[SHIFTING DISCHARGE, ALTERING RISK]

An exploratory study to assess the impact of the discharge distributions upon the flood risk of the upper-Rhine area of the Netherlands.

Elsbeth Brandsma, August 2016

Shifting Discharge, Altering Risk.

An exploratory study to assess the impact of the discharge distributions upon the flood risk of the upper-Rhine area of the Netherlands.

By

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Cover: Aerial photograph showing the bifurcation point Pannerdensche Kop, September 1990.
Retrieved from: https://beeldbank.rws.nl/MediaObject/Details/45346.

Abstract

The largest river in the Netherlands, the Rhine, bifurcates in several branches. The distribution of discharge amongst these branches is fixed by policy. As these distributions directly determine the water levels along the downstream river branches, they are expected to be an important factor in the risk of flooding during high water events.

In this thesis, the impact of changing discharge distributions amongst the branches of the Rhine is investigated. The impact is measured in terms of risk; expressed in the expected damage in Euros per year.

A literature study revealed that the current distributions originate from the 18th century, when they were established through constructions at the bifurcation points. Since then, little changes have been made to these points.

Focussing on the upper river area of the Rhine, the risk of the current situation was calculated, using a numerical tool that was developed for this purpose. This tool calculated the water load based on the discharge statistics obtained from GRADE2015. The strength of the dikes along these branches was calculated from fragility curves, taking in to account the failure mechanisms overflow/overtopping, macro-stability, and piping. The total risk was calculated using the damage data from the VNK study.

Starting from the current situation, the distribution of discharges was changed, calculating the risk for various distributions. This analysis showed that the total risk could be reduced by 35% when the distribution at the IJsselkop is modified, and 10% when changing the distribution at the Pannerdensche Kop.

Although the accuracy of the tool was limited, due to incomplete data, the results of this study make it worthwhile to investigate this further as it is likely that the total risk will change for a different discharge distribution.`

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Glossary

English	Dutch	Abbreviation	Description
Amsterdam Ordnance Datum	Nieuw Amsterdams Peil	NAP	A water reference level which is used in the Netherlands.
Bar	(zand)bank		An elevated region of sediment that has been deposited by the flow of the river.
Bifurcation point	Splitsingspunt		The location where a single stream river separates into two or more separate streams which continue downstream.
Bleeswerk	Bleeswerk		Construction for levee protection braided out of twigs, before it was sunk.
Breach	Bres		Any loss of material such that water could or does pass through the structure.
Catchment	Stroomgebied		The area from which precipitation and groundwater will collect and contribute to the flow of a specific river.
Climate change	Klimaatverandering		Refers to any long-term trend in mean temperature, wind speed, drift rate and its consequences on the mean se level, wave height, rainfall etc.
Cross-section	Doorsnede		Vertical section of the levee perpendicular to the levee course/line. It includes outside and inside sections and is measured by surveying elevations with ranges across the levee from landside to riverside (CIRIA, 2013).
Cumulative Distribution Function	Onderschrijdingskans	CDF	A function which describes the probability that a variate X (H as water level) takes on a value less or equal to a number x (h in this study). Another term for non-exceedance probability function. (Weisstein, n.d.-a)
Decimation height	Decimeringshoogte	DH	The increment of the water level associated with an increase or reduction in exceedance probability by a factor 10 (VNK2 project office, 2012).
Delta	Delta/Riviermond		A landform resulting from the deposition of sediment carried by a river as the flow leaves its mouth and enters another water body.
Design discharge	Maatgevende afvoer		The discharge at Lobith which corresponds to a certain recurrence time set by design
Design water level	Maatgevend hoog water	MHW	The water levels along the Rhine branches at design discharge.
Dike	Dijk		Raised, predominantly earthen, flood protection structure In this study dikes are geotechnical works, also described as (earthen) levees or flood defence embankments.
Dike circle	Dijkring		System of dikes (or high grounds) surrounding a polder, protecting this polder against inundation (Delft University of Technology, n.d.)
Dike section	Dijk Vak (VNK)		A part of a dike segment with homogeneous strength and load properties.
Dike segment	Ringdeel (VNK)		A dike stretch with virtually the same consequence, regardless the location of the breach.
Discharge	Afvoer	Q	Water which is transported from a water system per unit time. Discharge from the Alpen Rhine is inflow for the lower rhine. Discharge is usually noted as the letter ‘Q’ and expressed in cubic meters per second [m^3/s].
Discharge statistics	Werklijn		The relationship between the discharge and an exceeding frequency (Bisschop & Huisman, 2011).
Failure	Falen		Inability to achieve defined performance threshold for a given function, in particular for flood defence.
Flood	Hoog water		When the water discharge at Lobith exceeds approximately 4,000 m^3/s and the water does not only flow in the minor bed, but also through the floodplains. A flood is often described by its probability of not being exceeded, its hydrograph, max discharge, duration and volume.
Floodplains	Uiterwaarden		Part of the riverbed which is flooded during high river run-off. If the discharge at Lobith exceeds 4,000 m^3/s or is approximately 12 meters + NAP, the floodplains start discharging water.
Flood protection programme	Hoogwaterbeschermings-programma	HWBP	Flood protection programme of the Netherlands, responsible for the prioritizing of the maintenance of the dikes.

Flood risk	Overstromingsrisico		A function of the probability that an event will occur and the consequences associated with that event (CIRIA, 2013).
FlorRis	Veiligheid Nederland in Kaart	VNK	Flood risk study of the Netherlands, executed in 2006. (Ministerie Verkeer en Waterstaat, 2005)
Fragility	Kwetsbaarheid		The likelihood of particular defence or system to fail under a given load condition. Typically expressed as a 'fragility curve', relating load to likelihood of failure.
GRADE	GRADE		Generator of Rainfall and Discharges Extremes: A study in which the discharge statistics are calculated (Hegnauer et al., 2014)
Groyne	Krib		Hydraulic structures that are perpendicular to the landside. In rivers it functions to keep the river navigable. Historically the core of these groynes existed of a construction braided out of twigs and filled with sand.
Inner dike	Binnendijks		The region that is protected by a dike, often the dry side of a dike.
Levee height	Dijkhoogte		Vertical measured difference between the landside levee toe and the highest point of the levee crest.
Leveed area	Dijkring		Area behind the levee that is not flooded, or which the flooding is reduced or delayed due to the levee/flood defence system.
Macro instability	Instabiliteit binnen talud dijk		Failure mechanism in which sliding plain becomes saturated and starts to slide. Eventually this leads to a complete collapse of the dike.
Meander cut-off	Bochtafsnijding		The formation of a new main channel through the breach of a meander bend, which connects the two closest parts of the bend. This causes the flow to abandon the meander and to continue straight downslope.
Minor bed	Zomerbed		The main channel of a river, used for means of transportation. The only part of the river discharging water when the discharge at Lobith is lower than approximately 4,000 m^3/s.
Overflowing	Overlopen		Passing of water over the top of a structure as a result of a water level higher than the crest of the structure.
Overtopping	Overtoppen		Passing of water over the top of a structure as a result of wave action, surge or wind. The water level in front of the structure is lower than the crest level of the structure.
Piping	Kwel door pijpvorming		The creation of flow channels within a levee or the underlying ground as a result of seepage and continuing internal erosion. Piping can lead to the development of bois or breaches.
Probability	Kans		Measure of the change that an event will occur. Typically defined as the relative frequency of occurrence of that event out of all possible events and expressed as a percentage with reference to a time period e.g. one per cent annual exceedance probability. (CIRIA, 2013)
Probability Density Function	Kansdichtheidsfunctie	PDF	The derivative of the cumulative distribution function.
Recurrence interval	Terugkeertijd		The average number of years between floods of a certain size is the recurrence interval or return period. The actual number of years between floods of any given size varies a lot because of the naturally changing climate.
Return period	Terugkeertijd		For a given parameter (e.g. water level), the mean duration between two events where this parameter was observed. Inverse of the probability that a given event will occur in any one year.
Risk	Risico		A measure of the probability and severity of undesirable consequences or outcomes.
Room for the River	Ruimte voor de rivier	RvdR	Project with the main goal of creating more space in for the river in order to reduce the flood risk.
Scoop groyne	Schephoofdt		A groyne which is located in a way that it influences the water flow.
Stochastic event	Stochastische gebeurtenis		Unpredictable event due to the influence of random variables.
Truncated discharge statistics	Werklijn met aftoppen		The relationship between the discharge and an exceeding frequency, keeping in mind the physical limitations of the upstream river.
Uncertainty	Onzekerheid		Lack of sureness about something, caused by a natural variability (inherent uncertainty) or incomplete knowledge (epistemic uncertainty).

Upper river area	Bovenrivierengebied		The river area in the Netherlands, fed by the River Rhine and Meuse, east of the 'line': Schoonhoven-Werkendam-Dongemond. The water levels in the upper river area are not influenced by the tide of the North sea (Vergouwe & Sarink, 2014).
Water defence line	Waterlinie		Military defence line that was designed to keep out invaders by the controlled flooding of a chain of inundation fields.
Water level	Waterstand		Elevation of still water level relative to a datum.
Weir	Stuw		Hydraulic structure that is built across a river to raise the water level, divert the water or controls its flow.

1 Introduction

The kingdom of the Netherlands is located in a river delta area, and is characterized by an extensive coastline. Due to various reasons, such as rainfall, storms, and melting snow in the Alps, water levels in rivers can exceed their average values. These events of high water, called *floods,* can lead to *inundations*: water flows over areas of land where it is undesired, such as farmland or inhabited areas. 59 % of the surface of the Netherlands is threatened by inundation from either sea or rivers. More specifically, 29% of the area is threatened by river flooding, according to Planbureau voor de Leefomgeving (2009). Sadly, it does not come as a surprise that the history of the Netherlands is filled with numerous disastrous inundations, causing great personal and financial losses. These inundations were not only from the sea, but also from the rivers. To better withstand floods, and reduce the occurrence and damage due to inundations, many measures were taken throughout history. At first, these measures were taken on a local scale, aiming to reduce personal or communal risks. As early as the 12^{th} century, water authorities were founded, coordinating flood prevention in a more integral way, and protecting larger flood-prone areas of the Netherlands (Van Til, 1979). With the establishment of Rijkswaterstaat in 1798, flood protection was organised nation-wide (Van de Ven, 1976).

Despite improved flood protection, a probability that an area falls victim to inundation still exists. As much as inhabitants of a certain area would prefer to eliminate the probability that their area gets flooded, there is no such thing as 100% safety. As such, the level of achievable protection is, and will always be, a trade-off between the acceptability of a certain probability of a flood to occur on the one hand, and the costs and feasibility of the protective measures on the other hand. The former can be quantified in terms of the *flood risk*, which can be expressed as a function of the probability that a flood occurs per unit of time, and the consequences in case of a flood (Vrijling et al., 1997; Vrouwenvelder & Vrijling, 1987).

Calculation of the flood risk for a certain position is possible when certain parameters are known, such as the probability of a certain water level at that location, and the (local) strength of the protective measures taken (CIRIA, 2013). Naturally, these calculations change when newer insights and data become available. Recently, new data on the strength of river dikes has become available, allowing a re-assessment of flood risks. This study will use the new data to evaluate the discharge distribution along the branches of the River Rhine, and investigate if an alteration to the current distribution of the discharge along the branches of the Rhine results in a decreased flood risk.

1.1 Problem description

Due to the high population of the Netherlands, flooding will have severe consequences in terms of casualties and financial losses. It is therefore of utmost importance to prevent flooding, or at least limit the probability of flooding. One way to achieve this is through proper management of the river discharges. One of the most prominent rivers in the Netherlands is the River Rhine. This river enters The Netherlands in the East, close to the Dutch town Lobith. In the Netherlands, the river splits in multiple branches; Figure 1 shows these branches and displays their Dutch and English names. The location at which the river splits is called a *bifurcation point.* The red lines in the same figure indicate the location of the main dikes.

The discharge distributes over the branches at ratios determined by the properties of these branches, such as the river widths and slopes. Note that these ratios vary with water level of the downstream boundary, such as the North Sea or the Lake IJsselmeer. The distribution of the water over the branches is also modified through man-made structures such as weirs and dams, and through influences of retention areas. These structures modify the distribution to a ratio, which is based upon simulations and historical data.

Changes in the rivers morphology, either through natural or human causes, will change the discharge distribution. Historically, the rivers were far more unstable in terms of morphodynamics (Kleinhans et al., 2013). Nowadays, new measures in the River Rhine are extensively tested upon their influence on the discharge distribution (Kroekenstoel, 2014), and are designed such that they do not change the discharge distribution at design water level. Even flexible spillways have been built close to the bifurcation points in order to compensate for uncertainties in the discharge distribution (Schielen et al., 2008, 2007). This is based upon the fact that the current bifurcation point, with the corresponding discharge distribution fixed in policy, has historically proven to be stable. However, future flood waves and circumstances, such as wind, washing out of dunes upon the riverbed, might be of a nature or strength that has not yet been encountered. The response of the bifurcation points, and with that the discharge distribution, on these events is therefore unknown (Geerse, 2013).

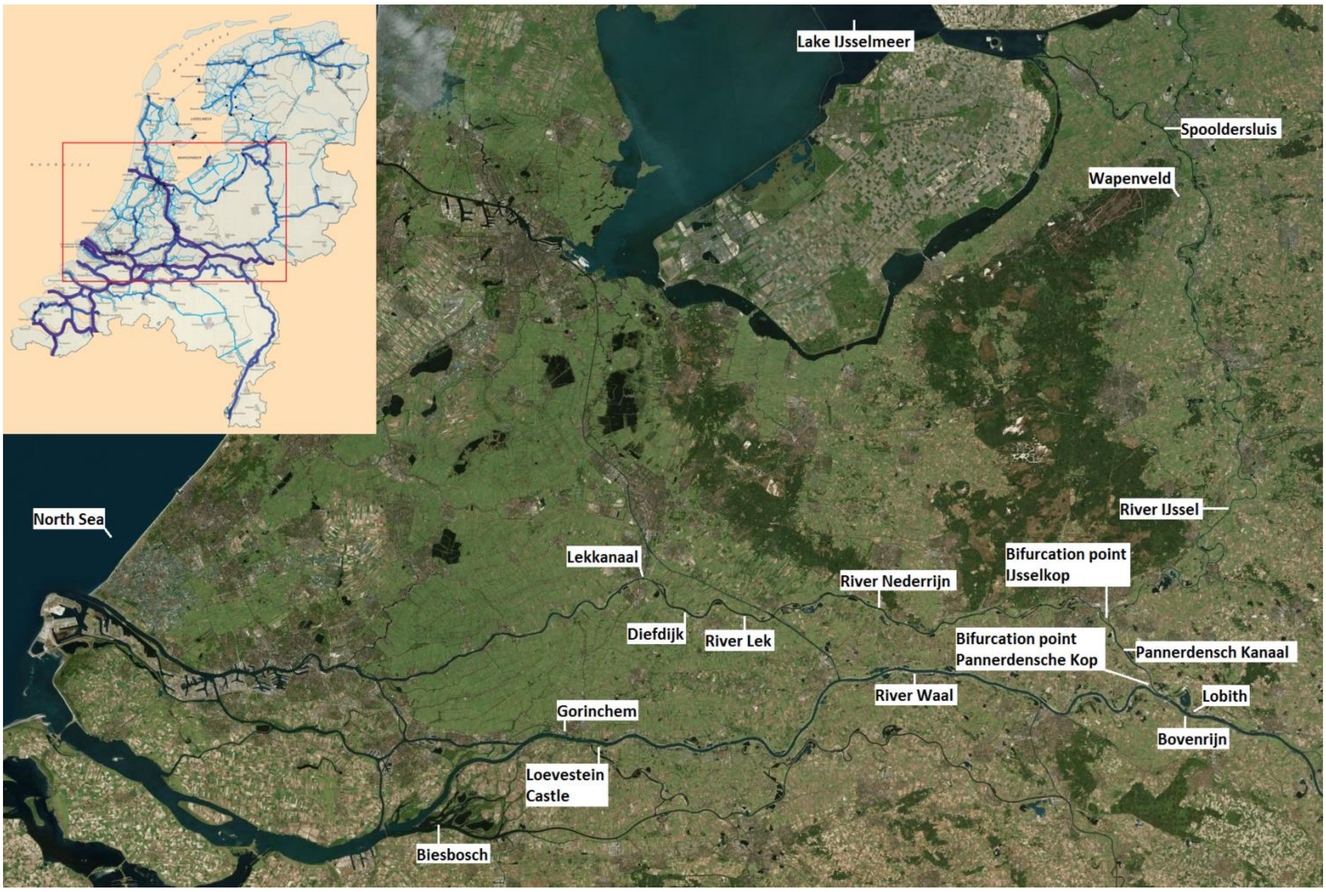

Figure 1: Reference map of the bifurcation points of the Dutch part of the River Rhine. Adjusted after 'Atlas van Nederland, deel 12: Infrastructuur. Vaarwegen.' (1984).

When the discharge distribution is altered, the water level and thus the loads on the dikes in the different branches change. These changes result in a different risk of inundation. Studies have been done in the past to evaluate the impact of different discharge distributions on cost efficiency (ten Brinke, 2013), however those studies mainly focus on either the failure probability at the design discharge (Ubbels et al., 1999) or the discharge at very low water levels, in times of drought (RIZA, 2005). Recently, the conditional failure probabilities (the risk of failure of a dike at a certain water height), of the dikes along the rivers Waal and IJssel have been measured more precisely. This resulted in new insights which can be used to investigate the cost-effectiveness of dike strengthening or broadening measures (Levelt et al., 2015; Van Rhee, 2013; Van Vuren et al., 2015) in the light of the new water safety policy (Ministerie van Infrastructuur en Milieu & Ministerie van Economische Zaken, 2016). These new water safety standards are based upon the flood risk approach for a fixed discharge distribution. Also the study about the flood risks of the Netherlands (VNK), driven by the

EU Floods Directive (European Commission, 2015), was done for the current *fixed* discharge distribution (Vergouwe, 2015). Amidst this strong emphasis on studies for fixed distributions, Arnold (2004) and Kok (2013) argued that water safety could potentially be increased by adopting an actively managed discharge distribution, rather than a fixed distribution.

1.2 Goal and research questions

This thesis assesses the change in total flood risk when an alternative discharge distribution is assumed along the Rhine branches. Utilizing the new conditional failure probabilities along the Dutch part of the River Rhine, the effect of the water level upon the flood risk is determined. This leads to the main goal of this research:

To investigate the impact of different discharge distributions over the Rhine branches on the total flood risk, determining whether it could be beneficial to change the discharge distributions, and re-evaluate the policy of the fixed distributions.

In order to achieve the objective, the following research questions have been posed:

- Due to which natural processes and human decisions and interventions became the bifurcation points and the discharge distributions as they are today?
- What is the flood risk of the current discharge distribution, expressed in Euros per year?
- What is the effect in terms of flood risk when the discharge distribution changes?
- How robust is the optimization for the uncertain factors in the calculation of the flood risk?

1.3 Scope and methods

The research questions posed above were answered through a historical study and a flood risk calculation.

- *The area of interest is roughly the upper river area (in Dutch: bovenrivierengebied) of the Rhine, i.e., there is no influence of the tide on the local water level of the river. This so-called upper river area is defined in section 3.2.1.2.*
- *Only primary dikes which are part of a dike section as defined by the Delta Programme were considered in this study: hydraulic structures and man-made water defences are not considered.*
- *2015 was taken as the reference situation: after the completion of the Room for the River projects but before the dike reinforcements of the 'HWBP' dike strengthening programme.*
- *The discharge statistics which are used as an input, were derived from the model GRADE (Generator of Rainfall and Discharges Extremes), known as 'GRADE 2015' for the situation 2015.*
- *A calculation method, based on the VNK calculation method was used to calculate the yearly probability of failure for a river section.*

1.4 Reading guide

In order to identify whether a change in the discharge distribution is beneficial, several steps are taken. First, in chapter 2, the establishment of the current discharge distribution of the Dutch Rhine branches is investigated. This knowledge is used in the same chapter to determine if (and how) the current distribution can be altered. Subsequently in chapter 3, the flood risk of the River Rhine area is calculated, which serves as the reference situation. In chapter 4, the impact of alternative discharge distributions on the flood risk is investigated, and compared to the reference situation. Then, in chapter 5, a sensitivity analysis is conducted to test the robustness of the flood risk calculation used in this study. The last chapter puts the findings in perspective, draws conclusions and answers the research questions, as well as giving recommendations for future work. And extensive appendix can be found at the end of this thesis, wherein more background information is provided.

2 History of the Discharge distribution over the Dutch Rhine branches

2.1 Introduction

This chapter reviews how the current discharge distributions along the branches of the Dutch part of the Rhine have been established. It provides a historical outline, tracing the establishment of the current bifurcation points and their respective discharge distributions. Subsequently, a description is given as to how the discharge distribution is determined by Dutch law. The last section of this chapter illustrates the functions of the River Rhine and the management of the river flow. This includes a highlighting of the uncertainties associated with changing discharge distributions.

The trajectory names of rivers change over time. The current names of the river branches are shown in Figure 1. As this chapter deals with current and past situations, the historical names of the (current) Dutch River Rhine from Spijk to Arnhem are featured in Figure 2. The bifurcation points of the Waal-Pannerdensch Kanaal and the Nederrijn-IJssel are called the Pannerdensche Kop and the IJsselkop, respectively.

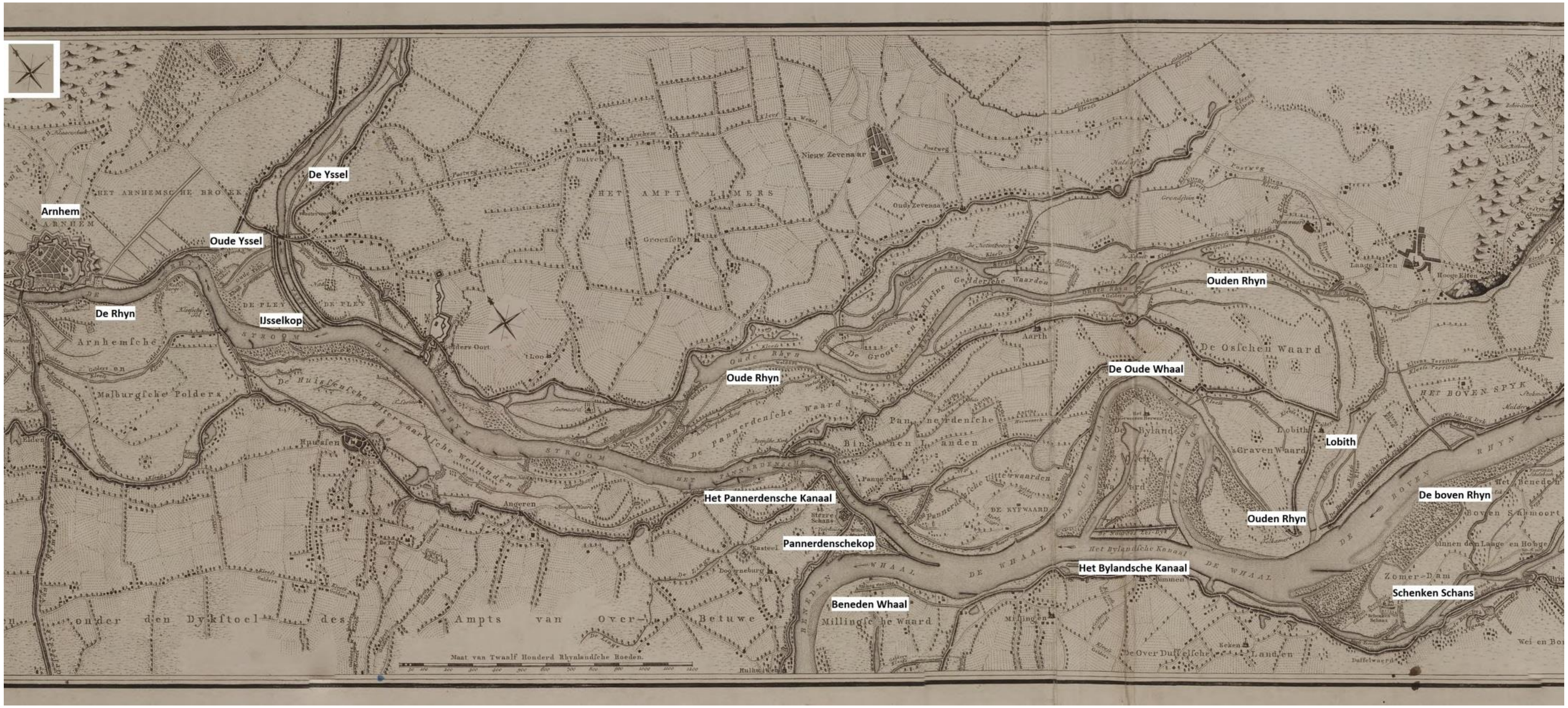

Figure 2: Part of the map 'Kaart van den Rhynstroom, van boven de stad Emmerik tot beneden de stad Arnhem'(Engelman, 1790).

2.2 History of the Dutch Rhine branches and their discharge distribution

The Rhine branches have shifted over time, not only through natural variation, but also as a response to human influences. The course of the Dutch part of the River Rhine has changed continuously in the last thousands of years, as illustrated in Figure 3. The light blue colour in this figure depicts the current river branches, while the red-to-green colour scheme depicts the riverbeds at times ranging from modern times to 7000 years ago.

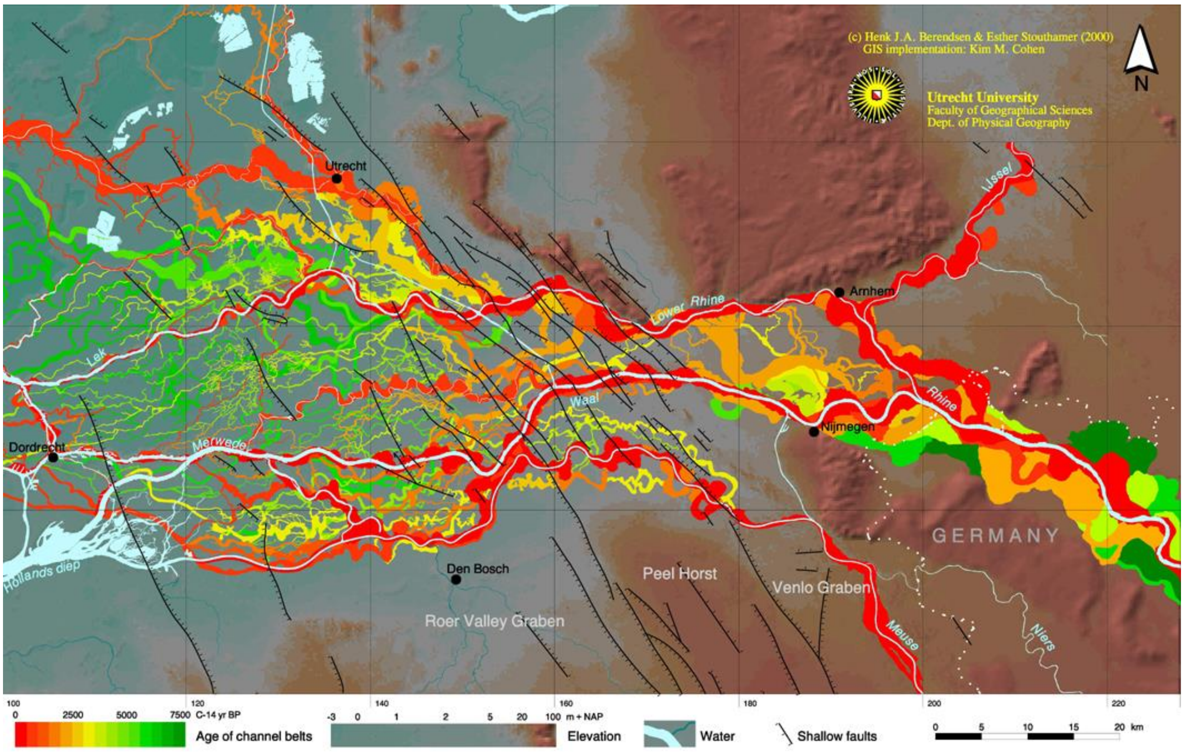

Figure 3: Age of Holocene channel belts in the Rhine-Meuse delta, the Netherlands (Berendsen et al., 2000).

Because the Netherlands is a densely populated river delta, flooding has always had severe consequences. Already in Roman times, human interventions have been applied to control flooding. For example, the Romans built a dam in the River Waal in order to prevent extensive floods in their north-western territory. By doing so, they diverted more water into the northern branch of the Rhine (Nederrijn-Lek). This diversion caused floods elsewhere on the Roman territory, and the current historical interpretation is that a channel was dug towards the IJssel to avoid these floods. In this way excessive water was directed towards the North, outside the borders of the Roman Empire (In de Betouw, 1787; ten Brinke, 2007).

In 1421 a notorious flood took place in the Netherlands: the Saint Elizabeth flood. This flood initiated the formation of the Biesbosch, a tide dominated pool tens of kilometres land inward from the Dutch coast, Figure 1. The flood also resulted in a change of the slope of the River Waal as compared to the Nederrijn-lek and the IJssel. It is believed that this flood caused the Waal to become the dominant discharging branch (Van de Ven, 2007).

Over the course of the 15th century, much sediment was deposited at the inlet of the Nederrijn branch, reducing the discharge to as little as 5% of the total flow in the Rhine. Often the Nederrijn did not carry any water at all. As a result of this, also the IJssel received a low amount of water from the Rhine; an undesired situation, as the river served multiple purposes. Apart from being a means of transportation, the IJssel was a fresh water supply, and an instrument to deter invading armies: in case of an attack, the dikes could be pierced to deliberately inundate an area to keep out the enemy's armies. An area inundated for this reason is called a *water defence line*. Low discharges and low water levels prohibited the use of this tactic. Low discharges also made the ports of cities along the IJssel inaccessible to merchant ships. This was detrimental to these cities as they gained much of their wealth from trade and (public) transport by ships. Amsterdam for example, being the centre of the international trade of the Republic of the Netherlands at that time, was hardly accessible via the Nederrijn, and ships had to make a detour via the Zuiderzee (now Lake IJsselmeer) (Van de Ven, 2007). On the other side, Amsterdam was also benefitting from the low waters of the River IJssel, as the trade in Asian goods was redirected from Deventer to Amsterdam.

From 1485 onward, many meetings were arranged, aimed to reach an agreement over the discharge distribution, but to no avail (Van Til, 1979). The first interference at a bifurcation point was established for military reasons: The bifurcation point of the Waal and Nederrijn was located at Lobith until approximately 1500, when it shifted and moved from Dutch territory to, (what is nowadays) Germany, named Schenkenschans. In the 17th century, a large part of the Southern Netherlands was conquered twice, once by the Spanish in 1629 and once by the French in 1672. These double invasions created willingness in the Southern provinces of the Netherlands to support a structural improvement of the discharge distribution.

This improvement encompassed the stabilization of the discharge distribution, safeguarding the functionality of the *water defence line*. However, this was a very difficult process, since the union of the Netherlands did not have the money nor the strength to decide themselves. Another major issue was the fact that all of the independent provinces had to cooperate. A difficult task, as the Netherlands in 1684 counted seventeen provinces.

On June 20th 1701, it was finally decided that a retrenchment had to be constructed between the Waal and the Nederrijn (Brunings et al., 1798; Ploeger, 1992). Since sediments mainly deposit in the inner bend of a river, this retrenchment was to be dug in the outer bend of the River Waal, to assure an inflow that would not be closed off by sedimentation. Although the shortcut was initially meant to be a retrenchment, it was decided in 1706 that this retrenchment should become the new main channel for the River Rhine. This new section, the Pannerdensch Kanaal, has been fully operational since the 14th of November 1707 (Van de Ven, 2007).

In 1711, the upper stream bifurcation point of the Waal and Nederrijn shifted and moved towards the West, (close to the village Spijk) and was then located a bit more to the East than the present location (In de Betouw, 1787).

On the 29th of July 1745 it was decided that the dikes near Spijk had to be reinforced. Along the Pannerdensch Kanaal, improvements were made so the Pannerdensch Kanaal would become the only channel towards the IJssel and the Nederrijn-Lek. The provinces of Holland, Utrecht (Utrech), OverIJssel (Overysel), and Gelderland (Gueldre) decided that the maximum discharge for the Pannerdensch Kanaal should be 1/3 of the Dutch inflow of the Rhine. If this was exceeded, a new conference could have been held to re-establish this arrangement (Van de Ven, 2007).

In 1745 it was decided that the 1/3 - 2/3 distribution of the discharges amongst the Waal and the Pannerdensch Kanaal had to determine the widths of the rivers: the Waal simply had to be twice as large as the Pannerdensch Kanaal (Hove van Gelderland, 1767).

Over time, several problems and challenges emerged, which led to agreements and alterations that could influence the discharge distribution. Appendix A provides a summary of the main decisions and corresponding actions that influenced the discharge distribution amongst the Dutch Rhine branches.

The discharge distribution that was decided upon in 1745 has been left largely untouched until today. In order to accomplish this specific discharge distribution, many measures were taken, with the most important being the reinforcement of the dikes around Spijk: In case of high water levels at Spijk, the old branch of the Nederrijn at Spijk (visible in Figure 2 between 'De oude Whaal' and 'Het boven Spyk') still discharged water towards the Nederrijn. As a consequence of this, the Nederrijn-Lek and IJssel received more than the desired 1/3 in case of high discharges. This resulted not only in more floods around the old branch, but also along the Nederrijn-Lek. The latter was caused by the fact that the water was not only coming from the Pannerdensch Kanaal, but also from the Old Rhine, caused by a partially clogged inlet of the IJssel. For this reason, between 1771-1777, a new IJssel mouth was excavated and maintained, with a new width of 1/3 of the Nederrijn-Lek.

It took years to establish a stable bifurcation point between the Waal and the Pannerdensch Kanaal: nowadays we can model the main behaviour of the rivers, but back in the 18th century, alterations of the river were done based upon "best practices": knowledge and experience gained over time through trial and error. In the 18th century, several practices existed to control the river. The most common practices were to reinforce the outer bend with layers of braided twigs, the so-called *bleeswerk.* The other one was to stabilize the riversides with *groynes*. The inner and outer bends of the Boven-Waal, just before the bifurcation point, were stabilized with *bleeswerk* in 1780. Construction of groynes to guide the flow was never executed because of the high water levels during the winter of 1780-1781.

In the spring of 1781, the site foreman of the water authority Rijnlanden, Christiaan Brunings, found out that ice drift had caused the sediment bar in the inner bend of the Pannerdensch Kanaal to shift towards the middle of the bifurcation point. A plan was constructed to use this sediment bar, and construct a giant scoop groyne on top of it. This groyne would be fitted with side groynes pointing downstream. Sediment would be deposited behind these side groynes, broadening the scoop groyne. The scoop groyne was designed such that the deposition of sediment was stronger at the Waal side of the groyne, making the Waal more narrow. This forced relatively more water towards the Pannerdensch Kanaal. The East bank of the Pannerdensch Kanaal was stabilized to create a stable separation point of the Rhine, or 'Boven Waal', bifurcating into the 'Beneden Waal' and the 'Pannerdensch Kanaal'. Since this construction, the bifurcation point has never been changed significantly.

Other main events in the Dutch river area included the normalisation of the rivers, between 1860 and 1930, and the canalization of the Nederrijn-Lek from 1954 to 1971. The normalization was at first implemented to accomplish a fast discharge of drifting ice (Ploeger, 1992), and was achieved by narrowing the main river beds, realizing sufficient depth for a constantly flowing main channel. Furthermore, sandbanks or islands in the rivers were connected to the riverbank, so the main stream was not diverted anymore. Other measures included cut-offs in sharp curves and the strengthening of the dikes and riverbanks along the rivers.

During the canalisation of the Nederrijn-Lek, multiple weirs were built. The weir at Driel is located close to the bifurcation point IJsselkop. This weir does not only influence the water levels at the River Nederrijn-Lek, but also maintains a constant flow towards the IJssel in a period of low flow. Therefore, it alters the discharge distribution during low flow.

2.3 Water Management of the Dutch Rhine Branches

As discussed before, rivers have multiple functions such as a providing a means for transportation, fresh water supply, agriculture, and they can be a line of defence against invasions of enemy armies. Over the last centuries, the River Rhine has been managed to provide benefit from each of these functions, while restricting the risk of inundations. The exact execution of the management often reflects a trade-off between the different functionalities on the one hand, and the flood risk on the other hand.

The management of the main rivers in the Netherlands is the task of Rijkswaterstaat, whose societal mission and core-businesses are: water safety, sufficient water supply, clean and healthy water, fluent and safe

traffic over water, and a sustainable habitat (Rijkswaterstaat Ministerie van Infrastructuur en Milieu, 2014). In the Netherlands, the discharge over the Rhine branches is only actively managed in case of a shortage of water (water scarcity) or an abundance of water (during a flood wave). Managing is done by steering controls, the main steering controls are depicted in Figure 4.
Not only during high water, but also during water scarcity, the distribution of water is forced by law. These distributions are realized by operation of the constructions shown in Figure 4. Since the risk of inundations is negligible during water scarcity (corresponding to low water levels), the discharge distribution during water scarcity is not taken into consideration in this study.

2.3.1 Management during high water

From a discharge of 2,300 m^3/s at Lobith, all the weirs at the Nederrijn-Lek are opened in order to discharge the water. Along with an increase of the discharge, the water level increases proportionally since the water is freely flowing (ten Brinke, 2004).

The outflow of the River Rhine towards the North Sea during high water levels, is influenced by the Maeslantbarrier, the sluices, water level-management of the Lake IJsselmeer, and the sluices of the Haringvlietdam (numbers 2, 3, and 4 respectively in Figure 4). These artificial constructions partially block the flow and are gradually more opened until full discharge capacity is realized. For example, the sluices in the Haringvlietdam (number 4 in Figure 4) regulate the discharge of the River Rhine and Meuse into the North Sea. At low discharges, these sluices are closed in order to retain a fresh water supply for agriculture, by diminishing salt intrusion from the sea. Starting from a discharge of 1,100 m^3/s at Lobith, these sluices gradually open until a discharge of 9,500m^3/s at Lobith. At this discharge, the sluices are fully opened and discharge 6,000 m^3/s (ten Brinke, 2004).

During high water in the rivers, the main concern is safety. The dikes in the upper river area are designed in such a way that they can withstand a flood with a recurrence time of 1 in 1250 years. The changes in the design water level and the design discharges, over the last century, are described in Table 1.

Table 1: Design water level or design discharges of the Rhine branches (discharge dominated).

Year	Rhine design water level or design discharge at Lobith[m^3/s].	Waal [m^3/s] %Design discharge	Pannerdensch Kanaal [m^3/s] %Design discharge	Nederrijn-Lek [m^3/s] (% Pannerdensch Kanaal)	IJssel [m^3/s] (% Pannerdensch Kanaal)	Cause of change/comment
Until 1953	Dikes were constructed with +1 meter above the highest known water level (of January 1926) (Van de Ven, 2007). This approximately corresponded to 12,000 without the old Rhine branches, and 13,500 with them.	8,250 (61.1%) Without the old Rhine branches.	3,750 (31.3%) inflow (without the old Rhine branches) and 5,000 outflow (with the old Rhine branches).	2,700 (54%)	2,300 (46%)	
1956-1977	18,000 (probability of event: 1/3,000)	11,400 (63.3%)	7,100 (39.4%)	4,200 (59.2%)	3,050 (43.0%)	The Dutch flood disaster of 1953. The discharges of the IJssel and the Nederrijn river exceed the inflow from the Pannerdensch Kanaal, since the old Rhine was still discharging water during high water levels.
1977-1992	16,500 (probability of event: 1/1,250)	10,400 (63.0%)	6,100 (37.0%)	3,575 (58.6%)	2,525 (41.4%)	The modification of the dikes for a design discharge of 18,000 [m^3/s] had severe impacts on the environment (Van Til, 1979). A commission led by the Minister of Transport, Public Works and Water Management decided that 16,500 [m^3/s] would be sufficient (Van Heezik, 2006). The discharge distribution is based upon the discharge distribution during recent high water levels (Dienst binnenwateren / RIZA, 1986).
1996-2001	15,000 (probability of event: 1/1,250)	9500 (63. 3%)	5500 (36. 7%)	3175 (57.7%)	2325 (42.3%)	Without sideways inflows and discharges.
2001-2006	16,000 (probability of event: 1/1,250)	10133 (63.3%)	6867 (36. 7%)	3386 (57.7%)	2480 (42.3%)	Same percentage as in 1996, but higher design discharge led to a different water distribution over the IJssel.
2006-2011	16,000 (probability of event: 1/1,250)	10165 (63.5%)	5835 (36.5%)	3380 (57.9%)	2461 (42.2%)	
2011-2016	16,000 (probability of event: 1/1,250)	10165 (63.5%)	5835 (36.5%)	3380 (57.9%)	2461 (42.2%)	The discharge distribution of 2006 remains unchanged.
2017-?	WTI2017 GRADE	±63.5%	±36.5%	±57.9%	±42.2%	Distribution remains approximately the same, only the design discharge is changed as a result of the use of GRADE.

2.3.2 Distribution management

Currently, the discharge is actively regulated for low water levels through constructions downstream, such as the weir at Driel and Amerongen (ten Brinke, 2004). At the same time, the distribution *at the design discharge* is determined in the Dutch policy. As stated before, the design discharge at Lobith is a flood event of a strength that occurs once every 1250 years, or in other words, the design discharge has a recurrence time of 1250 years. The value of the design discharge can be obtained from an extrapolation of the historical discharge statistics, and provides a design discharge of 16,000 m^3/s at the inflow of the Rhine at the Dutch/German border. Note that this is the discharge statistics described in 'HR2006' (Ministerie van Verkeer en Waterstaat, 2007), not the discharge statistics of GRADE 2015, which will be used in this study.

In view of the changing climate, it was proposed in the Deltaprogramme (Ministerie van Infrastructuur en Milieu & Ministerie van Economische Zaken, 2014) that the design discharge should be increased from 16,000 to 18,000 m^3/s, with the remark that for discharges above 16,000 m^3/s the discharge towards the Nederrijn-Lek branch should not exceed the current maximum discharge. This new design discharge of 18,000 m^3/s was converted to design water levels along the Rhine branches, which are fixed in the law 'wet op de waterkering' (Ministerie van Infrastructuur en Milieu, 1995). In order to meet the design discharges (stated in Table 2), it was assumed that the discharge distributions over the branches at the design discharge are fixed. This was done even though the design discharge has never actually occurred (Kroekenstoel, 2014).

The directive 'Rivierkundig beoordelingskader' states that a measure which changes the discharge distribution more than 5 m^3/s (at a discharge of 16,000 m^3/s at Lobith) needs serious review (Kroekenstoel, 2014). Although the discharge distributions are fixed for the design water level, the directive 'Rivierkundig beoordelingskader' states that a measure which changes the discharge distribution more than 20 m^3/s at a discharge of 10,000 m^3/s at Lobith needs approval, implying that not only the design discharge should be considered, but also lower discharges at which the separate branches are subject to discharges listed in the third column of Table 2 (Kroekenstoel, 2014).

Table 2: The discharge distribution set in accordance with policy, for the discharges at the Boven-rijn of 15,000 and 16,000 m3/s (Kroekenstoel, 2014).

River branch	Contribution (%)	Per branch (m^3/s)	Per branch (m^3/s)	Per branch (m^3/s)
Bovenrijn	100	10000[1)]	15000	16000
Waal	63.5	6473	9530	10165
Pannerdensch Kanaal	36.5	3527	5470	5835
Nederrijn–Lek	21.1	2077	3165	3380[2]
IJssel	15.4	1450	2305	2461[2]

1) This discharge is not set in accordance with policy.
2) The outflow from the gemaal at Kandia is included in this discharge (totally 6 m^3/s).

2.3.3 Artificial regulation of the discharge distributions

The water levels at the Rhine branches depend not only upon the discharge at Lobith, but also upon the water level at the downstream boundaries. Artificial constructions which have an impact on the water levels in the Dutch Rhine branches, (according to Hermeling (2004) and ten Brinke (2013)) are:

Outflow IJssel:
1. The inflating weir close to Ramspol
2. The discharge sluices at the ‘afsluitdijk’, which control the water level of the IJsselmeer .
3. The taps located in the floodplains of the bifurcation points Pannerdensche Kop and IJsselkop

Outflow Nederrijn-Lek & Waal:
1. The Maeslant barrier in the Nieuwe Waterweg
2. The Hartel storm-surge barrier in the Hartelkanaal
3. The storm-surge barrier Hollandsche IJssel
4. The discharge sluices in the Haringvliet.
5. The flood-control sluices in the Nederrijn-Lek.
6. The taps located in the floodplains of the bifurcation points Pannerdensche Kop and IJsselkop

These main steering controls of the Rhine branches are shown in Figure 4.

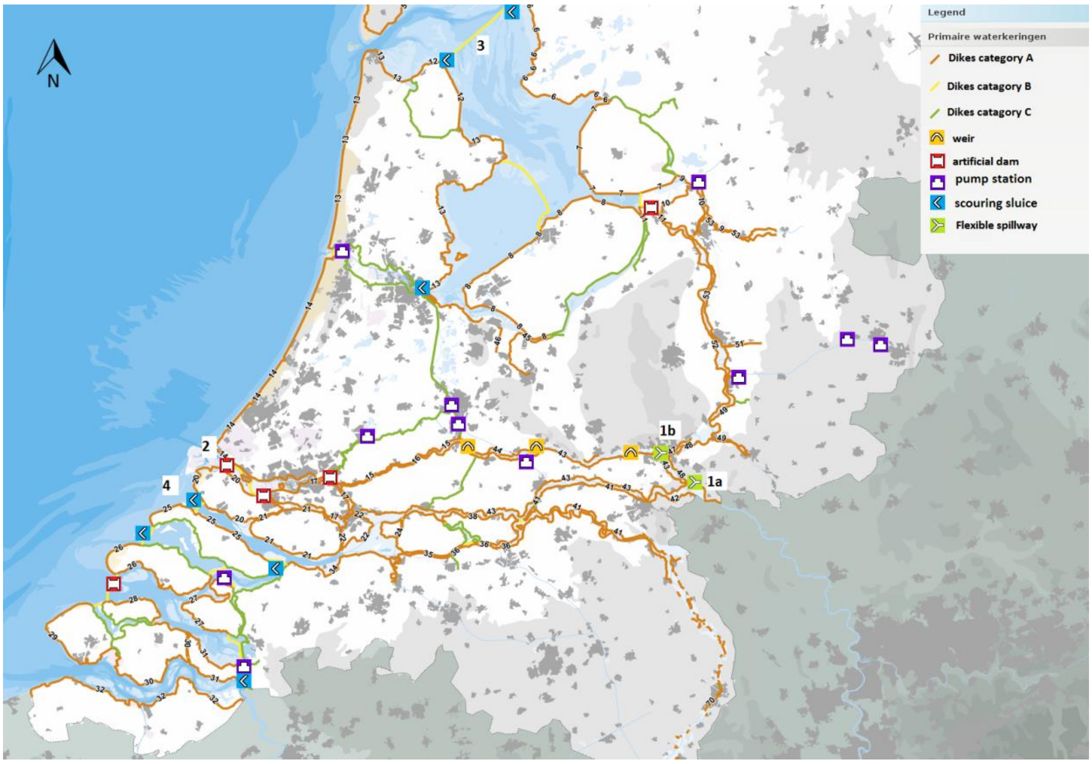

Figure 4: Artificial constructions in the branches of the River Rhine, adjusted after Rijkswaterstaat WVL (2015).

2.4 Uncertainties discharge distribution

Several processes influence the actual discharge distribution. Although the policy states that 63.5% of the inflow of the Rhine should flow via the Waal, it was found (during the high discharges between 1971 and 1995) that the Waal discharged more than 64% of the inflow (Ogink, 2006). According to ten Brinke (2013), this was due to several factors:

- The magnitude and the direction of the wind.
- The shape of the discharge wave and the subsequent morphological development of the riverbed.
- The failure of levees and spillways
- The change in river geometry.
- Roughness of the main river bed
- Roughness of the floodplains
- The subsidence of the riverbed.
- Uncertainty in the schematization of the model.
- Representativeness of the high discharges used for calibration.

Ogink (2006) arrived to the conclusion that the uncertainty in the discharge distribution at the Pannerdensche Kop is 500 m^3/s (±250 m^3/s) and for the IJsselkop 300 m3/s (±150 m^3/s) with a 90% confidence interval, corresponding to ±1.6% and ±2.5% of the discharge passing through those bifurcation points at design discharge. According to ten Brinke (2013), these numbers are still valid for the situation in 2013. The natural processes (listed above) have an influence on the discharge distribution, and are not precisely understood and therefore hard to quantify.

2.5 Stability discharge distribution

Policy assumes that the discharge distributions at the Pannerdensche Kop and the IJsselkop are stable. However, Kleinhans et al. (2013) state that a bifurcation point of a river transporting sediment simply cannot be stable. Sieben (2009) showed that the discharge distributions are changing over time due to sedimentation and erosion, and demonstrated that this process takes place for any given discharge rate. The Pannerdensch Kanaal erodes faster than the Waal and the IJssel erodes faster than the Nederrijn. The latter is mainly caused by the weirs in the Nederrijn.
In general, the river branch (or distributary) with the highest head/slope or the shortest path towards the sea grows at the expense of the other branch.
By the meanders in the rivers upstream of the bifurcation points and through spiral flow, sedimental sorting takes place at the bifurcation points. Sediment primarily moves towards the inner curve, therefore the Waal and the Nederrijn receive relatively more sediment. Coarse sediment, which is too large to join the spiral flow, rolls over the riverbed towards the Pannerdensch Kanaal and the IJssel. The abundancy of coarse sediment restrains further erosion of the riverbed (Kleinhans et al., 2013). However when the velocity of the water is high, an armoured layer can be eroded and the erosion can suddenly increase. Caused by excessive dredging in the 70's, and many weirs in the Rhine in Germany, the washed out sediment will not easily be replaced (Uwe Belz & Frauenfelder-Kääb, 2007).

2.6 Physical possibilities for alterations to the discharge distribution

Kleinhans et al. (2013) state that altering the discharge distributions is discouraged, or should happen really careful. If the discharge distribution is altered, it would be the easiest to do it in accordance with the natural behaviour of the river. The Waal and the IJssel are approximately the same length towards the outer water body. However, since the sediment load of the Waal is higher, the IJssel can potentially evolve to the most dominant river branch of the Rhine (Kleinhans et al., 2013). More discharge towards the River Nederrijn-Lek on the other hand, is not in alliance with the natural behaviour of the river since its slope is smaller than the slope of the River IJssel.

Any change in the flow of the water or the sediment near the bifurcation points can result in a different discharge or sediment distribution. This alteration in the sediment distribution can ultimately lead to another discharge distribution (Kleinhans et al., 2013). Arnold (2004) showed that actively steering of the discharge at the bifurcation point with a moveable threshold, will take days to adjust to the desired effect. Furthermore does it causes the water level upstream to go up.

2.7 Governmental possibilities for alterations to the discharge distribution

The discharge distribution is fixed by policy for the design water level, and was based on the law (Kok, 2013). The new water safety policy, defined in WTI2017 (Legal Testing Instruments for 2017), translates the standards set by the policy to a new design discharge. It aims to ease the work of the end user, such as engineering companies (Asselman, 2016). Since there are many uncertainties about the discharge distribution, the decision of altering of the discharge distribution is postponed towards 2050. Then it will be decided if the discharge distribution should be altered (in the future) or if the discharge distribution remains untouched.

2.8 Conclusion

The current discharge distribution at the Pannerdensche Kop is still related to the widths of the branches, based upon decisions taken back in 1745. Since the Old Rhine (Rijnstrangen) was still in use during high water levels, this implied nothing about the actual discharges of the Nederrijn-Lek and the IJssel at high water levels. Moreover, while it was agreed upon that maximum 1/3th of the discharge should flow towards the Pannerdensch Kanaal, the actual flow was much lower at the time (Van de Ven, 2007). This ratio could have been a political decision as a higher discharge through the Pannerdensch Kanaal was desirable for the flood prone areas located downstream the Waal, to protect the eastern defence line, and to increase navigational trade over the Nederrijn and IJssel. With those interests in mind, it is likely that this ratio was not chosen arbitrarily. Therefore, it can be concluded that the discharge ratio is not a product of hydraulic analysis of the branches and their capacities. The many floods of the 18th century close to the Pannerdensch Kanaal emphasize this (Van de Ven, 2007).

In 1767 this ratio was fixed by the widths of the river branches, not only for the Pannerdensche Kop, but also for the IJsselkop, which had been modified by the water authorities. No detailed calculations of the construction of the bifurcation points have been found. However, the 18th century water authorities were known to have competent and skilled supervisors, who gained their knowledge from their predecessors. Common river practices were based on years of experience and expert judgement (Van de Ven, 2007). Therefore it seems plausible that the construction of the *scoop* *groynes* of the bifurcation points, which determine the discharge distribution between the branches, was based upon trial-and-error. This is supported by the several attempts to improve the bifurcation points. If the former versions of the bifurcation points did not satisfy the needs, adjustments were made, until the bifurcation point appeared stable.

The current discharge distribution at design water level is fixed, approximately at the ratios set back in 1767. The current distribution is extrapolated from the values measured during high water levels in the first part of the 20th century (Dienst binnenwateren / RIZA, 1986). Although the exact discharge distribution at design water level is unknown, the discharge distribution is still (theoretically) fixed because the design water levels are fixed by law ('wet op de waterkering'). It is therefore that a fixed discharge distribution at the design discharge is assumed, also because this distribution is used as a boundary condition for the calculations and modelling of the flow at design discharge.

Although the policy states that 63.5% of the inflow of the Rhine should be discharged via the Waal, during the high discharges in the period of 1971 until 1995, the Waal discharged more than 64% of the inflow (Ogink, 2006). From this it might be concluded that although a fixed distribution is assumed and many alterations were made to the bifurcation points, the distribution, in fact, does vary as function of the discharge.

The bifurcation points are the result of natural processes and human interferences. Although the bifurcation points are declared to be approximately stable (Kleinhans et al., 2013), there is a large probability that the main river shifts if a breach occurs during high water.

3 Flood risk of the current distribution

3.1 Introduction

The final goal of this thesis is to evaluate how the flood risk changes with varying discharge distributions amongst the branches of the Rhine. To identify a change in risk, if any, the *current* situation must be assessed before we can proceed and change the discharge distribution.

The calculation of the current risk comprises multiple steps, some of which are non-trivial. The next section will describe this procedure, the data needed, and discuss the necessary assumptions. By the end of the chapter, the required tools are available, and the current risk, expressed in euros per year, can be calculated.

3.2 Flood risk

In simple words, the risk of an event expresses the combination of the likelihood that this event will occur, combined with the consequences in case that the event does occur. In short, one might write (Vrouwenvelder & Vrijling, 1987):

Equation 1: The definition of risk used in this study.

$$Risk = Probability * Consequences$$

To make this more specific for the current study, the flood risk depends on the probability that a dike will fail, times the consequences of the subsequent inundation of the hinterland. A sketch, illustrating how the risk, probability, and consequences of a flood are related to a number of factors is shown in Figure 5.

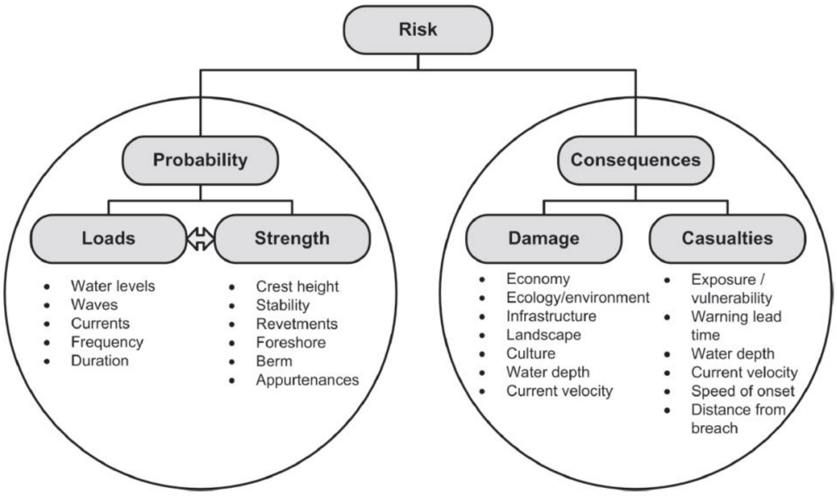

Figure 5 The components of flood risk (CIRIA, 2013) .

The probability and the consequences are not known with certainty, but can be predicted based on calculations. Over the years, these models and calculation methods have changed, due to progressive insight. In 2006, a new method has been applied to assess the flood risk in the so-called VNK or FloRis study (Ministerie Verkeer en Waterstaat, 2005; Vergouwe, 2015), as a response to the EU Floods Directive (European Commission, 2015), as described in Appendix A. This method provides the foundation of the risk calculation in this study. A flow chart, illustrating the steps required to calculate the risk is shown in Figure 6. Figure 5 and Figure 6 will be used for a step-by-step explanation in the next sections.

Furthermore the origin of the data is set out in Appendix B and cartoons which can clarify the background of Figure 6-C, Figure 6-D, Figure 6-E and Figure 6-F can be found in Appendix D.

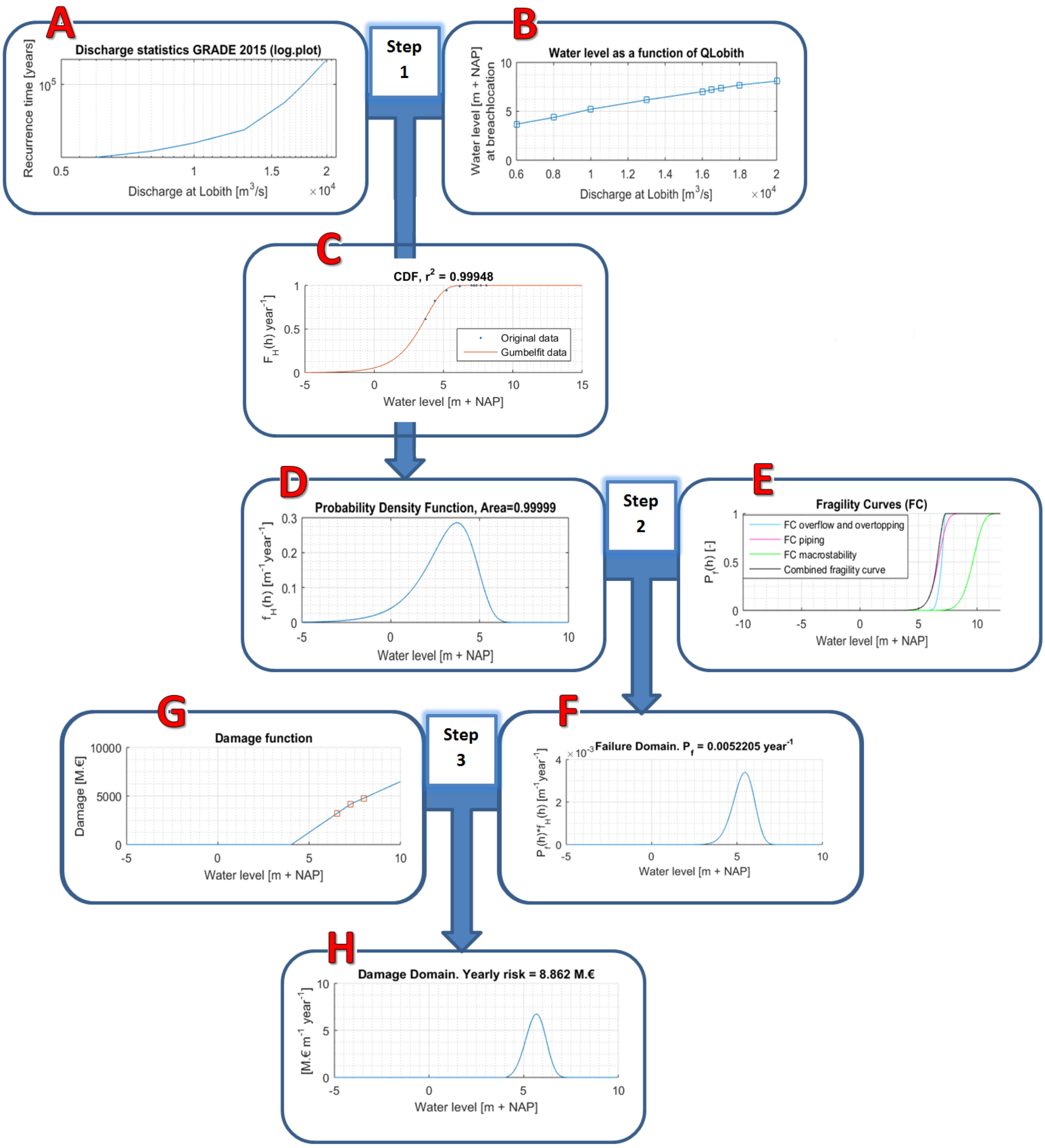

Figure 6: Schematic visualization of a flood risk calculation for one breach location.

3.2.1 Probability

As shown in Figure 5, the probability of a dike failure depends on two main factors: a certain load is exerted on the dike, which has a certain strength, which gives the dike a certain *chance* to withstand the load. Both the load and the strength must be determined, which will be the topic of the next sections.

3.2.1.1 Loads

The load, exerted on the dike is assumed to be only water level related. The water level changes over time, and can depend on many factors. By choosing the study area as the upper River Rhine area, the water level can be assumed to only depend on the rivers discharge, and independent of, for example, wind and tide. If, for example lower river area would be considered, the water level also depends on water levels in lakes or sea.

The water levels are related to discharge statistics of the inflow of the Rhine at Lobith, the statistics of which have been derived from the study GRADE, (reference year 2015, Hegnauer et al., 2014). These statistics provide the recurrence time of a certain discharge at Lobith, see the blue line in Figure 7. This discharge distribution is used as the first step in the risk calculation (Figure 6-A).

The water level at any downstream location depends on the discharge at Lobith, as drawn by the various lines in Figure 7. The plotted statistics can be found in Table 11 in Appendix F. Calculation of the risk at any specific downstream location thus requires the calculation of the local water level. This is done through the QH relations provided (appendix B), see Figure 6-B.

In step 1 of Figure 6, the recurrence time of the discharge at Lobith was combined with the QH relation for a specific downstream location. This resulted in the load of the water, expressed as a probability of occurrence of a water level per year (probability density function), see Figure 6-D.

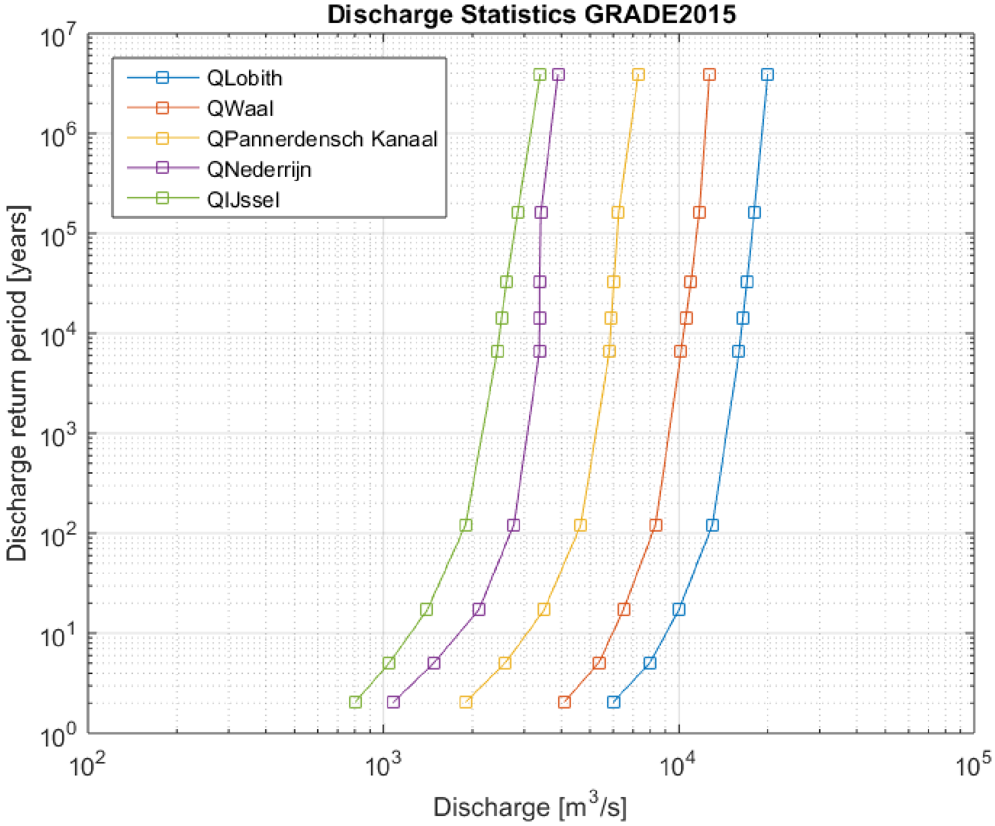

Figure 7: The discharge statistics (Hegnauer et al., 2014) used for the calculation. The distribution for the separate branches is documented in Stijnen & Botterhuis (2014).

The loads-component of Figure 5 consists of multiple factors. For this study's simplicity a stationary discharge calculation along the river is considered, instead of a flood wave. The duration of a peak water level is assumed to be 6 hours on average, as the strength of the dikes was determined for a steady load of 6 hours (Wojciechowska et al., 2015).

Furthermore it is assumed that the probability of occurrence of a peak discharge, does not depend on the last occurred peak discharge. Furthermore based upon Van Noortwijk et al. (1999) it is assumed that the time periods between the peak discharges of the Rhine is long enough such that are independent.

When the discharge statistics (Figure 6-A), and the discharge-water level relationship (Figure 6-B) are combined and fitted to a Gumbel distribution, a cumulative density function can be obtained as shown in Figure 6-C. The probability density function shown in Figure 6-D, is the derivative of the cumulative distribution function[1]. This probability density function represents the probability of occurrence of a peak water level in a year and is therefore the load of the water as shown Figure 5.

[1] This procedure is explained in more detail in Appendix F-I, more explanation about the fitting procedure is given in Appendix G.

3.2.1.2 Strength

Dikes can lose their ability to withstand water due to a multitude of reasons. The most obvious reason is when the water level exceeds the height of the dike, and water flows over the dike. This is illustrated in Figure 8A and B. Figure 8 also shows 10 other failure mechanisms that can lead to failure.

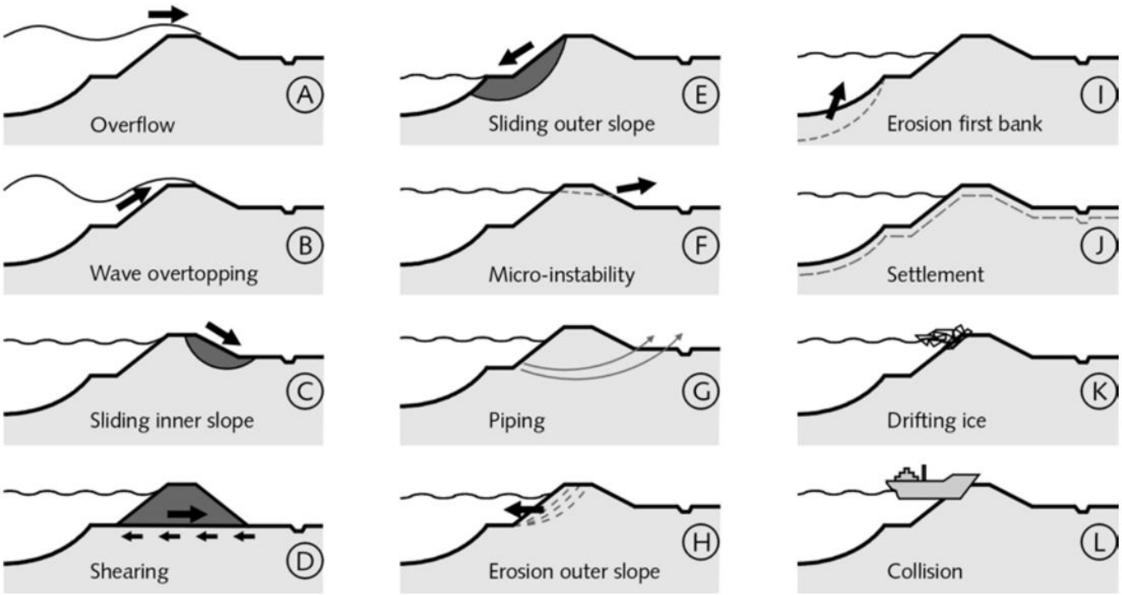

Figure 8: Different failure mechanisms (TAW Technische Adviescommissie voor de Waterkeringen, 1995).

According to the VNK2 study, the three failure mechanisms with the largest probability of occurrence for the upper Rhine river area are (Projectbureau VNK2, 2011):

- Overflow and Overtopping (Figure 8A and Figure 8B, respectively)
- Piping (Figure 8G)
- Macro-stability inner slope (Figure 8C)

This study will focus on these three failure mechanisms for the primary dikes in the upper River Rhine area of the Netherlands.

The Rhine river area is defined as the area fed by the Rhine. The upper region is the section of rivers in which the water levels are not influenced by the tide of the North sea. The boundaries of this area are the German border in the west and the line Schoonhoven-Werkendam in the east (Vergouwe & Sarink, 2014), shown in Figure 1. In practice however, it is common to define the upper River Rhine area by the boundaries of the dike ring areas (Ministerie van Verkeer en Waterstaat & Expertise Netwerk Waterveiligheid, 2007). The latter definition is also used in this study: the study area is illustrated in Figure 11 and the name of the branches is shown in Figure 1.

Table 3: The boundaries of the upper River Rhine area, as used in this study.

River branch	Riverside	Place	Kilometre number	Dike ring area
Waal	Right	Gorichem	955	43
	Left	Loevestein Castle	951	38
Nederrijn-Lek	Right	Lekkannaal	949	44
	Left	Diefdijk	943	43
IJssel	Right	Spooldersluis	981	53
	Left	Wapenveld	972	52

Primary dikes refer to the dikes which protect against floods by being part of a dike ring area (dike ring areas will be discussed later on), or are positioned outside of a dike ring area. Other artificial structures, such as dams and sluices are not considered in this study. Figure 9 shows a map of the upper Rhine area, and the minor bed and floodplains, which are restricted by the primary dikes.

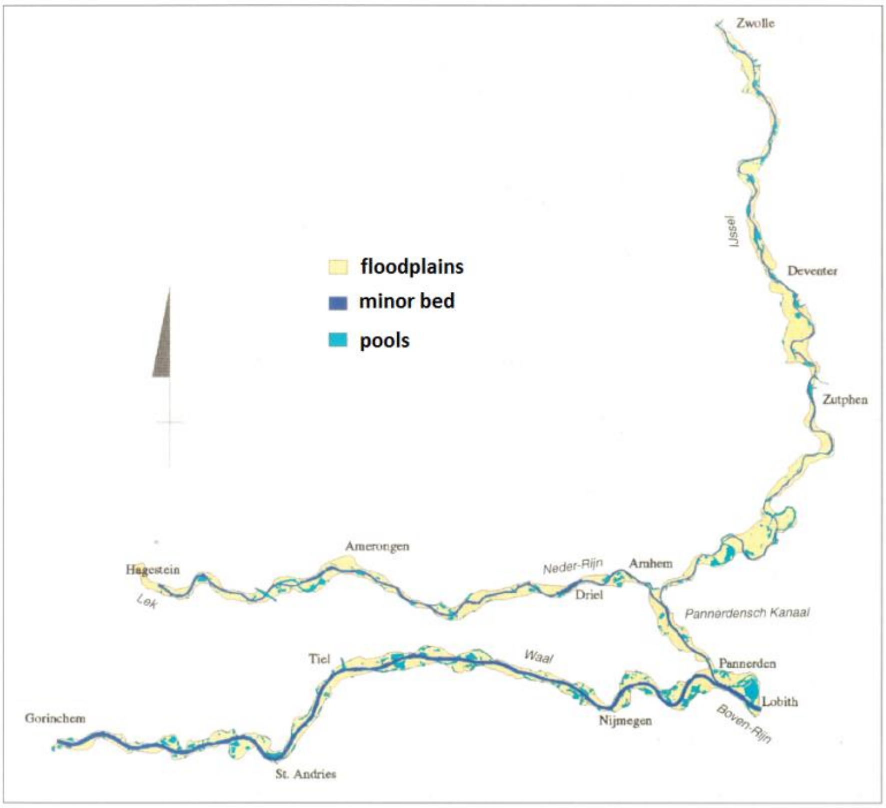

Figure 9: The minor bed and primary dikes along the floodplains of the upper River Rhine area (RIZA, 1996).

The ability of a dike to withstand the failure mechanisms described above depends on the water level and the dike properties. Because the properties of the dike, and with that its strength, vary with location, the local properties of the dike must be taken in to account.

The smallest portion of a dike that can be considered is a cross-section, which represents the dike properties, such as its height, of a dike section and has infinitesimal small length. A dike section is a small length of dike where the properties are uniform along its length, Figure 10. Several dike sections combine to a dike segment, where the consequences are the same for any breach along its length, Figure 10. Multiple segments combine to a dike ring area, which is depicted as the circle in Figure 10, and to a river branch. All the scales used in this study are shown in Figure 11.

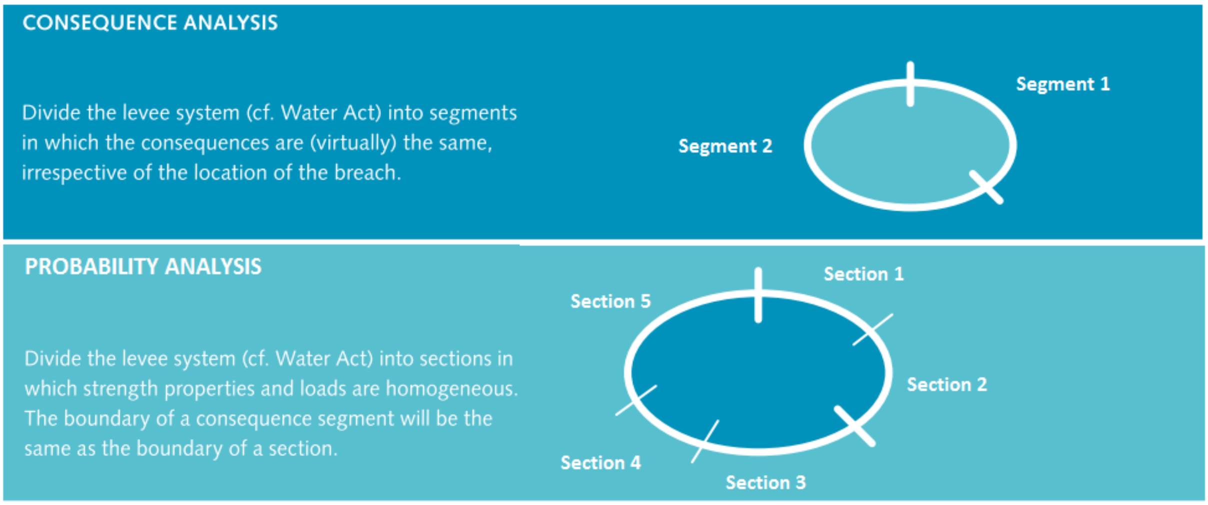

Figure 10: The difference between dike segments and dike sections. The circle represents a dike ring area. Figure adjusted after VNK2 project office (2012).

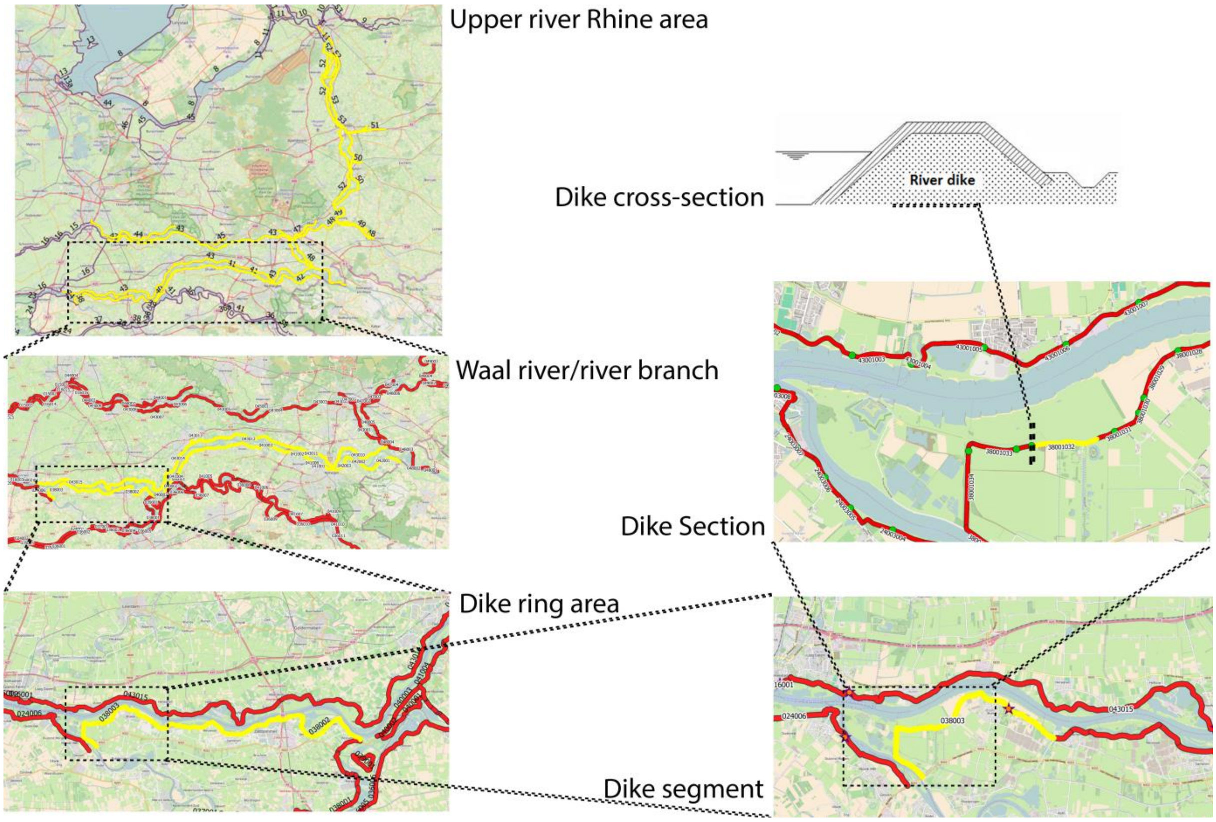

Figure 11: Different data scales used in this study. The name next to a figure refers to the yellow selection of that figure. The green dots in the 'Dike section'-figure represent the location of the cross-sections. The orange stars in the 'Dike segment'-figure represent the location of the breach.

The strength of a dike is thus evaluated per dike section and is expressed as a conditional probability of failure or a *fragility curve*. The quantitative shape of these fragility curves depends on many factors, such as the foundation of the dike and former failures. In this study, it is assumed that the fragility curves of the three failure mechanisms are independent. Therefore, the fragility curve for a specific failure mechanism for one dike segment is obtained[2] by combining the fragility curves of the sections of that section using the sum rule for statistical independent probabilities. This result is shown as the coloured lines in Figure 6-E. The combined fragility curve representing the fragility curve for all mechanisms, for the entire segment, is plotted in the same figure in black.

3.2.1.3 Calculating the probability

The product of the combined fragility curve of a dike segment and the probability density function of the load, explained in paragraph 3.2.1.1, provides the failure domain, as shown in Figure 6- F. The area under this graph corresponds to the annual probability of failure.

[2] The establishment of the fragility curves, as shown in Figure 6-E, is quite complicated. To improve readability, the details of this procedure are given in Appendix F.

3.2.2 Consequences

Now that the probability (left-hand part of Figure 5) has been explained, the next step is to discuss the calculation of the consequences (right-hand part of Figure 5). As the calculation of casualties falls out of the scope of this study, only damage was assumed to contribute to the consequences.

The consequences used in this study are derived from the VNK study. The VNK study calculated the damage as a function of different factors, such as flow velocity of the water inundating the hinterland and the size of the breach. From these factors mean damages for different recurrence times of the discharge at Lobith were calculated. These damages for upper rhine area were calculated for a recurrence time of 1/125, 1/1,250 and 1/12,500. The VNK study uses the situation in the year 2006 as the reference situation.

In this study it is assumed that a failure of a dike directly corresponds to damage. However, the failure of a dike does not directly have to result in a breach of the dike (Vrijling et al., 2010), as the many piping events during the flood of 1995 did not result in breaches or inundations (TAW Technische Adviescommissie voor de Waterkeringen, 1995).

The water levels of the river with these corresponding recurrence times, were calculated for every kilometre along the riverbank locations by Witteveen en Bos & RWS Waterdienst (2008). The location of this calculated water level was linked to the location of the breach (the representative point of a dike segment) based upon the shortest distance, see Figure 12. This resulted in an approximation of the water levels in the river for the three recurrence times and the corresponding damage calculations of VNK2, the red dots in Figure 6-G. Linear extrapolation of these three data points yields, the blue line in Figure 6-G.

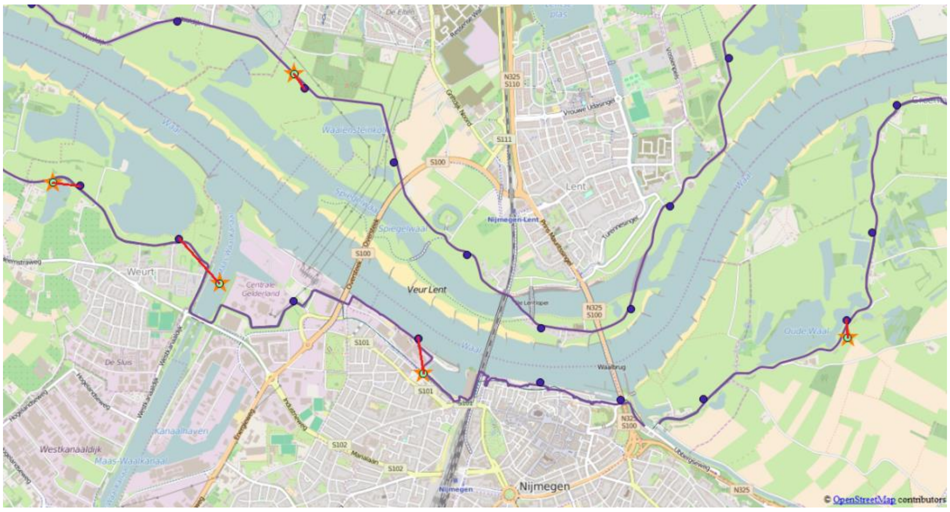

Figure 12: The combination of the location of the breaches of VNK2 (star shaped) and the water levels as calculated by Witteveen en Bos & RWS Waterdienst (2008).

In order to prevent that the total flood risk exceeds the total damage, the damage functions along the river branches were scaled. Therefore the 1/12,500 damage of all segments regarding one dike ring area are summed. Then a scale factor is derived from the ratio of the maximal scenario divided by the summed damage. When the summed damages are smaller than the maximal scenario, no scaling factor was used.

Equation 2: Damage scale factor

$$Scale\ factor\ dike\ ring = \frac{Maximum\ damage\ dike\ ring\ area}{\sum damage\ dike\ sections\ in\ dike\ ring\ for\ T = 1/12{,}500}$$

Maximum damages per dike ring areas, as calculated by the VNK2 study, are given in Table 5. The summed damages for all the branches is shown in Table 4. Note that the summation of the different branches exceeds the total damage of the upper River Rhine area, as shown in the last column in Table 4 .

Table 4: Total damage upper river area, considering the maximal scenarios of the dike ring areas along the corresponding river.

Name river branch	Bovenrijn	Waal	Pannerdensch Kanaal	Nederrijn-Lek	IJssel	Total damage upper River Rhine area
Damage M.€	8,513	33,716	23,783	46,495	24,161	83,675

Table 5: The maximum damage per dike ring, as calculated by the VNK2 study.

Dike ring area number	Max. Damage [M.€]	Corresponding river branches
38	6.03E+09	Waal
40	6.64E+07	Waal
41	9.24E+09	Waal
42	1.55E+09	Waal, Bovenrijn
43	1.68E+10	Waal, Nederrijn-Lek, Pannerdensch Kanaal
44	1.47E+10	Nederrijn-Lek
45	1.11E+10	Nederrijn-Lek
47	3.87E+09	IJssel, Nederrijn-Lek
48	6.96E+09	IJssel
49	5.73E+08	IJssel
50	1.86E+09	IJssel
51	2.26E+08	IJssel
52	1.42E+09	IJssel
53	9.25E+09	IJssel
+	8.36E+10	Total

3.3 Description of the flood risk calculation for one breach

The product of the damage function (Figure 6-G) and the failure domain (Figure 6-F) provides a damage domain: the yearly expected damage given a water level (Figure 6-H). The integral of this function provides the yearly flood risk for that segment in Millions of Euros per year.

The flood risks along the separate Rhine branches were only summed in order to derive the integral flood risk for that branch.

3.4 Results

The calculated flood risk for the reference situation is shown in the first row of Table 6. The total flood risk calculated for the upper river area does equal the summation of the flood risk of the separate branches. However, as shown in Table 5, multiple branches correspond to multiple dike ring areas. This study only incorporates the probability of failure and does not incorporate scenarios, which simulate combination of breaches. Therefore Equation 2 guard that the damage cannot exceed the total damage.

In Table 6 it can be seen that the flood risk of the River IJssel is a relatively high percentage of the maximum flood risk. The yearly flood risk is approximately 1/10th of the maximum damage, this is not in accordance with reality. This high flood risk is likely to originate from the strength component: a bad status of the dikes in the reference situation, or fragility curves which are calculated on the save side. Another cause of this can be the assumption that a failure of the dike directly results in a damage, while in practice this is not the case.

Furthermore note that the 'Total damage of the upper River Rhine area', as shown in Table 6 is not a summation of the maximal damage of the separate rivers. This is caused by the fact that multiple river branches border on the same dike ring area.

Note that the fragility curves for piping were not available for the Nederrijn-Lek branch, while it is the most important factor with regard to failure for the other branches.

Table 6: Flood risk per branch as a percentage of the maximum discharge.

	Bovenrijn	Waal	Pannerdensch Kanaal	Nederrijn-Lek	IJssel	Total damage upper River Rhine area
Flood risk [M.€/year]	44	2,183	1,083	31	2,421	5,763
Max. Damage [M. €]	8,513	33,716	23,783	46,495	24,161	83,675
Percentage of max damage [%]	0.52	6.47	4.55	0.07	10.02	6.89

4 Flood risk as a function of change in discharge

4.1 Introduction

In order to determine how a change in the discharge distribution affects the flood risk, the discharge distribution was altered. This was achieved by (virtually) altering the discharge distribution at the bifurcation points. To achieve this, discharge statistics were shifted resulting in a change in the recurrence time of a water level in the river downstream of the bifurcation point, thus simulating a change in the distribution.

By changing the discharge distribution in a step-wise fashion, and recalculating the flood risk (as described in the previous chapter) the relation between distribution and risk is found.

4.2 Risk calculation for different discharges

The calculation of the current flood risk is based on the discharge statistics at Lobith. Modification of the discharge distribution will change the discharge statistics of each of the branches. Given the river system that is studied, new discharge statistics have to be constructed for: 1) the Waal at the Pannerdensche Kop, 2) the Pannerdensch Kanaal at the Pannerdensche Kop, 3) the IJssel at the IJsselkop, and (4) the Nederrijn-Lek branch at the IJsselkop.

However, the simulations are based upon statistics and models that do not allow to simply change the discharge at each bifurcation point. Therefore, a work-around has to be devised, which comprises several steps. Key feature in this process is that the branches of the Rhine system are considered separately. The main constraints of this flood risk alteration were:

Equation 3: Discharge constraints at the bifurcation points.

$$Q_{Lobith} = Q_{Waal} + Q_{Pannerdensch\_kanaal}$$

$$Q_{Pannerdensch\_kanaal} = Q_{Nederrijn} + Q_{IJssel}$$

By considering the separate branches, for example the Waal, an increased flow towards this branch can be simulated by increasing the inflow of the Rhine at Lobith. This changes the flow Q_{Waal}

to a 'virtual inflow', denoted by Q'_{Lobith}. When the discharge distribution is changed, the water level statistics will change accordingly. For example, if the inflow of the River Waal is increased with 10%, a higher inflow (higher in terms of m^3/s) in the Waal will occur more often than in the reference situation. Therefore the recurrence period for the discharge in the Waal (Q_{Waal}), the red line in Figure 13) is shifted to the right, resulting in the yellow line Q'_{Waal}in Figure 13. This reflects how a relatively high inflow in the Waal will occur more often, while the inflow at Lobith (blue line in Figure 13) remains the same.

The next step is to relate the water levels along the branches to the new inflow statistics. These local water levels at any location along the river branches are related to the inflow at Lobith, for which the new statistics (the recurrence time of Q'_{Lobith} and Q'_{Waal}) were obtained in the previous step. Through linear interpolation, the new (virtual) statistics at Lobith corresponding to the new Waal statistics can be calculated. These results in Q'_{Lobith}, the purple line in Figure 13.

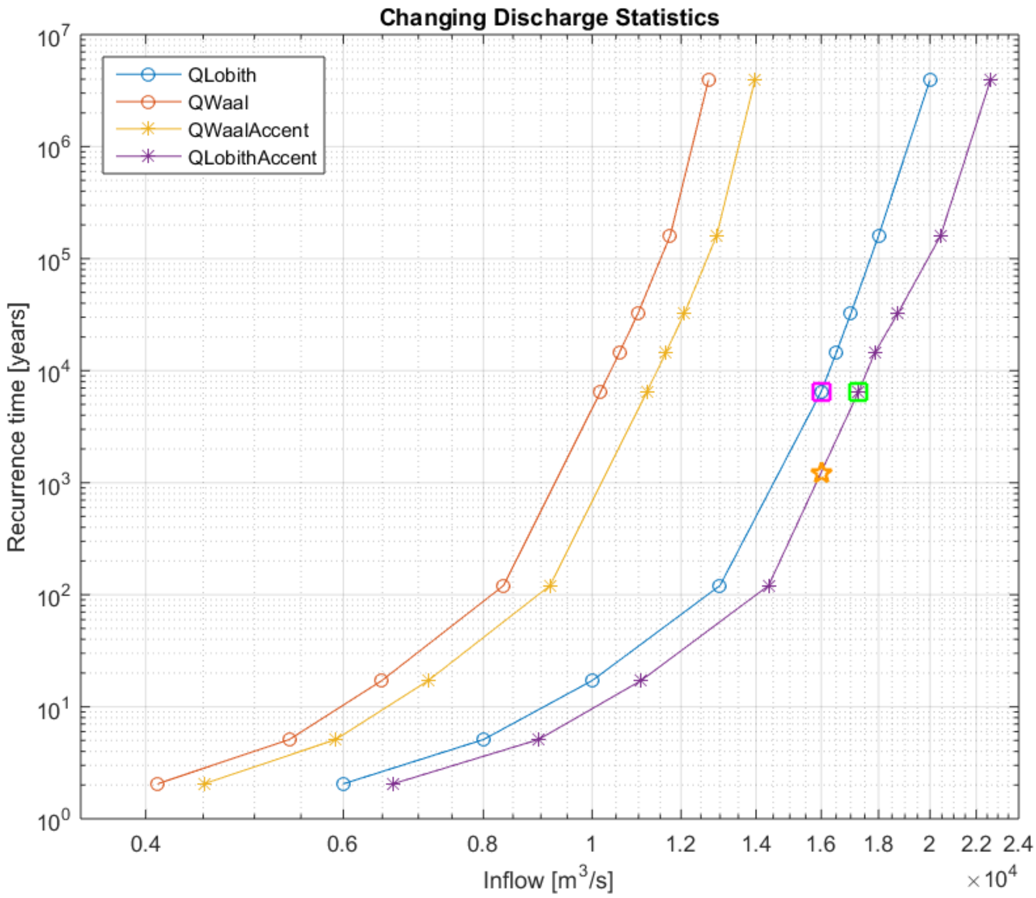

Figure 13: A 10% higher inflow into the River Waal, projected upon QLobith, noted as QLobithAccent or Q'Lobith.

The discharge Q_{Lobith} and Q'_{Lobith} corresponding to the recurrence time of approximately 6500 years are 16,000 m^3/s respectively 17,300 m^3/s (magenta and green square in Figure 13). Since the water levels are only given at nine values for a discharge, as shown in Figure 7, an interpolation step is required to obtain the recurrence time at more values of the inflow. The statistics corresponding to those discharges were derived by calculating the recurrence time of those nine predefined discharges for the new Q'_{Lobith} line. The recurrence time corresponding to 16.000 m^3/s, for an increased Waal inflow of 10%, can be found at the location of the orange star in Figure 13.

Since the real discharge statistics at Lobith does not change, the discharge statistics towards the Pannerdensch Kanaal becomes Q_{Lobith} minus Q'_{Waal}, Figure 13. This new inflow towards the Pannerdensch Kanaal, which is named $Q'_{Pannerdensch\ kanaal}$, can also be projected onto a new virtual Q'_{Lobith}. Since the local water levels are based upon the discharge at Lobith.

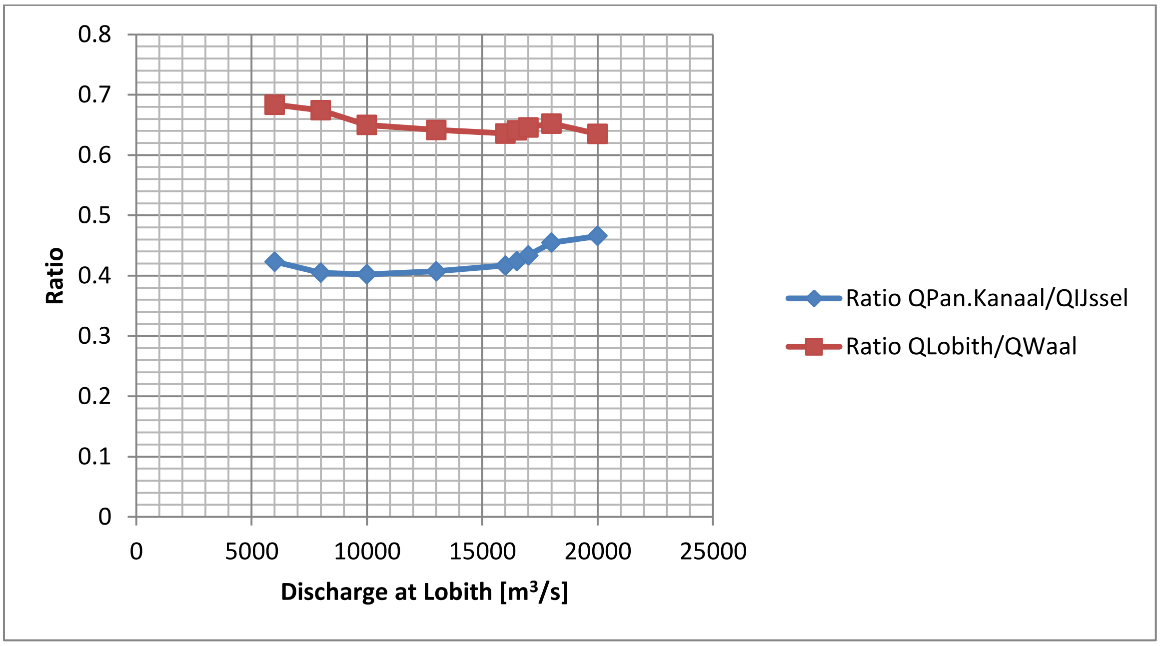

Figure 14: The discharge distribution ratios at the bifurcation points.

Although the discharge ratio is set fixed by law for the design discharge of 16,00 m^3/s, WAQUA simulations of the discharge in the Rhine river branches show that the ratio changes slightly for increasing discharge at Lobith. The ratios shown in Figure 14 are used as the discharge distribution in the reference situation; this division is called autonomous.

4.3 Calculation range

The most realistic, and therefore most interesting, range of change in the discharge distribution is the change closest to the reference situation. Although not realistic, one might wonder what happens when the whole range of discharges is taken into account: guiding all the water towards one branch and shift it toward the other branch.

The River Waal and the River IJssel are taken as a reference situation for the bifurcation points Pannerdensche Kop and IJsselkop respectively, this makes the risk calculation a function of the discharge towards these rivers. As stated above: the discharge distribution along the Pannerdensche kop is roughly 2/3 towards the Waal and 1/3 towards the Pannerdensch Kanaal, thus in the reference situation Equation 3 becomes:

Equation 4: The discharge for the Pannerdensch Kanaal at the Pannerdensche Kop.

$$Q_{Lobith} = Q_{Waal} + Q_{Pannerdensch_{Kanaal}}$$

$$= \frac{2}{3} Q_{Lobith} + \frac{1}{3} Q_{Lobith}$$

$$Q_{Pannerdensch_{Kanaal}} = Q_{Lobith} - \left(\frac{2}{3} Q_{Lobith} * 100\%\right)$$

If all the water is discharged towards the Pannerdensch Kanaal, $Q_{Pannderdensch_{Kanaal}} = Q_{Lobith}$. If all the water is discharged via the Waal, thus $Q_{Pannerdensch_{Kanaala}} = 0$, the maximum discharge in percentages towards the Waal becomes approximately +50%, as can be seen in Equation 5.

Equation 5: The maximum increase in discharge towards the Waal.

$$Q_{Lobith} = \frac{2}{3} Q_{Lobith} * (100\% + X)$$

$$\frac{3}{2} = 100\% + X$$

$$X = 50\%$$

The calculation is done with water level statistics, incorporating multiple discharges and the ratio does not remain the same for every discharge. In order to keep the constraints stated in Equation 3, the maximum change is 46%, resulting in a remaining discharge towards the Pannerdensch Kanaal as stated in Table 7. For an approximate ratio at the IJsselkop, the same procedure leads to Equation 6.

Equation 6: The maximum increase in discharge towards the IJssel.

$$\begin{aligned} Q_{Pannerdensch_{Kanaal}} &= Q_{Nederrijn} + Q_{IJssel} \\ &= \frac{3}{5} Q_{Pannerdensch_{Kanaal}} + \frac{2}{5} Q_{Pannerdensch_{Kanaal}} \\ &= \frac{2}{5} Q_{Pannerdensch_{Kanaal}} * (100\% + X) \end{aligned}$$

$$\frac{5}{2} = 100\% + X$$

$$X = 150\%$$

So since the total risk of the area is a function of the discharge towards the Waal, the change in discharge ranges from 0% to 146% of the reference situation of the Waal, see Table 7. For the IJssel, these numbers are 0% to 214% of the reference situation.

Table 7: Maximum discharges used in the flood risk calculation

Lobith [m³/s]	Waal [m³/s]	Waal * 146% [m³/s]	Remainder to Pannerdensch Kanaal [m³/s]
6000	4097	5981	18
8000	5370	7840	159
10000	6493	9479	520
13000	8338	12173	826
16000	10173	14852	1147
16500	10571	15433	1066
17000	10982	16033	966
18000	11736	17134	865
20000	12696	18536	1463

Pannerdensch Kanaal [m³/s]	IJssel [m³/s]	IJssel * 214% [m³/s]	Remainder to Nederrijn-Lek [m³/s]
1898	803	1718	179
2594	1050	2247	347
3501	1408	3013	487
4658	1897	4059	598
5825	2427	5193	631
5922	2512	5375	546
6025	2613	5591	433
6255	2844	6086	168
7300	3400	7276	24

For both calculations an increment size of 0.1% was chosen, and the results are shown in Figure 15 and Figure 16.

4.4 Results + Discussion

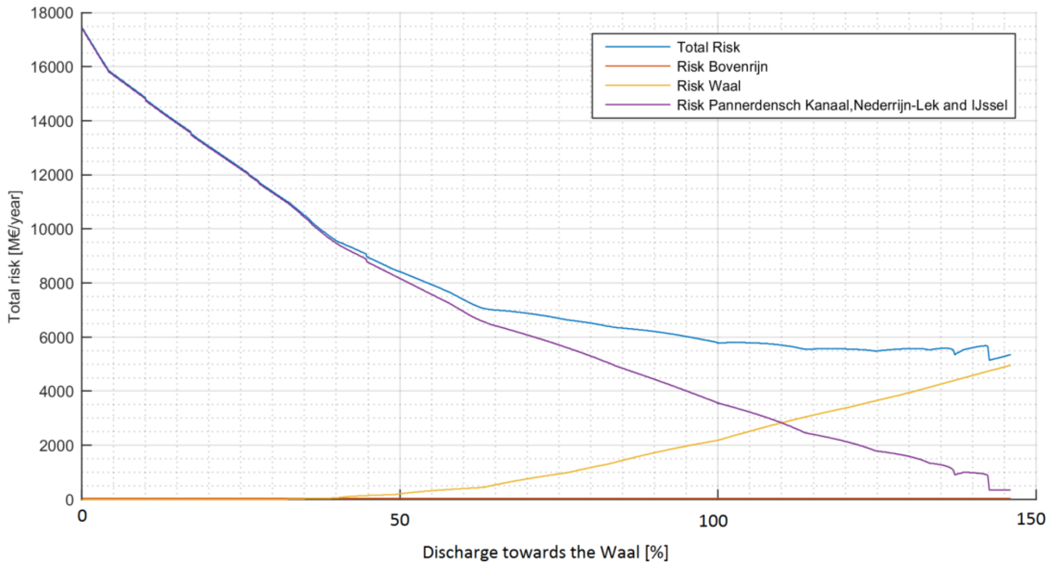

Figure 15: The yearly flood risk of the upper River Rhine area as a function of the discharge towards the River Waal at the Pannerdensche Kop.

In Figure 15 and Figure 16, 100% refers to the reference situation calculated in the previous chapter. 105% at the X-axis resembles a 5% increase with regard to the River Waal and the number 95% a 5% decrease of water towards the River Waal. The total calculated risk, drawn as the blue line in Figure 21, shows a decrease for a shift of the discharge distribution, directing more water in to the Waal.

Some bumps in the data, visible for example around 60% and 140% are peculiar: clearly visible in the total risk and the risk for the Pannerdensch Kanaal, Nederrijn-Lek, IJssel, but not so much in the data for the Waal. It is expected that these bumps originate from the data processing, rather than that they represent a physical (sudden) change in the risk. The cause of these bumps was investigated, where the discharge distribution and data fitting were checked for correctness. However, these steps were found not to be of influence.

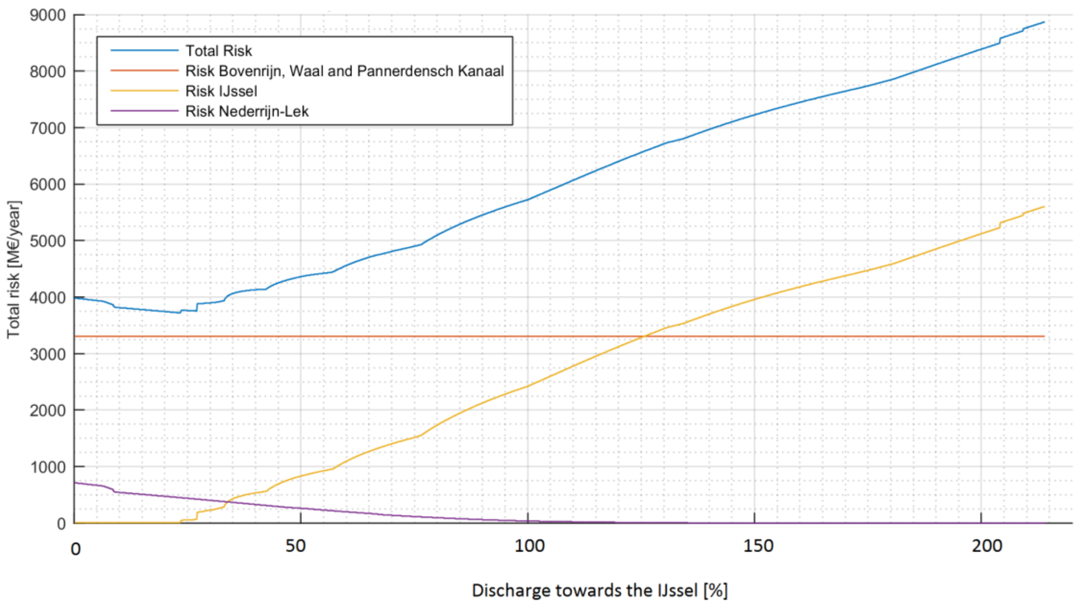

Figure 16: The yearly flood risk of the upper River Rhine area as a function of the discharge towards the River IJssel at the IJsselkop.

Figure 17 shows results analogue to Figure 16, but now for a shift of the discharge between the Nederrijn-Lek and the IJssel. Again, 100% on the horizontal axis indicates the reference situations. From Figure 17, it might be concluded that it is beneficial in terms of flood risk to send more water towards the River Nederrijn-Lek instead of the IJssel, as the total risk decreases for an increased discharge towards this branch. The total risk in figures 21 and 22 shows a minimum around 25% of the discharge of the IJssel in the reference situation, and increases again for even lower discharges towards the IJssel. As stated before, the risk calculation for the Nederrijn-Lek is based only on the failure mechanisms overflow/overtopping, as fragility curves for piping and macro-stability are not (yet) available.

The above figures provide interesting results and flood risk calculation for the distribution at the IJsselkop even shows a minimum. Note the instability of the flood risk calculation at a maximum discharge towards the Waal: this is probably caused by the extrapolation of the discharges towards a range which is not realistic.

5 Effect of input parameters on the calculated risk

5.1 Introduction

The calculation in the previous two chapters relies on many input parameters, such as the discharge statistics and the fragility curves. In order to gain insight in how sensitive the calculated flood risk is with regard to these input variables, a sensitivity analysis is conducted. In this analysis, the input parameters are varied, and the response of the calculated risk is measured.

5.2 Method

The calculation of the flood risk for different discharge distributions, as described in chapter 4 is repeated, but now with different input parameters. A new iteration of the calculation is executed for each change in input. As described previously, the risk calculation of this study consist of the load of the water, the strength of the dike, and the damage of an inundation. The following paragraphs describe the calculated risk when these aspects are changed.

5.3 Results

5.3.1 Other discharge statistics for 2015

One might wonder if the change in flood risk with regard to the discharge distribution, or the shape of the graph, still holds for other discharge statistics. Therefore the original flood risk calculation of GRADE2015 has been executed (left plots of Figure 17 and Figure 18) for comparison, next to the truncated GRADE2015 statistics and the statistics of the Delta Model for 2015 (Van Walsem, 2013). The recurrence time of the discharges at Lobith for the different statistics is shown in Table 8.

Table 8: The recurrence time for GRADE2015, Truncated GRADE2015 and the Delta model 2015.

Discharge at Lobith [m3/s]	GRADE 2015 [years]	Truncated GRADE 2015 [years]	Delta model 2015 [years]
	Recurrence Time		
6000	2.05E+00	2.05E+00	1.07E+00
8000	5.10E+00	5.10E+00	3.82E+00
10000	1.71E+01	1.71E+01	1.43E+01
13000	1.21E+02	1.21E+02	1.28E+02
16000	6.49E+03	6.49E+03	1.25E+03
16500	1.44E+04	1.44E+04	1.83E+03
17000	3.22E+04	3.22E+04	2.67E+03
18000	1.60E+05	1.60E+05	5.71E+03
20000	3.92E+06	1.67E+19	2.61E+04

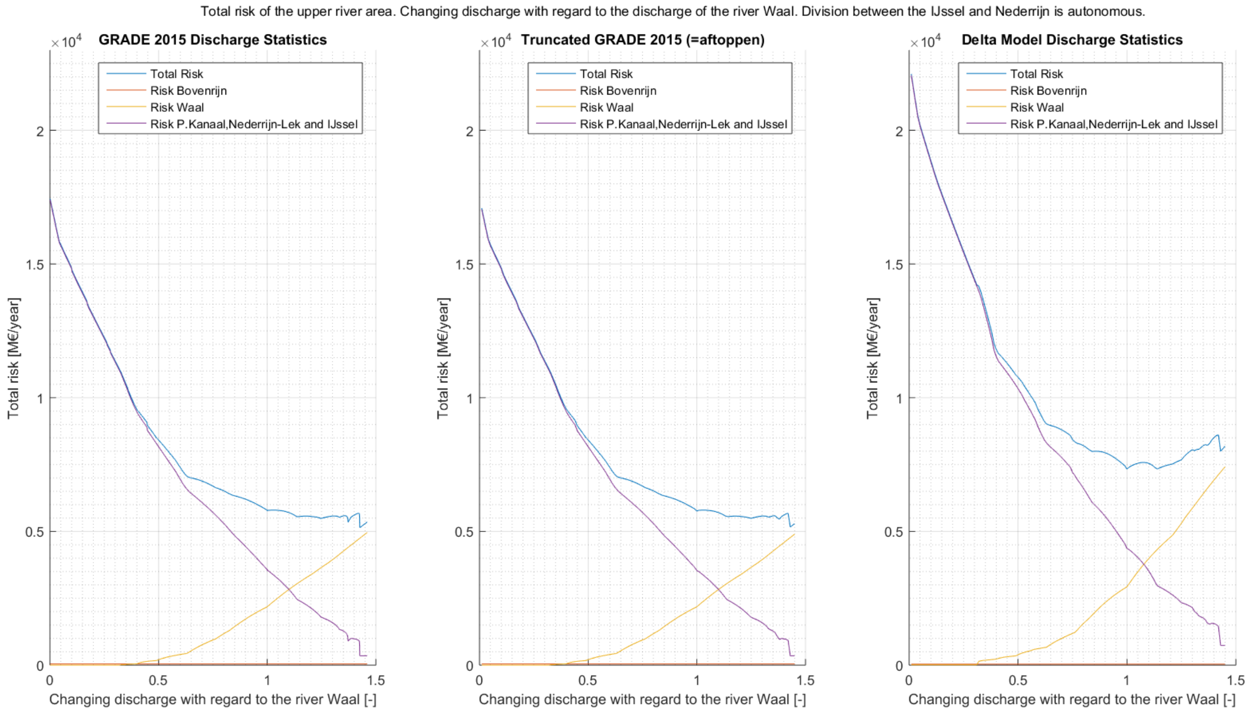

Figure 17: The yearly flood risk of the upper River Rhine area as a function of the discharge towards the River Waal at the Pannerdensche Kop for the truncated GRADE and the Delta model statistics. 1 represents the reference situation, a shift of 0.1 indicates a 10% change with respect to the reference situation.

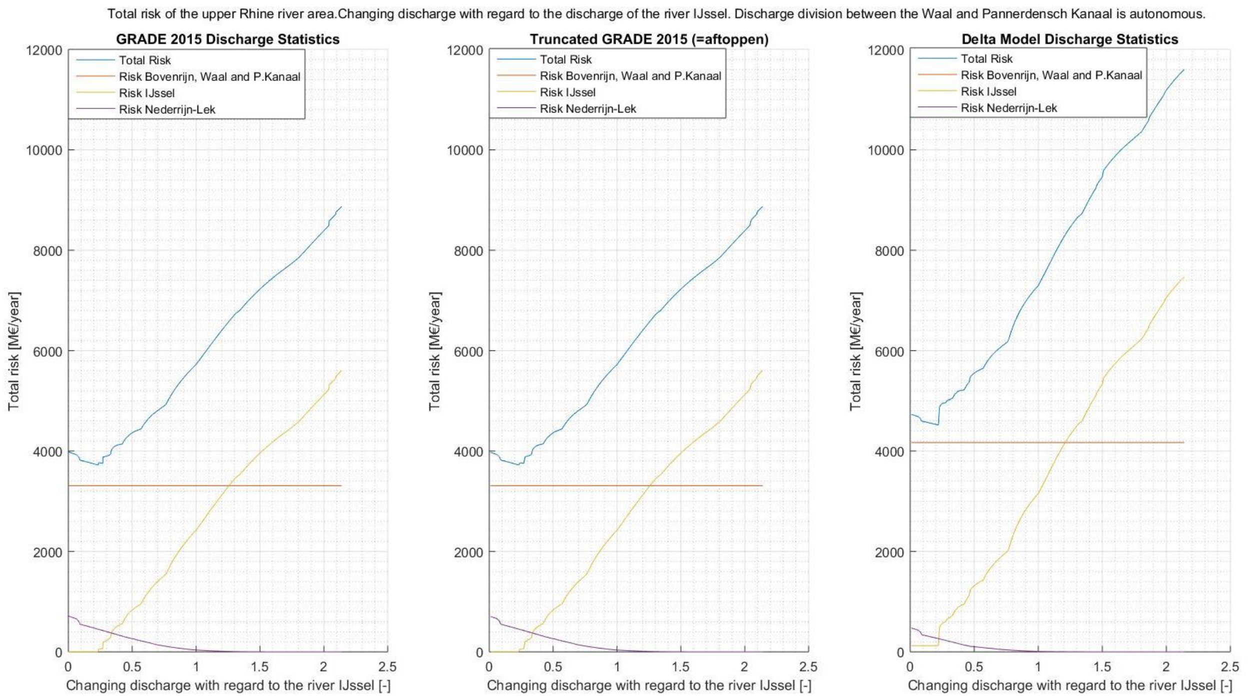

Figure 18: The yearly flood risk of the upper River Rhine area as a function of the discharge towards the River IJssel at the IJsselkop for the truncated GRADE and the Delta model statistics.

The truncated GRADE2015 statistics incorporate plausibility of an inundation or breach of the Rhine river upstream of Lobith. As can be seen in Figure 17 and Figure 18, the corresponding graph resembles the flood risk calculation of the normal GRADE statistics. This can be explained by the fact that the water levels are fitted upon a continuous statistical function. Since a truncation cannot be explained by the assumed statistical distribution of the water levels, the fitting for the truncated statistics is a little worse, but the outcome remains (practically) the same. For the discharge statistics of the Delta Model on the other hand, one can see that the expected flood risk is higher. This is caused by larger recurrence frequencies for the same floods. It should also be noted that the shapes of the flood risk as a function of the distribution are similar; a different discharge distribution still does affect the total flood risk. The figure corresponding to the discharge statistics of the Delta Model 2015 with respect to the Pannerdensche Kop shows roughly more an optimum in the flood risk than the original GRADE calculation.

5.3.2 The strength of the dike

As stated in Section 3.2.1.2, the strength of the dikes is derived from the fragility curves of the failure mechanisms overflow/overtopping, piping and macro-stability. From the fragility curves, macro-stability does often not really play a role in the combined fragility curve, which can also be seen in Figure 6-E. Therefor one might wonder how the flood risk function looks when only one failure mechanism is considered.

The flood risk calculation, only regarding the failure mechanism piping for the Pannerdensche Kop and the IJsselkop is shown in Figure 19, Figure 20 respectively. The discharge towards the Waal and IJssel ranges from 75% to 125% and the increment step is 0.1%.

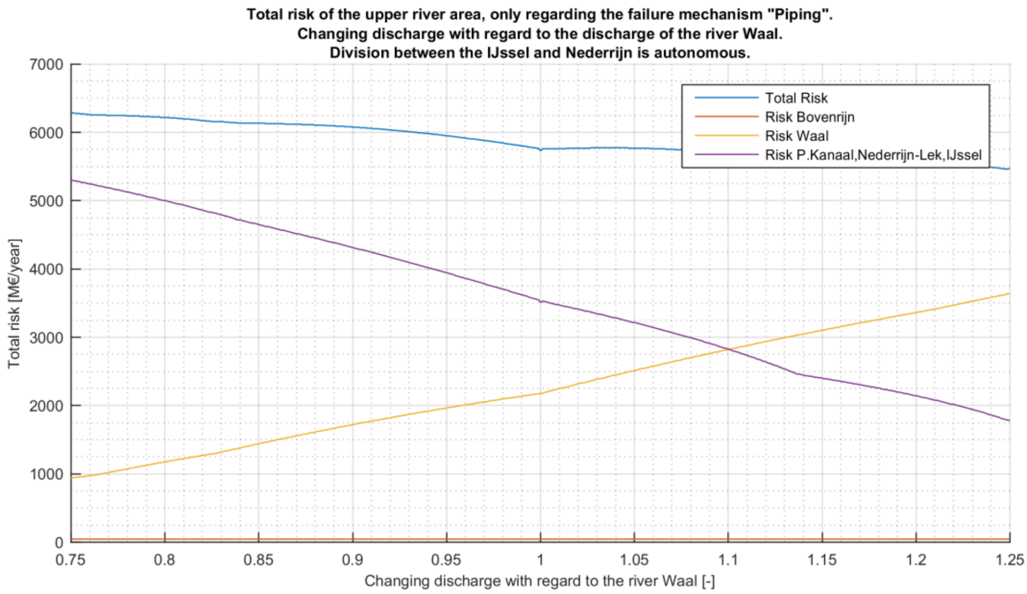

Figure 19: The risk of the upper river area as a function of the discharge distribution at the Pannerdensche Kop, only including the failure mechanism piping.

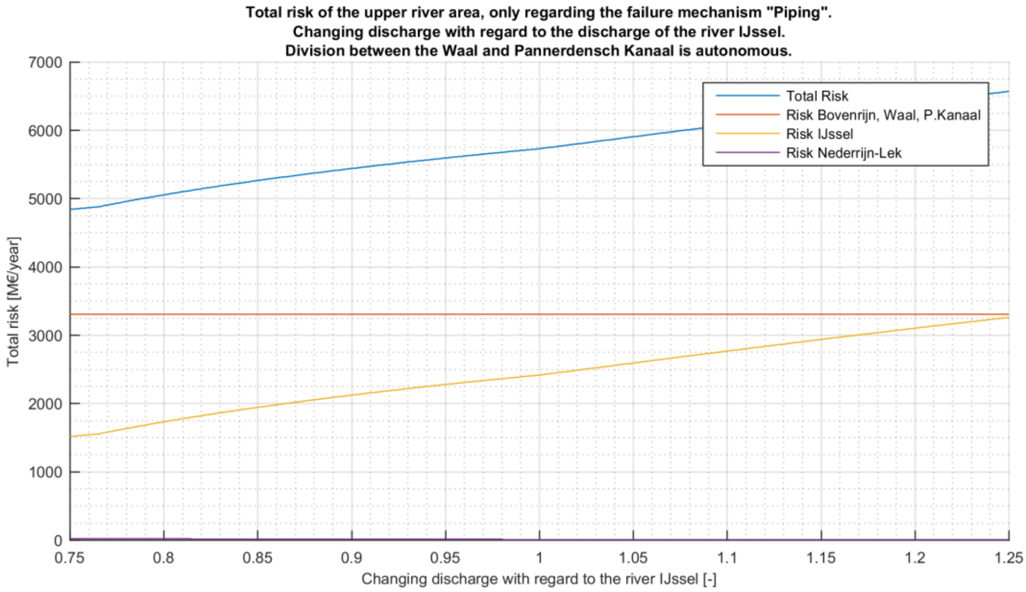

Figure 20: The risk of the upper river area, as a function of the discharge distribution at the Pannerdensche Kop, only including the failure mechanism piping.

When Figure 19 and Figure 20 are compared to Figure 15 and Figure 16 respectively it can be concluded that the shape and the magnitude of the yearly flood risk function of the upper River Rhine area as a function of the discharge distribution is mainly determined by the failure mechanism piping, which is thus the most dominant failure mechanism.

Although piping is the most dominant failure mechanism, overflow/overtopping is the only failure mechanism which is available for the whole study area, since piping and macro-stability data is not available for the Nederrijn-Lek branch. The flood risk calculation, only regarding the failure mechanism overflow/overtopping for the Pannerdensche Kop and the IJsselkop is shown in Figure 21, Figure 22 respectively. The discharge towards the Waal and IJssel ranges from 75% to 1.25% and the increment step is 0.1%.

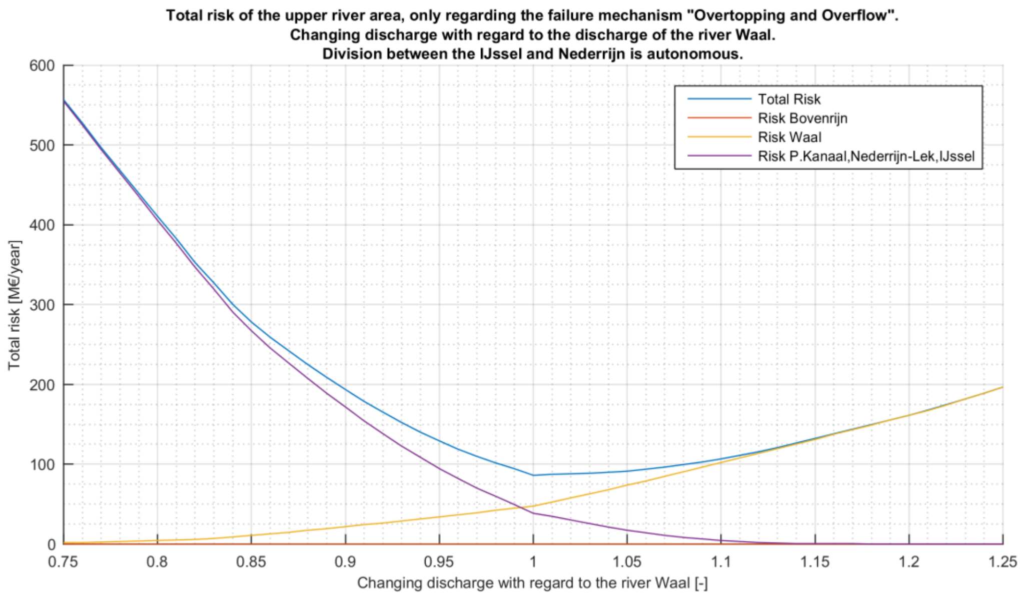

Figure 21: The yearly flood risk of the upper River Rhine area as a function of the discharge towards the River Waal at the Pannerdensche Kop, only concerning the failure mechanism overflow/overtopping.

When only the fragility curves of overflow/overtopping are considered, it can be seen that for the reference situation, the failure mechanism overflow/overtopping is only responsible for approximately 1.5% of the total calculated flood risk. Furthermore the optimum discharge distribution for the bifurcation point Pannerdensche Kop is located at the reference situation (1 on the x-axis of Figure 21), whereas the optimum discharge distribution calculated for all the failure mechanisms shows a decrease in flood risk when more discharge towards the Waal is realized, Figure 19.

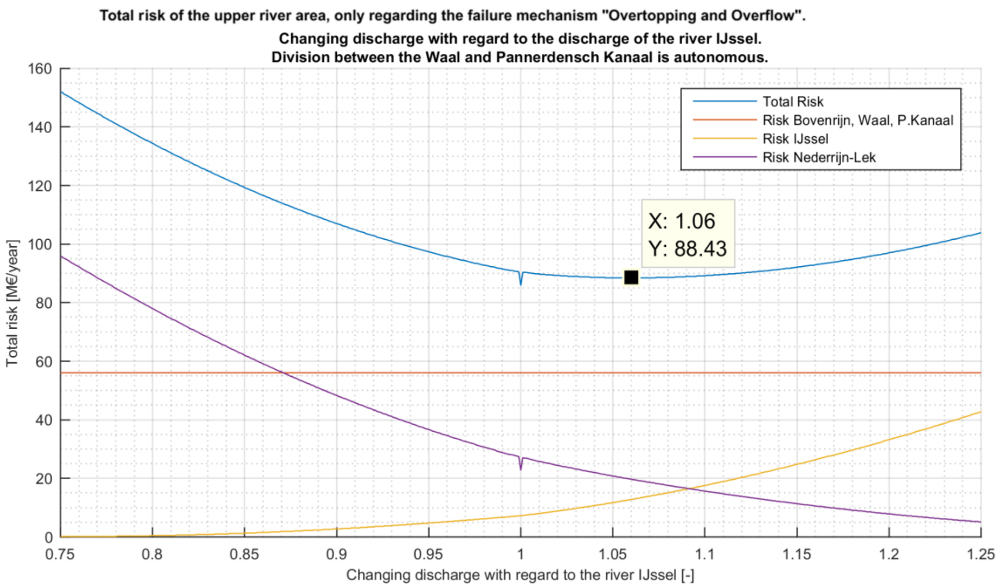

Figure 22: The yearly flood risk of the upper River Rhine area as a function of the discharge towards the River IJssel at the IJsselkop, only concerning the failure mechanism overflow/overtopping

The deviate flood risk at the reference situation, corresponding to 100% or 1 on the X-axis of Figure 22 is odd: only the smallest increase or decrease (±0.001%) shows the expected flood risk of the reference situation. Furthermore the reference situation in Figure 21 equals the discharge distribution close to 1, but not at 1. The cause was investigated, but due to time restraints, the cause could not be identified. If this data point is neglected, the minimum flood risk can be identified at a 6% increase of the discharge of the Waal, as shown in Figure 22.

5.3.3 The influence of the piping mechanism

The fragility curves for piping, as used in this study, (determined by Deltares) were said to be calculated with a generous safety margin. Therefore the conditional probability of failure due to piping is probably higher than in reality. Since piping is the most dominant failure mechanism, the sensitivity of altering the fragility curves can have a high impact on the overall flood risk. Changing the fragility curves ±3%, ±1% and ±0.1%, ranging 75% until 145% for the Waal with regard to the Pannerdensche Kop, with an increment step of 1% is displayed in Figure 23.

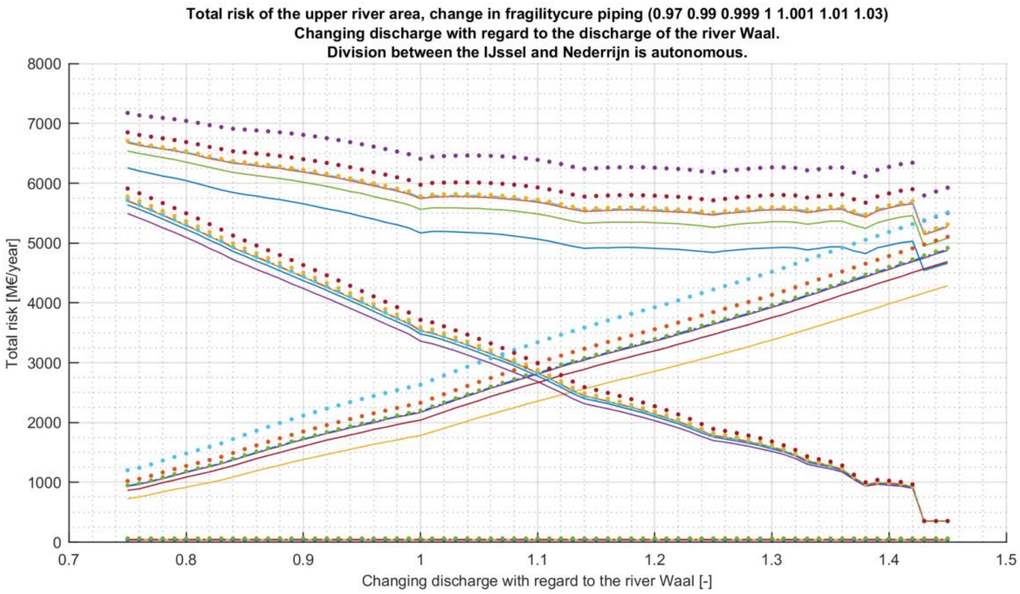

Figure 23: Sensitivity analysis for the failure mechanism piping upon the flood risk for the bifurcation point Pannerdensche Kop.

A 3% increase of the fragility curve, shows approximately 12% increase of the total risk of the study area for the reference situation. On the other hand, a 3% decrease of the fragility curves for piping shows approximately an decrease of 10% of the total risk of the study area for the reference situation. The sensitivity analysis of the fragility curve clarified the importance of having accurate fragility curves for piping, as the flood risk calculation is dominantly sensitive to this failure mechanism.

5.3.4 Consequences and damage

In order to draw a conclusion about the robustness of the risk calculation, not only the factors that determine the probability component, but also of the factors that contribute to the consequence component, such as the scaling factor, should have been taken into consideration. Furthermore, the water level-damage relationship derived from the VNK study of 2006 does not incorporate all Room for the River measures, and the calculated damages are from a decade ago and thus changes for the situation 2015. Although these investigations would have been interesting, due to time constraints this was not possible to do.

6 Discussion, conclusions, and recommendations

The goal of this work was to determine if an altered discharge distribution among the branches of the river Rhine results in a different flood risk.

First, the historical background of the current distribution was investigated. It was found that the first modifications to the river could have been made as early as two millennia ago. The current distribution amongst the branches of the Rhine was determined roughly 300 years ago, and was not changed significantly since then. The distribution was chosen with more emphasis on maintaining the various functions of the river branches, such as a means for transportation and a line of defence, rather than on safety concerns. The current distribution is fixed by policy, but has shown to vary in reality: certain discharges can result in different distributions.

A numerical tool was made to calculate the flood risk for the current situation. Only primary dikes along the upper river area of the Rhine were considered in this model. The loads on the dikes were calculated using GRADE2015. The strengths of the dikes were calculated using the fragility curves, only the failure mechanisms overflow/overtopping, piping, and macro-stability were taken into account. The load and strengths were combined with the damage information provided by the VNK study in order to obtain the flood risk.

The numerical tool was demonstrated to work, even though the calculated flood risk was much higher than expected. This is likely due to the fact that for high water levels multiple dikes can be breached, causing damage in multiple areas. In reality, a breach in one dike will lower the water level in the river branch, reducing the load on other dikes and therefore the flood risk. As said, this mechanism is currently not accounted for, and should be added to the tool to provide quantitative results.

Multiple interpolation steps were required to calculate the risk, as some input data contained few data points. The interpolation steps were needed to allow for many calculations. The error introduced by the fitting could be reduced by obtaining more data points.

Calculation of the strengths of some of the dikes was hindered, as fragility curves were only available for part of the dike sections, or were only available for a single failure mechanism. Most notably, there was no strength data for the Nederrijn-Lek, so fragility curves for the overflow/overtopping failure mechanism had to be calculated. Clearly, more complete data has to be acquired to provide more reliable numbers. Also, the calculation did not take into account ongoing measures and debates to reduce probabilities for flooding, or reduce the consequences (Asselman, 2016), both of which influence future flood risks. It would be interesting to identify the area with the highest risk and virtually reduce this risk, for example by strengthening the dike.

Calculation of the consequences was done based on financial damage, which should be made more realistic and up-to-date, as the VNK study is based on data and water levels for the year 2006. Moreover, including casualties in the consequence analysis in addition to financial damage can provide another view upon result of a shift in discharge. This would be especially important when also the lower river area is included in the study, as this region is densely populated. The latter step is also required if a complete calculation of the risk is to be obtained.

Despite these limitations and difficulties, a method to vary the discharge distribution amongst the river branches was found. This method allowed to answer the main question of the thesis: Yes, an altered discharge does change the flood risk. Keeping in mind that the provided data was incomplete, the calculations predict a reduction of 35% in the damage with respect to the current situation when the discharge towards the IJssel is changed. By increasing the discharge towards the Waal, a reduction of almost 10% in the total risk is predicted. Although the simulation suggests to almost entirely block the IJssel, it should be considered that only water safety is regarded in this study and no attention is given to the other functions of the rivers.

A brief step was taken to explore the influence of different discharge statistics. Future work should work this out in more detail, and determine if a reduction in total flood risk can still be obtained by shifting discharges. An excellent test case would be to use discharge statistics for the year 2100 as an input.

The fact that the risk in the current work changes with altering discharges is promising. Even though the calculations should be improved, it shows that it is worth the effort to investigate, and possibly reconsider the policy of the fixed discharge distributions.

Implementing a different, or even a variable discharge distribution in reality will be an tremendous task. The current work did not focus on legal possibilities for doing this, or on the social implications. Much of the existing (theoretical) work on the Rhine relies on the fixed distribution as a boundary condition, and should be reconsidered if this boundary condition changes.

The practical implication of a different or variable discharge can be expected to interfere with the sediment: erosion and deposition of sediment can alter the distributions, which has proven to be unpredictable in the past, resulting in even more variables in an already complex problem.

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Appendices

A. Detailed chronological historical background of the formation of the discharge distribution.

Table 9: Main decisions and works in the Dutch Rhine branches, which had implications for the discharge distribution at the Pannerdensche Kop or the IJsselkop, from the construction of the Pannerdensch Kanaal in 1707.

Date	Stakeholders	Decision	Motivation for this decision	Implications for discharge distribution
Aug 1745	Holland, Utrecht, OverIJssel, Gelderland.	A maximum of 1/3 of the Rhine inflow is allowed to flow through the Pannerdensch Kanaal.	Many floods in the old Rhine area (Rijnstrangen). For cities along the River IJssel and Nederrijn-Lek, which heavily depend upon trade, it was desirable to have more inflow towards those branches.	Since this date the maximum discharge distribution is fixed at the Pannerdensche Kop. However, since the Old Rhine riverbank still carried water used during high water levels, it did not say anything about the amount of discharge in the Nederrijn-Lek and IJssel (Van de Ven, 2007).
1767	Furstendom Gelre en Graafschap Zutphen.	Fixed width of the rivers Waal (40 Rijnlandse roeden ≈ 151m), Nederrijn (20 Rijnlandse roeden ≈ 75m), IJssel (10 Rijnlandse roeden ≈ 38m) (Hove van Gelderland, 1767).	Many flood events caused by the gaining of extra land next to the river.	Widths are being translated into discharge ratios. The Waal discharges 6/9th of the Rhine, consequently the Pannerdensch Kanaal 3/9th. The discharge between the Nederrijn and the IJssel where 2/9th respectively 1/9th (Van de Ven, 2007).
1771	Pruisen and the Netherlands.	The Arnhem Treaty was signed by Pruisen and the Netherlands where the discharge distribution was fixed.	The water distribution was not only important during high inflow, but also because higher discharges provided better navigability of the river in times of drought.	The discharge ratios as described above were fixed in an agreement: 2/3th of the inflow of the Rhine is discharged via the Waal and 1/3th is discharged via the Pannerdensch Kanaal. Afterwards, 2/9th had to flow through the Nederrijn and 1/9th through the IJssel.

Date	Stakeholders	Decision	Motivation for this decision	Implications for discharge distribution
1775	Van Hugenpoth (Main inspector construction of the Bijlands kanaal) and Brunings (Water manager at water authority Rijnlanden).	A *neck cut-off* was realized in the Boven-Waal, in order to keep the discharge distribution stable. This resulted in the Bijlands kanaal in February 1775.	A meander of the upper-rhine shifts in the direction of the bifurcation point. If this shift would have been continued, the primary flow of the water would have been flowing through the Pannerdensch Kanaal instead of the Waal.	The discharge distribution remains the same. However, since the flow velocities downstream of the Bijlands kanaal in the main stream is increased, more sediment is transported.
1782	Christian Brunings, first foreman Rijkswaterstaat (1798).	In the spring of 1781 the site foreman of the water authority Rijnlanden, Christiaan Brunings, found out that the sediment bar (*point bar)* in the inner bend of the Pannerdensch Kanaal was shifted towards the middle of the bifurcation point, caused by ice drift during winter. A giant scoop groyne was constructed in the length of this bank with side groynes, the latter were directed downstream, where sediment deposits should take place, so this bifurcation point would become stable.	As a result of the neck cut-off, more sediment was deposited in the inner curve between the Boven-Waal and the Pannerdensch Kanaal, which resulted in a sandbank *(point bar)* and the discharge was increasingly flowing to the Waal: the bifurcation point was unstable in terms of sediment.	This scoop groyne provided more inflow towards the Pannerdensch Kanaal and a stable separation point of the Rhine, or 'Boven Waal', bifurcates into the 'Beneden Waal' and the 'Pannerdensch Kanaal'. This is still the bifurcation point as it is present today (Van de Ven, 2007). Although this bifurcation point is stable, the water distribution was not yet stable. This was caused by the bad conditions of the rivers (Van Heezik, 2006).
1798	Rijkswaterstaat.	Rijkswaterstaat was founded.	Managing the large rivers in the Netherlands was done by local water authorities. This resulted in difficulties with maintaining the whole river system.	Rijkswaterstaat was founded in order to manage the large water bodies.
1850-1888	The Kingdom of the Netherlands.	Normalization of the upper Rhine branches, to keep the river stable (Silva et al., 2000). The total length of the rivers was shortened.	Smooth discharge of high water levels and ice, in order to prevent the probability of flooding (Ploeger, 1992).	Improvement of the discharge distribution between the rivers (Van Heezik, 2006).

Date	Stakeholders	Decision	Motivation for this decision	Implications for discharge distribution
1869	The Kingdom of the Netherlands.	Castle 'Pannerden' was built at the bifurcation point in the flood plains.	Fear of the damming of the Pannerdensch Kanaal and therefore no water inlet towards the Holland Defence Line.	Consequently, at a high discharges, the inflow in the Waal would decrease. This was compensated by the construction of a spillway at the Millingerwaard (Van de Ven, 2007).
1888-1890	The Kingdom of the Netherlands.	Second normalization of the upper Rhine branches, to keep the river stable (ten Brinke, 2004).	Keeping the rivers into position and improving the rivers for navigational purposes.	Improvement of the discharge distribution between the rivers (Van Heezik, 2006).
10 Nov 1900	The Kingdom of the Netherlands.	Law 'Waterstaatswet 1900' pointed out the general rules of the government and responsibility of the rivers (from 2009 integrated in the 'Waterwet').	Flood disasters were not centrally regulated.	The goal of the law 'Waterwet' is to govern the design water levels related to the safety standards which are fixed in the law 'Waterwet'.
1908	The Kingdom of the Netherlands.	Law 'Rivierenwet' operational (Van de Ven, 2007) (From 1999 this law is integrated in the 'Wet beheer Rijkswaterstaatwerken'.)	The responsibility of the maintenance of the levees and floodplains were not clear.	The Kingdom of the Netherlands bears the final responsibility for the maintenance of the rivers.
1912-1934	The Kingdom of the Netherlands.	Third normalization of the upper Rhine branches, to keep the river stable (ten Brinke, 2004).	Keeping the rivers into position and narrowing the main channels in order to increase the navigable depth.	Improvement of the discharge distribution between the rivers (Van Heezik, 2006).

Date	Stakeholders	Decision	Motivation for this decision	Implications for discharge distribution
1953	The Kingdom of the Netherlands.	In order to permanently close off the old Rhine river, the capacity of the Pannerdensch Kanaal needed to be improved. The river became 160 meter shorter and the discharge capacity of the main channel was increased. The floodplains were enlarged and new dikes were constructed. (Frings & Kleinhans, 2002).	Maintaining the levees in the Old Rhine area was costly and the regulation of the discharge via the old Rhine appeared not straightforward to manage.	Higher discharge capacity of the Pannerdensch Kanaal.
May 1958	Dutch Parliament.	In order to improve the safety, the Deltaplan was introduced.	Triggered by the flood disaster of 1953, improvements for flood protection were introduced in the parliament.	Because of the higher safety standards, it was more important to know exactly what the discharge distribution would be.
1994 1995	The Kingdom of the Netherlands.	“Deltawet grote rivieren” and later “Wet op de waterkering” (Van de Ven, 2007)	In order to guarantee and maintain a high safety level.	The safety of the primary water should be tested regularly. Furthermore Netherlands is divided into ‘dike ring areas’, which all have their own safety standard.
2007	European Union.	The EU Floods Directive (European Commission, 2015).	This Directive requires Member States to assess if all water courses and coast lines are at risk from flooding. Furthermore it requires its member states to map the flood extent and assets and humans at risk in these areas and to take adequate and coordinated measures to reduce this flood risk.	No specific direct measures for the bifurcation points.
2009	Dutch government.	The EU Floods Directive is adopted in the Dutch law. These are converted to the ‘Waterwet’, ‘Waterbesluit’ and the regulation of risk maps.	Consistency between the Dutch and European laws.	No specific direct measures for the bifurcation points.
2015	Delta-programme.	It is decided that a change of the existing discharge distribution is not desirable.	Uncertainties exist around the discharge distribution at the distribution and the adjusting of the discharge distribution (Asselman, 2016).	The existing agreements about the discharge distribution will be endured until 2050. Discharges higher than 16,000 m^3/s at Lobith will not discharge a higher amount towards the Nederrijn-Lek branch.

B. The origin of the datasets used for the flood risk calculation.

Table 10: The origin of the datasets used for the risk calculation.

	Dataset	Description Data	Data	Data used at scale	Source
Discharge statistics	**A**	The exceedance frequency of a certain inflow of the Dutch Rhine at Lobith	Qlobith [m³/s] with a corresponding recurrence frequency [year^{-1}]	Dutch Rhine system	GRADE (Hegnauer et al., 2014)
Discharges Rhine locations	**B**	The inflow of the branches of the River Rhine.	Qlobith [m³/s] with a corresponding Qbranch [m³/s], for the Waal, Pannerdensch Kanaal, Nederrijn-Lek and the IJssel.	Dutch Rhine system	WAQUA 2015
Location Dike segments/reaches and Dike sections.	**C**	The exact locations of the Dike segments and the Dike sections.	Shapefile of the Dike segments, Dike sections and the location of their representative points.	Dike segments and sections	Delta Programme / HWBP
Dike segments/reaches and Dike sections.	**D**		The ID numbers of the Dike segments and the accompanying ID numbers of the Dike sections.	Dike segments and sections	VNK Database
Local water levels of the dike sections	**E**		Qlobith [m³/s] with the corresponding water levels [m + NAP] for the representative points of a dike section.	Dike sections	Hydra-NL
Fragility curves Overflow/Overtopping	**F**	Conditional probability of failure for a dike section and a water level.	Fragility curves along the branches [-].	Dike sections	Deltares
Fragility curves Piping	**F**	Conditional probability of failure for a dike section and a water level.	Fragility curves along the branches [-]. Data not available for the Nederrijn-Lek branch.	Dike sections	Deltares
Fragility curves Macro-stability	**F**	Representative conditional probability of failure for a dike section given a water level.	Fragility curves along the branches [-]. Data not available for the Nederrijn-Lek branch.	Cross sections	Deltares
Consequence data	**G**		Damage in Million € for a breach relating to a discharge with a recurrence time of 1/125, 1/1,250 and 1/12,500	Dike segments	VNK Databases, LIWO (Rijkswaterstaat Ministerie van Infrastructuur en Milieu, 2015)
Water level dike segment	**H**		The water levels [m+NAP] for the river branches at the riverside locations of the VNK calculation	Dike segments	TMR calculations (Witteveen en Bos & RWS Waterdienst, 2008)

C. The graphical output of the calculation tool, given a dike section and a dike segment.

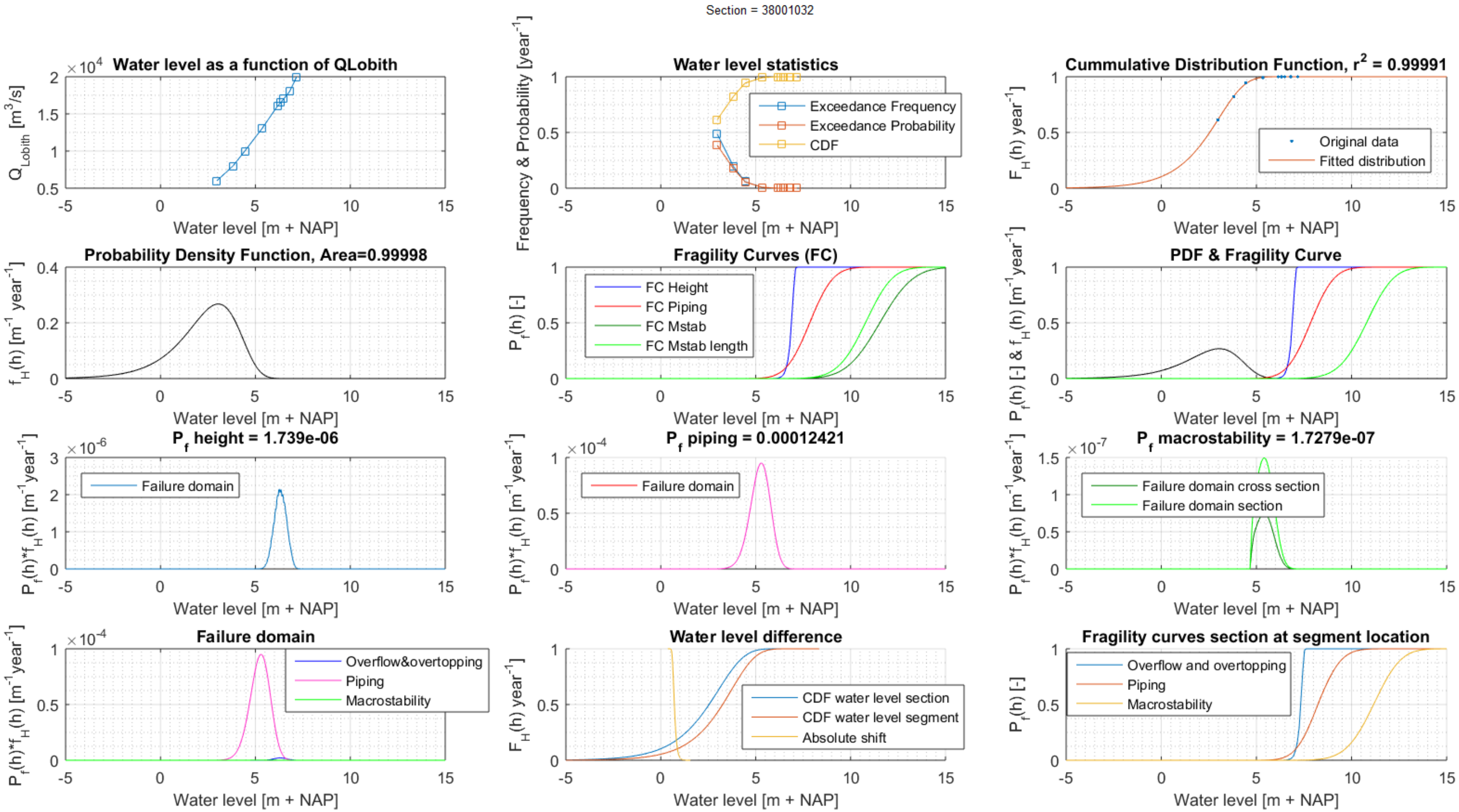

Figure 24: The calculations with respect to a dike section. Note that the 4th, and the 6th until 10th graph are not used for the flood risk calculation. These graphs only serve an illustrational purpose.

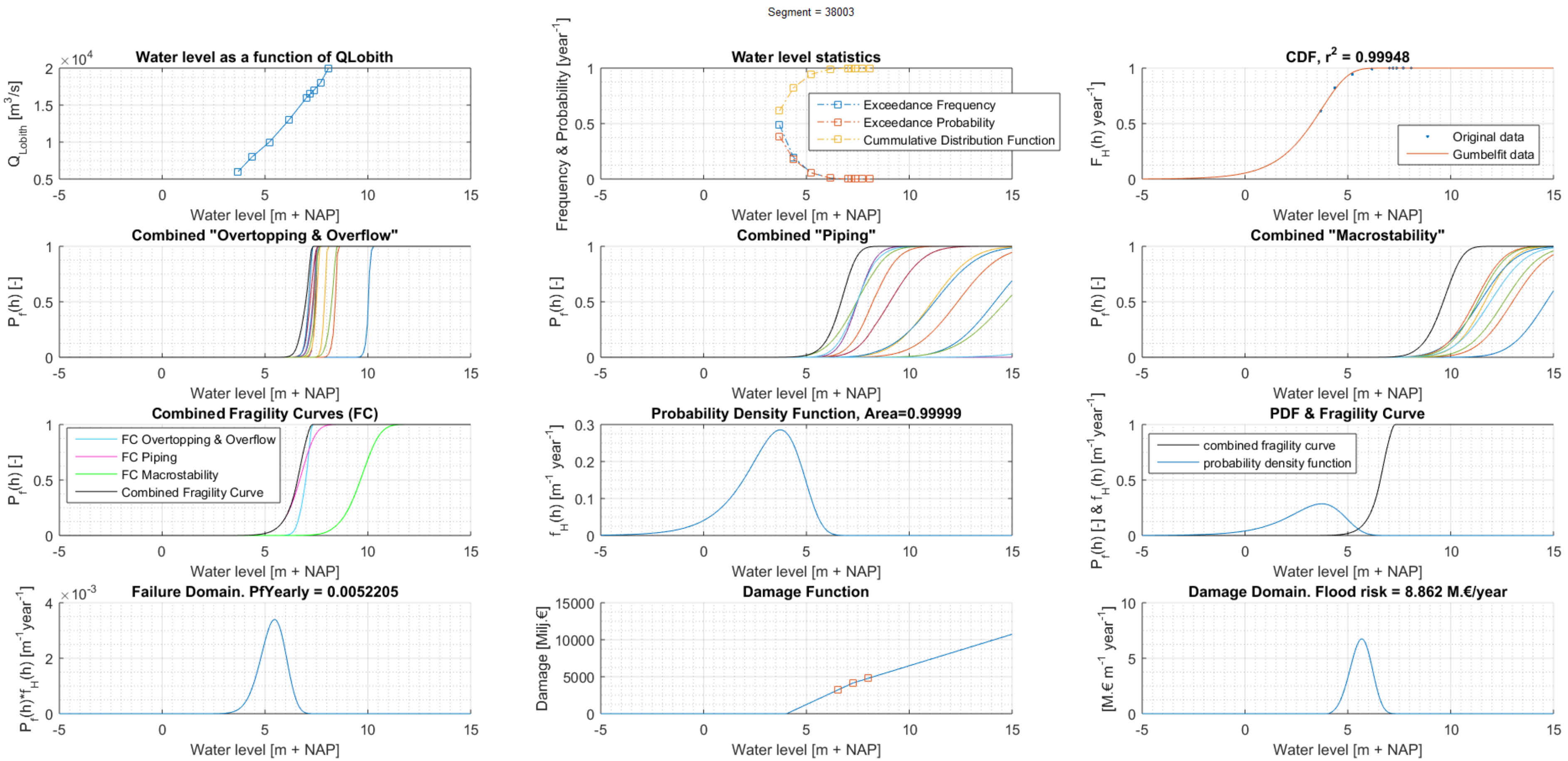

Figure 25: The calculations with respect to a dike section. Note that the damage function has not yet been scaled in this example, therefore the flood risk only holds for this specific dike section and not to a river branch.

D. Figures regarding the origin of the data of the flood risk calculation

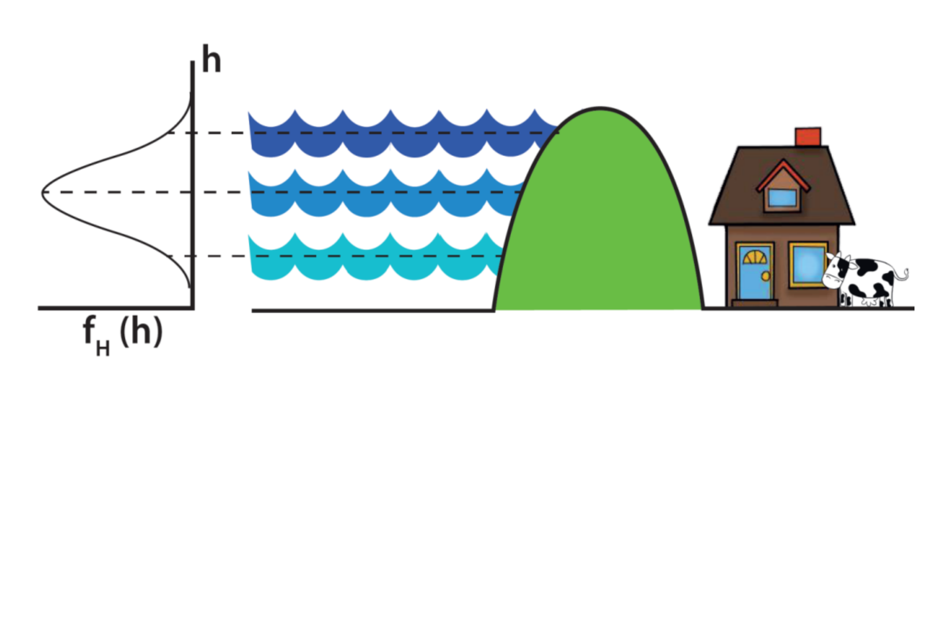

Figure 26: The probability of occurrence of a water level per year. The graph is turned 90 degrees to the left in order to visualize the origin of the data.

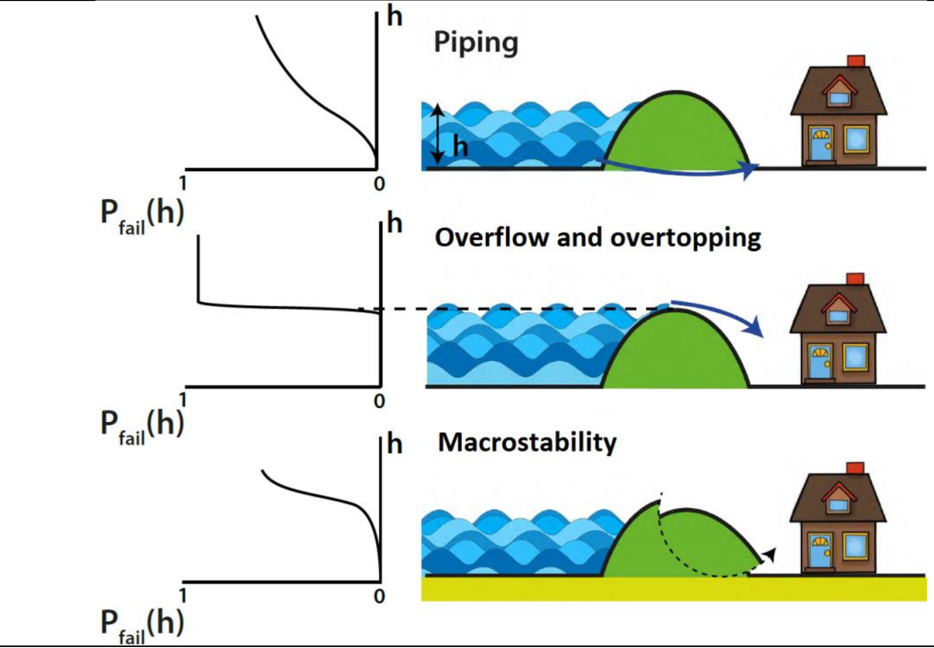

Figure 27: The conditional probability of failure given a water level for three different failure mechanism. The graph is turned 90 degrees to the left in order to visualize the origin of the data.

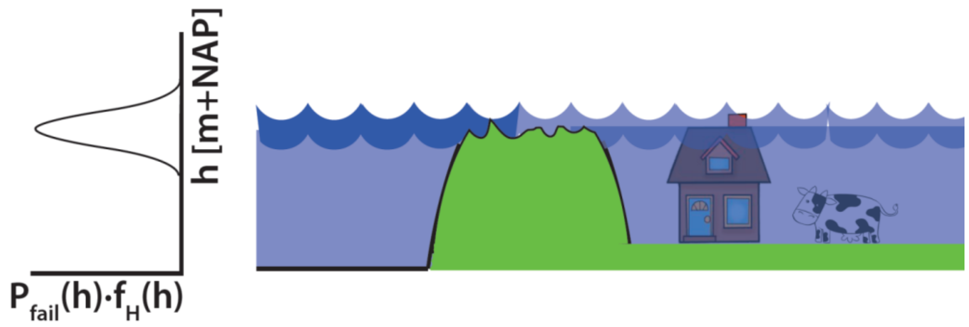

Figure 28: The failure domain for a given water level; the integral of the graph is the probability of failure. The graph is turned 90 degrees to the left in order to visualize the origin of the data.

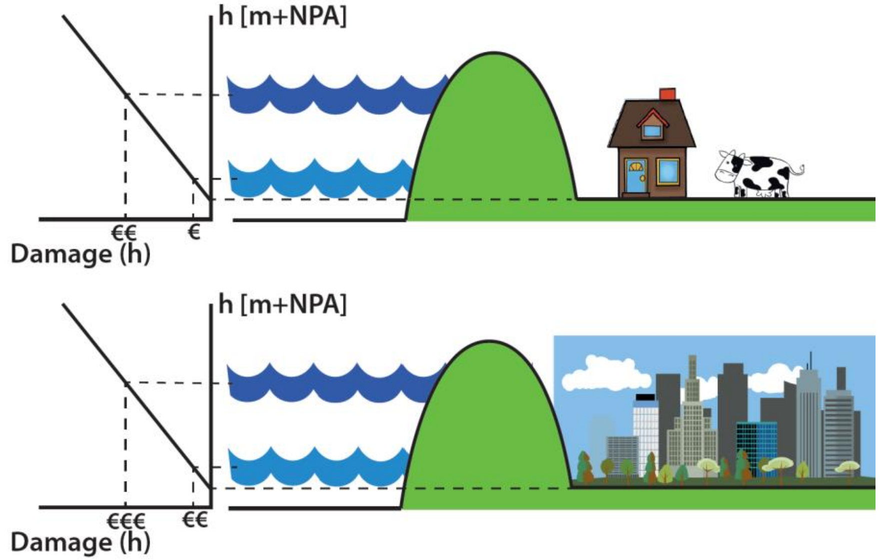

Figure 29: The damage as a function of the water level. The graph is turned 90 degrees to the left in order to visualize the origin of the data.

E. Other model runs

Flood risk as a function of the discharge towards the other river.

Since the discharge of the other branches depend upon the change in discharge of the Waal and IJssel branch, one might wonder if the behaviour of the flood risk depends upon this factor. For the IJsselkop, the discharge of the IJssel is changed in order to calculate the flood risk of the IJssel. Then the discharge of the Nederrijn was calculated trough subtracting the IJssel discharge from the discharge of the Pannerdensch Kanaal, altering values provided in Table 7.

Caused by the limited amount of data points, extensive extrapolation is necessary in order to determine the flood risk of the corresponding discharge. Therefore the flood risk calculation was also done with a change in discharge with respect to the Nederrijn river. Figure 30 shows approximately the same results as the change in discharge with respect to the IJssel, small deviations aside. Therefore this figure confirms the validity of the model with respect to this factor. Therefore this is not repeated for the bifurcation point Pannerdensch Kop.

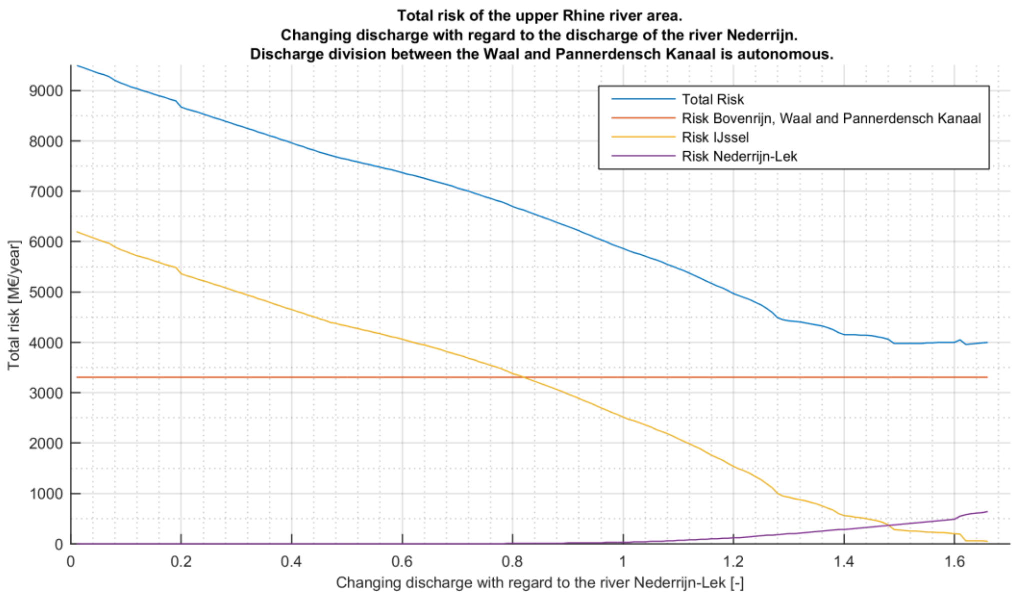

Figure 30: Flood risk calculation as a function of the change in discharge towards the River Nederrijn-Lek.

Flood risk as a function of an absolute shift in discharge.

The flood risk calculation as described in the main report is calculated based upon a precentral shift with regard to one of the distributaries. This is done since it clarifies the magnitude of the shift with regard to the discharge of the branch. When the calculation is done with an absolute shift, the flood risk calculation is shown in Figure 31 and Figure 32 for the Pannerdensche Kop and the IJsselkop respectively.

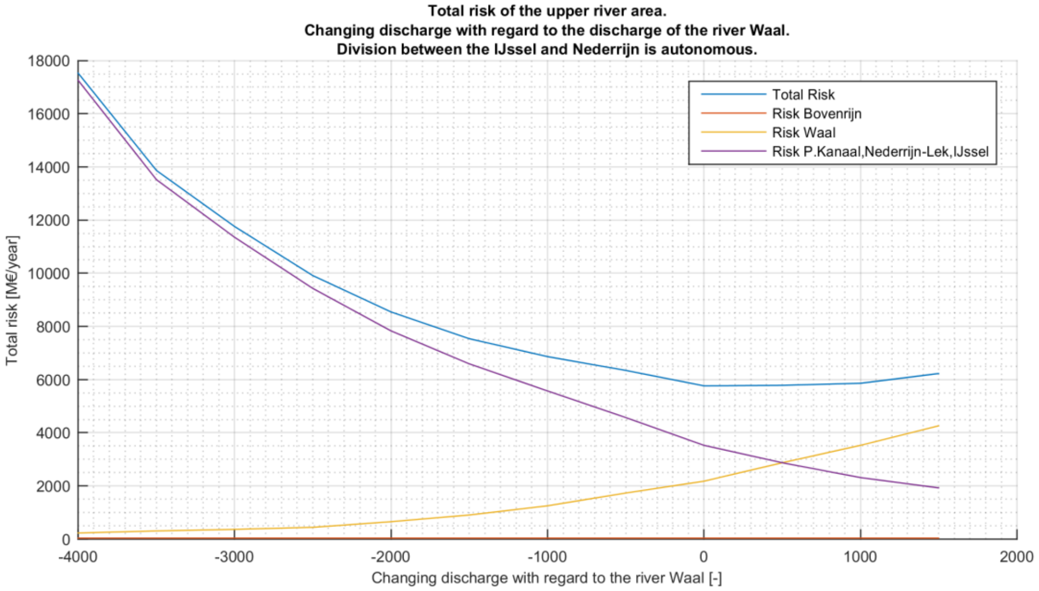

Figure 31: The flood risk calculation with an absolute shift towards the River Waal for the Pannerdensche Kop.

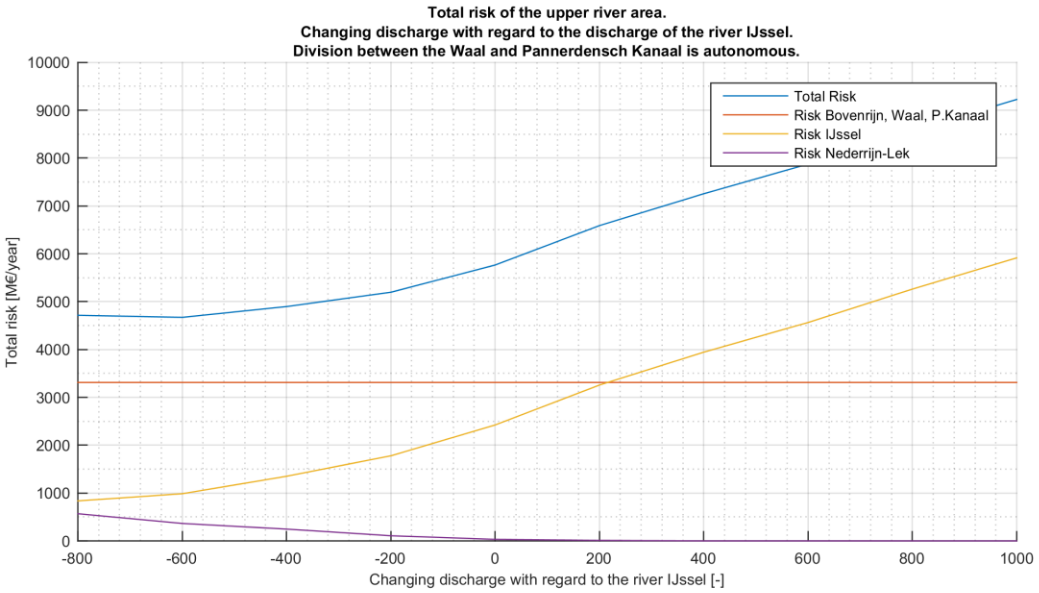

Figure 32: The flood risk calculation with an absolute shift towards the River IJssel for the IJsselkop.

F. Calculation fragility curves

The fragility curves shown in Figure 6-D correspond to one breach location, and thus represent the dike at segment level. However in Appendix B it is stated that the conditional failure data is provided at cross-section or section level, Figure 11. It therefore requires calculations to derive the fragility curves shown in Figure 6-D, which represents the fragility curve at section level. The procedures to transfer the fragility curves from one level to another are explained in this appendix. Figure 33 shows the steps that have to be taken to translate the probability and the corresponding data levels. These levels are separated in that figure by the orange dotted lines.

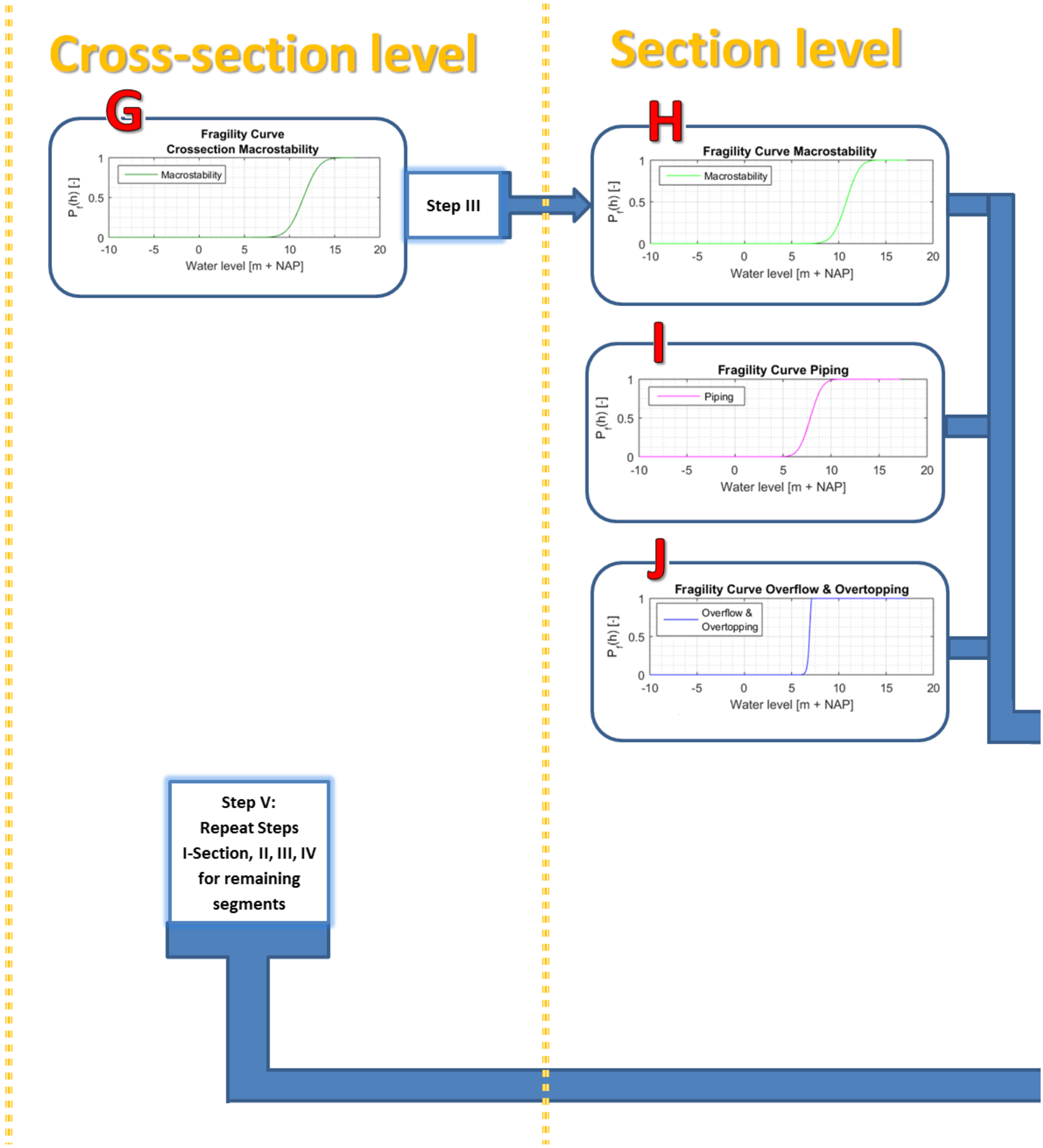

Figure 33 (Part 1 of 3): The derivation of the fragility curves at segment level. Arrows continue in Part 2 of this figure.

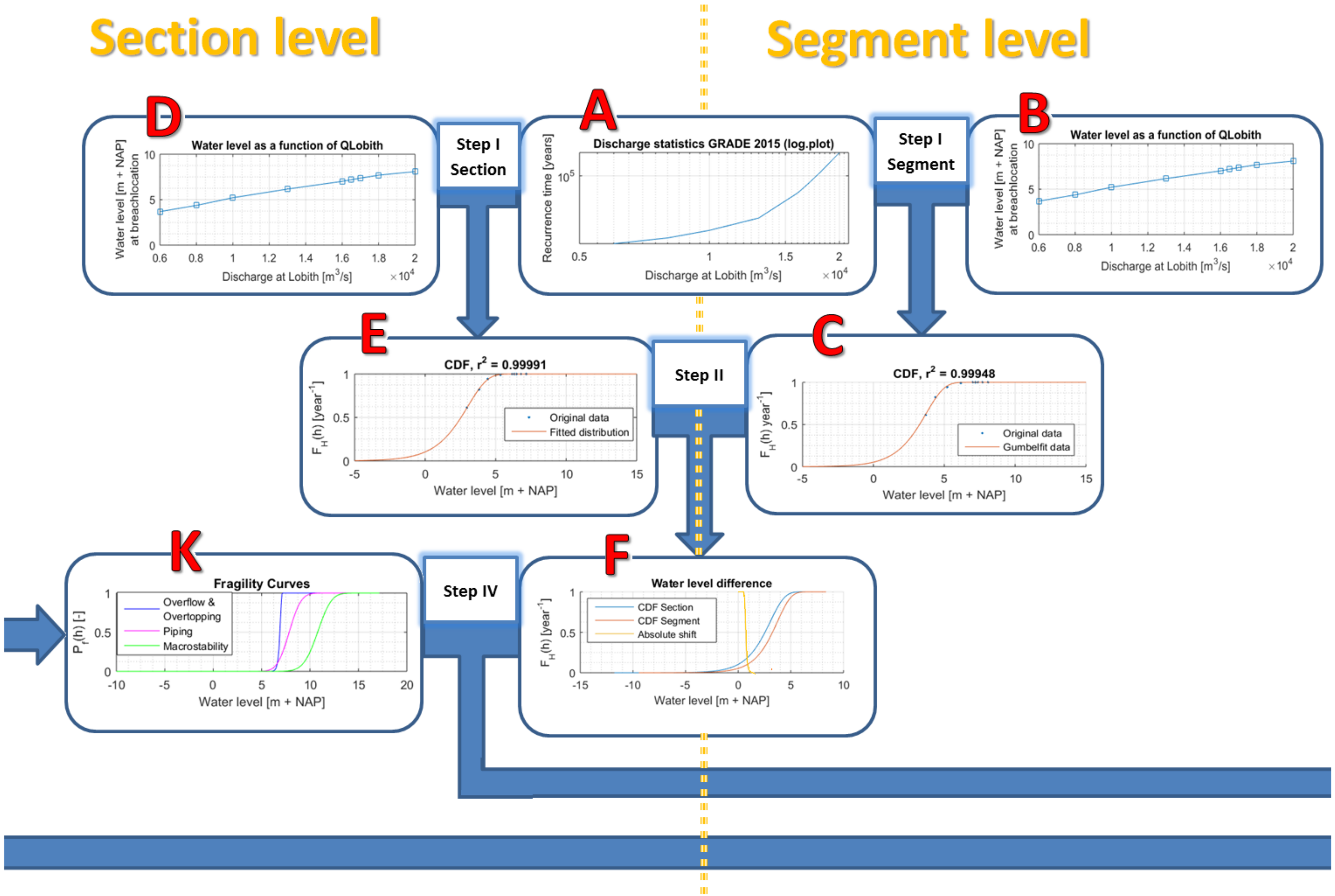

Figure 33 (Part 2 of 3): The derivation of the fragility curves at segment level. Arrows originating from Part 1 and continue in Part 3 of this figure.

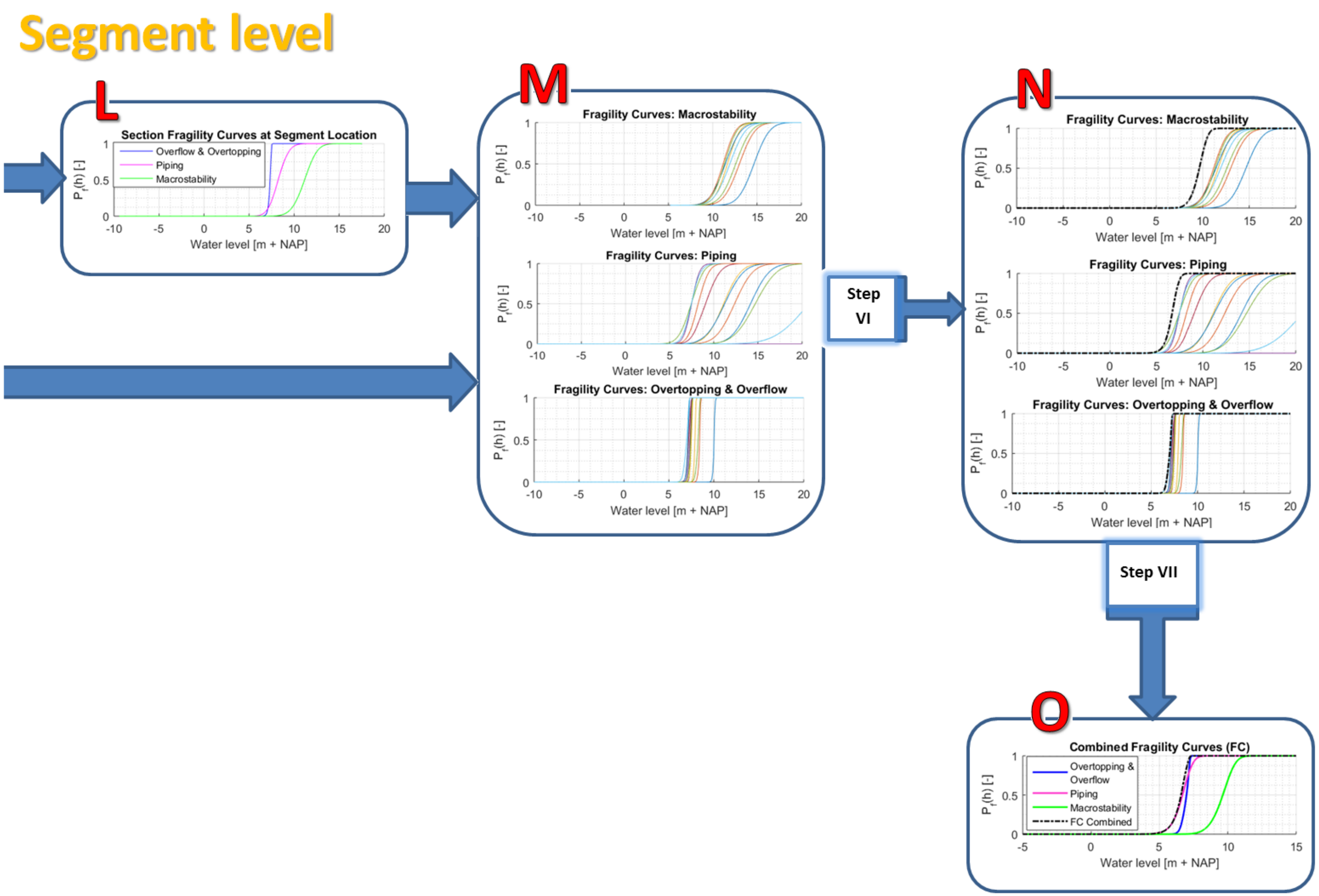

Figure 33 (Part 3 of 3): The derivation of the fragility curves at segment level . Arrows originating from Part 2 of this figure.

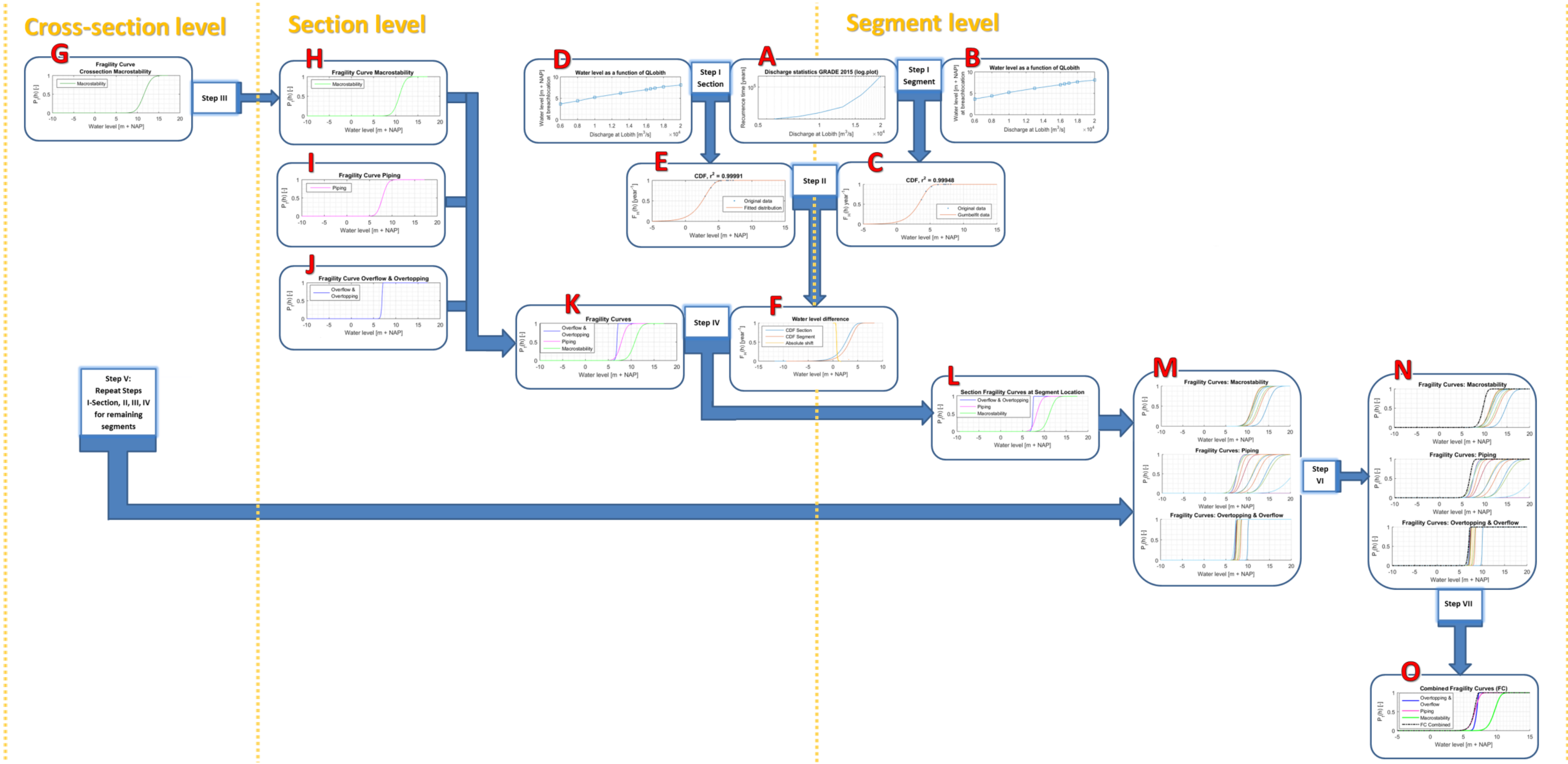

Figure 33: Overview figure of the process of deriving the fragility curves at segment level.

I. Cumulative Density Function

In order to transfer the fragility curves from section level to segment level, the cumulative density function of the water level had to be calculated for both the representative section location and the representative segment (breach) location. The derivation of this CDF is not only necessary for the calculation of the load component, but also for the strength component since it is used in the conversion of the fragility curves from cross section level to section level. Step I-segment, described in the next 3 sections: A, B and C (segment level), is identical to Step I-section. Where figures A, B, and C for the section, correspond to figures A, D, and E for the section procedure.

A. GRADE Discharge statistics

The discharge statistics provides a discharge at Lobith with a corresponding recurrence time for that discharge. The yearly exceedance frequency of occurrence of a certain flood is defined as:

Equation 7: relation between frequency and recurrence time

$$F(X \geq x) = \frac{1}{T}$$

With:

$T =$ Recurrence time [years]

$X =$ Statistically distributed event, e.g. the discharge at Lobith.

$x =$ A specific event corresponding to the recurrence time T, e.g. the discharge at Lobith of 15,000 m^3/s.

For an event with a discharge at Lobith of 15,000 m^3/s (Q), according to GRADE reference year 2015 as shown in Figure 7, Equation 7 becomes:

Equation 8: Applied relation between frequency and recurrence time for discharges

$$F(Q \geq 15{,}000) = \frac{1}{1{,}250}$$

Equation 8 is given for 9 discharge values. This was done in order to keep the amount of data of the local water levels, described in the next section, manageable. The nine discharges at Lobith are given in the first column of Table 11.

Table 11: The discharges of GRADE2015 and the corresponding statistics used for the reference situation.

Discharge [m3/s]					Corresponding statistics			
Lobith	Waal	Pannerdensch Kanaal	Nederrijn	IJssel	Recurrence time	Exceedance frequency	Exceedance Probability	Non-exceedance Probability
6000	4097	1898	1082	803	2.05E+00	4.88E-01	3.86E-01	0.6139757
8000	5370	2594	1487	1050	5.10E+00	1.96E-01	1.78E-01	0.8218478
10000	6493	3501	2104	1408	1.71E+01	5.84E-02	5.67E-02	0.9432914
13000	8338	4658	2763	1897	1.21E+02	8.28E-03	8.25E-03	0.9917542
16000	10173	5825	3374	2427	6.49E+03	1.54E-04	1.54E-04	0.9998458
16500	10571	5922	3379	2512	1.44E+04	6.96E-05	6.96E-05	0.9999304
17000	10982	6025	3380	2613	3.22E+04	3.11E-05	3.11E-05	0.9999689
18000	11736	6255	3411	2844	1.60E+05	6.24E-06	6.24E-06	0.9999938
20000	12696	7300	3900	3400	3.92E+06	2.55E-07	2.55E-07	0.9999997

B. Discharge water level relationship

The water levels along the branches are calculated for the inflows at Lobith. In order to calculate the probability of occurrence of any water level along any branch, the water levels are matched using their recurrence time.

As can be seen in Figure 33-B, the water level for this particular dike segment at an inflow at Lobith of 15,000 m^3/s is approximately 7 m + NAP. And thus, related to a water level, Equation 8 becomes:

Equation 9: Applied relation between frequency and recurrence time for water levels

$$F(H \geq h) = \frac{1}{T} \qquad F(H \geq 7) = \frac{1}{1250}$$

With H: a statistically distributed water level

h: a specific value of statistical distributed water level with a recurrence time T.

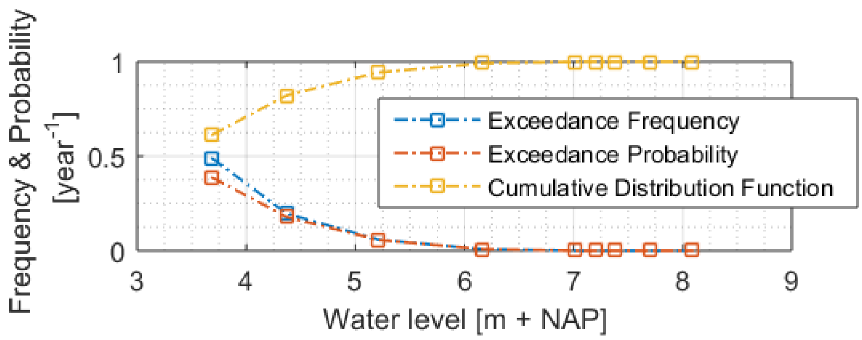

Figure 34: Exceedance frequency, Exceedance probability and the Cumulative Distribution Function for Segment 38003.

This exceedance frequency is the blue line in Figure 34.

As the peak discharges are assumed independent, it can be stated that the recurrence time of the peak discharges is exponentially distributed (Van Noortwijk et al., 1999). If the average recurrence time is an amount of years, indicated with the letter x, the cumulative probability distribution of the recurrence time T for one year can be written as:

Equation 10: The relationship between frequency and probability for one year.

$P(T \leq 1) = 1 - \exp\left(-\frac{1}{x}\right)$ (Van Noortwijk et al., 1999)

This relation between frequency and probability is shown in Figure 35. The calculated probability for the water levels is shown as the red line in Figure 34 and Table 11.

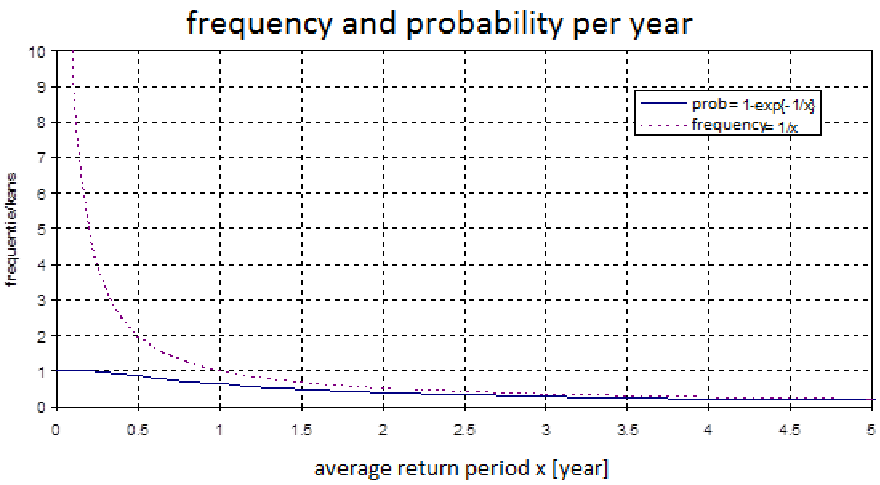

Figure 35: The relationship between frequency and probability visualized, after Van Noortwijk et al. (1999)

By the definition of probability:

$$P(\Omega) = 1$$

With $\Omega = the\ event\ space\ of\ all\ possible\ events$

the probability of all the possible events is one. Therefore, the probability of non-occurrence ('onderschrijdingskans') or cumulative distribution function, yellow line in Figure 34 and the blue dots in Figure 33-C, is:

Equation 11: The relationship between probability of exceedance and the probability of non-exceedance (PDF).

$$P(H < h) = 1 - P(H \geq h)$$

With: $H =$ statistically distributed water level.

$h =$ The water level corresponding to the recurrence time T.

C. *Cumulative Distribution Function*

Since the cumulative distribution function exist of nine data points, which do not cover the whole range of plausible water levels, extrapolation of the data is needed to obtain a finer data mesh for further use. However, since linear extrapolation does not give realistic results, another extrapolation method was applied: statistical data fitting upon the Cumulative Distribution Function of the water level.

The statistical best fit for all the given data is the Gumbel distribution, with the smallest sum of squared deviation from the function overall. The data points were therefore fitted to a CDF with Equation 12:

Equation 12: Gumbel distribution

$$F_H(h) = 1 - \exp\left(-\,exp\left(\frac{h-\alpha_{fit}}{\beta_{fit}}\right)\right)$$

With α_{fit} and β_{fit} as fitting parameters: $-\infty < \alpha_{fit} < \infty$ and $\beta_{fit} > 0$.

Where α_{fit} is the location parameter and β_{fit} is the scale parameter (Weisstein, n.d.-b). The fitting is shown in Figure 33 -C.

II. *Transfer Fragility Curves*

The fragility curves are a conditional probability of failure, given a water level. Since the fragility curves are provided at section level and the consequence data is provided at segment level, they cannot be readily compared. One cannot simply shift the fragility curves and add or subtract one value for the water level shift to shift the curves from section location to the breach location.

This is caused by the different shape of the QH-relation, which is the relation between the discharge at Lobith and the water level at any point along the river branch.

Therefor the CDF of the segment (Figure 33-C) is subtracted by the CDF of the section (Figure 33 -E). This results in an absolute shift of the water level statistics with regard to the recurrence time, the orange line in Figure 33 -F. Shifting the fragility curves over this absolute difference therefore yields the fragility curve at segment level.

III. From Cross-section to Section level

Fragility curves general

Fragility curves are conditional probabilities of failure for a given water level (Wojciechowska et al., 2015). As stated in section 3.2.1.1, the three failure mechanisms considered in this calculation are: 1) overflow and overtopping, 2) piping and 3) macro-stability.

The properties of fragility-curves are given by:

Equation 13

$P_f(h) = P(Z \leq 0|h)$ with: $P(Z \leq 0) = P(S \geq R)$ and $Z = R - S$

Where:

$P_f(h_T) = P(Failure\ and\ water\ level\ h_T)$: The probability of failure given a water level (h [m+NAP]) with the corresponding recurrence time of T [years].

Z: The reliability function of the dike, with limit state $Z = 0$.

R: The resistance of the dike, against failure.

S: The load of the water, which advances the failure.

The dike will fail when the resistance of the dike against failure is smaller than the load of the water. Thus $P(Z \leq 0|h)$ is the probability that the load is larger than the resistance, given a water level.

Fragility curves specific

As shown in Figure 33 -G, the fragility curves for the failure mechanism macro-stability were provided at cross-section level. Therefore this fragility curve first had to be adjusted in order to be a representative fragility curve for the dike section.

The conditional probability of failure for a cross-section is smaller than (or equal to) the conditional probability of failure of a dike section. In order to scale a fragility curve at cross-section level towards dike section level, the length of the dike section and the properties of the failure mechanism have to be taken into account. These factors determine the number of times the sum rule of independent probabilities has to be applied upon the conditional probability of failure.

The probability of failure for a dike section, given a known probability of failure of a representative cross-section, can be calculated by the so-called 'length-effect' . The number of times the sum rule should be applied for a cross-section is determined by the so-called N-factor (Ministerie van Infrastructuur en Milieu Directoraat-Generaal Water Waterdienst Rijkswaterstaat, 2015):

Equation 14: Length effect factor N.

$$N = 1 + \frac{\alpha * L_{section}}{\beta}$$

With:

α The fraction of the length of the section that is sensitive for the dike failure mechanism. [-]

β The length of the independent, equivalent parts of the dike section for the failure mechanism. [m]

$L_{section}$ The length of the dike section. [m]

For the upper River Rhine area considered in this study, the parameters α and β are given for the failure mechanisms piping and macro-stability in Table 12.

Table 12: Values for alpha and beta (Ministerie van Infrastructuur en Milieu Directoraat-Generaal Water Waterdienst Rijkswaterstaat, 2015).

Failure mechanism	Parameter α [-]	Parameter β [m]
Piping	0.90	300
Macro-stability	0.033	50

The amount of cross sections in one dike section is approximated with the rounded integer N. The failure properties of the cross-sections in one dike section are equal; since one dike section is chosen in a way that it has approximately the same probability of failure, see Figure 10. If one dike section can be represented by two cross-sections (N=2), we want to know the probability of one of them fails. Therefore we call the event of failure of the first represented cross-section A, and the other B. The probability of failure that A or B fails, given a water level (h) is:

Equation 15: The sum rule of independent probabilities

$$P_f(A \cup B|h) = P(A|h) + P(B|h) - P(A \cap B|h)$$

$$= P(A|h) + P(B|h) - P(A|h)P(B|h)$$

With: $P(A|h) = \frac{P(A*h)}{P(h)}$ and $P(B|h) = \frac{P(B*h)}{P(h)}$

An arbitrary visualization of this procedure is shown in Figure 36: where the sum of failure of A and B will become 2, which is not possible by the definition of a probability, the values calculated with the sum rule, as described in Equation 15 do not exceed 1.

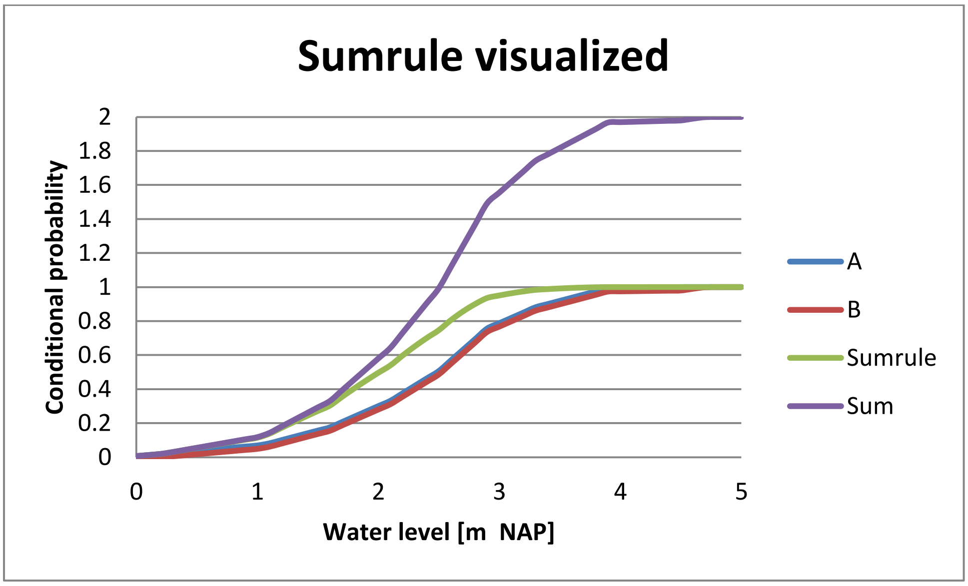

Figure 36: The sum rule visualized for not mutually exclusive independent probabilities.

And thus with this scaling factor, the probability of flooding of a dike section can be calculated. The fragility curve for a cross-section and a section for macro-stability are shown in Figure 33-G, respectively H.

IV. From Section to Segment level

The fragility curves shown for the 3 failure mechanisms shown in Figure 33 -H, Figure 33 -I and Figure 33 -J are the fragility curves for macro-stability, piping and overflow/overtopping respectively. In order to transpose these fragility curves to segment level, the shift in water level, as calculated in step II, is added to the corresponding water level of the fragility curve.

After this shift, the fragility curves on section level, shown in Figure 33 -K are shifted to the breach location (segment level) in Figure 33 -L.

V. Repeat for remaining sections

Most dike segments consist of more than one dike section. Therefore the former steps (except step-I section) have to be repeated for the other dike segments of the dike section. For example, this dike segment 38003, consists of 12 sections. When these are shifted towards the breach location, they are plotted together in Figure 33 -M.

VI. Segregate Fragility Curves Failure mechanisms

A levee system can be regarded as a serial system: it fails if one of its elements fails.

The combined probability with regard to the failure mechanisms piping and macro-stability, were derived by taking the sum for all independent probabilities given a water level, given in Equation 15, generalized to unions of arbitrary numbers of events. This results in black dashed line in the first and second graph of Figure 33 -M.

The probability that a dike segment will fail with regard to overflow/overtopping will be the largest where the crest of the dike is the lowest, relative to the water level. Therefore the combined fragility curve for dike segment j consisting of i is the black line shown in the third graph of Figure 33 -M.

Equation 16

Combined probability for overflow/overtopping: $P_{f,j}(h) = Max(P_{f,i}(h))$

VII. *Segregate Fragility Curves*

Finally the combined fragility curves of the different failure mechanisms are combined, with the sum rule, to one fragility curve: 7th graph of Figure 25.

Equation 17

$$\begin{aligned} P(A \cup B \cup C|h) \\ &= P(A|h) + P(B|h) + P(C|h) - P(A \cap B|h) - P(A \cap C|h) - P(B \cap C|\, h) \\ &+ P(A \cap B \cap C|h) \\ &= P(A|h) + P(B|h) + P(C|h) - P(A|h)P(B|h) - P(A|h)P(C|h) \\ &- P(B|h)P(C|h) + P(A|h)P(B|h)P(C|h) \end{aligned}$$

With: $P(A|h) = \frac{P(A*h)}{P(h)}$ and $P(B|h) = \frac{P(B*h)}{P(h)}$ and $P(A|h) = \frac{P(B*h)}{P(h)}$

G. Fitting of the water level statistics

As stated in Appendix F-I: the water level statistics are fitted upon a cumulative distribution function defined by Gumbel (Weisstein, n.d.-b). The probability density function, indicating the yearly load of the water, is the derivative of this cumulative distribution function. This function is stated in Equation 18.

Equation 18: The probability density function.

$$f_H(h) = \frac{1}{\beta_{fit}} \exp\left(\frac{h - \alpha_{fit}}{\beta_{fit}} - exp\left(\frac{h - \alpha_{fit}}{\beta_{fit}}\right)\right)$$

With mean and variance:

$\mu = \alpha_{fit} - \gamma\beta_{fit}$ where $\gamma = 0.577$, known as the Euler-Mascheroni constant.

$\sigma^2 = \frac{1}{6}\pi^2{\beta_{fit}}^2$

By the definition of a probability density function Equation 19 and Equation 20 should hold (Mood et al., 1963):

Equation 19

$$\int_{-\infty}^{\infty} f(h_T)dh = 1$$

Equation 20

$$f(h) \geq 0 \; for \; all \; values \; of \; h$$

When applied for a range in water level of 4σ from the mean in positive and negative direction, 99,994 % of the expected values are within this interval. This is shown in the 4th graph of Figure 24 and the 8th of Figure 25. The total area under the graph as calculated is shown in the title.

Once the distribution is changed from the reference situation, fitting takes place upon the new water levels. Due to this new fitting, other values for $\alpha_{fit}$and β_{fit} can be calculated as more optimal, which results in a changing load. Since only 9 data points are available , these parameters highly depend upon the interpolation or extrapolation of these data points. In order to reduce this factor, only the location parameter α_{fit} is changed, whereas the scale parameter β_{fit} was kept the same as the reference situation.

Dankwoord

Na een intensieve afstudeer periode nadert het einde van mijn master *Civil Engineering and Management* aan de Universiteit Twente. Dit betekent niet alleen het einde van mijn studententijd in Enschede, maar ook het aanbreken van een nieuw tijdperk. Mijn afstudeeronderzoek gaat over de invloed van het verschuiven van de afvoerverdeling op het overstromingsrisico. Doordat het onderzoek vrij omvangrijk is, ben ik zelf ook veel bezig geweest met de invloed van het verschuiven van mijn aandacht naar bepaalde componenten van het onderzoek om een zo compleet mogelijk beeld te vormen. De afgelopen tijd heb ik dan ook veel geleerd, niet alleen op wetenschappelijk gebied, maar ook op persoonlijk vlak. Met dit dankwoord wil ik me richtten tot allen die me hebben geholpen met het tot stand komen van dit afstudeerrapport.

Allereerst wil ik graag mijn afstudeercommissie bedanken voor hun aanzienlijke aandeel in dit onderzoek. Andries: bedankt dat je, ondanks je drukke schema, toch altijd momenten wist te vinden om met mij te sparren en me te bestoken met vragen over mijn onderzoek. Ik heb veel van je geleerd. Kees: bedankt voor het vertrouwen dat je mij gegeven hebt. Ik heb veel gehad aan je scherpe blik en doorgrondende vragen: je wist de studie altijd in een groter perspectief te zien en daarbij de details niet uit het oog te verliezen. Ralph: bedankt voor de oprechte interesse en je enthousiasme over mijn onderzoek, dit heeft me altijd erg positief gemotiveerd. En tot slot Denie: bedankt voor je feedback, je hulp in het wetenschappelijke deel van het onderzoeksproces en het stimuleren van het verbeteren van mijn schrijfproces.
Het was een waar genoegen om met jullie samen te mogen werken, hoofdzakelijk dankzij jullie heb ik heb mijn afstuderen als erg prettig ervaren.

Verder wil ik graag alle medewerkers van *HKV lijn in water* bedanken voor de leuke en leerzame tijd op kantoor in Lelystad. Omdat mijn onderzoek raakvlakken heeft aan meerdere expertises, was de benodigde kennis of informatie vaak over verschillende experts verspreid. De vele gesprekken waren dan ook niet alleen nuttig voor mijn onderzoek, maar ook reuze interessant. Bedankt aan ieder die heeft bijgedragen aan mijn onderzoek en een speciaal woord van dank voor Joost Pol, voor het meedenken en zijn bijdrage aan de overstromingsrisicoberekeningen.
Naast de medewerkers van HKV, ben ik ook Nathalie Asselman van Deltares en Robert Vos van RWS dankbaar voor het meedenken over mijn onderzoek in de oriëntatiefase.

Ten slotte wil ik graag mijn familie en vrienden bedanken voor hun liefde en steun gedurende mijn studie. Een speciaal woord van dank voor de onvoorwaardelijke steun van mijn ouders en jullie nimmer aflatende vertrouwen, en voor Erik, voor je begrip en je oneindige geduld. Ik bin tige wiis mei dy!

Elsbeth Brandsma,
Enschede, 24-8-2016