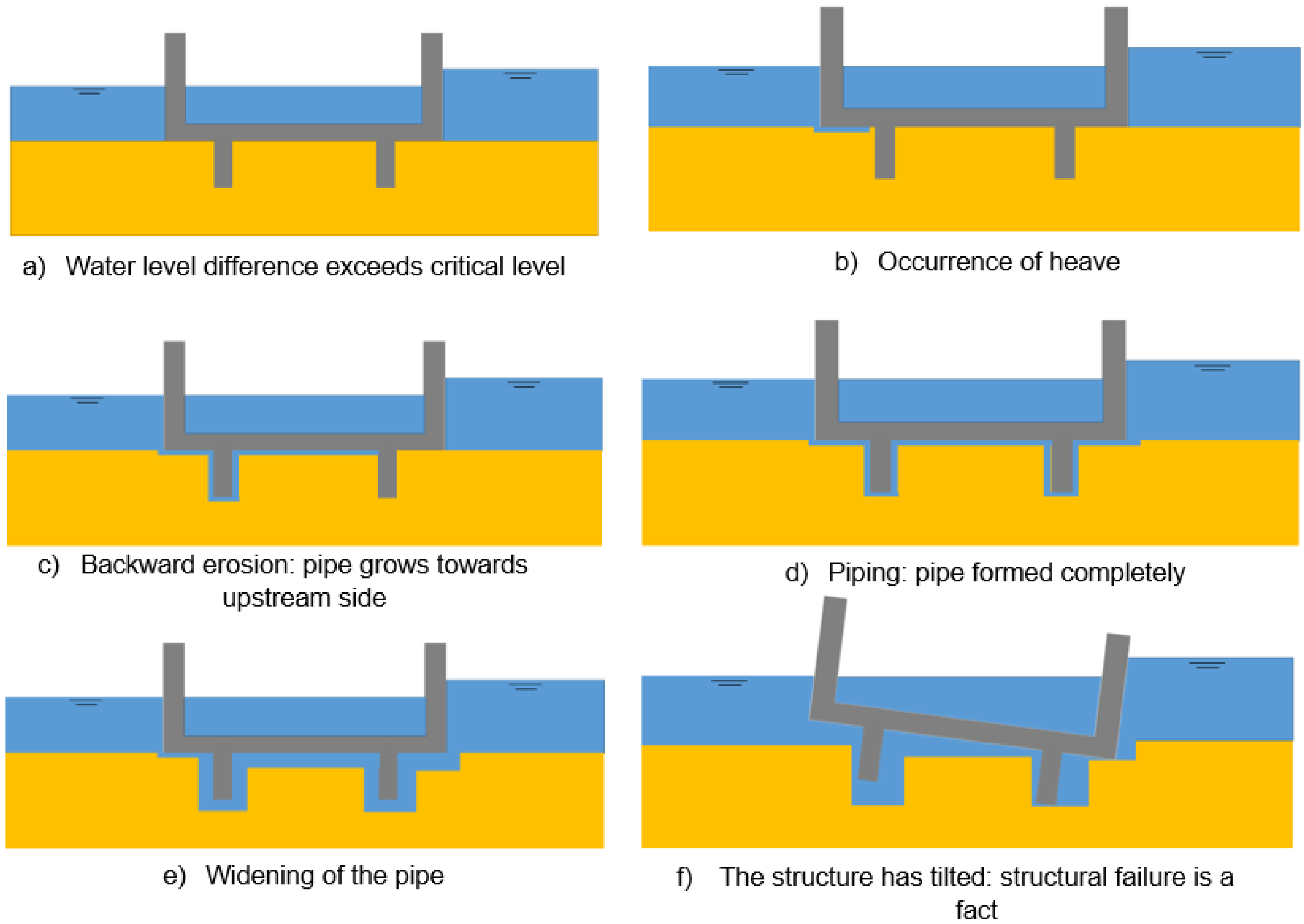

Figure 42 Graphical representation of structural failure of a hydraulic structure as a consequence of piping. The structure has tilted but no breach in the dike trajectory has formed

F. Glossary of terms

This thesis contains references to literature that is available in Dutch only. This glossary contains an overview of the translations of key terms from this literature.

English to Dutch

Chamber lock	Schutsluis
Creep	Kruip
Custom test	Toets op maat
Decimate height	Decimeringshoogte
Detailed test per dike reach	Gedetailleerde toets per vak
Detailed test per dike trajectory	Gedetailleerde toets per traject
Dike reach	Dijkvak
Dike ring area	Dijkring
Dike trajectory	Dijktraject
Event tree	Gebeurtenissenboom
Exceedance graph	Frequentielijn
Failure[24]	Falen
Failure mechanism	Toetsspoor
Failure probability budget	Faalkansbegroting
Fault tree	Foutenboom
General filter	Algemeen filter
Hydraulic structure	Civiel kunstwerk
Liquify	Fluïdisatie
Length effect	Lengte effect
Lower limit value	Ondergrens
Phreatic level	Freatisch niveau
Piezometric head	Piëzometrisch niveau / stijghoogte
Piping with both horizontal and vertical seepage path elements	Onderloopsheid
Piping with solely horizontal seepage path elements	Achterloopsheid
Primary flood defense	Primaire waterkering
Safety judgment	Veiligheidsoordeel
Seepage screen	Kwelscherm
Shallow foundation	Fundering op staal
Simple test	Eenvoudige toets
Statutory Assessment tool 2017	Wettelijke Beoordelingsinstrumentarium 2017 (WBI 2017)
Warning limit value	Signaleringswaarde
Water law	Waterwet

[24] In this thesis failure means that a flood defense loses its water retaining function.