DELFT UNIVERSITY OF TECHNOLOGY & HKV LIJN IN WATER

The economic risk for inland water transport due to uncertainty in morphodynamic behaviour

by

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Preface

In front of you lies the thesis *The economic risk for inland water transport due to uncertainty in morphodynamic behaviour*. It was written in partial fulfilment of the requirements for the degree of Master of Science in the field hydraulic structures and flood risk. The possibilities of a risk assessment method for morphodynamic behaviour were investigated and applied to consequences for navigation and uncertainty in discharge. The research was set-up as part of a larger investigation to develop an integral risk assessment for morphodynamic behaviour.

It was a long road with its ups and downs, but I was fortunate enough to do the research at HKV, where Saskia van Vuren and Joost Pol were always prepared to help me in any way possible. Also the other colleges there made my time writing much more fun with the occasional joke and lively lunches. I would like to thank everyone at HKV, but especially Saskia and Joost their great guidance and support these couple of months. Likewise Andries Paarlberg was a great help in setting up the model and helping me figure out how it worked.

I wish to thank my supervising committee for helping me broadening my view and putting theory into practise. They were always prepared to answer any enquiries and give constructive criticism when needed. My gratitude also goes out to Charta software, IVR and Panteia who provided me with the data and tools needed to complete my research.

Last, but most certainly not least, I would like to thank my family and friends for being there when I needed to vent about the work not going as planned, motivating me and distracting me with fun activities. Especially my mother and Wouter always stood by my side, I appreciate it immensely.

I hope you enjoy reading my final thesis.

Mirjam Flierman

Delft, March 9th 2017.

Summary

River systems are vastly changing environments, not only due to natural process, but also human intervention play an important role. With the many different river functions at play, large amounts of maintenance budgets are associated with river management. One of these river functions, which is sensitive to morphodynamic changes, is inland water transport. Due to insufficient depths ships have to unload or may not travel at all, inducing economic losses for the shipping companies. For decision makers it is important know expected costs or losses and the spread therein to decide on mitigating measures. Therefore the question asked and answered in this thesis is *How can economic risk for navigation related to uncertain morphological behaviour be quantified and how can it be used to support decision making?*. Risk is defined as the product of the probability of an undesired physical morphodynamic state times the consequences. This thesis proposes a risk assessment method that quantifies uncertainties in morphodynamic behaviour and its consequences for inland water transport. The risk assessment method has been applied to a nourishment case study to investigate potential of the approach for decision making.

The first step to answer this question is the evaluation of uncertainties in morphodynamic behavioural modelling. From literature it was found that the main uncertainty sources are the discharge variation, hydraulic roughness and bedform prediction and the parameters of sediment transport formulae. The discharge variation was selected as an uncertainty source to be investigated in this study. The propagation of uncertainties from uncertain discharge variation into morphodynamic states and depth conditions has been done by the means of Monte Carlo Simulations (MCS) using a morphodynamic model of the Dutch Rhine system. The model concerns a quasi-3D morphodynamic Delft3D model of the Upper Rhine and Waal, hereafter called DVR model. The principle of MCS is to run the deterministic morphodynamic DVR-model repeatedly, each time with a different set of statistically equivalent model inputs. In this case a discharge time series. To that end, 100 times a discharge-time series of 20 years duration have been synthesised and were imposed as input for the morphodynamic DVR-model. The model outputs were compared with measurement data and the different aspects of morphodynamic behaviour were discussed.

The Least Available Depth (LAD) somewhere along the Upper Rhine and Waal was deduced for all time steps and for all model simulations. The statistics of the LAD showed a large variation even for the same discharge level. The 90% confidence interval for the middle discharges is about 0.4m. The variation was mostly caused by differences in morphology and bedforms, showing the importance of including morphology in risk assessments. A comparison with a deterministic simulation using the DVR hydrograph, which is usually used with the DVR model, was done. This hydrograph only contains one peak each year and a different discretisation. The comparison showed the importance of using a full range of discharges since otherwise some low depths never occur.

The results from the morphodynamic simulations, as a LAD-time series, were used to determine the consequences for navigation. The consequences of undesired morphodynamic behaviour and consequently restricted navigation depth consist of welfare losses due to unloading (or even not travelling) and dredging costs.

The welfare loss is defined as the change in consumer surplus between a situation without depth restrictions and one with depth restrictions. The consumer surplus is the difference between what the

consumer is willing to pay and what they did pay: their benefit of the transaction. These benefits decrease due to higher prices, because of ships not being able to navigate fully loaded. The initial costs and some other variables (transported quantities per ship as a function of time, load factors, etc.) needed for this calculation are extracted from a database created by BIVAS: a network shipping model that calculates shipping costs.

The dredging costs are directly determined using dredging volumes that form output of the DVR-model times a unit price. A dredging module is integrated in the Delft3D model, which is similar to the way dredging activities are done in reality. In addition to the dredging costs per volume, also the distance between dredge and dump location was considered. The dredging costs turn out to be small compared to the welfare loss: less than 1%. The expected total yearly costs (welfare loss + producer loss + dredging costs) amount to 68.3 million euros with a skewed 90% confidence interval of 185.4 million euros. (Jonkeren et al.; 2007) found an expected yearly loss of 28 million. They also used the concept of economic surplus, but applied it to a historic data set of water, price and load data. The median of the costs is similar to this value (43.9 million euro), but due to some extreme years the expected costs are higher.

The last part of the risk assessment method is dealing with risk, where a case study applying nourishments on the IJssel, Boven-Rijn and Waal was compared with the reference case. The nourishments are applied for mitigation of the bed degradation on the Dutch Rhine. While bed degradation might seem favourable for flood safety due to lower water levels, it forms a risk to stability of structures. Bed degradation might also cause local reductions in navigable depth due to fixed layers that do not lower with other parts of the river, while the water level does. The decrease in welfare loss due to application of nourishments was small. However, the expected benefit of the measure in 15 years was 1.4 million euro and continued to be more profitable in time. Since the nourishments serve more than one purpose they will probably be even more profitable if an integral risk assessment, including more river functions, was applied.

The risk assessment method developed and applied gives a first estimate of the risk for navigation due to morphodynamic behaviour. However, it is computationally expensive (simulations take in the order of weeks on the Dutch national super computer), but the importance of the use of MCS or other uncertainty propagation was proven in the study. The method can be used by decision makers to compare mitigating measures. The full profits of the nourishment case could not be shown since only the river function navigation was investigated.

Therefore, it is recommended that in future studies other river functions such as flood safety, cables and pipelines and environment and recreation are taken into account. Also the other uncertainties: hydraulic roughness and bedform prediction and the transport parameters should be investigated and the Delft3D DVR model needs to be validated. It is also recommended that the supply and demand of inland water transport on either market or single trip scale is further investigated. Finally, it is also recommended that the definition of welfare loss for different stakeholders is further investigated and an estimate for the indirect costs is made to assess whether they should be included in the method.

Contents

Abbreviations

ALD	**A**greed **L**ow **D**ischarge
ALW	**A**greed **L**ow **W**ater
DVR	**D**uurzame **V**aardiepte **R**ijndelta (Sustainable Fairway Rhinedelta)
EH	**E**ngelund **H**ansen
FORM	**F**irst **O**rder **R**eliability **M**ethod
LAD	**L**east **H**vailable **D**depth
LHS	**L**atin **A**ybercube **S**ampling
MCS	**M**onte **C**arlo **S**imulations
MPM	**M**eyer **P**eter **M**uller
NURG	**N**adere **U**itwerking **R**ivieren **G**ebied
RftR	**R**oom **f**or **t**he **R**iver
rkm	**r**iver **k**ilo**m**etre
WFD	**W**ater **F**ramework **D**irective

Symbols

A	dredging price	€$/m^3$
C	costs of freight transport	€/tonne
C_w	waterplane-coefficient	
C	Chzy coefficient	$m^{1/2}/s$
D	demand function	
D_50	median grain diameter	m
d	vessel draught	m
EC	economic welfare loss	€
f	load factor	
L	vessel length	m
p	price of freight transport	€/tonne
Q	discharge	m^3/s
q	transported load	tonne
t	time	years
$TPCMI$	tonnes per centimetre immersion	tonne/cm
u	flow velocity	m/s
W	vessel width	m
α	calibration coefficient	
Δ	relative density	
ϵ	price elasticity of demand	
ϵ	hiding and exposure factor	
θ	shields parameter	
θ_{cr}	critical mobility factor	

Chapter 1

Introduction

1.1 Background

Before civilization as it exists now, the first settlements were often near rivers and in many countries this is still the case. The reason people tend to live near rivers seems obvious at first: for drinking water and agriculture. However, there are many more rationales behind it: rivers can be used for transportation, river deltas are more flat and thus the area is easier to use and rivers used to be a good way of transporting waste. Unfortunately living near a river can also have negative consequences, such as risk of flooding.

In the Netherlands over many years adjustments have been made to the river system to make optimal use of its functions and to protect the country against flooding. One of the largest changes is the normalization of the rivers in the late 19th century: river bends were cut off, groynes were implemented and less room was made available for floodplains. A reason for this was improvement for navigation, but it was also done in order to create more room for people living close to the rivers and on the flood plains. As consequence the system needs to find a new equilibrium bed for the shorter river. The new equilibrium has a smaller slope since the sediment transport increased due to higher flow velocities, while the sediment supply upstream did not increase. Moreover, in Germany the use of river power was introduced and dams were built for this purpose. This resulted in entrapment of sediment and together with the consequences of normalization the Dutch Rhine branches started eroding. Also in both Germany and the Netherlands sand mining took place, increasing the erosion process. This large scale erosion process has been going on for two centuries and will go on for at least one more century if no measures are taken. The current amount of bed level decrease in the Dutch Rhine branches is 1 to 3 cm/year (Huthoff et al.; 2015).

Some of the consequences of bed degradation and some small scale morphological behaviours are:

- Nautical bottlenecks will be formed at fixed layers (locks, fixed beds)
- Insufficient ground cover (cables, pipelines)
- Stability problems for foundations of structures (bridges, banks)
- Changes in discharge distributions at bifurcations causing water shortages and changes in large scale sediment distribution
- Desiccation of floodplains

Currently a few measure programs are being implemented to improve flood safety, but also reduce bed level decrease. One of the large programs is Room for the River which includes many measures such as decreasing groyne height, side channels and floodplain adjustments. The main reason for these measures is to increase discharge capacity and thus flood safety. Many of the measures establish the increase in discharge capacity by redirecting flow in side channels ore over flood planes and groynes. This takes discharge out of the main channel slowing down the flow in the main channel and causing sedimentation. At the location where the flow joins the main channel again the opposite happens: erosion. As a consequence accretion and erosion waves propagate through the river system, possibly causing problems for navigation.

All measures have positive and negative consequences for the previously described river functions. The effects should be analyzed when designing the river system, while keeping in mind that different functions might have different requirements of the river system.

To predict future river behaviour and acquire an understanding of the long-term morphodynamic processes, morphological models have been developed. These models are often used deterministically to compare consequences of different measures and reference situations and predict the impact on various river functions. Still a lot of uncertainty exists in the models, not only the accuracy of the models themselves, but also the input parameters and boundary conditions exhibit a large range of uncertainties. Some of these uncertainties cannot be decreased by more knowledge: they are due to natural variability (inherent uncertainties). The uncertainty in river bed behaviour needs to be quantified for decision makers to get a good understanding of morphological response and its uncertainty to make well informed decisions.

1.2 Problem description

To be able to make balanced decisions about handling issues caused by morphological behaviour and counter measures an accurate cost-benefit estimation is needed. However, the costs and benefits are uncertain since the future circumstances are unknown and depend on many uncertain variables. Therefore, a wide range of possible morphodynamic states, their probabilities and consequences should be assessed to get an impression of the risk related to (autonomous) river behaviour. Or in other words a systematic integral approach trailing the uncertainties to the consequences that can be used by, for example, river managers will be developed.

The start of a risk assessment is the systematic analysis of the uncertainties involved. While some studies have investigated the uncertainties that influence morphological behaviour, there are still large knowledge gaps in this area. The uncertainties can be quantified using measurements (Van Vuren; 2005), probabilistic approaches (van der Klis; 2003; Van Vuren; 2005) and expert judgement (Warmink et al.; 2011). They generally agree on what the significant factors are (roughness, bed topography, sediment transport parameters and discharge) and how they propagate as uncertainty through the morphodynamics of a river system. Often studies that analyse the uncertainties only partly link them to consequences and risks for the functions of the river, but not fully translate them to economic costs. Moreover, in present-day engineering practice a deterministic approach is used. The step of trailing uncertainties to the consequences and risk is of importance for policy makers. Therefore, a risk assessment method for morphological behaviour and a method for cost-benefit considerations of risk reduction measures is needed.

1.3 The scope of this thesis

A risk assessment generally starts with a system definition. While the overall aim of this study is to develop a versatile morphological risk assessment method, it is first applied to one specific uncertainty which is trailed to the consequences for one river function. In the last part of this research is analysed how the method can be broadened to include multiple river functions, other study areas, be used in operational context and the future. This method for the research was chosen to be able to do an in-depth study of a risk assessment method without getting lost in the many different uncertainties and river functions. The chosen river function is inland water transport and the uncertainty for the in-depth analysis is discharge variability in time.

Discharge uncertainty is an significant contributor to the variation in morphological behaviour of a river system (van der Klis; 2003; Van Vuren; 2005; ; n.d.). An interesting aspect of discharge uncertainty is its inherent variation in time and that it affects almost all river processes. Since already large amounts of discharge data is readily available, the uncertainty in discharge variation can be easily estimated.

As mentioned earlier the autonomous bed degradation and other morphological changes have large effects on the following river functions: flood safety, water supply, cables & pipelines (and stability of structures), navigation and environment & recreation. Since the knowledge on flood safety is currently quite advanced, the investigation of morphological risk of water shortage involves an in-depth study of the bifurcations and for the subject of cables and pipelines more data needs to be acquired before being able to develop a risk assessment methods, the most suited river function for this study is navigation.

Especially on the subject of decision making a lot is still unknown and the welfare loss for navigation can be quite large: in 2003 the welfare loss is estimated to be € 91 million due to insufficient water depths (Jonkeren et al.; 2007).

The study area of this thesis will be limited to the Boven-Rijn and Waal. This study area was chosen because the Boven-Rijn and Waal are important routes for navigation: sea to hinterland (Ruhrgebied). Also already some detailed morphological models and measurement data for this area are available.

1.4 Research questions

The aim of this study is to develop, analyse and apply an integral risk assessment and to quantify morphological risk for navigation in monetary values. The steps of the risk assessment method are incorporated in the first 4 sub-questions shown below. The last question looks at the possibilities of using the developed method in a broader context.

Research question:

How can economic risk for navigation related to uncertain morphological behaviour be quantified and how can it be used to support decision making?

Sub-questions:

1. How can morphodynamic behaviour of the Rhine system be modelled and which uncertainties are most important therein?
2. How can uncertainty in discharge be quantified and how does it affect uncertainty in morphodynamic behaviour?
3. How can the economic consequences and risk related to uncertain river behaviour be quantified for navigation?
4. How can the risk assessment method support decision makers?
5. How can the developed risk assessment and management method be applied in a broader context?

1.5 Research outline

The contents of this thesis are shown in Figure 1.1. In black the main chapters are given, only these chapters are elaborated on below.

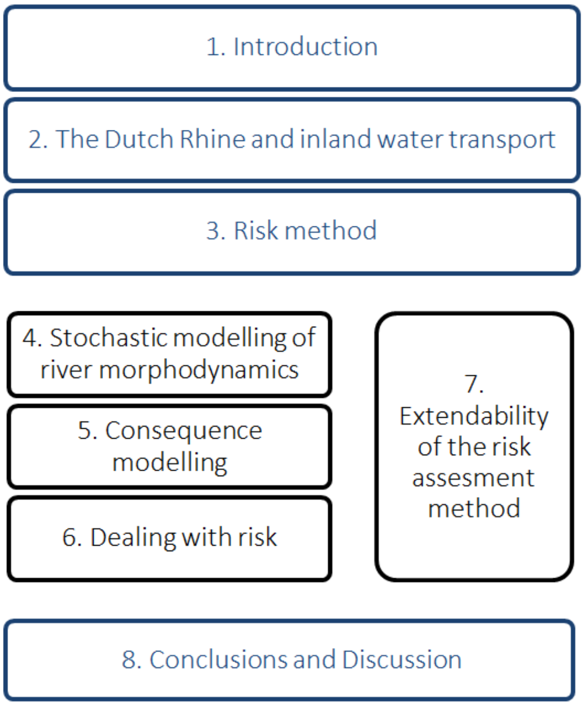

FIGURE 1.1: The chapters of this thesis.

Stochastic modelling of river morphodynamics The important processes for morphodynamic river behaviour with regards to navigation are listed and a model is selected that includes these processes. Since there is much uncertainty in prediction of morphodynamic behaviour, the important uncertainty sources are described and the uncertainty in discharge is elaborated on in more detail. Some methods for propagation of uncertainties through the model are discussed and again one is selected. After which a general description of the modelled morphological behaviour is given and the shortcomings of the model are relayed.

Consequence modelling The next step in the risk assessment is translating the morphodynamic system response to its implication for water depth developmet dynamics and asses its consequences for the river function navigation (dredging and navigability). This is then translated to monetary consequences with probabilities. This chapter concludes with an estimate of the monetary risk for navigation due to morphodynamic behaviour.

Dealing with risk The possibilities for evaluation of risk are discussed. After which a case study including a measure is chosen and compared with a reference case. Also a cost-benefit analysis is done comparing costs for different stakeholders for a situation with or without measure.

Extendibility of the risk assessment method The larger aim of this study is to develop a integral risk assessment for morphology. The applicability of different uncertainties and river functions, and also the possibility of extending the method in operational context, the future and for different types of study areas are discussed.

Chapter 2

The Dutch Rhine and inland water transport

The aim of this study is to develop a risk assessment approach for morphology in order to support decision making. To be able to apply the approach a study area is chosen and a river function to be investigated for consequence modelling was specified to limit the scope of this study. In this chapter a general account of the study area: the Boven-Rijn and Waal is given in morphological sense as well as for inland water transport

The Rhine flows through six countries from the Swiss Alps to the North Sea in the Netherlands. The Rhine flows into the Netherlands from Germany as the Boven-Rijn and soon splits into the Waal and Pannerdensch Canal (see Figure 2.1). The inflow location is near Lobith and the discharge there is often used as reference in this study. The average discharge of the Rhine at Lobith is 2200 m^3/s and approximately two-thirds of this discharges is distributed to the Waal.

2.1 Inland water transport

The port of Rotterdam is the largest port in Europe and water transport is an important part of the Dutch economy. The port of Rotterdam forms a good connection between the sea and the Western-European hinterland; the Boven-Rijn and Waal are important parts of this connection. The Netherlands has by far the largest fleet of the Western-European countries: 10.200 out of a total of 18.400 ships (IVR; 2016). Moreover, approximately 1/3 of the transported tonne-kilometres in Western-Europe goes through the Netherlands.

The large advantage of water transport over other modes of transport is that it is cheap and delays do not occur often. For inland water transport sufficient depth is of utmost importance to prevent

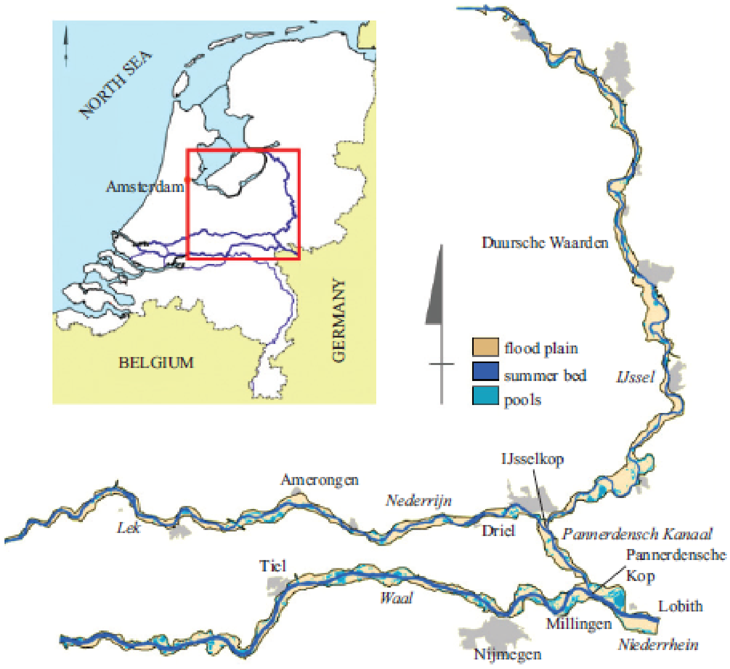

FIGURE 2.1: The Dutch Rhine branches. (Van Vuren; 2005)

delays. In 2003, a dry year, the economic loss due to insufficient depths is estimated to be €91 million (Jonkeren et al.; 2007). Therefore the economy benefits from well maintained fairways.

The Rhine countries have agreed on certain requirement for depth and width during low water for different river sections. The Boven-Rijn and Waal have a depth requirement of 2.8m and a width requirement of 150m during low flow conditions. The minimum water depth should be available for a discharge at Lobith that is exceeded 95% of the time: the Agreed Low Discharge (ALD, in Dutch: Overeengekomen Lage Rivierafvoer (OLA)). At the moment this discharge is 1020 m^3/s at Lobith. The discharge can be translated to a water levels: Agreed Low Water (ALW, in Dutch: Overeengekomen Lage Rivierwaterstand (OLR)). If the depth measured from this reference plane does not meet the requirements, the government is obligated to dredge. The government subcontracts this using performance contracts, where the subcontractor must proof every two weeks that they meet the requirement.

2.2 Morphology of the study area

The study area consists of four sections of the Dutch Rhine: the Boven-Rijn, Upper Waal, Middle Waal and Lower Waal (Figure 2.3 & Table 2.1). The first section: the Boven-Rijn, lies between Lobith, where the Rhine flows into the Netherlands and Pannerdensch Kop, where it splits in the Waal and Pannerdensch Channel. Due to bend sorting in the bend just before the bifurcation the side of the Pannerdensch Canal consist of courser sediment, preventing it from silting up. Sediment segregation

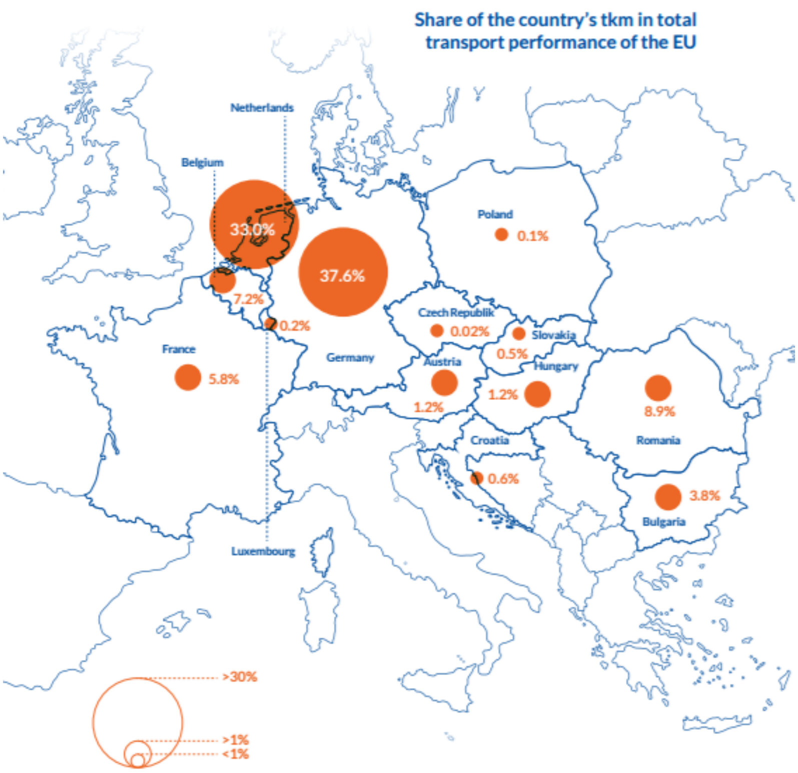

FIGURE 2.2: Transport performance defined as amount of tkm given in percentage per country. (*Market insight inland navigation in Europe: fall 2016.*; 2016)

can occur in three dimensions, e.g. downstream fining, bend effects and bed armouring, which all occur in the Rhine (Van Vuren; 2005). Due to human intervention the Rhine has become a coarser river partly due to normalisation. The coarser sediment-inflow from Germany causes the lower Rhine to erode. The reason for this is the coarser sediment being less mobile and therefore the transport capacity downstream is larger than upstream.

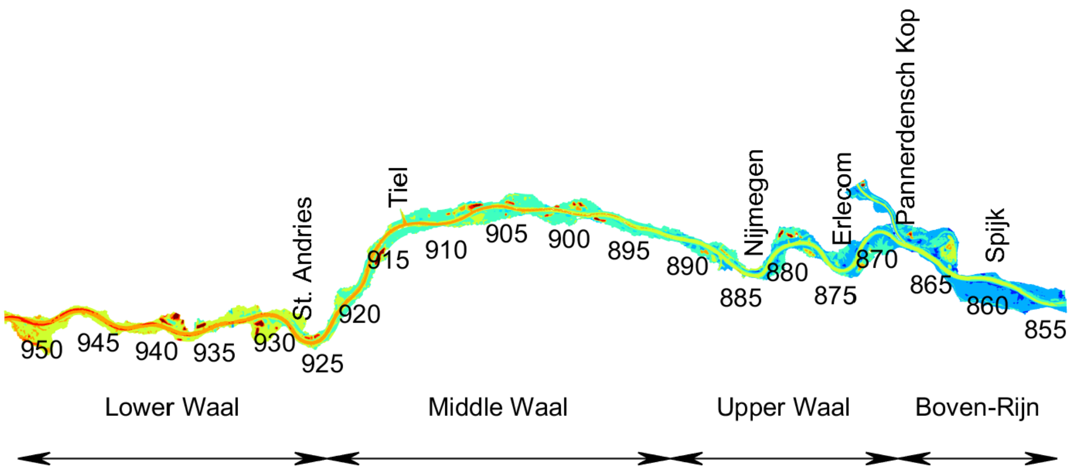

FIGURE 2.3: Impression of the initial bed level for the study area. In black the Rhine kilometers (rkm).

After Pannerdensch Kop the river continues as the Waal. On the Waal many large stretches of groynes are applied, however, recently many have been lowered. While groynes were designed in order to keep the main channel sufficiently deep and wide, they can also cause problems during low flows. Due to vortex shedding from the groyne heads erosion holes develop, which move into the main channel and downstream form these scour holes sedimentation occurs.

Table 2.1: Locations of important river sections of the study area (left). Locations of the four river stretches and Pannerdersch Kop (right).

Location	**Begin**	**End**	
Spijk	858	861,5	rkm
Rijnwaarden	862,5	873,6	rkm
Millingerwaard	867,6	873	rkm
Erlecom	873	876	rkm
Lent / Nijmegen	881,5	886,6	rkm
Druntense waard	906	913	rkm
Tiel	913,5	915	rkm
St. Andries	925	929	rkm

Section	**Begin**	**End**	
Boven-Rijn	854	867,6	rkm
Pannerdensch Kop	867,6		rkm
Upper Waal	868	891	rkm
Middle Waal	892	924	rkm
Lower Waal	925	952	rkm

The upper Waal is the first section which reaches to just downstream of the bend near Nijmegen. This section is dominated by bend effects such as point bars, pools and crossings. Due to curvature induced secondary flow in bends transverse slopes and point bars and pools can develop in inner and other bend resp.. Due to circular flow near the river bed a current towards the inner bend develops and sediment is transported towards the inner bend. To find the transverse slope and equilibrium between gravitational force down-slope and the up-slope drag force is formulated. At the crossings between opposite bends a shoal can be formed in the middle of the river: a risk for navigation. In geometrically complex reaches shoals and deep parts can be formed due to the 3D flow patterns.

The middle Waal is a quite straight reach until Tiel, where there is a bend. In the straight reach non-stationary effects such as dunes and other bedforms become apparent. Ripples are small scale bedforms that are strongly related to the sediment grain size and are little influenced by flow conditions. Contradictory to ripples, dunes are largely influenced by flow conditions such as water depth and velocity, while much less affected by grain size. Their scale is also large: instead of order centimetres and decimetres, they can become a few metres high. In the Rhine different bedform scales are superimposed.

The lower Waal has a sharp bend near st. Andries and some milder bends, therefore both the bend effects and the non-stationary bedforms can be noticed. The effects described are mirco and meso scale (Table 2.4), however, there are also some macro scale processes, such as the geometry of the bifurcation which was mentioned earlier.

scale levels	morphological phenomena
micro	- bedforms, such as ripples and dunes - vertical segregation of sediment fractions
meso	cross-sectional profile evolution: - transverse bed slope and pointbar/pool formation in bends - crossings between opposite bends - formation of shallow and deep parts in geometrically complex reaches - bank erosion - overbank sand deposition - local scour in groyne fields and formation of so-called groyne flames - local scour e.g. around bridge piers
macro	- longitudinal profile evolution - evolution of geometry at river bifurcations

FIGURE 2.4: Morphodynamic processes in the Rhine for different scale levels. (Van Vuren; 2005)

It is expected that erosion occurs in the Boven-Rijn and Upper Waal due to the tilting of the bed as explained in chapter 1. The tilting point lies near Tiel where the erosion changes to accretion of the bed. Near Spijk, Erlecom, Nijmegen and St. Andries measures have been taken to prevent bodem erosion in the bends (Table 2.1). These locations won't erode with the rest of the river and because the water level does decrease will form bottlenecks for navigation. St. Andries and Nijmegen are locations were aften the least available depth occurs (LAD, Dutch: minst gepeilde diepte).

Chapter 3

Risk method

In this chapter the risk assessment and dealing with risk approach is explained in general terms. A more precise account of the methods is given in the first section of every subsequent chapter. The method is divided in three steps: stochastic modelling of river morphodynamics, consequence modelling for inland water transport and risk evaluation and management. The first section of the current chapter explains the general risk assessment method that is applied. After which a small introduction is given for each of the steps. The last part of the research: the broader applicability of the method, is only discussed in its chapter.

3.1 Risk assessment

As explained in the first chapter, the aim of this study is to develop a risk assessment method for morphodynamic response. Or in other words one would like to know the probability of a certain morphodynamic response linked to its consequences in order to support decision making. Jonkman (2015) described a general method for risk assessment (Figure 3.1). In chapter 2 the river system is described in general terms, i.e. the morphological processes are briefly discussed and the importance of inland water transport on the Rhine in the Netherlands is touched upon. By making a qualitative analyses the importance of different processes is found and what method is best for quantification of the risk is determined. When the risk is assessed the following question is whether the risk is acceptable and to what extend risk reduction should be applied.

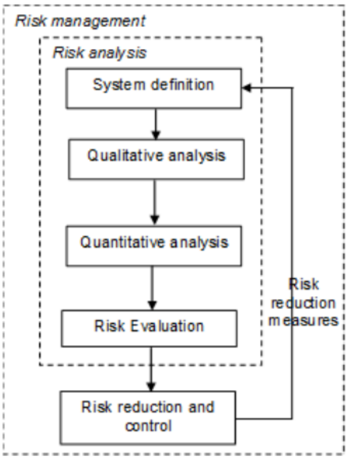

FIGURE 3.1: Flowchart risk management and risk analyses (Jonkman; 2015)

Following this scheme for morphological risk results in the risk assessment approach as given in Figure 3.2. In the first arrow the morphological processes are described and possible methods predicting morphodynamics are discussed and compared. Also the uncertainty in predicted morphological behaviour plays an important role in the probability of occurrence of different morphological states. The next step is the modelling of consequences belonging the morphological states, resulting in a risk estimate. The final part of the risk approach is the evaluation of the risk in order to support decision making.

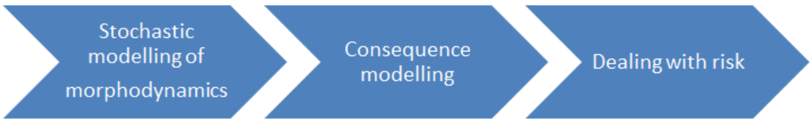

FIGURE 3.2: The general risk assessment approach to developed in this study.

3.2 Stochastic modelling of river morphodynamics

The complex interaction between flow, sediments and topography results in morphodynamic behaviour. As seen in the previous chapter, there are many different processes involved in morphological behaviour and the temporal and spacial scales widely differ. To model these processes, for example, rules of thumb, numerical and analytical models can be applied. Most often the numerical models are used since morphodynamics shows strong non-linear behaviour and the principle of superposition of modes of behaviour cannot be applied.

A defining characteristic of morphological models is the number of dimensions in which the equations are solved. 1D models are the most widely used, since they are simple and can give a first estimate of the wanted parameter without too much effort. However, for morphological research 1D models often do not suffice, because morphological behaviour has components in all three dimensions. Van Vuren (2005) investigated the difference between the riverbed statistics of a 1D and a quasi-3D model of the

Waal. She found that while the 1D would suffice for large scale (order of entire river basins) initial investigations, the 2D model would be better suited for more advanced studies, looking at smaller, local scales. The 1D model showed significantly less statistical variation and underestimates the uncertainty range.

The current study aims to locate morphological issues and determine consequences, which involves local, small scale morphological response, therefore a 1D model is deemed inappropriate. The main disadvantage of using fully 3 dimensional models is the computational time, therefore for this research applies a 2D (depth averaged) model, included parametrised 3D effects. In this study the DVR schematisation of the Rhine branches, incorporating Delft3D, is used (Vuren et al.; 2006). The DVR schematisation is model of the Dutch Rhine branches including current measures.

Predicting morphological behaviour is associated with many uncertainties. The main uncertainties related to the model can be categorised in model parameters, model input and model concept uncertainties. The types of uncertainties and an estimate of their importance from literature is given and the chosen uncertainty is discussed further. The quantification of the effect of uncertainties on output is done with uncertainty propagation. Methods such as sensitivity analysis, uncertainty analysis and using the correlation between uncertainties are generally applied. For stochastic modelling two methods are often applied: the First Order Reliability Method (FORM), Monte Carlo Simulations (MCS). The latter is not applied since the morphodynamic behaviour in time is an important aspect of the risk assessment. The two other methods are compared and one is chosen to be used on the DVR Rhine model.

3.3 Consequence modelling of morphodynamic behaviour

To determine the risk, the morphodynamic behaviour is translated to consequences. In engineering practice quantification of consequences in monetary values is often done, since it can be easily compared with costs of risk reduction measures. Unfortunately, not all consequences can be quantified. For example loss of life in flood risk assessments or indirect consequences for nature. In this study the focus lies on the monetary consequences and the non-monetary and indirect consequences are only discussed briefly.

The consequences depend on the river function that is looked at. To make a full risk assessment all consequences for all functions should be taken into account. First the parameters resulting from morphodynamic modelling and their effect on the river functions are analysed. Flood safety has as main parameter water level, while cables and pipelines has local bed level and inland water transport has water depth. The next step is finding the consequences for when the values of these parameters are not as wanted. In some cases a failure tree is used to couple different parameters and aspects of the problem to determine a probability of failure. Per failure mode or scenario the costs resulting from it are determined and linked to their probability to determine the risk. The expected costs over the entire time period can be used for comparison of the measure and reference case later on.

3.4 Dealing with risk

The last steps in risk management are risk evaluation, reduction and control (Figure 3.1). The evaluation of the risk is different per type of risk. Generally either a threshold is set or an economic optimum approach is used. The first can be used for non-quantifiable costs or hazards, therefore in flood risk a threshold of maximum probability is set depending on the consequences for the area. In the shipping industry a similar rule exist, namely: a minimum depth of 2.8m in the reference situation (for ALW) is guaranteed by the government. Even though it is possible to dredge when this level is exceeded it might be more feasible to look at more structural solutions. When different solutions are proposed an optimum based method can be applied. This paragraph discusses how optimum based decision making can be incorporated after the currently developed risk assessment method. A case study is used to apply the risk management method and analyse the results.

In the chapter 6 the chosen case study: river nourishments, is elaborated on. The nourishments are applied for mitigation of the bed degradation on the Dutch Rhine. While bed degradation might seem favourable for flood safety due to lower water levels, it forms a risk to stability of structures. Bed degradation might also cause local reductions in navigable depth due to fixed layers that do not lower with other parts of the river, while the water level does. River nourishments were chosen since the cost can be calculated relatively easily, they can change in time and they are relatively easy to implement well in the Delft3D model. As is often done in decision making, multiple alternatives are compared with a reference situation and each other; in this study only one alternative is analysed and compared.

A possible technique for decision making is simply comparing the expected costs of the reference situation and the measure case. However, it doesn't take into account the spread in risk. How to handle the spread in risk depends on the stakeholders and the consequences of the case. Moreover, these types of methods for river morphodynamics often only take into account one river function. To include these aspects would require another separate study to be done. Therefore, a simple comparative risk assessment is done between the two cases. The cases are compared based on the final expected costs over 15 years as well as the development of the costs during these years. The costs for dredging and nourishments and the economic loss are also analysed separately to take into account that the risk bearing party is not the same as the investing party.

Chapter 4

Stochastic modelling of river morphodynamics

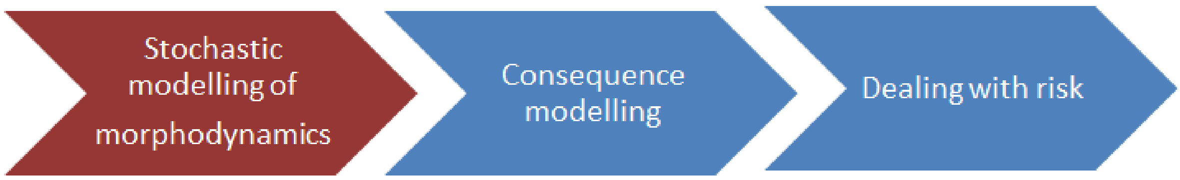

FIGURE 4.1: The general risk assessment approach to be developed in this study.

This chapter discusses the morphodynamic processes and models proceeded by the methods for stochastic modelling and the important uncertainty sources. Also the model and uncertainty source examined in the following parts of the study are discussed in more detail. A general account of the morphological behaviour in the simulations is given and the water depth and dredging amounts and their probabilities are discussed. The last part of this chapters is a discussion of the model and its shortcomings and sensitivities.

4.1 Morphodynamic modelling

Morphodynamic models should incorporate the interaction between water motion, sediment transport and bed topography well. Figure 2.4 gives an impression of the morphodynamic processes in the Rhine and their scales levels. In this research the Delft3D model as developed for Sustainable Fairway Rhinedelta is used (Vuren et al.; 2006). As explained in the previous chapter it was selected because it is a quasi-3D model and modelling the important morphodynamic process requires 3 dimensions

4.1.1 Delft3D DVR model

Delft3D Rhine schematization

The schematisation includes Room for the River (RftR), Water Framework Directive (WFD) and NURG (Nadere Uitwerking Rivieren Gebied) measures, Nourishments at Millingen aan de Rijn and dredge and dump capabilities. When the requirement of 2.8m at ALW is not met dredging occurs up to the required level plus 0.5m. The requirement is tested against a reference plane which simulates ALW when the discharge at the upstream boundary is between 1200 and 2250 m^3/s. The dredge and dump module is further discussed in Appendix A. Also a list of measures can be found in this appendix.

The computational domain of the model is given in Figure 4.2. The baseline schematization is based on the knowledge in 2012, including measures that where planned to be constructed, actualisation up to 2011 and the RftR measures as described in the RftR pakkettoets P2012-I. The WFD measures consist of side channels and removal of bank protection at various locations. The reader is referred to Sustainable Fairway Rhinedelta II (Ottevanger and Giri; 2015) for more information on the locations and implementation of these measures (see also Appendix A). The nourishments at Millingen aan de Rijn (rkm 862 to rkm 864.5) in 2016 and 2019 have a target bed level of ALW -4.0 m and the layer thickness is 0.30 m. For the nourishment 50%-50% granite and fine gravel is placed between the tips of the groynes. For the last part of the risk assessment: risk management, extra nourishments are added to the reference case. The implementation of the nourishments is described in chapter 6.

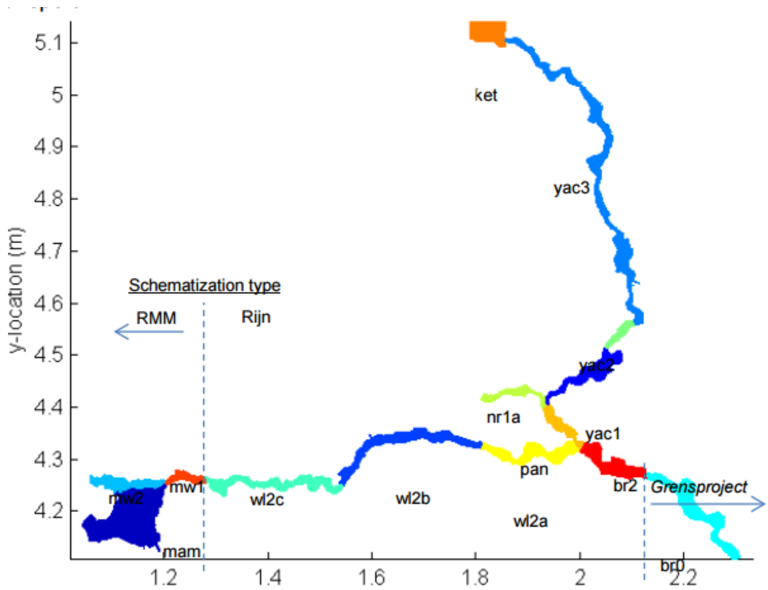

FIGURE 4.2: Computational domain of the DVR Delft3D model. (Ottevanger, 2015)

Basic equations

Delft3D solves the (depth averaged) 2D shallow water equations consisting of the continuity and the equations of motion. The model is sometimes called quasi-3D, since it includes parametrisations for curvature-induced secondary flow. The flow conditions are used to determine bed morphology through sediment transport formulations. A morphological factor is applied to account for the different time scales of hydrodynamics and morphology.

Transport formulae

Sediment transport is related grain size, flow velocity and hydraulic roughness. For the Boven-Rijn and the upper Waal (and Pannerdensch Canal) a definition using graded sediments is applied, while for the finer lower and middle Waal the modified van Rijn (1984) formula is used (see Appendix A). The choice for this sediment transport formula was justified by (Yossef et al.; 2008); they also calibrated the transport formula. The formulation includes bed load as well as suspended transport and has the advantage that they can be calibrated with separate parameters.

Dune, bedform and roughness predictors

Hydraulic roughness and dune height play an important role in the prediction of hydrodynamics as well as morphology. In Delft3D a choice can be made between the following dune height predictors: Van Rijn (1984c), Fredsoe (1982) based on Meyer-Peter-Muller (MPM) or Engelund-Hansen (EH) and a power relation. These formulas give relations for dune height in equilibrium conditions, therefore, a advection relaxation equation is added to account for the adjustment time. In the current Delft3D schematisation Fredsoe (1982) based on MPM is used for dune height, van Rijn (1984) for dune length and also van Rijn (1984) for roughness prediction. See Appendix A for the formulae.

Boundary and initial conditions

The upstream boundary condition at the start of the Boven-Rijn is the discharge in time defined per grid cell. The downstream boundary conditions at the end of each river branch are water levels. Except for part of the Neder-Rijn there a discharge is applied to create the effect of the weirs in the Neder-Rijn and Lek. The water levels are defined per time and originate from the baseline schematisation in which bed level measurements are included. For closed boundaries or land-water-boundaries the velocity normal to the boundary is zero.

Due to the discharge condition in the Neder-Rijn an instability at this boundary was found. Water levels and flow velocities show unexpected behaviour (Appendix G). During this is study it was not possible to find a solution for this problem. A possible explanation for it is the morphology and flow at the bifurcations influence the distribution as well as the boundary condition and no stable equilibrium between the two exists.

The morphological boundary condition is a bed level decrease at the Nieder-Rhein of 1.5 cm/year, based on historic data. For the Neder-Rijn bed level decline is assumed to be zero.

The initial conditions consist of initial topography including structures, flood plains etc., but also roughness and dune heights. Initial water level is created by an initial period of flow based on the boundary conditions. The two upstream domains in the study area (br2 and wl2a, until rkm 892) have graded sediment based on measurements and the other two domains apply uniform sediment which does becomes finer downstream.

Delft3D calculation process

The Delft3D DVR schematisation uses a quasi-stationary approach to decrease computation time. This approach uses discrete discharge levels and the flow regimes of the discharge-levels are saved to a database to be reused. To be able to apply this the discharge-series are discretised to steps of $500m^3/s$ with a minimum level of $750m^3/s$ and a maximum of $9750m^3/s$ (see Figure 4.3).

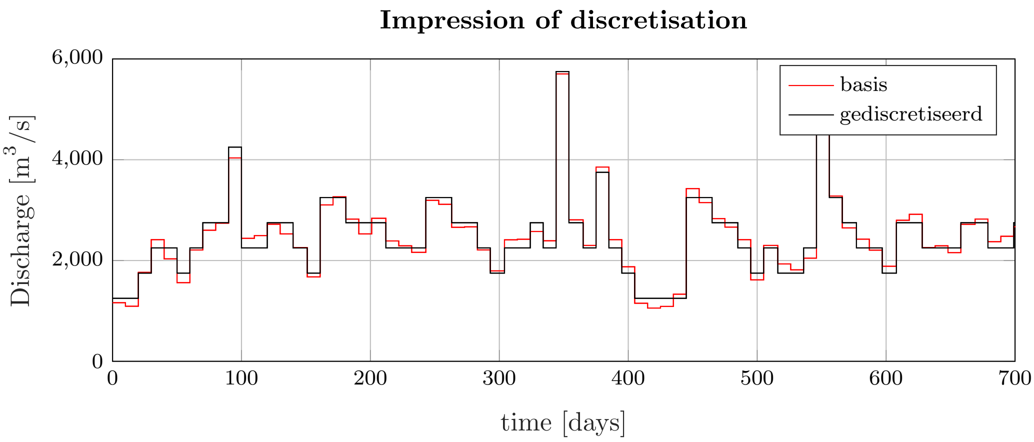

FIGURE 4.3: Example of discharge-series discretisation.

4.2 Stochastic modelling

Since modelling morphodynamic behaviour is uncertain in this section methods to include this uncertainty are discussed and also the main uncertainty sources are briefly touched upon.

4.2.1 Uncertainty propagation

The to be used method depends on the type of equations that are used to describe the problem. In morphological response modelling the system consist of multiple non-linear equations. While analytical uncertainty propagation is sometimes used it's generally too time consuming to be used for complex models. Therefore, only two methods that are reasonably applicable for morphological models are discussed here.

Method of moments

The most common form of the method of moments is the first-order second-moment uncertainty analysis (in hydraulic engineering sometimes referred to as First Order Reliability Method (FORM)). This method only involves the first two moments: the mean and the variance. The third moment is skewness, while there are methods to involve this they are quite complex and therefore generally other methods are preferred. Furthermore, this method requires that the model has a linear or close to linear relation between in- and output, the uncertainty in the input is small and the mean and standard deviation of the probability distributions exist. Unfortunately these requirements are often not fulfilled in river morphology modelling. However, it can be used for the (simpler) equilibrium state estimations (van der Klis; 2003).

Monte Carlo Simulations

The Monte Carlo method is a method where the model doesn't need to be simplified or linearised. The base of the method is creating different realizations of the model output by getting realizations from every uncertain variable using their probabilistic distribution functions and using them as input for the model. This is repeated until there are enough model outputs to analyse the uncertainty of the outputs. The variables are chosen in such a way that each simulation has equal probability. While this method keeps the model intact and uses the underlying probability distributions, the computing time increases, since instead of one time (deterministic approach) the model should be run a large number of times. This might cause a problem since models for river morphology are already quite complex and time consuming to run.

Crude sampling

Often when applying MCS crude or simple sampling is used, where realizations of the variables are generated from a prescribed probability distribution. A number of random values between 0 and 1 are generated and the inverse cumulative distribution function is used to find the corresponding values of the variable. This method is often used because it's easy to implement, standard statistical techniques can be applied to the output and the input variable can be any magnitude or have any variance as long as they can be described statistically.

Latin Hybercube Sampling

To reduce computation time different sampling methods have been made, one of these is Latin Hypercube Sampling (LHS) (McKay; 1997). For all variables the cumulative probability distribution will be divided in a number (n) of bins (intervals) of equal probability. For example the first 20% (0-20%) is one bin, then the next 20% (20-40%) is another bin etc. (see Figure 4.4). In each bin one realization is chosen at random. If multiple variables are involved, for each variable a bin is chosen and the variables are combined to create a set of model inputs. Every bin is only used once, resulting in n (the number of bins) outputs. A method was designed by Iman and Helton (1988) to include correlations in the process.

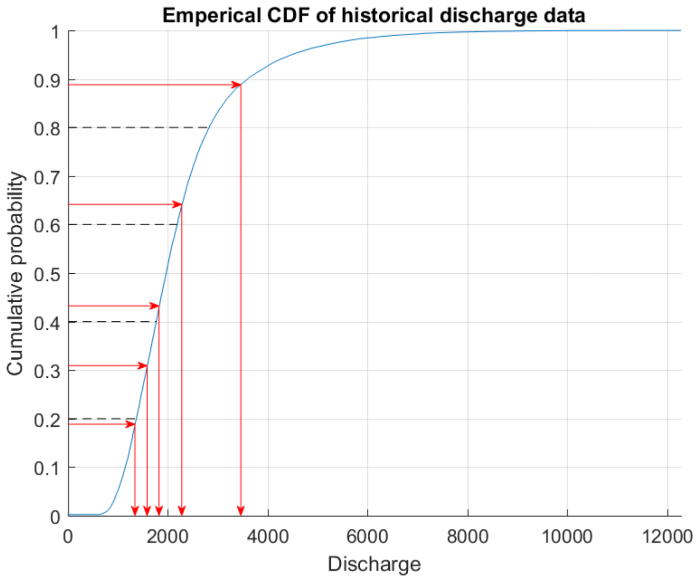

FIGURE 4.4: Illustration of Latin Hybercube Sampling. Per interval of probability 0.2 (black dashed lines) one value is randomly chosen to represent that interval (red arrows).

For monotonic problems the accuracy of the LHS method is better for the same number of runs as the crude sampling method (van der Klis; 2003), however, it is not known whether this holds for non-monotonic problems such as river morphology models. For crude sampling a method exist to estimate beforehand how many realizations are needed for a certain accuracy, unfortunately, such a method does not exist for LHS. Also since the outputs of LHS are not completely independent the method for estimating accuracy is not reliable (Morgan and Henrion; 1990).

4.2.2 Uncertainties

In this subsection the uncertainties that influence morphological behaviour are discussed. A distinction is made between aleatoric and epistemic uncertainties, where the latter is due to a lack of knowledge and thus the amount of uncertainty can be decreased by doing research. Aleatory uncertainty is said to be the natural variability of the system. Both types of uncertainties are found in morphological development modelling and therefore the uncertainty sources should be analysed carefully. The uncertainty sources are subdivided in the categories:

- Model uncertainties
- Model parameter uncertainties
- Model input uncertainties

The first category gives an general overview of the methods of analysis model uncertainties. In this thesis these types of uncertainties are not further elaborated on, however, it's recommended that this is done in later studies. The other two types depend largely on the chosen model, and are therefore only analysed for the Delft3D DVR schematisation.

4.2.3 Model uncertainties

A model is a simplification of the real world and it often has many limitations and assumptions in the model structure, because schematisation is difficult and often not enough knowledge about the processes involved exits. These uncertainties in the model are generally not quantified and the model parameters are only calibrated to fit known data. According to Refsgaard et al. (2006) the availability of data is the first question in the classification of assessing conceptual model uncertainties. In the case were data does exist part of it is used to calibrate the parameters and a different data set is used to validate the model.

To account for the uncertainties in the model concept there are two methods: increasing parameter uncertainty to account for structural uncertainty or estimation of the structural uncertainty term (Refsgaard et al.; 2006). A method that is for instance used in climate change calculations is using multiple conceptual models, and the difference in outcomes gives a measure of uncertainty of the model structure. Another option when there is not enough data available is asking expert opinions, this was done for example by Warmink et al. (2011) for the uncertainties in a hydrodynamic river model.

4.2.4 Model parameter uncertainties

Transport formula parameters

The most used formulas in the Netherlands are Meyer-Peter Muller (MPM), Van Rijn and Engelund Hansen (EH). Van Vuren (2005) showed that the difference between MPM and EH is negligible in Delft3D. In a later study by Yossef et al. (2008) different formulations were examined and a modified van Rijn (1984) formula was proposed, calibrated and validated.

In her global sensitivity analysis Van Vuren (2005) found that the critical shields parameter and power of the bed shear stress were the most important parameters (applied in the MPM formula) (Figure 4.5).

Hydraulic roughness and bedform predictors

Hydraulic roughness is used to model momentum and energy dissipation and depends on the bed material and the flow conditions. An empirical formula is used for calculating the hydraulic roughness and it's a large source of uncertainties, since it's a simplification of multiple processes that dissipate flow energy (Warmink et al.; 2013). One of the factors for the hydraulic roughness calculation is the material or vegetation, generally aerial photographs are used for determining the type of vegetation in flood plains.

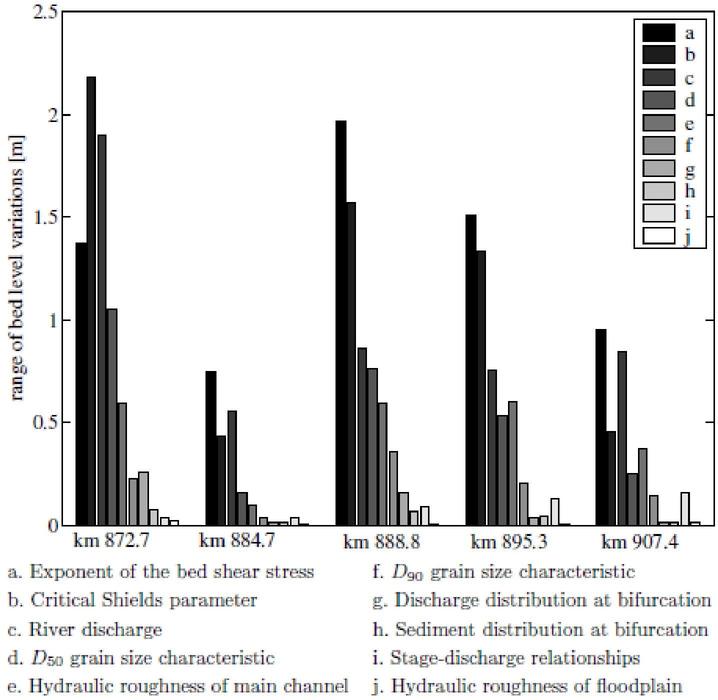

FIGURE 4.5: Results of the global sensitivity analysis using a 1D Rhine model. (Van Vuren; 2005)

In Delft3D the predictor for river dunes and bedforms is used for bed roughness. Haitel et al. (2004) investigated the influence of climate change on bedforms in the Rhine and the consequences for navigation. They concluded that the influence of bedforms is much smaller than the discharge effected for climate change. For future low waters (2100) the dune height would be only 1% (0,05m) of the water depth, for high waters this can be more than 10%, but in that case the water depths are also much larger (order of 10-15m). However, it should be kept in mind that there is a time lag between discharge and dune height, possibly resulting in larger dune height during low flows. There is a strong relation between previous and current discharges, therefore it also takes time for a discharge to decrease and sudden drops are less likely. Van der Mark et al. (2008) found quite different results for one measurement set at Lobith the dune height was between 0,25 and 1,3m for 8m water depth. The standard deviation of the dune height is 0,47 times the mean for these values. The latter investigation agrees more with the results of Delft3D DVR simulations.

4.2.5 Model input uncertainties

While Van Vuren (2005) found, using global sensitivity and uncertainty analysis, that the most significant uncertainties lie in transport formula parameters; uncertainties in discharge, grain size and hydraulic roughness could not be neglected either. However, the grain size was found to have no significant effects on the morphological response variation.

Bed materials

Bed materials vary in space and time in the river due to natural variability as well as sorting processes. The special variations can be caused by bedforms, local armouring and bend effects. Especially if the problem requires a high resolution morphological results than the bed materials are important. Van Vuren (2005) found that the effect of uncertainty in bed material on morphological response was small compared to contribution of uncertainties in discharge. In more recent versions of the Delft3D Rhine model the grain size can vary in space, which most likely decreases uncertainty effects significantly.

Initial conditions

The initial bed topography has significant influence on morphological effect studies. The bed topography is generally found by bathymetry soundings of the river. In the current model the flood plains have fixed bed and are schematised separately form the main channel.

Boundary conditions

van der Klis (2003) concluded that the influence of the boundary and initial conditions depends on the distance between the boundary conditions and the area under investigation and the time period. Since for the current study a large initial period is used, it is expected that initial and boundary conditions do not have large effects on the morphological response.

Discharge

The importance of discharge variation on morphology is generally understood, since flow velocity is a large factor in sediment transport. Van Vuren (2005) showed that it is one of the three most important variables. In comparison to many other uncertainty sources many measurements have been done and therefore a large historical data set is available, which makes obtaining an accurate probability distribution or resampling records more reliable.

Choice of uncertainty

Unfortunately due to time restrictions and complexity of the model it is not possible to take into account all uncertainties, therefore, one has to be chosen. Van Vuren (2005) showed that the morphological response is sensitive to transport parameters and river discharge variations. Previous bedform investigations for the Rhine showed a large difference in bedform height and therefore there is a significant amount of uncertainty involved in the determination of bedform height.

The discharge is chosen as main uncertainty variable for this thesis, since no knowledge exists on uncertainties in transport parameters for the current model and recently a some improvements have been made (Yossef et al.; 2008). Discharge is one of the main factors for issues navigation and therefore not including it in this study would underestimate the cost significantly. While it is clear from previous investigation that the uncertainties in bedform formulations are significant and they have a large influence on navigational bottlenecks, it would require an entirely separate study to investigate bedforms, because too much is still unknown.

Other variables such as hydraulic roughness, bathymetry (bedforms) and sediment transport formulations should not be forgotten and it is recommended that these uncertainties are assessed in follow-up research.

4.2.6 Selected uncertainty source: discharge

Discharge hydrograph synthesis

One source was chosen to be investigated in this study, namely: discharge. The discharge time series used in the model needs to be created, since having a non-constant discharge has a significant effects on morphology. It's important to have a hydrograph as realistic as possible. In Van Vuren (2005) five methods are described for synthesis of discharge hydrographs. These methods are multivariate log-normal distribution, bootstrap resampling, nearest-neighbour resampling, a statistical description using four parameters and Bootstrap sampling combined with a flood event predictor. Van Vuren (2005) found that the multivariate log-normal distribution doesn't fit the tail of the data well and overestimates the possible high discharges, even simulating discharges that are not physically possible for the Waal. The statistical description using four parameters did not perform well either, most likely due to its strong simplification of the discharge hydrographs. Therefore, only nearest-neighbour and bootstrap resampling are elaborated on.

Bootstrap resampling

Approximately hundred years of discharge data is available for Lobith (the location where the Rhine enters the Netherlands). Bootstrap resampling resamples from this data to create a new discharge series, but instead of random sampling 10 day discharges one year samples are chosen to keep the seasonal variation and neighbour correlation intact.

Nearest-Neighbour resampling

The nearest-neighbour resampling technique searches for the next interval by finding intervals similar to the current and previous one and randomly selecting their neighbour. To prevent the loss of the seasonal cycle only 7 intervals in the same period of the year can be chosen from, but since this is done for every year it still leaves 700 options. Within these options a period of two discharge is sought that is similar to the current discharge and the one before that.

Van Vuren (2005) also investigated how well the different discharge hydrographs preform in a hydrological model. She found that the difference between Bootstrapping without flood predictor and nearest-neighbour resampling is small. Bootstrapping with flood event predictor, has a little more variation in morphological response than the original bootstrapping method and the overall effects are small. in this study the Nearest Neighbour resampling technique is used, after which the results are discretized to values of 500 m^3/s.

Discharge hydrographs

In the DVR model a schematisation discharge series of historical data is used. The discharges are discretized to certain values which have important statistical information. The peak is often near March, which is the case in reality as well (Figure 4.6, bottom). Figure 4.6 also gives an impression of the statistical properties of the 100 discharge hydrographs that are used in the MCS. While the peak discharge is often in the first half of the year, it is far from always the case. However, in reality the timing of the peak can also vary.

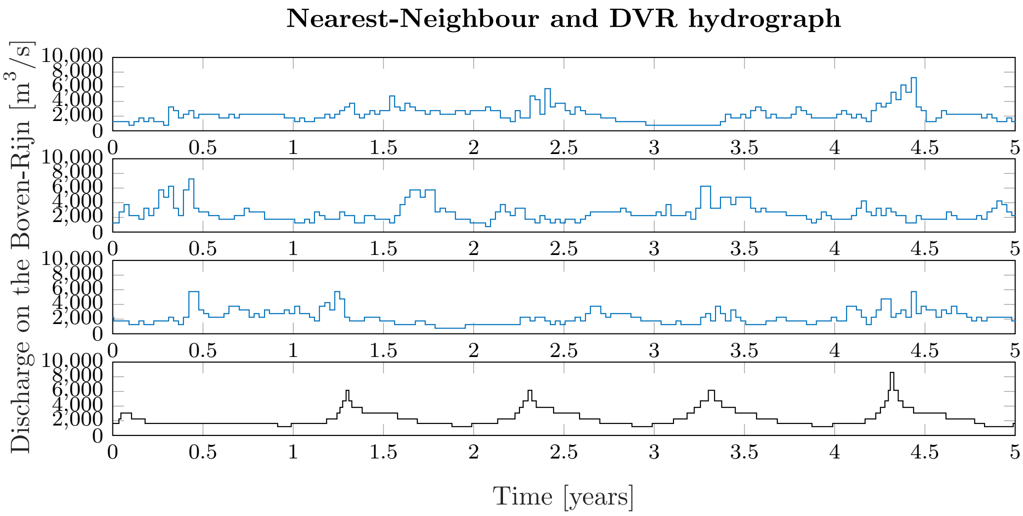

FIGURE 4.6: Hydrograph 1, 2 and 3 of the 100 created using Nearest-Neighbour sampling (blue) and the DVR hydrograph (black) for the first 5 years.

As explained in the previous subsection the discharge hydrograph for MCS have been created by Nearest Neighbour resampling, a method based on Bootstrap resampling of historical discharge data. The method of resampling doesn't guarantee the same ALD will occur, in this case the probability that the discharge in less than 5% of the time is lower than 1020 m3/s is 0.68. ALD occurs therefore less than expected, which might results in a slight underestimation of the final costs. Figure 4.7 shows the probability density functions of discharge occurrence for the input discharge and historical data.The input data has more occurrences in the middle-high range, while the original data has a higher probability of having large or small discharges.

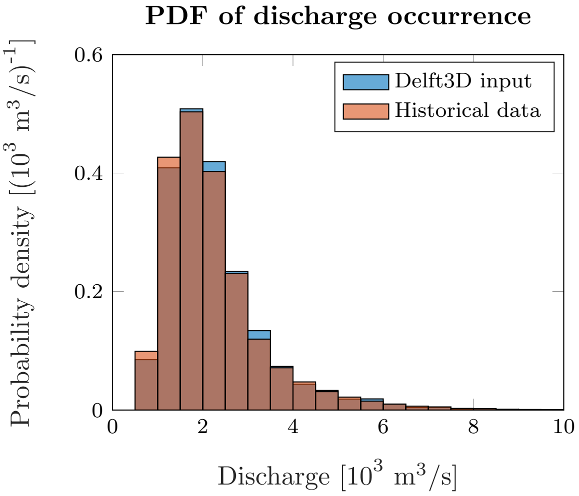

FIGURE 4.7: Probability density functions of discharge occurrence for the data used in Delft3D and the historical data.

4.3 Morphodynamic behaviour

For navigation two morphological results play an important role: width and depth. The width is given as fairway width and the depth depends on the following aspects of the morphological computation:

- Bed level (-)
- Dunes and Bedforms (-)
- Dredging (+)
- Water level (+)

These factors are discussed on the following paragraphs, after which the depth is analysed as well. The water level mainly depends on the discharge, the roughness and the river morphology, because all these subjects are discussed separately, no section is dedicated to this subject. Finally also the depth and width are tested against the requirements given by the Dutch government.

4.3.1 Initial period

Most models have an initial period due to the conditions in the model not being exactly equal to the ones in reality. Due to these discrepancies, the model needs to find a new equilibrium bed level that corresponds to the applied input variables and parameters. For example the simulated sediment transport and the real transport might not match. This also holds for the DVR model and adding to this is the fact that the initial bed level is from 2012 and the measures for 2015 (as planned in 2012). Moreover, the model was never validated properly and even if it had been the discrepancies would still exists in smaller amounts.

The first years of the simulations indeed show different behaviour from the other years. Figures 4.8 and 4.9 show the behaviour in the first 5 years and the 5 thereafter. The first 5 years are quite unstable and show much variation within a small distance. The second five years look more stable, but have a larger 90% confidence interval. This is most likely due to each simulation having a different initial bed level and in time the number of possible states increases.

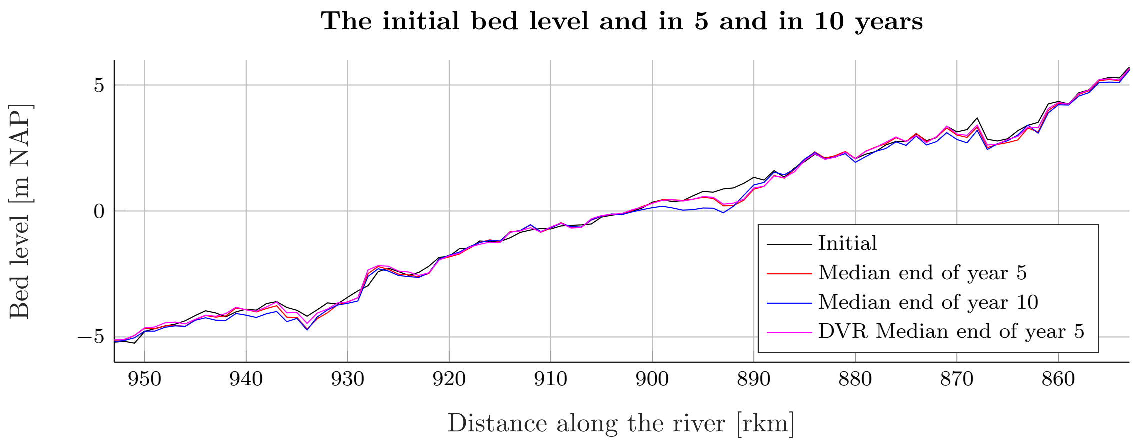

FIGURE 4.8: The bed level initially and after 5 years (left) and between year 5 and 10 (right). Bed level averaged over width between groynes ('normal width') and per rkm.

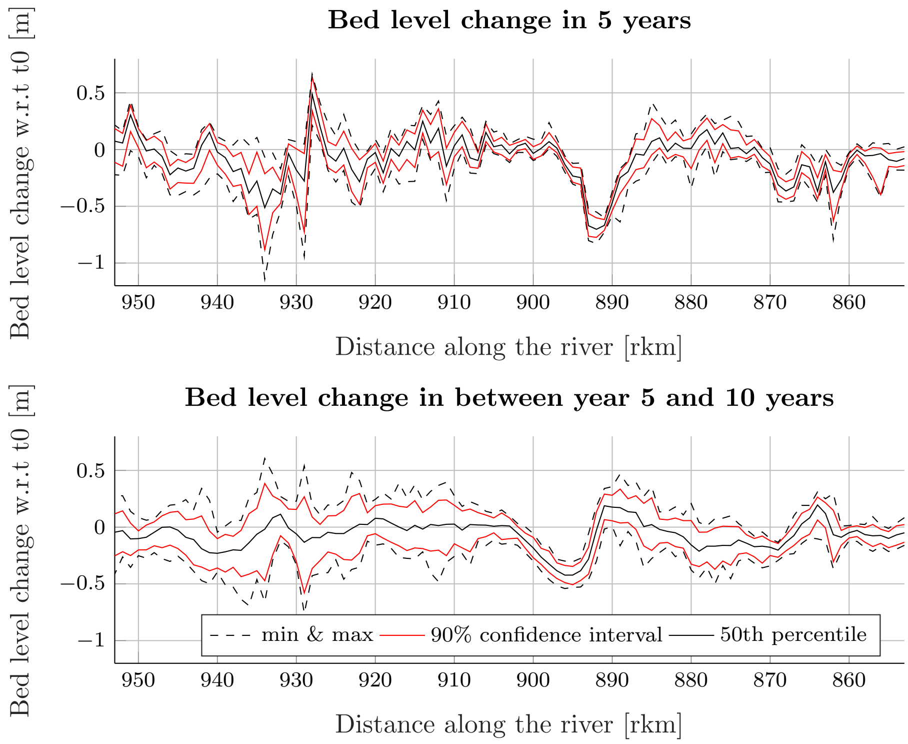

FIGURE 4.9: Bed level variation w.r.t. to t0 in the first 5 years (upper) and year 5 after 10 years (lower) averaged per rkm. Bed level averaged over width between groynes ('normal width') and per rkm.

Initially three locations show large decline: approx. rkm 867-872 (After Pannerdensch Kop), rkm 887-897 (after Nijmegen) and rkm 930-940 (Zaltbommel). Over time the initial erosion spreads downstream with approximately 1 km/year which is expected according to RIZA (2005). Table 4.1 shows the bed level decrease in for the three locations in the first 5 years and 5 thereafter compared with measurements from Blom (2016). The second 5 years are significantly closer to the measurements.

TABLE 4.1: Median initial bed level decrease for 3 locations averaged over 5 years. The measurements (1999-2010) are read from Figure 1 of Blom (2016)

	rkm 870	rkm 890	rkm 935	
Deterministic	4.4	12	6	cm/year
MCS 5 years	6	14	10	cm/year
MCS 5-10 years	3	4	4	cm/year
Measurements	2	3	4	cm/year

An explanation for the erosion at rkm 892 is that a boundary between two domains of the model is situated there. Two changes occur at this domain boundary: the transport formula and sediment size goes from graded to uniform. The bed shear stress does not change significantly near this location, but the sediment diameter and bed load transport does. However, the change already starts before the domain boundary, where also a smaller grain diameter is found.

After approximately 5 years the behaviour changes, most likely indicating the end of the initial period (Figure 4.10). However, the deterministic simulation has a significantly longer period of different behaviour. An explanation for this is that the first 6 years have low discharges. Also due to the lack of variation the initial period possibly takes longer. In the last 7 years the deterministic simulation starts to behave similar to the MCS, indicating most likely the end of the initial period.

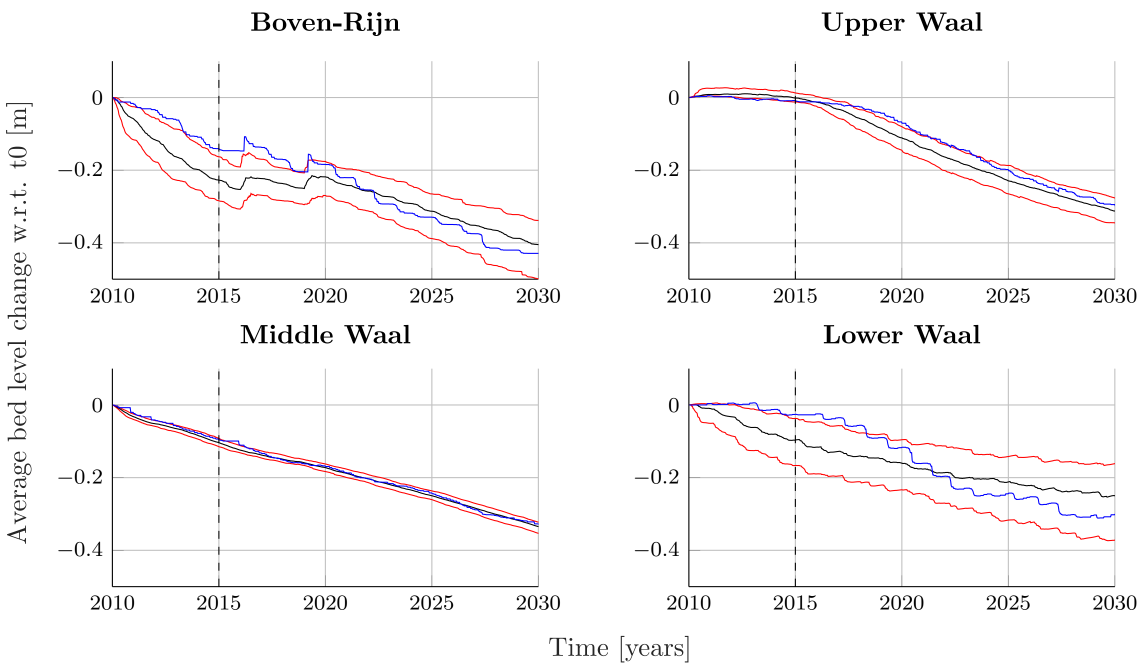

FIGURE 4.10: The bed level change averaged over the length river domain and the fairway width. Black: 50th percentile, red: 90th percentile, blue: deterministic simulation.

The dredging amounts in the first 5 years and especially the first year are significantly larger and are decreasing in time. For the above named reasons the first 5 years are taken out of the following analysis, as well in this chapter as all following. Therefore the sixth year of the simulation is named 2015 and is from now on the first year. It should be kept in mind that the deterministic simulations the initial period is not yet over.

4.3.2 Bed level

The first part in calculating the depth is the bed level. As expected the bed level decreases in time (Figure 4.11). In the Boven-Rijn the nourishments in 2016 and 2019 are clearly visible. Measure data shows that in recent years the Boven-Rijn has stopped decreasing (Blom; 2016). An explanation for the Boven-Rijn not stabilising in the simulations is the upstream boundary condition of 1.5cm bed degradation per year. The speed at which the bed level decreases in time differs per domain; it stays relatively constant for the Boven-Rijn, upper and middle Waal, while it starts to decrease for the lower Waal. For the lower Waal this is expected due to the tilting point near Tiel (rkm 915). The bed level should be increasing but doesn't, most likely due to the sand mining from rkm 925 onward.

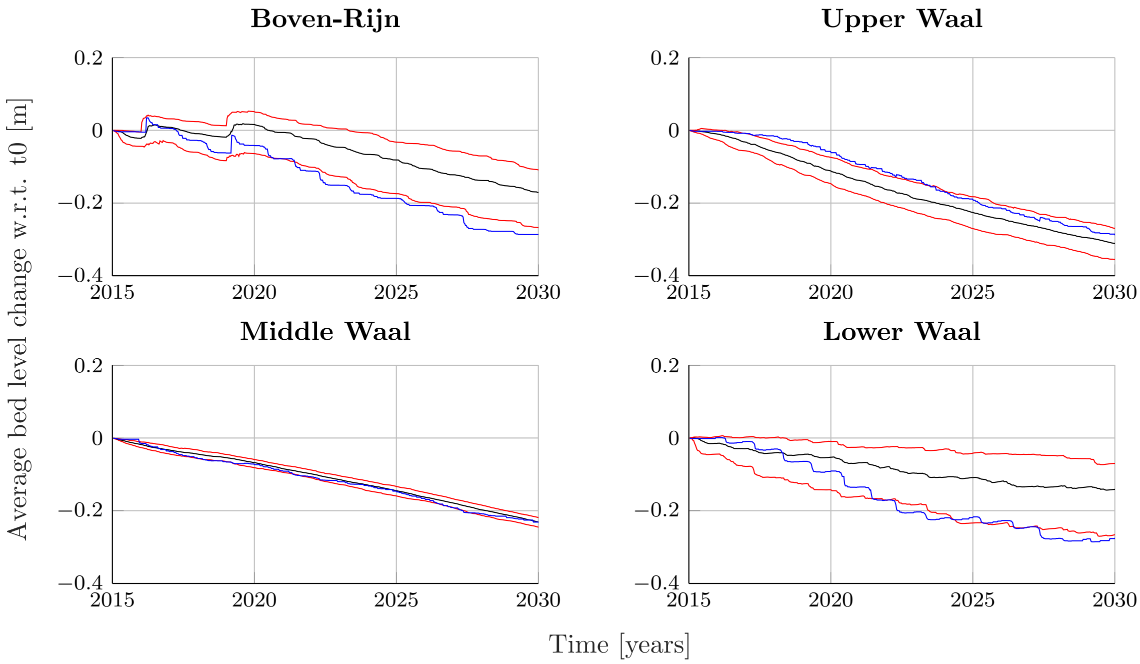

FIGURE 4.11: The bed level change averaged over the length river domain and the fairway width. Black: 50th percentile, red: 90th percentile, blue: deterministic simulation.

Measurement data from Sieben (2008) shows a decrease in the erosion in more recent years (1999-2006, Table 4.2). However, it should be noted that this period is to short to draw solid conclusions. The middle Waal decreases much more than to be expected, most likely due to the large amount of erosion near rkm 890.

Table 4.2: The averaged bed level change over river length and fairway width per year. First three columns form measurements as given by Sieben (2008) and fourth column the averaged over the periods. The last column shows the averaged simulated values over 15 years.

Section	**rkm**	**1950-1973**	**1970-1999**	**1999-2006**	**Avg.**	**Simulations**	
Boven-Rijn	859-867	-3.0	-3.0	-0.1	-2.7	-1.2	cm/year
Upper Waal	868-886	-1.0	-3.0	-1.7	-2.1	-2.1	cm/year
Middle Waal	887-915	-1.0	-1.0	-0.5	-0.9	-1.6	cm/year
Lower Waal	916-951	-1.0	-2.0	0.4	-1.3	-1.1	cm/year

Figures 4.13 and 4.12 show the bed level change in space. Again the erosion around rkm 890 is clearly visible, it seems to be a dominant phenomenon since the 90% confidence interval is small. Another reason for the small confidence interval is that the reach is relatively straight. The bends near rkm 879 and rkm 862 show large amounts of erosion, however, it is quite local. Around the bends near Spijk, Erlecom, Nijmegen and St. Andries the 90% confidence interval is also larger, with a maximum of more than 1m.

In the schematisation 90.000 m^3/yr is allowed to be mined between rkm 925 and 953; this could explain the decline of the end of the Lower Waal. Due to the implementation of the mining in the model, in some years the amount that is mined is larger than the allowed volume (see Appendix A.6). The averaged mined amount over all MCS is 85,900 $m^3/year$ and in 4.4% of the 1500 simulated years the amount was higher than the allowed volume. For the DVR hydrograph the average is 91.400 $m^3/year$ and in 40% of the years the allowed volume is exceeded. This might explain the significantly larger bed level decrease on the lower Waal in the deterministic simulation. The initial period of the deterministic simulation is possibly not yet over, causing more erosion.

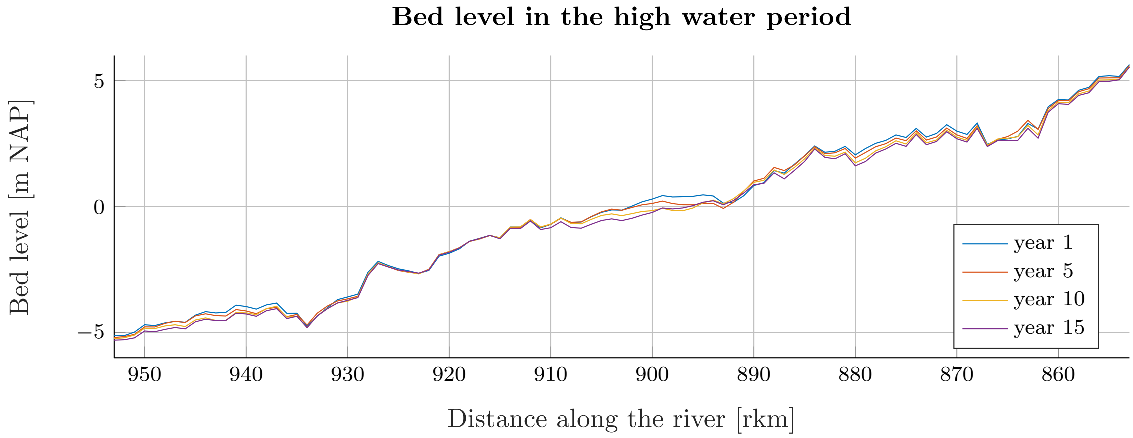

Figure 4.12: Median bed level change in time during the high water period. Approximate time of high water discharge: March.

4.3.3 Dunes and bedforms

Dunes can play an important role in the minimum depth, because dunes show a late response to flow conditions, therefore the maximum dune height occurs after a discharge wave when the water level

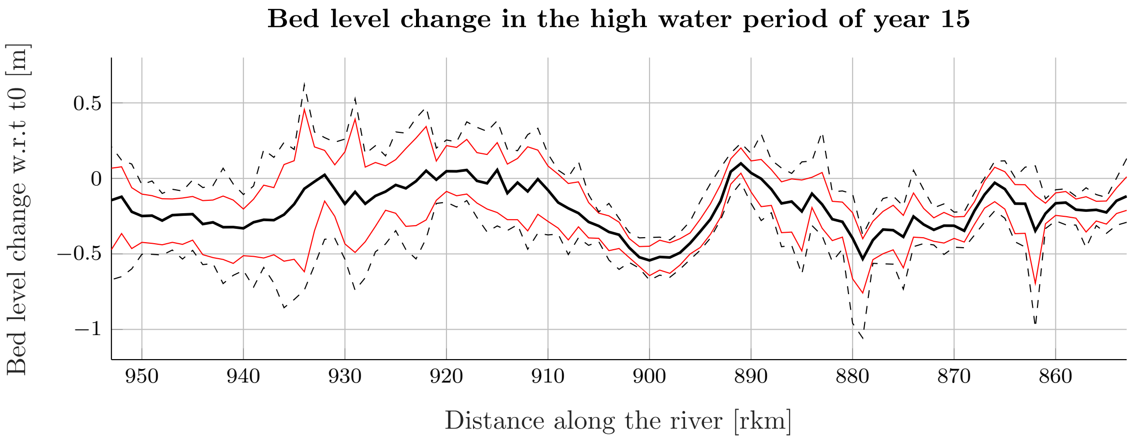

FIGURE 4.13: Bed level variation w.r.t. t0 in the high water period of year 15. Approximate time of high water discharge: March. In black the median value, red: 90% confidence interval and black dotted the minimum and maximum.

is already decreasing. This is indeed the case since the correlation between the dune height and the discharge on time step earlier is larger than for the same time step. In the low water period the dune height can still be as large as 2.2m (Figure 4.14). The median for the low and high water periods are quite similar, but the high water period shows an approx. 30% larger 90% confidence interval. The confidence interval increases with distance going downstream.

The dune height shows the expected variation in space (Chapter 2): larger dune heights in the more straight middle Waal. The lower dune heights on the Boven-Rijn and upper Waal are most likely a result of larger sediments: less mobility (Sieben; 2008). The bends near Erlecom (rkm 875) and Nijmegen (rkm 885) have higher dune heights than surrounding areas, possibly due to the higher water level in the outer bends.

When comparing the simulation results with the average of 7 years (1999-2006) of measurement data it is found that the overall dune height is lower for the simulations, except for rkm 930 and further downstream (Sieben; 2008) (Figure 3.3). The decrease in dune height after rkm 930 is most likely due to the larger normal width and thus smaller shear stress according to Sieben (2008). The difference between the measurements and simulations is in the order of 1 or 2 decimetres, again with the exception of the downstream end. However, it is difficult to draw solid conclusions since it is not known when the measurements were done and therefore the discharge is unknown, which is an important factor in dune height.

4.3.4 Water level

Agreed Low Water level

The agreed low water level is used as a reference plane for the dredging activities. It is based on a discharge of 1020 m^3/s at Lobith, which is exceeded approx. 95% of the time. In the model the reference plane or ALW is updated every five years. The first reference plane is the actual plane known

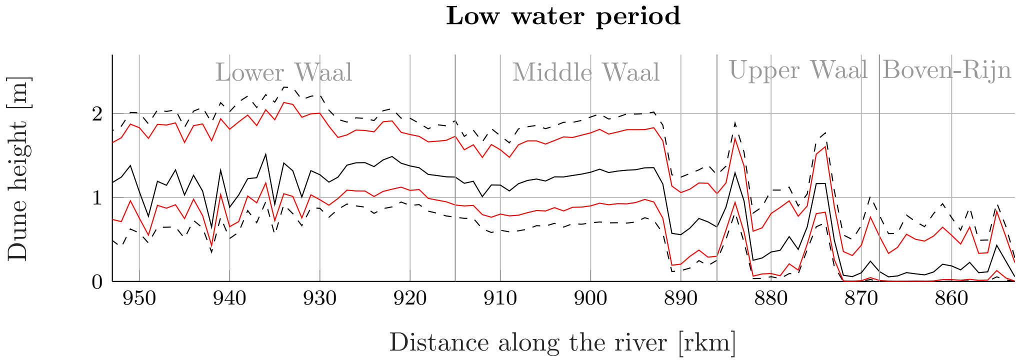

FIGURE 4.14: Dune height variation in the low water period of year 10 averaged per rkm and over the fairway width. Approximate time of smallest discharge: the beginning of August.

in 2012. At the first time step the water level for Q = 1020 m^3/s is calculated and the difference between this plane and the plane at the start of year 6 is added to the original reference plane to create the new one. Since the official reference plane is lower than the calculated one (Figure 4.15, left), this is a conservative approach. The difference can be explained by a number of things: model errors, measurement errors, assumptions made in determination of official reference plane etc.. As to be expected the change in reference plane shows a clear relation with the bed level change (Figure 4.15, right). For example the erosion near rkm 890 is also shown in a decrease in reference plane and moves downstream as well.

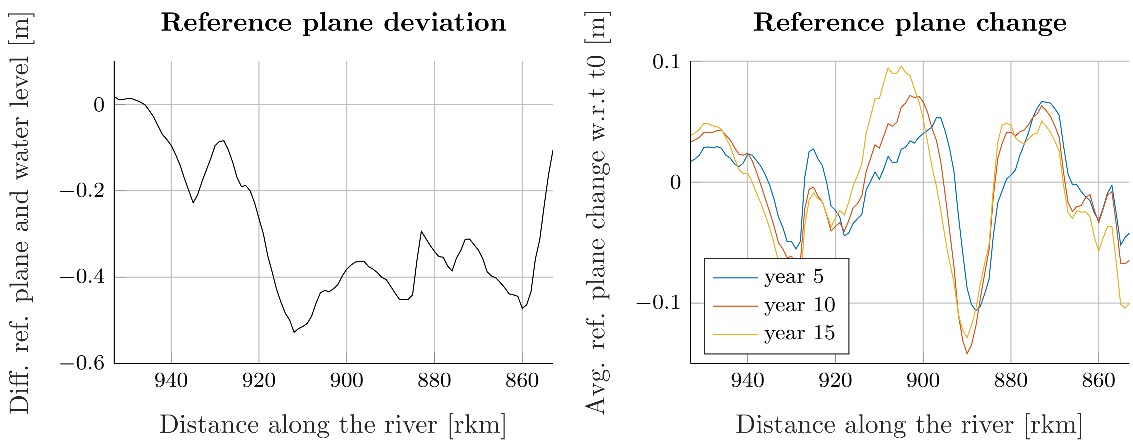

FIGURE 4.15: Left: official reference plane minus calculated water level for $1020 m^3/s$. Right: averaged change in reference plane in space for the three updates. Averaged per rkm.

Discharge distribution

During a conference in 1771 it was decided that the water distribution of the Rhine in the Netherlands should be 2/3 to the Waal and 1/3 to Pannerdensch Channel (Rijkswaterstaat; n.d.). Later (1989) Rijkswaterstaat was founded to uphold this rule, which now only holds for extreme discharges. At IJssel Kop 1/9 goes to the Neder-Rijn and 2/9 to the IJssel. For low water discharges at the Boven-Rijn less water flows to the Neder-Rijn, but a minimum of 25 m^3/s is maintained. For low discharges more water is distributed to the Waal, which is expected due to the policy. For discharges at the Boven-Rijn of above approx. 3700 m^3/s the control structures no longer affect the distribution (Rijkswaterstaat; 2011).

For intermediate discharge values at the Boven-Rijn in the simulations a fraction of approximately 0.7 flows to the Waal (Figure 4.16). It should be noted that an error in discharge at the Neder-Rijn of up to 60 m^3/s occurs in the model due to the discharge boundary (see Appendix G). This error is mainly found between discharges of 2250 and 3750 m^3/s on the Boven-Rijn. If the entire error is directed to the Waal instead of the Neder-Rijn then the error in this discharge range would amount to 0.9% to 1.4%.

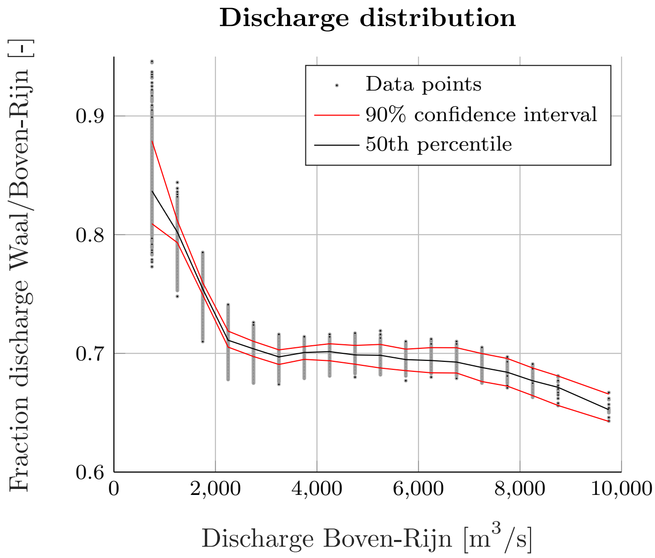

FIGURE 4.16: Discharge distribution (Waal/Boven-Rijn) for the discharge levels at the Boven-Rijn.

4.3.5 Dredging

The Dutch government is required to dredge the depth to 2.8m from a reference plane. The area's were the largest amount of dredging occurs are after the fixed layers at Erlecom (rkm 876), Nijmegen (rkm 887), and St. Andries (rkm 925-929). Also a problem area is around the Druntensche Waarden (rkm 906-913) and Tiel (rkm 913,5-915) were multiple river management measures are taken.

Bardoel (2010) shows using measurements that the needed yearly dredging amount to keep the Waal navigable (requirement ALW -2.8m) is 400,000 to 450,000 $m^3/year$. This amount is 'hopper' volume (beun in Dutch) not in situ volume therefore it should be dived by approx. 1.4 (Mosselman et al.; 2007). The in situ volume is than about 300,000 $m^3/year$. While this is the same order of magnitude as the simulated volumes it is still at least a factor three larger. Adding the mined 90.000 $m^3/year$ at the downstream end of the lower Waal decreases the difference. Moreover, the dredging volumes also decline in time (Figure 4.18).

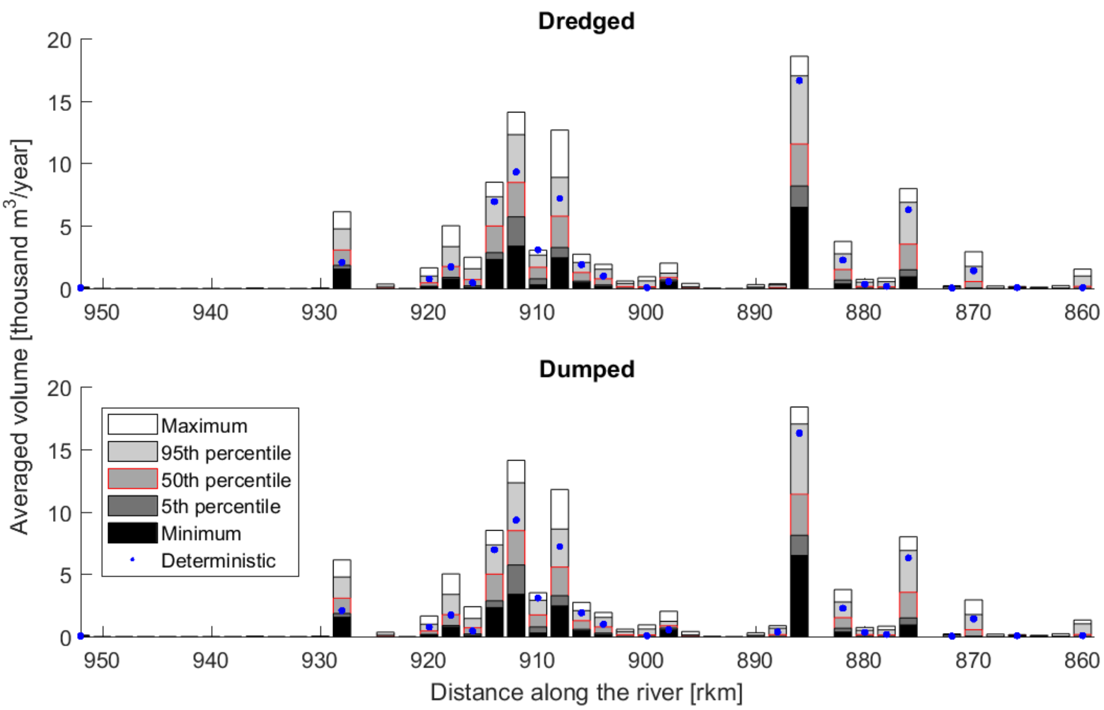

FIGURE 4.17: Dredging and dumping volumes in 15 years (in situ).

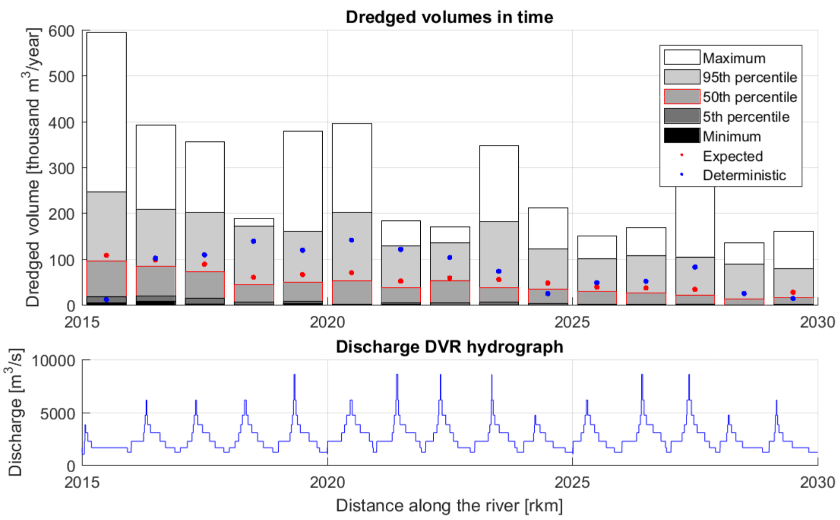

FIGURE 4.18: Dredging volumes per year (in situ). Statistics from MCS and the deterministic values using the DVR hydrograph. Bottom plot: DVR hydrograph.

The deterministic calculations using the DVR hydrograph show a little different behaviour, but almost always within the 90% confidence interval of the MCS. The behaviour of the deterministic simulation is related to the magnitude of the peak in that year (Figure 4.18). The dredging amounts found using the DVR hydrograph are generally high compared with the median values of the MCS. An explanation for this is the longer initial period, but also the lack of variation in discharge can have influence. The deterministic hydrograph has one peak per year and only decreasing discharge thereafter, though the MCS hydrographs show more variation, possibly eroding some of the sedimentation away. The last 7 years of the simulation the deterministic volumes are closer to the expected values of the MCS.

4.3.6 Navigational requirements

The navigational requirements for the Boven-Rijn and Waal are a minimum depth for a discharge of 1020 m^3/s at Lobith of 2.8m and a minimum width of 150m. This minimum width is not further assessed in this research, however in Appendix A a section is dedicated to the width of the fairway and how the depth is calculated using this width.

It is expected that due to the ongoing erosion and thus water level decrease, the fixed layers will become more significant problem areas in the future. Even though indeed the water level dropped near for example Nijmegen (Figure 4.15), no significant trend of LAD in time was found.

LAD most often occurs at the fixed layers near Nijmegen (rkm 884) and St. Andries (rkm 925) (Figure 4.19). They are bottlenecks for similar amounts of the time. For high discharges (which occur less frequent) the location of LAD often lies near the downstream end of the lower Waal (rkm 953). The location of LAD moves downstream: Nijmegen for 500-1100 m^3/s, St. Andries for 1100-1800 m^3/s and rkm 953 for higher discharges (Figure 4.20 right). This is explained by an increase in discharge having less effect on the water level downstream than upstream due to the influence of the sea downstream.

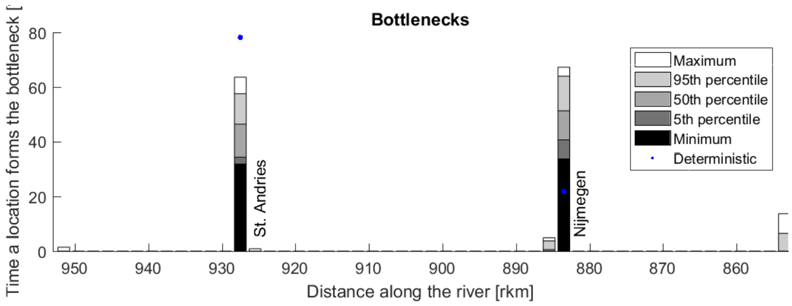

FIGURE 4.19: The locations of LAD and the percentage of time they occur.

Finally a cumulative probability function is made of all LADs that have occurred in the data set of 100 x 15 years (Figure 4.20). Three steep slopes in the CDF can be distinguished, the corresponding depths have a large probability of occurrence The deviation between the first two depths at approx. 2.2m and 3.2m is most likely caused by the discretisation of the discharge levels to steps of 500 m^3/s, as can be seen in the scatter plot on the right: the first and second grouping do not overlap w.r.t. LAD. The same goes for the second and third discharge grouping, however, they overlap a little. Moreover, the spread in minimum depth per discharge (90% confidence interval) is on average 0.4m. For lower discharges the value is higher and for higher discharges smaller, as can be seen in 4.20.

The deterministic results for dredging volumes were high, but w.r.t. LAD the opposite is the case: shallow depths occur less frequent. The deterministic simulation almost never reaches below the required

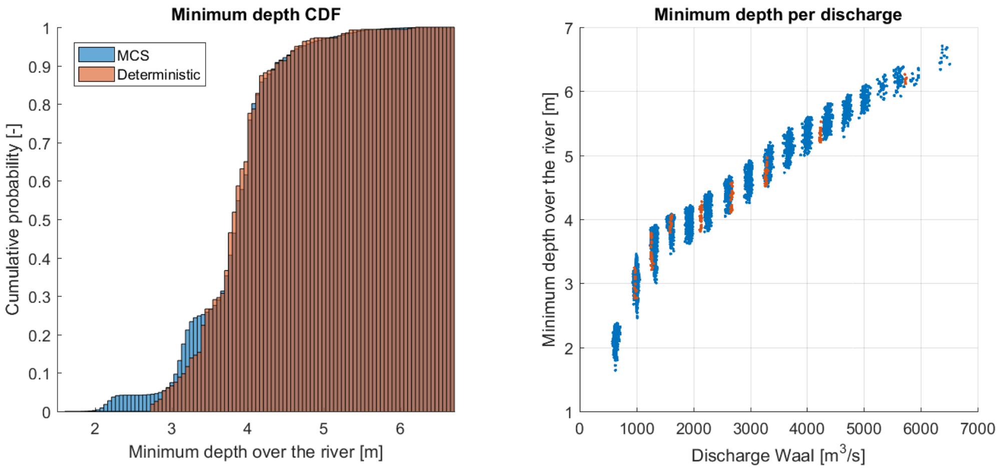

FIGURE 4.20: Left: Cumulative distribution function of the minimum depth over the river length and fairway width. Right: Minimum depth over the river length and fairway width plotted against the discharge on the Waal.

2.8m, because the smallest discharge is 1203 m^3/s. To be able to use the DVR hydrograph for drought related studies a more full rang of discharges must be used, including low discharges. Moreover, for every low depth dredging occurs in the deterministic simulations, while for the lowest discharge in the MCS (750 m^3/s) dredging cannot be done. This further increases the difference between MCS and deterministic.

4.4 Discussion

The Delft3D model preformed reasonably well for the larger morphological processes when compared to measurement data of Sieben (2008). However, still some problems were encountered with the model.

First of all, it has never been validated properly, even though different aspects were compared with measurement data. For example it was found that the calculated reference plane would differ up to 0.5m from the official reference plane. Moreover, the model has an initial period were it needs to adjust the bed level to a mismatched sediment transport.

Another issue with the Delft3D DVR model is the instability of the boundary condition at the Neder-Rijn. The error in discharge caused by this instability had a maximum value of 60 m^3/s, but was still increasing in time. It seems that most of this error is taken on by the Waal, but it is difficult to say so conclusively.

The deterministic simulation underestimated the expected minimum depths, partly due to the hydrograph not including discharge values below 1203 m^3/s and it not containing much variation: one peak per year. The depth determined without morphology (for the initial bed level) shows the effects of the differences in hydrographs between the MCS and deterministic simulation (Figure 4.21). Moreover, the lack of variation in the DVR hydrograph and the low discharges in the first 5 years cause a long initial period.

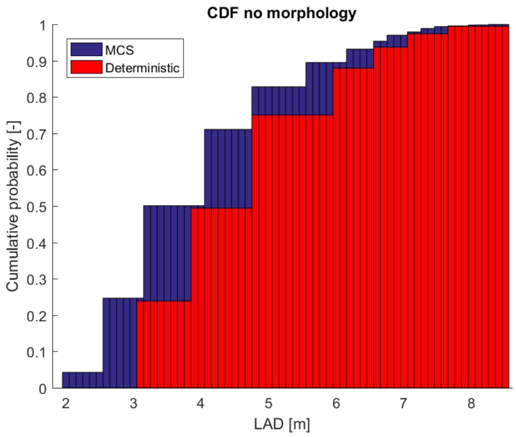

FIGURE 4.21: The CDF's of LAD for the MCS and deterministic simulations without morphology.

Even though there are still some issues concerning the DVR model, it preforms reasonably well for predicting large scale phenomenon. Bed degradation and dredging amounts are generally in the same order of magnitude as measurements, however the dredging volumes differed a factor 2 with measurements. Therefore, the model is less suitable for direct prediction of bed levels and dredging volumes, but can be used for comparison of measure cases. The necessity of uncertainty analysis was proven since large differences between the deterministic and MCS results were found.

Chapter 5

Consequence modelling of morphodynamic behaviour

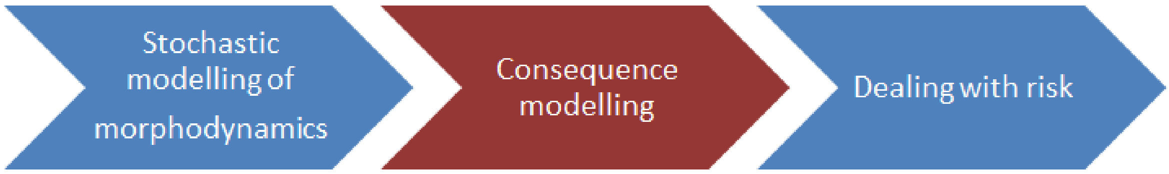

FIGURE 5.1: The general risk assessment approach to developed in this study.

In this chapter the link is made between morphology and economic welfare loss (through water depth). Due to small water depths the amount of load that can be transported will be decreased, which results in one of three things: load is transported with other vessels, the load is transported with an other modality or it is not transported at all. Al these options result in additional costs or losses for the consumer and/or producer. In this case the consumer is the client paying to use inland water transport and the producers are the shipping companies.

The first section of this chapter explains the method of calculating monetary consequences. This is separated in two parts: economic welfare loss and dredging costs. The economic welfare loss is determined based on the concept of change of surplus (their benefit of the transaction). The initial costs and some other variables (transported quantities per ship as a function of time, load factors, etc.)needed for this calculation can be extracted from a database created by BIVAS (a network shipping model that calculates shipping costs) or a simplified method can be applied. The dredging costs are calculated with a simple rule of thumb (Mosselman et al.; 2007).

5.1 Method

5.1.1 Theory of economic surplus and application

Jonkeren et al. (2007) described a method of calculating the price and quantity changes due to a difference in water depth. This method is based on the economic concept of surplus, well known in micro economics. It uses supply and demand functions, which describe the price per unit as a function of transported load (Figure 5.2).

Its assumed that in a fully saturated market the price for which transport is supplied is equal to the production cost. The costs, and thus price, at equilibrium for the reference situation is C_0 (=p_0). The new cost, which is increased due to restrictions in water depth (hereafter named restricted price), is C_1 (=p_1). The demand function (D) gives the maximum amount of product q the consumer wants to transport for a price p (see Figure 5.3 and Figure 5.2). The elasticity of the demand function ϵ, i.e. the slope of the demand line, needs to be estimated.

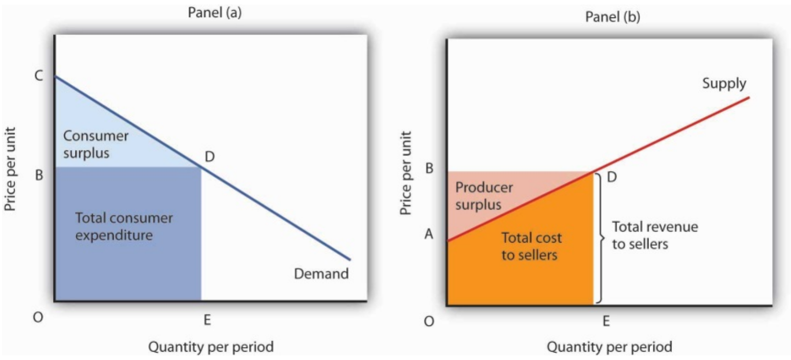

FIGURE 5.2: Consumer and producer surplus. (Rittenberg; 2008)

A measure for the consumer benefits is consumer surplus. The market price or equilibrium price (point D in Figure 5.2) is lower than what a consumer is willing to pay (point C). The difference between what they are willing to pay and did pay for the goods is the consumer surplus (area B-C-D). If the same is done for the producer the producer surplus is found. In this case the producer surplus is zero since the supply-function is constant.

The welfare loss is equal to difference in surplus between the unrestricted and the restricted situation (see Figure 5.3). The welfare loss (WL) is therefore defined as the difference between the unrestricted and restricted consumer surplus: the grey area in Figure 5.3. The equation for this area is:

$$WL = (p_1 - p_0)q_0 - 1/2(p_1 - p_0)(q_0 - q_1) \tag{5.1}$$

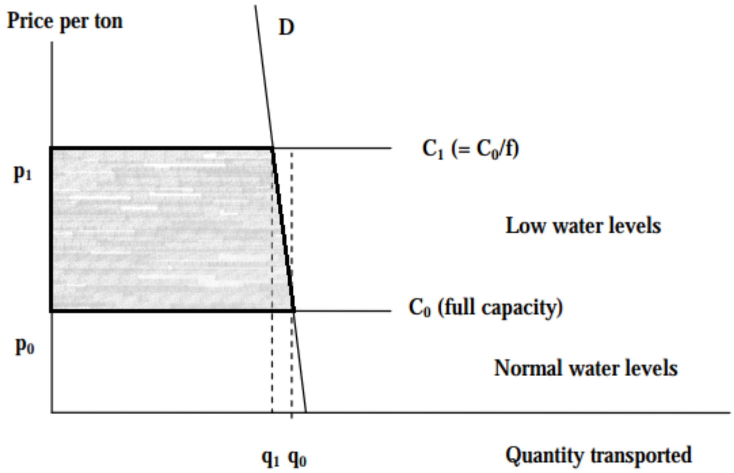

FIGURE 5.3: Change in economic surplus. Adapted from Jonkeren et al. (2008).

Using the elasticity of demand $\epsilon = \frac{\Delta q}{q_0} / \frac{\Delta p}{p_0}$ the following formulation for welfare loss can be found:

$$WL = (p_1 - p_0)q_0(1 + \frac{1/2\epsilon(p_1 - p_0)}{p_0}) \tag{5.2}$$

The supply line is per definition equal to the marginal costs in a situation of perfect competition. The assumption of perfect competition is reasonable because many companies operate the market and it is relatively easy to join the market. It is also assumed that the demand is relatively in-elastic. The assumption of in-elasticity is justified by the fact that water transport is already cheap compared to other modes of transport, therefore a little price increase doesn't have a large effect on the quantity supplied. Secondly consumers using water transport often rather pay more than have their production process delayed, since shipping costs are generally only a small part of the total process (Jonkeren et al.; 2008).

Application

To apply the welfare loss calculation based on economic surplus the following information is needed:

Initial transported load	q_0
Initial price per tonne	p_0
Restricted price	p_1
Elasticity of demand	ϵ

The water depth has a strong time dependent character, but as well does the demands for goods. Therefore, the input variables for the welfare determination are time dependent as well. The restricted price also depends on the type of ship used and the amount of load transported by this ship, thus per ship movement (trip) information about price, load and ship type is needed. In this subsection the possibilities for getting the trip information are examined.

There are different ways to obtain the above named input variables. A simple method can be found in the work of Bosschieter (2005) (hereafter named the simple method). This method uses information on the entire fleet to obtain the initial transported load and uses ship dimensions to find the load factor. The load factor is used to determine the restricted price from the initial one. The initial price is estimated from historic data. Since the application of the welfare loss method needs trip information in time an estimate of how often ships pass the study area is made. Bosschieter (2005) assumed that on average a ship would return every 5 days. Since this estimate is quite rough the fleet is compared with a database of trips and the return period calculated from this. The simple method is explained further in Appendix H and is only used as a comparison for the following advanced method.

A model can be used for determination of the initial and restricted price of transport. Not many shipping models exists, but two were found in literature for the study area: BIVAS (BInnenVaart Analyse System) and TRANS-TOOLS (Krekt et al.; 2011). BIVAS is a network model which determines the best route for every ship in a database given waiting times for structure, fairway intensity, hydraulically circumstances etc.. It also calculates a loadfactor if ships can not sail fully loaded and the routes and costs for the extra ships. TRANS-TOOLS is a transport model that takes into account all modes of transport and also has an economic aspect. Since this investigation focusses solemnly on the economic consequences for the inland water transport sector, and not other modalities, BIVAS is investigated for use in his study. In the box below the model is explained and after the use in this thesis is discussed.

BIVAS

BIVAS (BInnenVaart Analyse Systeem) is an inland shipping network model for the Netherlands. The basis for the model is a journey-database and a network of the Dutch (and some other) fairways. In this database (IVS90) journeys made are recorded by counting stations, such as locks, and saved with variables such as date, vessel class, origin and destination, freight type, loading capacity, and ship dimension. In the current version of BIVAS three base scenarios are included: the years 2011, 2013 and 2014.

The network and journey-database together form the base scenario. It is possible to add economic growth scenarios, fleet changes and water condition seasons. While adding future scenarios might give more realistic outcomes, during this study it will not be used, because it makes it more difficult to compare results over time and trace results back to their cause. Once the scenario is constructed, the journeys are allocated.

For each journey in the database the Dijkstra-algorithm (an algorithm for finding the shortest path) is used to find the optimal route for the vessel based on costs (or travel time depending on the settings). This process is iterative since the costs depend on travel time, which depends on fairway intensity, which depends on the taken route. The transport costs are a summation of cost of sailing and for loading and unloading and fixed costs. The transport cost for sailing are subdivided in fuel usage, repairs and maintenance, labour and material costs. The cost data used is obtained from yearly data from businesses, surveys and other investigations.

If there is no route available for a vessel with a certain load, an analyses is made whether it can sail without load. For the possible routes that follow from this analysis the maximum draught is calculated and the load is adjusted to this draught. If the load factor is adjusted more trips are created to compensate for the loss in transport capacity. These extra trips are allocated again in the same manner as the original journey. However, no extra empty journeys are included for these extra trips. The minimum load factor is set to 30%, with less freight the trip is not feasible.

Two major problems arise when applying BIVAS with MCS:

1. The computational time of BIVAS per year is approx. 1.5h, resulting in extremely large total computational times for the risk assessment (total: 1.5h (comp) x 15 (years) x 100 (MCS) = 2250h ≈94 days)
2. If only input data on a few branches exist, ships take other routes were no data exists, because the have larger water depths. Therefore, either water data about the other branches should be obtained or manually a selection should be made of fairways and trips.

For these reasons it is not feasible to apply BIVAS during this study. The databases of trips that are used by BIVAS have detailed information about the ships which can be used in a simplified method without journey allocation Since the initial databases do not include prices, BIVAS is applied to calculate the costs without depth restriction. This is done for the three databases: 2011, 2013 and 2014. For each day in a year the trips from the year with the highest discharge on that day are put into a new database. This database has therefore the least effects of depth restrictions.

Figure 5.4 shows the method for welfare loss determination using this database step by step. The equations used are briefly discussed below, but assumptions made are elaborated on in the following subsections, where the needed inputs are discussed.

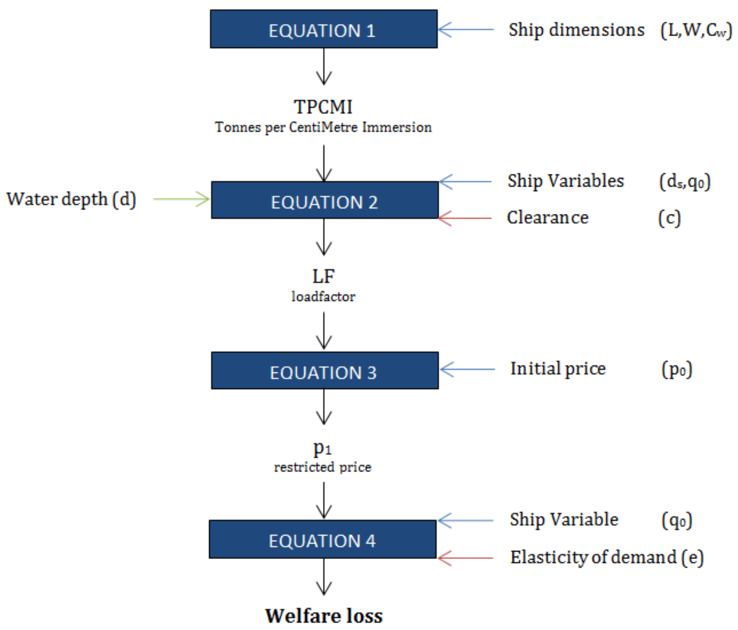

FIGURE 5.4: The welfare loss calculation method as applied in this thesis. The blue arrows show input from the database, the green arrow is input obtained from the Delft3D calculations and the red arrows are assumptions.

EQUATION 1 To obtain the load factor the change of load with a decrease in depth is needed: TPCMI (tonnes per centimetre immersion). Or in other words the number of tonnes that needs to be unloaded per centimetre less depth. The inputs for this equation are the ship dimensions width (W), length (L) and the water plane coefficient (C_w, Dutch: blockcoëfficiënt). The water plane coefficient is the water plane area divided by length and width, for most ships estimated to be 0.9. The equation is based on Archimedes: the 0.01 originates from the fresh water density and a change in units (m to cm).

$$TPCMI = 0.01WLCw \tag{5.3}$$

EQUATION 2 The load factor is the decreased load due to unloading divided by the initial load. This depends on the possible draught of the ship (d_s) for a certain water depth (d). The draught of a ship should be smaller or equal to the water depth (d) plus the clearance (c).

$$f = \begin{cases} 1 & d + c \geq d_s \\ \frac{q_0 + 100TPCMI(d_s - (d+c))}{q_0} & d + c < d_s \end{cases} \tag{5.4}$$

EQUATION 3 The initial price (p_0) divided by the load factor (f) results in the restricted price (p_1).

$$p_1 = \frac{p_0}{f} \tag{5.5}$$

EQUATION 4 Finally the concept of economic surplus is applied to obtain the welfare loss. The formula is repeated here.

$$WL = (p_1 - p_0)q_0(1 + \frac{1/2\epsilon(p_1 - p_0)}{p_0}) \tag{5.6}$$

Since the four steps are applied to each trip individually the welfare loss is also per trip and is later summed over all trips per time period. Below the inputs needed are further discussed.

5.1.2 Input - Initial load

The initial load is obtained form the database created by BIVAS. The BIVAS data shows a clear variation within each week, this is most likely caused by the labour expenses being larger in the weekend and less availability of labour as well. Also a clear yearly trend is seen with less demand of transport in the summer and around the holidays in the winter and January (Figure 5.5). For more information on the used database see Appendix B.

5.1.3 Input - Initial price

As explained above, the initial price is calculated by BIVAS and the resulting three databases are combined. Approximately half of the costs per trip are fixed costs, the other half is variable. The fixed costs always made, even if the ship doesn't sail.

Empty trips

The recorded trips without load can't be taken into account in the current method since it's based on the change in load factor, which is already (nearly) zero for empty trips. However, the costs made for these empty ships is part of the costs of a trip and should be included. Unfortunately it's not possible to link empty trips back to their original trips, thus a different method needs to be applied. In this study the extra costs added to the individual trips (C_{empty}) are calculated as follows:

$$C_{empty} = C_{var}\frac{x_{trip}}{x_{class}} + \frac{C_{fix}}{N_{class}} \tag{5.7}$$

where C_{var} and C_{fix} are the summed variable and fixed costs of empty trips in the considered CEMT class, x_{trip} the distance traveled during the considered trip, x_{class} is the total travelled distance in the considered class and N_{class} the number of trips in the class. In this formula a dependence on distance traveled and CEMT class can be recognized, since these two factors play an important role in the costs of the empty trips.

5.1.4 Input - Restricted price

The final welfare loss determination consists of four steps, as shown in Figure 5.4. The restricted price is calculated using the load factor. When the price increases due to the ships not being loaded to full capacity due to lower water depths, the costs increase with a factor $^1/_f$, where f the load factor. This is exact if a $^1/_f$ is an integer, however if it is not then the whole constant costs and $^1/_f$ times the variable costs is the correct value for the 'restricted' costs. The assumption is therefore made that ships only sail with maximum possible load and that partly loaded ships give load to each other to make a smaller number of maximum loaded ships.

When the too little is transported the trips are financially not feasible for consumers because the price is to large. To prevent consumers from choosing another modality shippers offer a lower price to still have income and not have their ships lie idly by and lose even more money. Therefore a minimum load factor of 0.3 is applied. The choice of using a minimum load factor instead of a maximum price was made because of the large difference in price between different trips, setting one maximum would be inappropriate. The following equations apply when the a load factor of below 0.3 is reached ($p_1 \neq c_1$).

$$p_1 = {}^{p_0}/_{0.3} \tag{5.8}$$

$$loss_{consumer} = WL(p_1) = (p_1 - p_0)q_0(1 + \frac{1/2\epsilon(p_1 - p_0)}{p_0}) \tag{5.9}$$

The consumer loss equation is the same as without price ceiling, however now the maximum price instead of the costs are used.

$$c_1 = {}^{p_0}/_f \tag{5.10}$$

$$loss_{producer} = min(C_{fixed}, q_1(c_1 - p_1)) \tag{5.11}$$

For the producer loss a minimum between fixed costs and the loss due to applying a lower price than costs is applied, since the shipper would choose the option with the least loss. This has no influence on the consumers loss because it is assumed that if the shipper will not sail an other modality will take the job for the same price.

5.1.5 Input - Elasticity of demand

An important variable to determine the total surplus is the demand elasticity. This factor determines the change in transported load with increasing price. Jonkeren (2009) used regression analysis to find the price elasticity from daily data. The data originates from the village Kaub, the German Rhine. Water level was used as the instrument variable, because it influences the price but not the transported load. This resulted in an average elasticity of demand of -0.60 with and standard deviation of 0.27. Similar analyses have been done for other rivers and for different goods, but most were for the United

States. However, it is assumed that the results from Jonkeren (2009) are more applicable to the current study, since the geographical area (Rhine) is roughly similar and the results were acquired with a the same goal in mind, namely economic welfare loss calculations.

5.1.6 Example welfare loss calculation

In this box an example of the welfare loss calculation is given. Three types of inputs are needed for the calculation: the database with information on trips (blue), the Deflt3D modelling results (green) and some assumptions (clearance and elasticity of demand, red).

In Figure 5.5 an impression of the database and Delft3D results is given. A clear weekly trend of high and low values of transported load is shown. Near the winter holidays there's a decrease in transport and also the month January and the summer show decreased transport.

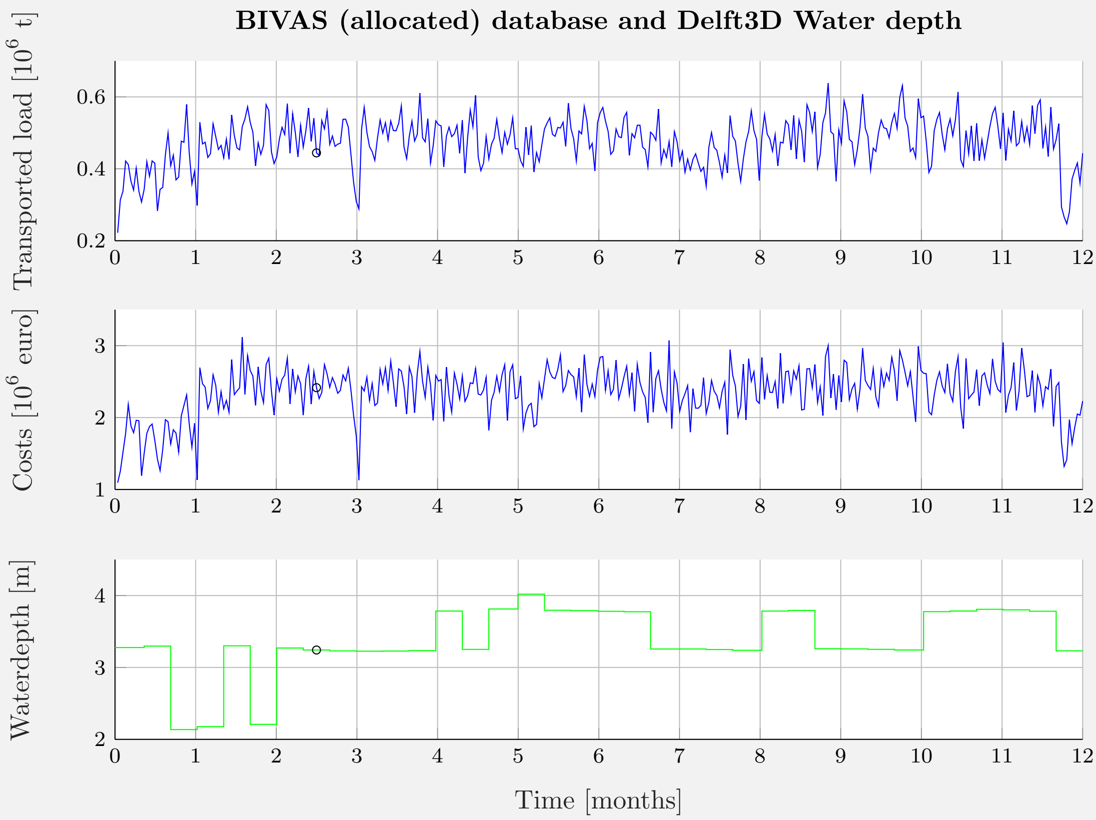

FIGURE 5.5: Impression of the BIVAS database and one year of water depth from the Delft3D simulations. Black circles: values for March 16th.

For each trip the process of calculating the welfare loss from Figure 5.4 is done. Therefore, one day is chosen for this example: March 16th (Table 5.1). On this day some restriction occurs, but it is not an extreme situation.

TABLE 5.1: Values from the database and a water depth for March 16th and the two assumptions.

Date		**16 March**	
Water depth	d	3.24	m
Number of trips		216	
Transported load		4.4 10^5	t
Costs		2.4 10^6	€
Clearance	c	0.3	m
Elasticity of demand	ϵ	-0.6	

The determination of welfare loss is done for one trip on this day. In addition to the water depth and assumptions some variables that are related to the specific trip are needed (Table 5.2).

TABLE 5.2: The values for the parameters needed to calculate welfare loss (and more) for one trip. Since trip ID's are only unique per database (2011, 2013, 2014) and the method uses a compiled database, the database year is named.

Trip ID		**295062**	
Database		2013	
Distance		145	km
Tavel time		8.1	hr
Transported load	q_0	3,656	t
Vessel Width	L	16.8	m
Vessel Length	W	135	m
Waterplane coefficient	C_w	0.9	
Vessel Draught	d_s	3.35	m
Initial Costs	C_0	66,856	€
Initial price	p_0	18.29	€/t

Now that all data has been acquired the four steps of the calculations of welfare loss are done, resulting in Table 5.3. In Figure 5.6 an impression of the welfare loss calculation is given with the supply and demand lines. It also shows a quite large maximum price (48.8 €/t), which is the reason why for the maximum price a minimum of f = 0.3 was chosen.

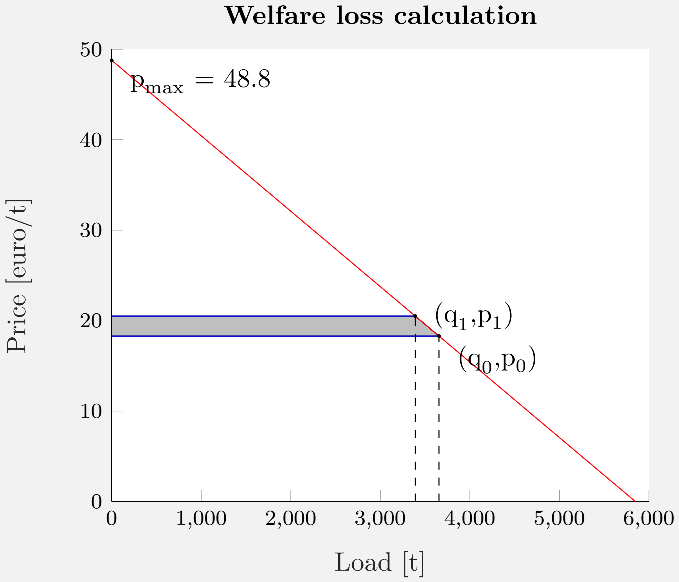

FIGURE 5.6: Impression of the supply and demand lines for trip ID 295062 (2013) on March 16th, with a water depth of 3.24m. Red line: demand function and blue lines: supply functions.

TABLE 5.3: Results of the welfare loss calculation (and it's sub-steps: the four equations) for trip ID 295062 (2013) on March 16th.

EQ	**Variable**		**Value**		**Input**
1	TPCMI	Tonnes per cm immersion	20.5	t/cm	L, W, C_w
2	f	Load factor	0.89		TPCMI, d, d_s, q_0, c
3	p_1	Restricted price	20.50	€/t	f, p_0
4	WL	Welfare loss	7,793	€	p_1, p_0, q_0, ϵ

In order to get a feeling for the size of costs and the other parameters involved Table 5.4 shows the costs, price, transported load and surplus for the initial and restricted situation and the difference between these. If the change in demand with higher price would not have been taken into account the costs would rise with the same percentage as the price. The surplus itself is in the same order of magnitude as the costs.

TABLE 5.4: The costs, price, transported load and surplus for the initial and restricted situation and the difference between these for trip ID 295062 (2013) on March 16th.

		Initial	**Restricted**	**Difference**	**%**
Costs	€	66,856	69,503	2,648	4.0
Price	€/t	18.29	20.50	2.21	12.1
Transported load	t	3,655	3,390	-265	-7.3
Surplus	€	55,713	47,920	-7,793	-14.0

5.1.7 Dredging costs

In the Netherlands dredging is done by dredging companies hired by the government. These companies work under a performance contract, in which the company must prove that the required depth is exceeded; this should be checked at least every two weeks. Due to the use of this type of contract it's difficult to determine the costs per cubic meter of dredged material. In the report 'Case studies Duurzame Vaardiepte Rijndelta' (Mosselman et al.; 2007) the following formula is used for determination of dredge costs:

$$Cost = A * 1.4V \tag{5.12}$$

where A is the price per cubic metre: $A = 2 + 0.6L$ with a maximum of 20 €$/m^3$, L the distance between dredge and dump locations and V the volume of dredged material. The volume and distance can be found from the Delft3D DVR Rhine model. Since this formulation is already a number of years old it needs to be adjusted to the current price levels.This can be done by multiplying the inflations over the years following 2007. Table 5.5 shows the inflation and the final percentage price increase over the years combined.

TABLE 5.5: Inflation per year between 2007 and 2015. (CBS)

Year	2007	2008	2009	2010	2011	2012	2013	2014	2015	Total
%	1.6	2.5	1.2	1.3	2.3	2.5	2.5	1.0	0.6	16.6

5.1.8 Discounting costs

In civil engineering projects a discount rate is used to account for the loss of value of money over time. This can be seen as someone investing in the project, who would of course want at least the capital market interest. In 2007 the risk free discount rate was set to 2.5% by the Dutch government, but due to the crisis this has been decreased to 0% in 2015. This discount rate doesn't account for the risk of the investor for this an extra discount is added: the risk discount. The risk discount is the same as previous years: 3%. The total discount rate is then 3%. Based on recommendations from the government it is assumed to be constant on the long-term as well as short-term (Discontovoet; 2015). The discount rate can be applied as follows:

$$C_d(t) = C(t)\frac{1}{(1+i)^{t-t_0}} \tag{5.13}$$

$$C_d = \sum C(t)\frac{1}{(1+i)^{t-t_0}} \tag{5.14}$$

where C_d the discounted costs, C the original costs, t and t_0 the time and base year respectively and i the discount rate. This method can be used to compare the one-off investment cost now with the dredging costs or the total economic loss over 20 years.

5.2 Results consequence modelling

The method explained above is applied. In this section the results are discussed, first separately for the different cost sources and later the total costs are elaborated on. The welfare loss due to depth restrictions is calculated applying the concept of economic surplus and an extra element is added: producer loss. The total costs are found when adding these two costs sources and the dredging.

5.2.1 Welfare loss

Freight price

When the water depth data is linked to the trip information, the load factor and then the restricted price is calculated. The fraction of price change can be linked back to the minimum depth on a certain day, resulting in a relation between the two (Figure 5.7). Around a depth of about 2.5m there is little data due to discretisation of the discharge (Chapter 4). The median price increase at a minimum depth of 2.8m is 24.4% and at 4m it is almost zero (1%). The 90% confidence interval of the relation is quite large (up to 35%), most likely due to the use of the trip database. Between a depth of 3 and 3.5m a large spread is found. This is most likely due to large ships that do not occur in the whole database, only in some periods. The deterministic results follow the MCS well. An exponential fit is made to the data which fits the data well until a depth of approx. 4.5m (Figure 5.7 right). The formula for the fit is $22.89e^{-1.47X} + 0.80e^{0.041X}$ and r^2 is 0.98.

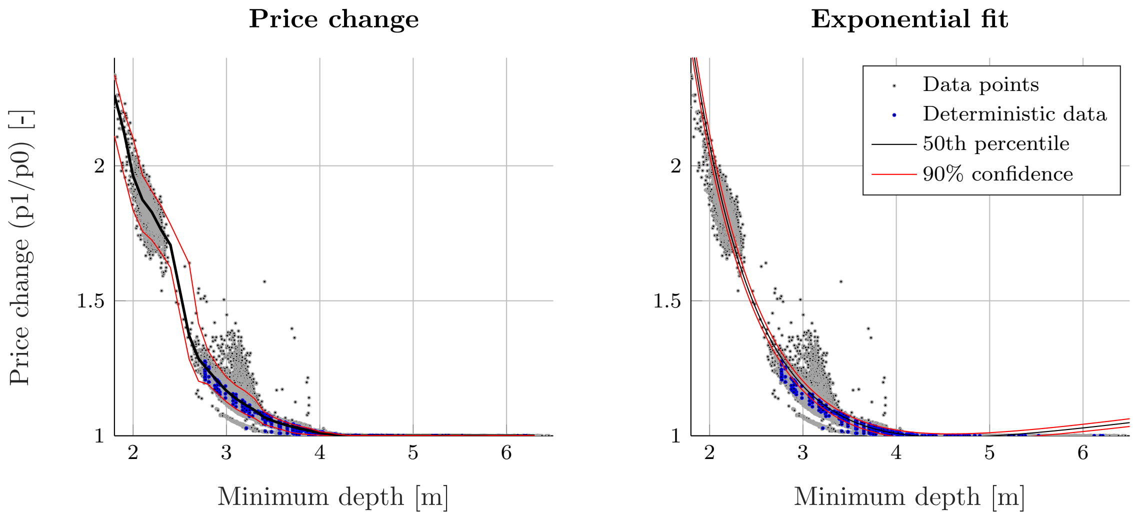

FIGURE 5.7: The average price change per decade related to the minimum depth in the study area (left). The same data exponentially fitted (right).

Figure 5.8 shows the price change as well as the discharge in time. Next to the clear relation between low discharge and price change for the statistic values show a skewed distribution. The confidence interval has an almost rectangular shape most likely due to the discretisation of the discharge.

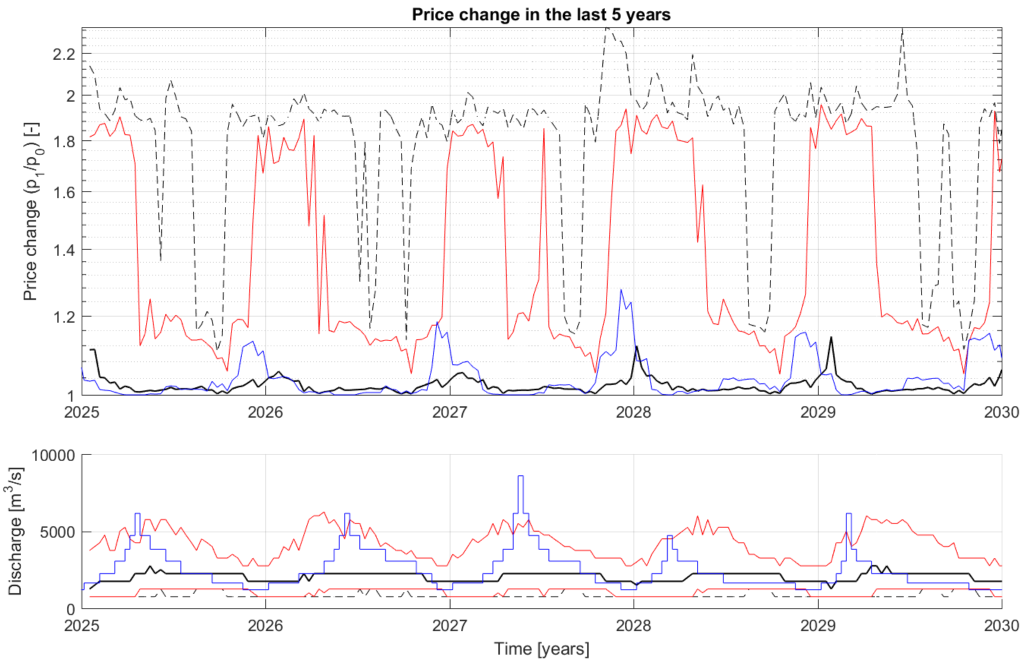

FIGURE 5.8: The average price change per day defined as new price divided by the initial price for the last 5 years of the simulation. Black: median, red: 90th percentile, black dotted: minimum or maximum and blue: deterministic.

Welfare loss

No clear trend of depth in time (other than due to discharge) was found (Chapter 4), thus the same holds for the welfare loss (Figure 5.9). The expected unrestricted costs per year are 974.6 million euro and the expected welfare loss per year is are 65.3 million euro: an increase of 6.7%. The deterministic simulation shows expected costs of 34.8 million euro (3.7%), the median of the MCS is closer to this value (43.1 mil. €) due to the skewed distribution (Figure 5.9). Therefore the deterministic simulation underestimates the expected welfare loss.

In the determination of welfare loss a maximum price is applied when the load factor is smaller than 0.3. The average initial price for trips where the maximum was applied 11.39 €/tonne and the averaged maximum was thus 37.97 €/tonne ($= p_0/0.3$). The average restricted price would have been 103.23 €/tonne if the price ceiling was not applied. Due to the price ceiling the producer makes a loss when transporting goods or when they choose not to sail: fixed costs. These costs are significantly smaller than the consumer loss (4.2%): 2.7 million euro expected loss. Again the distribution is skewed with a 90% confidence interval of about 10 million. The producer loss shows similar ups and downs in time as the welfare loss, as expected.

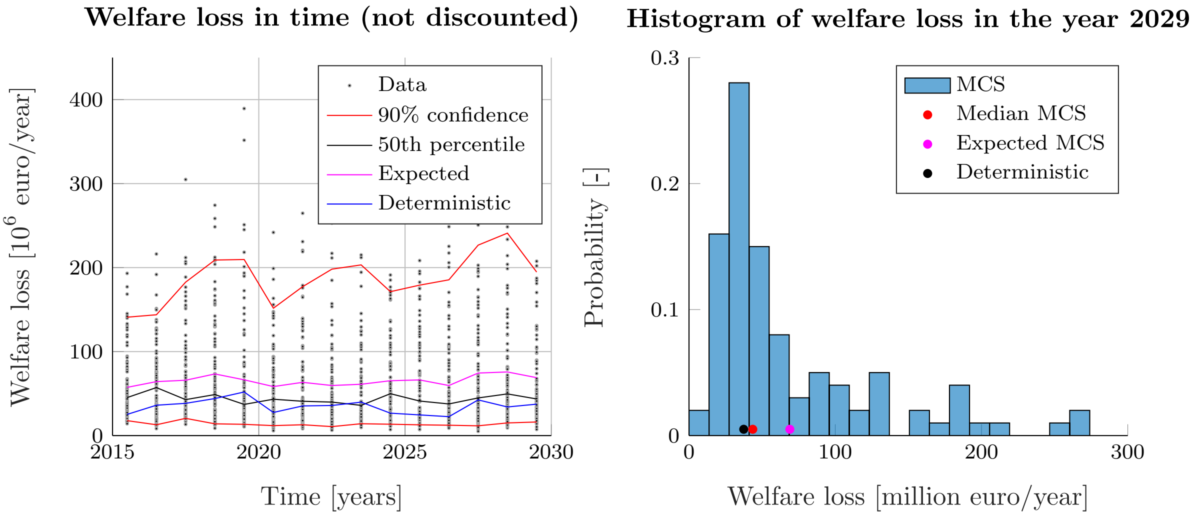

FIGURE 5.9: Non-discounted welfare loss in time (left). Probability function of the welfare loss in year 2029 (right).

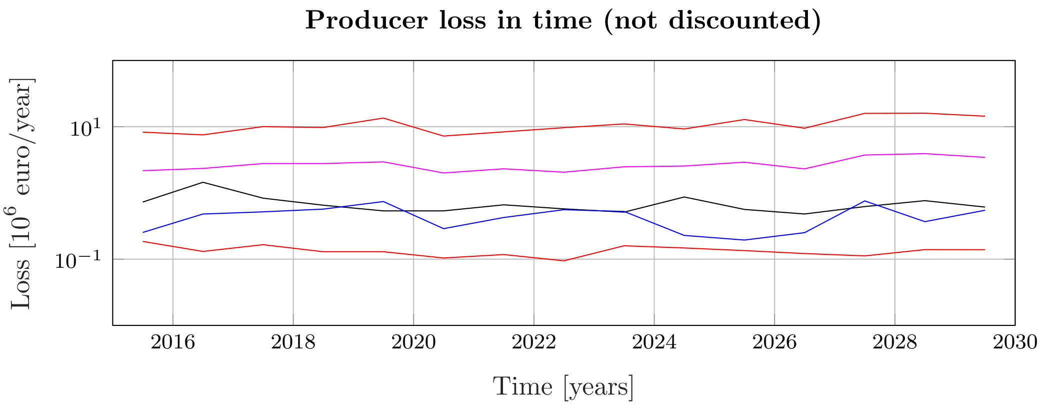

FIGURE 5.10: Producer loss in time on a logarithmic scale. Red: 90% confidence interval, black: 50th percentile, magenta: expected value and blue: deterministic simulation.

5.2.2 Dredging costs

To calculate the dredging costs the price has to be determined using Mosselman et al. (2007), which depends on volume and transported distance. This results in an average price of 3.41 €$/m^3$ in situ, because the dredged material is often dumped in the same or a nearby river kilometre. The trend in time is the same as for the dredging volumes (decreasing) since the price does not differ much in time.

5.2.3 Total costs

Summing up the welfare loss, the producer loss and the dredging costs results in the total economic costs. As to be expected the for the probability distribution of the discharges the costs have also a skewed distribution. The distribution of the deterministic simulation is not as skewed, probably because extreme low values do not occur due to the discretisation and dredging. Another reason is that less data points are available (15 years) and therefore extremes might not occur in the set.

TABLE 5.6: Total (non-discounted) costs per years per source: expected values and 5th and 95th percentile values.

	MCS		Deterministic		
	Expected	Percentiles	Expected	Percentiles	
Welfare loss	65.3	13.7 - 189.1	34.8	23.0 - 49.9	million €
Producer loss	2.72	0.13 - 11.25	0.45	0.20 - 0.75	million €
Dredging costs	0.20	0.01 - 0.57	0.27	0.04 - 0.53	million €
Total costs	68.3	14.1 - 199.5	35.5	23.4 - 51.1	million €

The dredging costs are by far the smallest contributor; a reason for this could be inaccurate dredging amount: the values were a factor 2-4 too small when compared with measurement data. The producer loss is the intermediate contributor ans the largest cost is the welfare loss. The difference in magnitude of the contributors is shown in Figure 5.11. The deterministic simulation shows a significantly smaller welfare loss, most likely due to the discretisation of the discharge hydrograph. The total costs in time shows similar behaviour to the welfare loss, because of its large contribution (Figure 5.12). Due to discounting and the dredging costs decreasing in time the total cost also decrease with time.

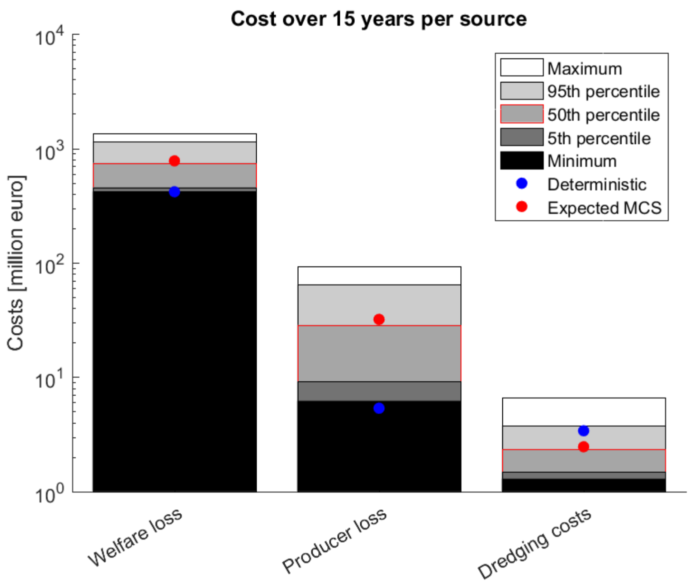

FIGURE 5.11: Total discounted costs in 15 years per source.

Figure 5.13 shows the final result of the risk assessment: a probability density function of the total costs in 15 years. It has an almost symmetric shape with a small upper tail. The upper tail is smaller than in previous graphs because this graph sums 15 years and thus some highs and lows equal out. The expected value is 811 million euro in 15 years for MCS and 425.8 for the deterministic simulation. The large difference is due to the lower welfare loss and producer loss of the deterministic simulation. Part of the reason for this might be the discretisation of the discharges: there is no discharge below 1203 m^3/s.

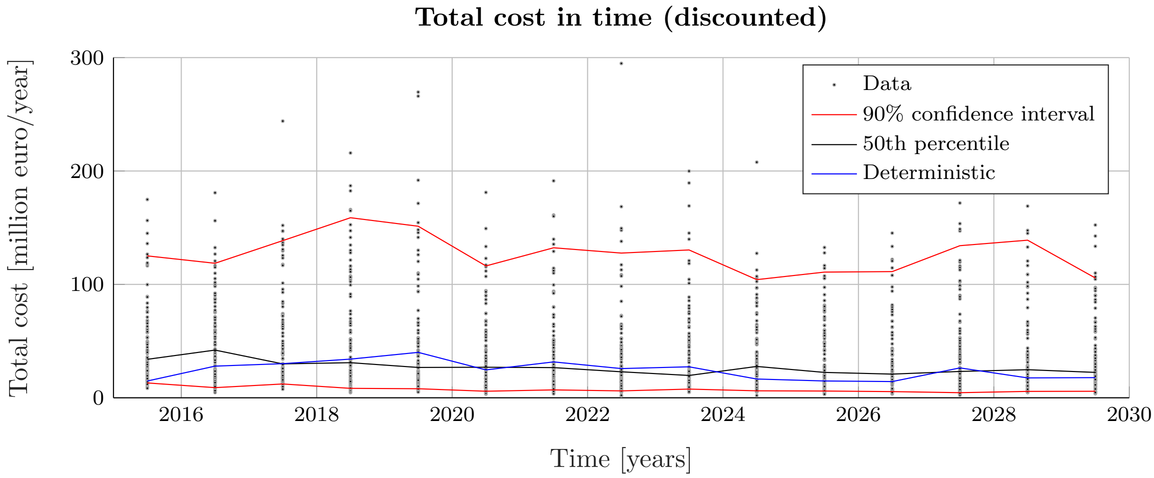

FIGURE 5.12: Dredging costs in time.

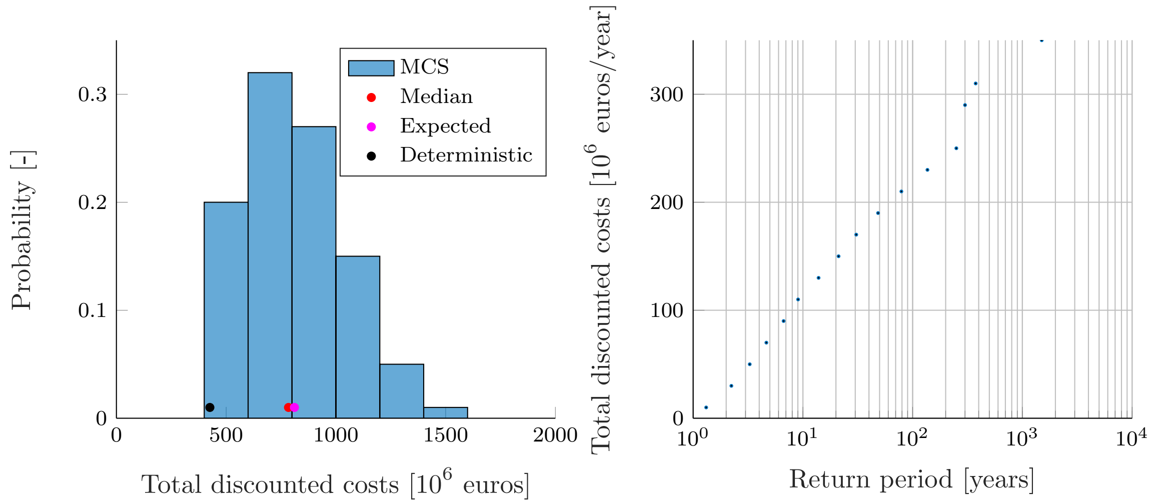

FIGURE 5.13: Left: Probability distribution of total discounted costs in 15 years. Right: return periods of costs per year on semi-logarithmic scale.

5.3 Discussion consequence modelling

5.3.1 Comparison with literature

H. Havinga (personal communication) shows that the expected loss in 30 years due to unloading is 480 million euros. In a dry scenario were a year such as 2003, which normally occurs every 10 years (Bosschieter; 2005), happens every year the value increase to 960 million. The assumption is made that the bed degradation is 0.6m in 30 years, in the DVR model it is approx. 0.5m. The non discounted expected costs in this method are 980 million for 15 years. While the values are in the same order of magnitude there is still a large difference in cost: approx. a factor 2-4. Havinga assumes that the load that cannot be transported times the price in the summer period (20€/tonne) is the loss, therefore does not include welfare loss in the same manner as done in this study. Hence, it is expected that the values differ.

Jonkeren et al. (2007) investigated price and water data and using the theory of economic surplus he found a yearly expected loss of 28 million euro, which is less than half of the 68.3 million found in this study (non discounted). Moreover, he also concluded that in the year 2003 a loss of 91 million would have occurred. The once in 10 year value found in this thesis is about 110 million euro, thus reasonably close to the value for 2003 of Jonkeren et al. (2007).

5.3.2 Simple method

The simple method as explained earlier in this chapter and discussed further in Appendix H, does not use BIVAS and therefore has no relation for demand in time. The method uses information about the entire Dutch Rhine fleet from IVR Schepen Informatie Systeem. Also some assumptions were made about the return period of the ships and the amount of fixed costs. The initial price is 8.76 €/tonne, which was obtained from data provided by Panteia.

The result of the method is lower discounted costs in 15 years: 776.6 million €for the advanced method and 690.7 million euro applying the simple method. The 90% confidence interval was also smaller, most likely due to the more crude assumptions including less variables. The two results from literature and the results from the simple method all show a smaller total loss. To conclude if these three values are underestimations or the advanced method gives an over estimation, more research is needed into the validity of the assumptions.

5.3.3 Consequence determination

The method uses a database of trips that occurred in reality. Even though an optimal database was created using 3 different years of data, in the database still signs of depth related phenomenon are found. However, the use of a database was needed to obtain information on the demand in time.

Another uncertainty in the welfare loss calculation is the elasticity of demand. A value of -0.6 was selected from literature data, but Jonkeren (2009) showed that the standard deviation of this value is 0.27. Moreover, in other papers significantly higher demand elasticities were found. The risk assessment method was applied for two other elasticities of demand: -0.6 minus and plus the standard deviation, resulting in a decrease of 6% and an increase of 2% respectively. The influence on the welfare loss is neglectable, because it is most likely much smaller than the effects of other uncertainties.

It is difficult to say how accurate the method is when looking at one single trip at the time: the formula for supply used is for a company. Moreover, when the price is increased due to insufficient the consumer decides to transport less goods (negative slope of the demand function), but then less ships are needed thus the price decreases again. Therefore using a non-constant supply line would be more accurate, however, not enough knowledge of the cost exist to do so. Research on the behaviour of shippers (producers) and consumers is needed to obtain a better understanding of the supply and demand of a single trip (or the market in general). Also the assumption that the restricted price is equal to the initial price divided by the load factor might not be accurate and needs further investigation.

Moreover, not al costs are paid for by the same stakeholder: the welfare loss mainly comes to consumers, dredging cost are for the government (indirect consumer and producer) and the also producer has separate losses. The shipping company would also have losses due to changes in modal split, but these are not taken into account in this study (Appendix B).

Chapter 6

Dealing with risk

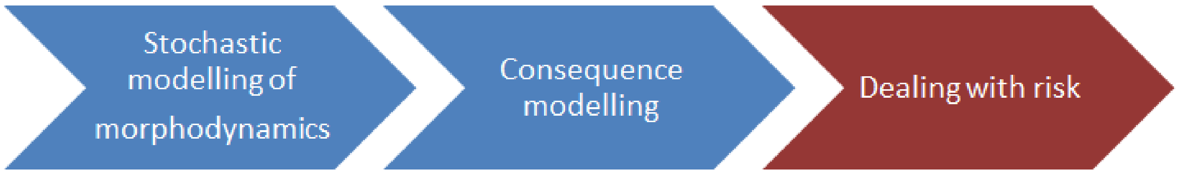

FIGURE 6.1: The general risk assessment approach to developed in this study.

The aim of the developed risk assessment approach is to aid decision makers. To this end an example situation is used to test how well the method preforms in this aspect. A simple question for a decision maker is introduced: is it more cost effective to not take any measures against bed degradation or to implement river nourishments in the manor suggested by the report 'Sustainable Fairway Rhinedelta II' (Ottevanger and Giri; 2015). Of course in reality the issues for decision makers are much more complicated and there exist multiple methods for decision making other than comparison as suggested here.

In the first section of this chapter the method is discussed and the choice for a simple comparison is justified for this investigation. Then the selected case study is further elaborated on, mentioning as well the theory behind nourishments as the implementation in Delft3D. After which the morphological behaviour and monetary costs are compared between the reference and nourishment case.

6.1 Methods for decision making

There are multiple methods of quantifying consequences, the most universal being translating consequences to monetary values. The monetary loss in navigation can be found be looking at the increasing costs due to less availability of the channels, or in other words the increase in price times the quantity. This welfare loss can then be related to the costs of a measure to mitigate the shipping problems, e.g.

dredging. However, most measures do not just serve one river function and it is therefore difficult to say what the costs of a measure just for navigation purposes would be. For this reason no cost-benefit analyses can be done in this thesis, since it only focuses on one function. Meanwhile it is possible to compare different measures using their economic consequences for shipping and dredging costs (which main purpose is keeping the river navigable).

6.2 Nourishment case

By Ottevanger and Giri (2015) a number of mitigating measures are investigated. In this study only one is analysed to be able to properly link the results back to the measure. A choice was presented between a case were river nourishments were applied and one were training walls are implemented. The nourishment case was selected, since it has cost in time and the costs are easier to calculated.

Nourishments are implemented to mitigate the ongoing bed degradation on the Dutch Rhine. While bed degradation might seem favourable for flood safety due to lower water levels, it forms a risk to stability of structures. Bed degradation might also cause local reductions in navigable depth due to fixed layers that do not lower with other parts of the river, while the water level does.

Nourishments are an often used non-permanent or flexible solution. A nourishment is an addition of sediment in an eroding area; the problem with this type of solution is that the extra sediment moves downstream, which can cause problems in other parts of the river and makes the effect at the desired location only temporary. Coarser materials are a better option for nourishments because they have a lower travel speed, due to their decreased mobility, and thus are effective longer. A risk of using coarse sediment for nourishments is that bed degradation downstream of the nourishment is increased. The coarse sediment has less mobility and therefore the transport capacity downstream is higher than at the nourishment, causing erosion (Blom; 2016).

6.2.1 Implementation in Delft3D DVR schematisation

As explained earlier the case chosen for the risk management part of this research is the nourishment case, where at the locations shown in Figure 6.2 bed stabilisations in the form of nourishments are applied. The nourishments are placed on the entire Pannerdensch Canal, the first 40km of the IJssel, the Neder-Rijn up to Driel and the Waal just after the bend near Nijmegen.

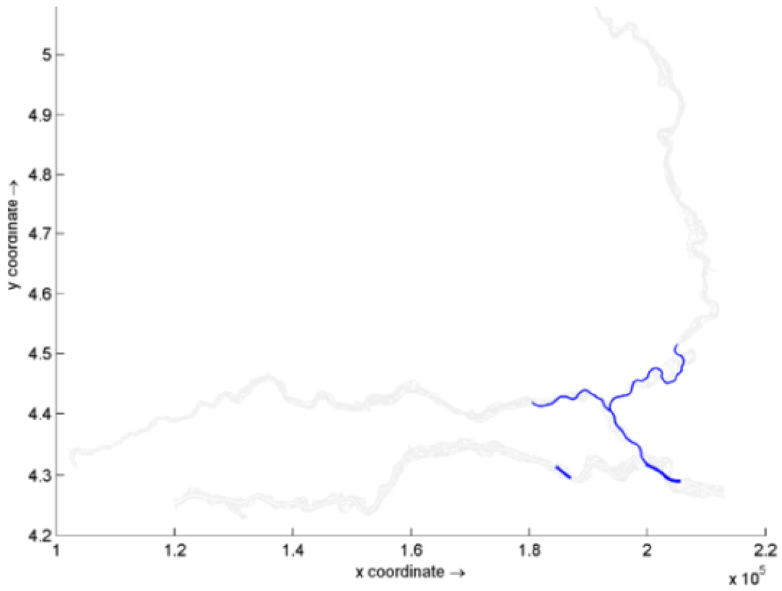

FIGURE 6.2: River nourishment locations. (Ottevanger and Giri; 2015)

The target level for the nourishments is set to the level it was in 2010 (the first year of the simulation), therefore the nourishment is only applied when the averaged depth is below this level (Figure 6.3). The second requirement is that the nourishment is only applied when the depth satisfies the shipping requirements. A layer of sand with a thickness of 0.3m is placed within the polygon width (or fairway width). The applied sediment is a 50/50 mixture of granite ($D_{50} = 5mm$) and fine gravel ($D_{50} = 7mm$). The nourishments are applied yearly from 2020 onwards.

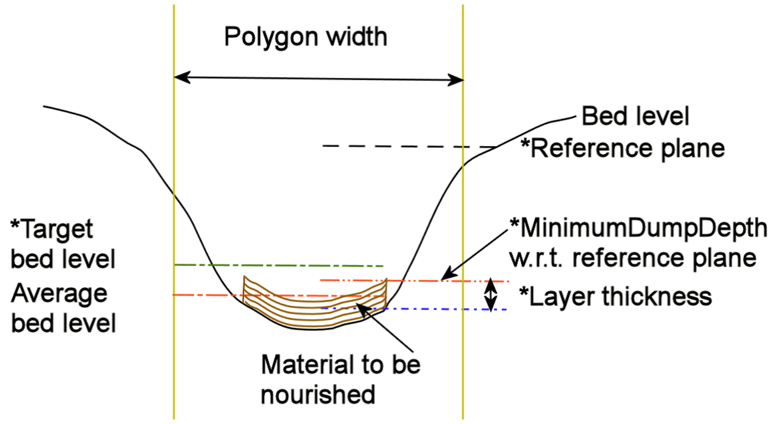

FIGURE 6.3: Definition sketch of nourishment criteria. (Ottevanger and Giri; 2015)

6.3 Morphological behaviour of the cases

6.3.1 Bed level

Due to nourishments on the Boven-Rijn and Waal downstream of Nijmegen the bed level in the nourished areas is expected to decrease less than in the reference case. For the Boven-Rijn the results are clear: the bed level increases with respect to the reference situation (Figure 6.4). For the Boven-Rijn it is enough to compensate the already occurred erosion in the first 10 years (2010-2020), but for the upper and middle Waal it is not enough to stop further degradation. The lower Waal erodes with approx 1 millimetre more than in the reference situation, insignificant compared to the general degradation in the order of decimetres. An explanation for this is the sediment upstream being coarser due to the nourishments and thus causing a difference in transport capacity between up and downstream.

The deterministic simulation shows more accretion for the lower and middle Waal in the first 2 years. Around this time the behaviour starts to change from seemingly initial behaviour to behaviour more similar to the MCS, which could explain the differences.

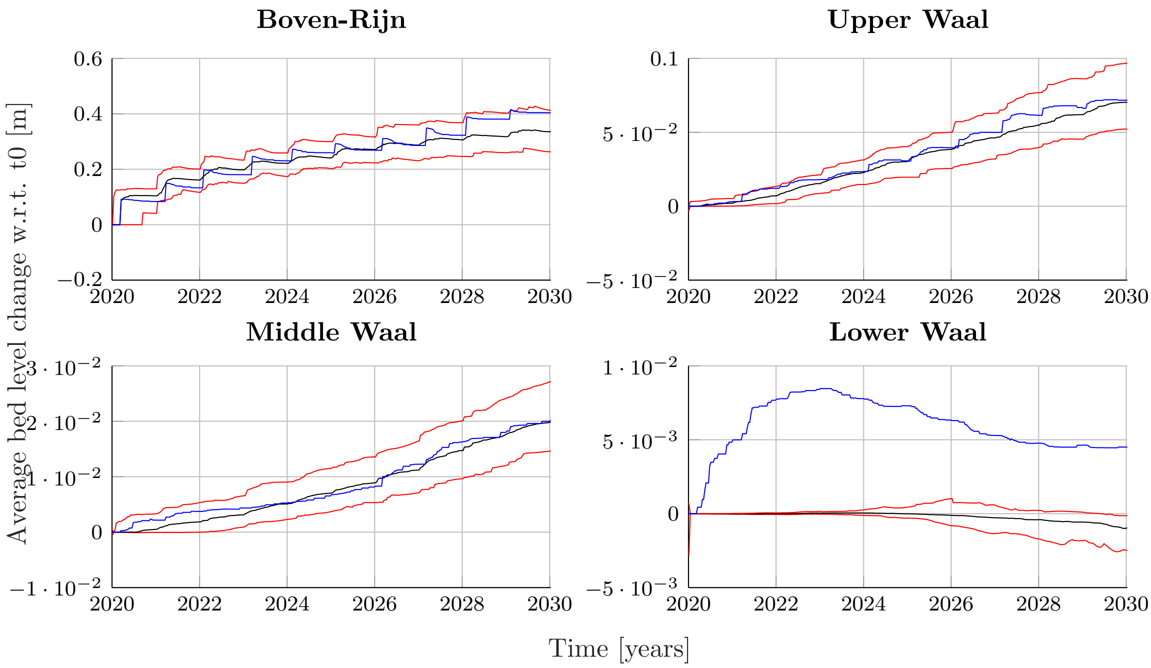

FIGURE 6.4: The difference in averaged bed level over the 'normal width' between the nourishment and reference case averaged per river section. Black: 50th percentile, red: 90th percentile, blue: deterministic simulation.

The Boven-Rijn accretes (Table 6.1), the reason for this is that the first 5 years of the simulations are not taken into account, but the nourishments are based on the initial bed level. The upper and middle Waal have less erosion per year, but are still eroding. Again the change in the lower Waal is insignificant. Even though the reaches on average show improvement the results are quite local (Figure 6.5). This is expected since the nourishment are local as well.

TABLE 6.1: The averaged bed level change in time for the last 5 years of the simulation and the measurements from 1999-2006 (Sieben; 2008).

	Avg. measurement	**Reference**	**Nourishments**	
Boven-Rijn	-2.7	-1.2	1.0	cm/yr
Upper Waal	-2.1	-2.1	-1.6	cm/yr
Middle Waal	-0.9	-1.6	-1.4	cm/yr
Lower Waal	-1.3	-1.1	-1.1	cm/yr

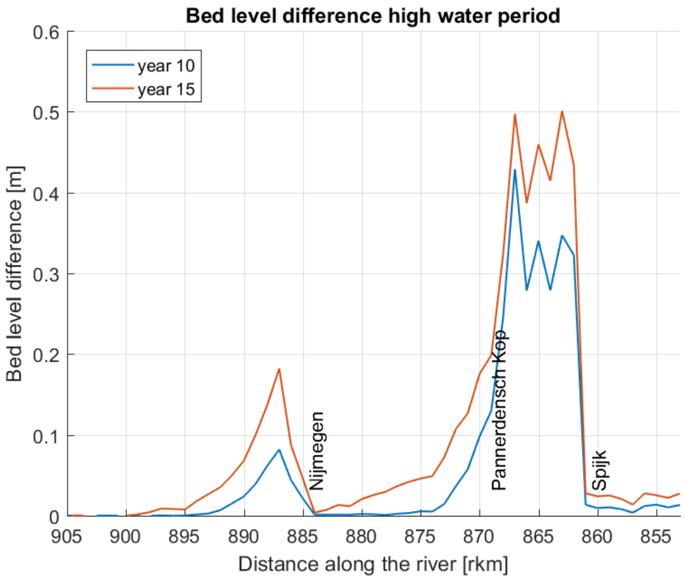

FIGURE 6.5: Average bed level difference between the reference case and the nourishment case for year 10 and 15 during the high water period (March). Averaged per rkm.

6.3.2 Discharge distribution

The discharge distribution is influenced by the nourishments. The first parts of the Neder-Rijn and IJssel are nourished and the entire Pannerdensch Canal, but the Waal only near Nijmegen. A larger amount of water is therefore distributed to the Waal (Figure 6.6). However, the minima of the distribution are lower for the nourishment case than for the reference. This is possibly caused by sedimentation at the entrance of the Waal due to Boven-Rijn nourishments.

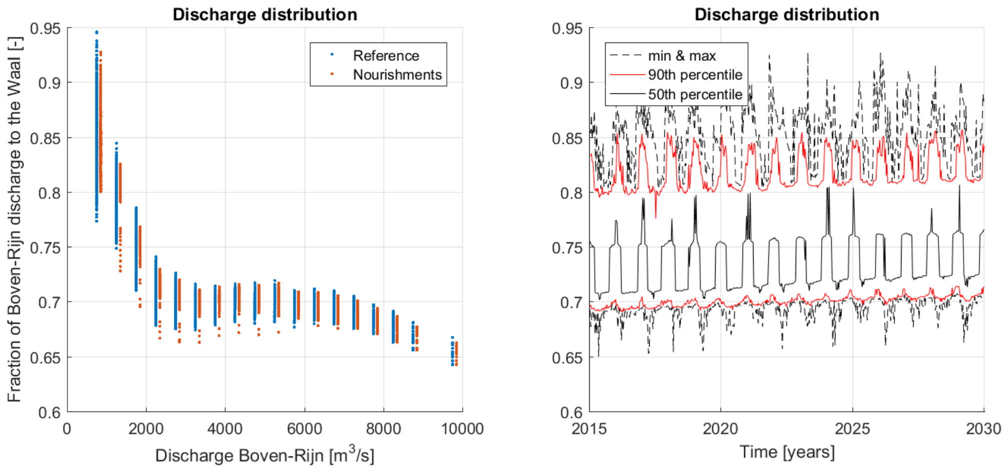

FIGURE 6.6: Left: Discharge distribution (Waal/Boven-Rijn) per for the discharge levels at Lobith. All data included (100 x 15 years). Right: Discharge distribution (Waal/Boven-Rijn) in time. All data included (100 x 15 years)

6.3.3 Dredging

The dredging amount for the reference and nourishment case do not differ much (Figure 6.7). Near Nijmegen a little more is dredged, but it is insignificant compared to the total dredging volumes.

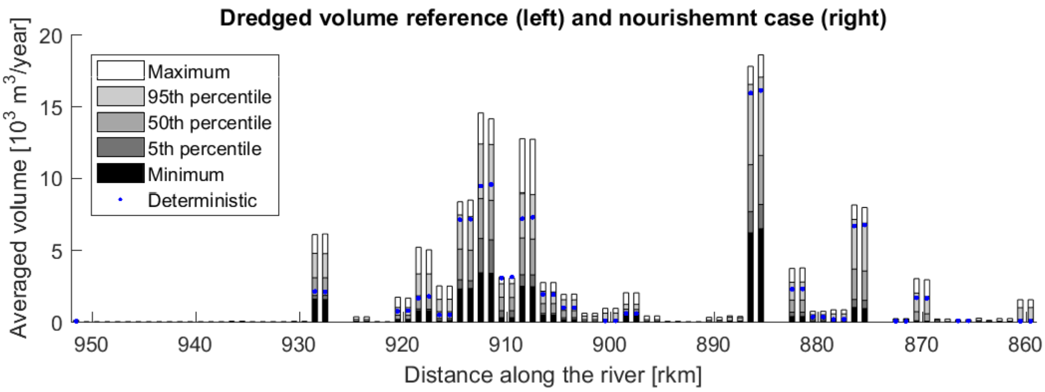

FIGURE 6.7: Dredged volumes on the Waal and Boven-Rijn for reference (left) and nourishment case (right). Bar colours from dark to light: minimum, 5th percentile, 50th percentile (red edge), 95th percentile, maximum. Blue: deterministic simulation.

6.3.4 Nourishments

The expected yearly nourished volume is 167,000 m^3: the same order of magnitude as dredging and mining over the entire Waal and Boven-Rijn. The amounts of nourished sediment are larger on the Boven-Rijn than the Waal, as expected since the bed decrease in of the Boven-Rijn is larger and only a small part of the Waal is nourished. The deterministic simulation shows for especially the Boven-Rijn larger volumes. An explanation for this is the faster decline of the Boven-Rijn in the deterministic simulation than the MCS.

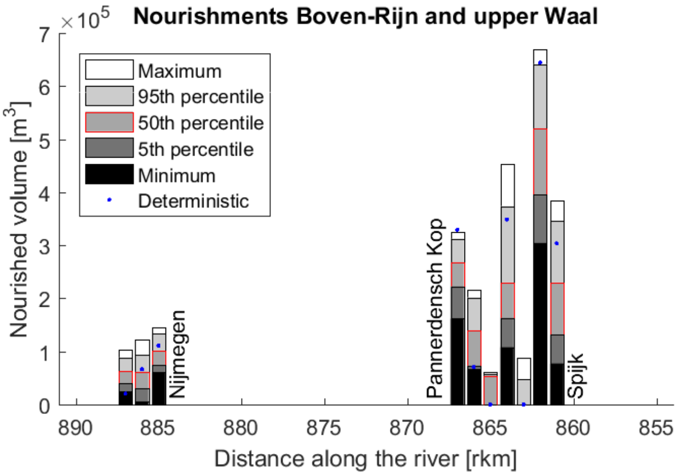

FIGURE 6.8: Nourished volumes in 10 years (2020-2030) on the Waal and Boven-Rijn. Averaged per 2 rkm.

6.3.5 Navigational requirements

The Nourishment case generally has a higher LAD, therefore increasing navigability (Figure 6.9). The location where LAD is 1% more often at St. Andries, but near Nijmegen is still the location where LAD occurs most of the time. Thus the issues with the fixed layer and ongoing bed degradation decrease when applying nourishments after this location.

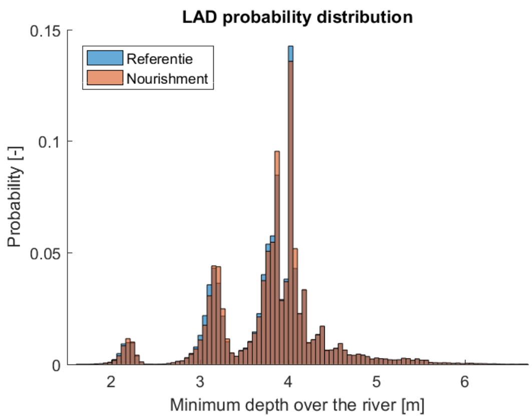

FIGURE 6.9: Probability distribution of LAD for the reference and nourishment case.

6.4 Economic consequences

No estimates of the costs for river nourishments have been found in literature. The costs for coast nourishments lie between 4 and 6 €$/m^3$ (Rijkswaterstaat; 2008; STOWA; n.d.). RWS estimated the costs of nourishments to be 5 €$/m^3$ when using dredged material and 10 €$/m^3$ if new sediment is brought in (S. Quartel, personal communication, 28-02-2017). In the simulations new material is added to the river and therefore the price of 10 $€/m^3$ is used.

As discussed in the previous chapter the depth differences are small and therefore also the difference in results for the for the contributors are small, thus only the final results are compared. Figure 6.10 shows that indeed the nourishment case has less welfare and procurer losses. Also the distribution between two is a little different, but for both cases shows a little larger upper tail.

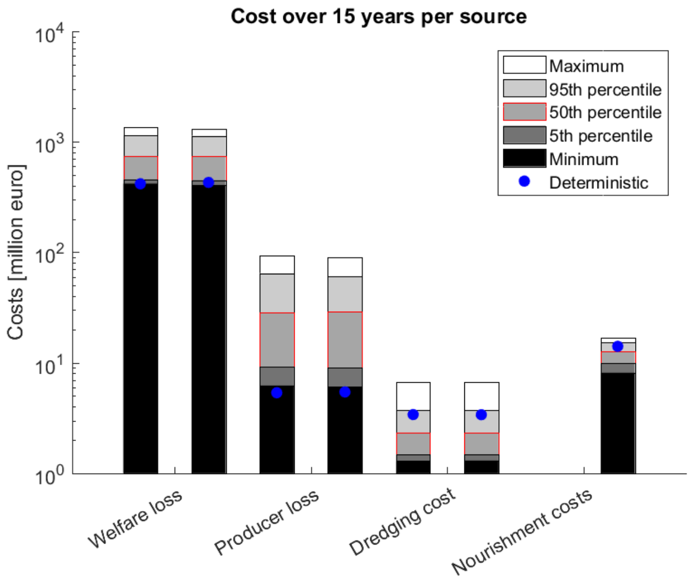

FIGURE 6.10: The total costs in 15 years per contributor. Left: reference case. Right: nourishment case.

The nourishments have to catch up to the amount of erosion already occurred in the first 5 years and therefore in the first years the nourished volumes are larger, thus nourishment costs show a decline in time. The expected costs decreases from 3.1 to 1.2 million euro per year, due to discounting costs the difference increases even further in time. Figure 6.11 shows the total discounted costs in time. The deterministic simulation does not make a profit even until the last year of the simulation, therefore in 15 years the case also made a loss. For the MCS the improvement was shown near Nijmegen, which only forms a bottleneck for the lowest discharges which are included less in the DVR hydrograph.

The expected yearly costs of the nourishment case are smaller, therefore a profit is expected. Also the 90% confidence interval has decreased, as to be expected since the nourishments compensate for some differences in bed levels. However, the probability of making a loss still exists (Table 6.2). Generally simulations that do not make a profit already had relatively low welfare and producer losses, because the discharges were also lower, in these cases the necessity of was smaller. As expected cases with a smaller bed level decrease often make a profit since the nourishment costs are higher.

FIGURE 6.11: The difference in costs between the reference case and the nourishment case per year. Black: median, magenta: expected, red: 90% confidence interval, blue: deterministic.

TABLE 6.2: Comparison between the statistical values of the costs of the reference case and the nourishment case divided by cost contributor. In million euro over 15 years.

	Reference case		Nourishment case		Difference	
	Expected	Percentiles	Expected	Percentiles	Expected	Percentiles
Welfare loss	776.6	457.1 - 1146.8	764.1	449.8 - 1132.3	12.5	3.3 - 25.6
Producer loss	31.9	9.2 - 64.6	30.5	9.0 - 60.7	1.48	0.01 - 5.11
Dredging costs	2.46	1.48 - 3.74	2.46	1.48 - 3.70	0	-0.03 - 0.03
Nourishment costs	0	0	12.6	9.9 - 15.3	-12.6	-15.3 - -9.87
Total costs	811	469.1 - 1219.8	809.6	474.6 - 1215.8	1.38	-9.76 - 18.94

The final result of the risk assessment: the probability distribution of the difference in total discounted costs is presented in Figure 6.12. While the expected value indeed indicates a profit the median value is a loss. The loss for the deterministic simulation is large, because already no extremely low depths occurred and therefore the benefits do not outweigh the costs.

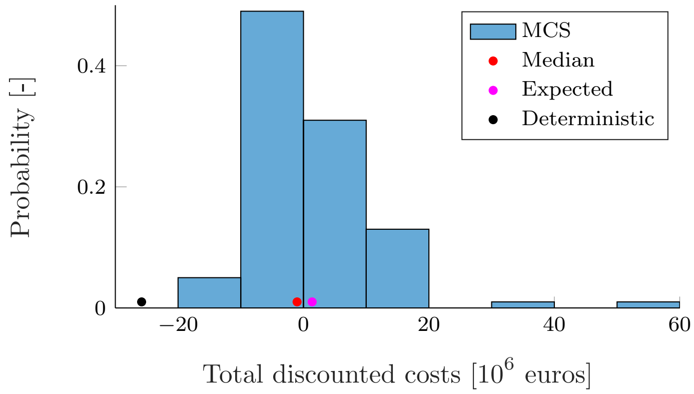

FIGURE 6.12: Probability distribution of difference of total discounted costs in 15 years between reference and nourishment case.

6.5 Discussion

In this chapter a comparison was done between the reference case and a case including nourishments. Decision makers need to know whether it is expected that a measure will make a profit and also the chance that it does not. Also the time it takes to make a profit is important for investors, even if they do not directly get the returns, such as with the government.

Only two new assumptions were done in this chapter: the implementation of the nourishments and the costs per cubic metre. However, the method of implementation of the is of extreme importance to the effects. In future studies an estimate of the effect of applying nourishments in a different way should be investigated. Because the profits are low the costs of the nourishments have a significant influence on whether a profit is made, but not much knowledge on and only rough estimates of the price exist.

In river management all river functions should be assessed when investigating the costs and benefits of a measure. In this case only navigation was taken into account and the measure was already expected to make a profit. Since most other river functions benefit from nourishments as well, e.g. cables, pipelines and the stability of structures, it is likely that it will also make a profit when doing a broader assessment. However, in this assessment indirect and unquantifiable consequences and stakeholder opinions should also be taken into account.

Chapter 7

Extendibility of the risk assessment method

In the previous chapters a method was developed for risk assessment and dealing with risk. The method included stochastic modelling of morphodynamic behaviour, the translation to consequences and risk and a case study of a measure. This risk assessment method was developed in order to create a broader risk assessment for river morphodynamics. Therefore, in this chapter some possibilities for extending the method to other contexts are discussed.

7.1 Integral risk assessment method for morphodynamics

The current study is done with a larger research plan in mind: the development of an integral risk assessment method for morphodynamics in order to support decision makers. Or in other words: including all (important) river functions and uncertainties in the developed method. The general method that was applied in this thesis is can be easily extended to include more uncertainties and river functions. However, doing so would result in an extremely time-consuming method. This is explained further in this section and some possibilities for improvement of the method are presented.

The first part of the method is stochastic modelling of morphodynamic behaviour, in which uncertainty is introduced. MCS with crude sampling of discharge hydrographs is used on a morphodynamic model, resulting in a set of output variables that change in time. MCS can be used for multiple uncertainty variables, however, in order to get the same accuracy the number of simulations would increase significantly. Therefore it should be investigated if for multiple uncertainties a different method for resampling is more feasible because of shorter computational times, for example Latin Hypercube Sampling.

A large advantage of this method that it that it returns sets of output variables instead of each variable being determined separately. Because of this the correlation of the variables is maintained. For the next step: determining consequences, these variables are used. Each river function needs different variables, e.g. while water depth is the main factor for water transport, the local bed level is of importance for cables and pipelines. For each river function a suitable way of determining consequences from the morphodynamic variables must be found.

Flood risk

In the Netherlands a national method exist for the determination of flood risk. However, the details of the methods can differ between authorities. Risk is a multiplication of probability and consequences; below the steps taken for these subjects are given (after VNK2 (2012)).

Probability

1. Decomposing levee system into elements or sections.
2. Schematisation of sections and failure probability calculation.
3. Calculate scenario probabilities.

Consequences

1. Defining consequence segments.
2. Producing flood propagation models.
3. Defining scenarios.
4. Selecting consequence estimates for each scenario.

To calculate the probability of failure the defences are tested for a number of failure modes using the hydraulic loads water level and wave action. Both the strength of the defence and the loading conditions use a probability distribution instead of design values. To include morphology a large number of possible bed levels can be selected from the Delft3D DVR model to which discharges with different probabilities are applied. Also waves should be included in the model. This can result in new probability distributions for water levels. It should be investigated whether using the Delft3D model or the current method (TMR 2006) with the Delft3D DVR model bed levels is more accurate for water level calculations.

On the consequence side of the national flood risk method morphology should be included on the second step: the flood propagation models. Important variables in flood propagation are the flood characteristics, e.g. water depth, velocity and rise rate. The flood propagation models are produced by the provincial authorities. For each of these models it should be investigated whether and how (river) morphology can be included.

Cables and pipelines

For cables and pipelines no risk assessments exist at the moment, but some initial studies have been done. The Dutch government did an analysis on the groundcover of cables and pipelines (Rijkswaterstaat; 2014). The study concluded that for a lot of them it is unknown exactly where they are and therefore field investigations are necessary. Stijnen and Nicolai (2012) made a framework for risk assessment for pipelines using a failure tree to determine probability of failure of the system. Morphology is of importance because due to erosion the cables and pipelines can lay bare or even become free spanning.

The study of risk for cables and pipelines requires local knowledge of bed level changes: order of metres, while in the Delft3D the smallest direction is in the order dozens of metres and the longest direction as large as a 100m. However, for an initial estimate this most likely is prices enough; it should be investigated further whether this is true indeed.

The consequences of damage to cables or pipelines differs per type, e.g. cable or pipeline damage can cause lack of power, telecommunication, or water and even hazardous chemicals getting loose in the environment. Not much has been published on the subject of consequences for river crossing cables and pipelines, however, insurance companies most likely have knowledge about this subject. Some studies were done on the risk for sea cables and pipelines (Veritas; 2010; Bai and Bai; 2012; Yoon and Na; 2013). While they are partly applicable to river crossing cables and pipelines some aspects might differ significantly, e.g. type of cover, failure modes or types of damage and resulting consequences. Moreover, non of these studies are able to quantify the consequences of failure, but qualitative assessments are made for loss of life, environmental impact and material damages.

Water supply

It is of utmost importance that during low waters enough water is available for drinking water and preferably for agriculture and industry. As a result of climate change more and longer dry periods will occur and the sea level will rise. The consequences of the first are clear: water shortage, not only the amount of water but also the distribution thereof is essential to water supply. Sea level rise causes salt intrusion to move further landward, creating more brackish water which needs to be desalted before use. Most of these issues have little to nothing to do with river morphology, however, the water distribution and water intake are affected by morphology.

The latter is effected by large scale bed degradation which causes water level decrease and therefore a large height difference needs to be overcome. This will happen over a large time scale and the most logical solution is to increase the power of the pumps used for intake. Developing a risk assessment for this is deemed unnecessary due to the nature of the problem.

The morphology of the bifurcations and the large scale behaviour of the branches has an effect on the water distribution. Using Delft3D the water distribution can be determined. Evaluation of the consequences involves a large scale investigation of the losses for industry, agriculture and water treatment plants. A few examples of consequences are decrease in productivity, failed or smaller harvest and investments for water storage or extra treatment. Quantification of the consequences is difficult since there are many indirect consequences. Also the consequences are largely dependent on the water supply chain: governmental policy. Haasnoot et al. (2014) proposed a method for determination of crop damage in fractions. Using a similar method for economic loss calculations as done for inland water transport, it would be relatively easy to translate this to monetary values (if demand curves are known). The economic impact of lack of water supply for industry requires more knowledge on the type of industries, how much they use, the exact water supply chain and the demand and supply of the product.

Decision making

The most applied methods for risk evaluation and management are using a threshold or an (economic) optimum. Of course there are also more complex methods, which for example weigh the importance of stakeholder opinions and indirect consequences. Applying these is outside the scope of this thesis, since it focusses on monetary risk. For the four above named river functions both an threshold and an optimum method can be applied. When comparing measure alternatives, the ones that do no reach the threshold are dropped or adjusted after which residual measure alternatives are compared by expected costs. This can be done either over a certain period, e.g. the sum after 15 years, or at each point in time. To find optimal solutions the latter is better since extra measures can be added at points in time where the costs are large.

7.2 Applicability for different study areas

The current study relied for a large part on the hydraulic and morphological model for the Boven-Rijn and Waal. However, if the developed risk assessment and management method was to be used in other countries or for different study areas, some problems might arise. For example accurate discharge information, a model and fleet data might not be readily available in all countries.

For Europe a model for shipping and other transport costs already exists: TRANS-TOOLS. If the scale of the morphology is expanded to Europe the model cannot be as detailed as the current model and most likely a 1D model would be applied.

The current study was applied to a lower river delta and a river that has been canalised over time. The results in quite mild and controlled flow patterns. In the lower Rhine the river bed is made of sand and gravel, but other possible study areas might have a bed made of (larger) rock. These factors change the nature of the morphology and therefore completely different factor might play an important role. In the lower Rhine river ice is quite uncommon, but for other areas the influence of it should be taken into account. A short description on river ice is given in Appendix E.

7.3 Risk assessment in operational context

Note: the current section is only applied to discharge uncertainty and consequences for shipping.

The current method focuses on long term risk assessment and management; an option to improve the current method is to include an operational aspect.For this the emphasis should be on short term knowledge of water depth and thus morphology and hydrodynamics. With this information shippers can decide whether to change their planning, route, speed or used ship type for example. Also dredging activities can be timed better if more precise knowledge exists on future water levels and morphological behaviour.

The current uncertainty analysis of morphological response gives a range of variation and cannot be used for bed level information on an exact date. A method for short term morphological behaviour would involve more recent information on bed levels and discharge predictions instead of resampling historic data. The discharge data can be determined using rainfall predictions in a hydrological run-off model.The discharge predictions should be related to water and bed levels. This can be done using the current method with a detailed hydraulic and morphological model, however, due to computational time and the time involved to set up the input this might not be a realistic solution. A possible solution is looking at possible morphological developments under discharge for different initial beds; this should be done for locations that are often bottlenecks.

Currently multiple organisations commissioned by Rijkswaterstaat are doing research into the use of under-keel clearance echo sound measurements for bed level information for other vessels. A large amount of vessels already measure the under-keel clearance, however, this data is not saved and only used at that moment; the new projects aim is to collect this data. The under-keel clearance needs to be adjusted using draught, water depth, vessel speed and direction to be used for bed level measurements. When this initial bed level data becomes available it's possible to use it as input for morphological predictions.

For each time step (over a period of approx. 10 days) a PDF for the depth can be determined using the previous method. This can be used for the following purposes:

- More accurate planning
- Choice of route
- Vessel speed
- Optimisation of fuel use
- Reliable travel time estimates
- More precise unloading

Based on the probability of a depth occurring at a certain time it can be considered whether to not sail, wait, unload, decrease speeds or change routes. For the options of not sailing, unloading and change of routes the consequences are relatively easy to quantify (see chapter 5); if assumed that at the moment of sailing the water depth is certain. If that is not the case extra margins need to be taken to prevent ships getting stuck.

7.4 Indirect and non-quantifiable consequences

Note: the current section is only applied to consequences for shipping.

The developed method only takes into account direct consequences that are relatively easily quantifiable. To get a complete few of the risk the indirect and non-quantifiable consequences should be added to the method. Important herein are the delays in transport. Base products arrive late, which may stop production for some time. The costs of this are difficult to quantify, because every case is different, i.e. depending on planning, industry, contract etc..

The Dutch government determined values for costs of extra travel time and less reliability: Value of Time (VoT) and Value of Reliability (VoR) respectively. These were determined from questionnaires with shippers, transporting enterprises and unloaders. Table 7.1 shows the values based on the prices of 2010. Reliability is defined as the benefits of decrease in spread in travel time (*De maatschappelijke waarde van kortere en betrouwbaardere reistijden*; 2013).

TABLE 7.1: Values of Time and Reliability. Given in euro's per hour for prices of 2010. (*De maatschappelijke waarde van kortere en betrouwbaardere reistijden*; 2013)

Location	VoT	VoR
Quay	76.7	27
Lock/Bridge	338	28.3

These values can be used to determine extra costs due to increase of fairway intensity because of extra ships. BIVAS can be used to find the extra travel time. However, this is only a part of the costs: the time the industry has to wait for their goods is not accounted for.

7.5 The future

The risk assessment method is based on a period of 15 years, because of the time consuming nature of the Delft3D model. 15 years is still a short period when assessing river management measures.

Therefore a longer period should be applied, to do so some other future effects such as climate change and economic growth must be included.

Inland water transport

With road transport becoming more costly due to the increasing amounts of congestion of the roads, the modal split might change in favour of inland water transport. However, the data found about water transport dates no further back than 2007 (CBS, Eurostat); the economic crisis of 2008 and teh period thereafter have a large influence on this data, so no long term trends can be found.

Over time ship sizes have increased and the fairway has been adjusted ot facilitate larger vessels. Figure 7.1 shows the year of built for different CEMT-classes of the current western European fleet. It should be kept in mind that for vessels further in the past the number of ships is less correct since some may have retired. However, it is clear that CEMT-class V has become increasingly popular, while CEMT-classes I, II, and III are barely built any more (Source: IVR Schepen Informatie Systeem).

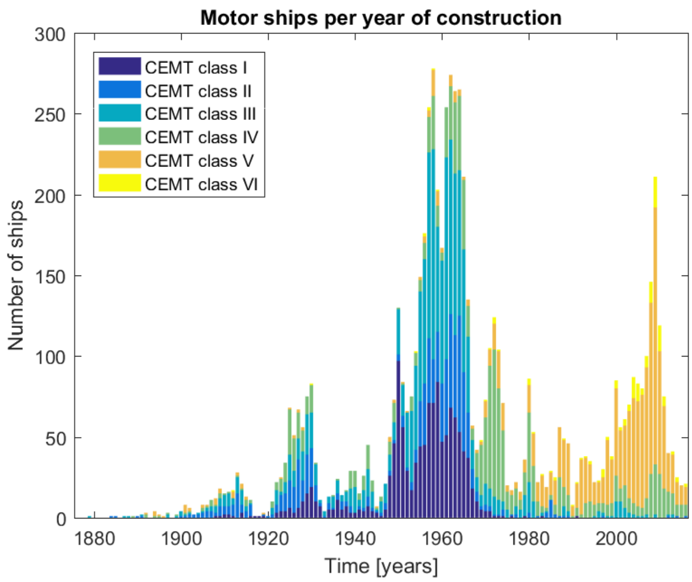

Figure 7.1: Number of ships and year of built per CEMT-type for the western European fleet. Source: IVR Schepen Informatie Systeem

The ship size is not likely to increase in the near future since many of the current fairways in western Europe are not even class VI yet. However, if data on the fleet change becomes available: it is possible to include this and economic growth scenarios in BIVAS.

Chapter 8

Conclusions and discussion

8.1 Conclusions

In this thesis a method was developed for morphodynamic risk assessment and decision making. It is specified to the uncertainty in discharge and the risk for navigation, with the research question: *How can economic risk for navigation related to uncertain morphological behaviour be quantified and how can it be used to support decision making?*. The results are discussed in five parts that match the sub-questions: morphodynamic behaviour and uncertainties, stochastic morphological modelling, consequence modelling, decision making and the extendibility of the risk assessment method. Starting with uncertainty in discharge through stochastic morphodynamic modelling and consequence modelling the risk for navigation was determined. How to deal with this risk is the next step and finally some possibilities for extending the method are given.

8.1.1 Morphodynamic behaviour of the river system

'How can morphodynamic behaviour of the Rhine system be modelled and which uncertainties are most important therein?'

In the Dutch Rhine a number of different phenomenon are found such as bend effects, bed forms, and grain sorting. To model navigation risk, a morphodynamic model should be able to simulate these phenomenon. Therefore, a 3D model is deemed most appropriate. Unfortunately, a full 3D model demands large amounts of computational time and does not exist for the Rhine yet. A quasi-3D model, a 2D model with parametrised 3D effects, is a good compromise. Thus the morphodynamic model selected is the Delft3D DVR model of the Dutch Rhine, hereafter DVR model. This model also includes graded sediments in the river sections were this is most important.

Due to models being a simplification of reality they always come with uncertainties. Not only the model itself, but also the inputs come with uncertainties. From literature it was found that the three major contributors to uncertainty are discharge variation, hydraulic roughness and bedform prediction and the transport parameters. Uncertainty in discharge was selected because it is inherent uncertainty and therefore will always exist and it has the interesting aspect of time.

8.1.2 Stochastic morphodynamic modelling

'How can uncertainty in discharge be quantified and how does it affect uncertainty in morphodynamic behaviour?'.

For quantification of uncertainties MCS (Monte Carlo Simulations) was chosen, because it keeps the non-linearity of the model in tact and is easy to implement. The concept of MCS is that multiple realizations with the same probability are created by sampling input variables.

The discharge data is resampled using the nearest neighbour resampling method to 100 time series of 20 years as suggested by (Van Vuren; 2005). The discharge data was discretized to levels of 500 m^3/s starting at 750 and with a maximum of 9750 m^3/s. Moreover, the MCS are compared with a deterministic simulation using a hydrograph that is generally used with the DVR model. The DVR hydrograph has one peak per year and is discretized to a small number of statistically important values (1203-8592 m^3/s).

The water depth, which is the most important variable for navigation, depends on bed level, dune height and discharge. The bed level showed in general bed degradation, even on the lower Waal where it is expected to accrete, due tot the tilting of the river. This is possibly caused by the 90.000 m^3/s that is mined downstream from St Andries (rkm 925). Large amounts of erosion occur after Nijmegen (rkm 890) and after rkm 930. The average bed level decrease over the entire study area is 36cm in 15 years with a 90% confidence interval of 16cm. The deterministic simulation generally showed larger amounts of erosion. An explanation for this is that it has a longer initial period, it later shows similar behaviour to the MCS. The longer initial period is most likely due to the lack over variation in discharge and the low discharges in the first 6 years of the hydrograph.

Another issue for navigation is the limitation of water depth because of dune heights. The dune heights found where highest in the middle Waal as expected, but also in the outer bends near Nijmegen and St. Andries large dunes where found. The maximum dune height is about 2.2m metre and the 90% confidence interval increases in downstream direction with a minimum of about 0.5m and a maximum of approx. 1.2m in the low water period.

Combining these results the water depth can be found. The least available depth (LAD) showed a clear relation with the discharge, as expected. The 90% confidence interval of this relation was 0.5m, showing the range of morphological behaviour. In the deterministic simulation low depths were less frequent, partly due to the discretisation of the discharge having a minimum of 1203 m^3/s, while the discharge minimum for the MCS is 750 m^3/s.

The dredging amounts that were found were in the order of 100,000 $m^3/year$ plus sand mining of approx. 90,000 $m^3/year$. The amount of dredging needed decrease in time to a median value of 17,000 m^3 in year 15 and the 90% confidence interval decrease from about 230 to 80 $m^3/year$. The value of 100,000 $m^3/year$ is low compared to measurement data which points to 300,000 $m^3/year$ (Bardoel; 2010), but in the same order of magnitude. The above gives confidence in the model since it simulates the lare phenomenon, however, it can best be used for comparison of cases and not direct calculation of bed levels.

8.1.3 Consequences modelling

How can the economic consequences and risk due to uncertain river behaviour be quantified for navigation?

The economic consequences were divided in two parts: dredging/measure costs and welfare loss. The dredging costs were calculated with a simple formula depending on the distance between the dumped and dredged location (Mosselman et al.; 2007). The welfare loss is the change in economic surplus (benefits of a transaction) due to depth restrictions. The cost increases due to depth restrictions are directly translated to price increase through the load factor ($p_1 = p_0/LF$). Except when load factor is below 0.3, then a maximum price is applied, therefore the producer (shipper) makes a loss. To use this method, an initial load, an initial price and a new price are needed. These were taken from a database created by BVIAS: a navigation network model. For each ship movement (trip) in this database the loss calculation is made by linking the depth to the database in a time step.

Thus the first step is the calculation of the 'restricted' transport costs or price. The fractal price change showed a relation with the minimum depth and thus the discharge. Above an LAD (least available depth) of approximately 4m the price change almost zero ($< 1\%$). At 2.8m the median price increase is 24.4%. The spread in price change per depth is large (up to 35%) due to the differences in time of the trip database.

Table 8.1 shows the results of the loss calculations. The largest contributor is the welfare loss, then the producer loss and last the dredging costs. The deterministic simulation shows much lower values due to the lack of low depths, because of the discretisation of the hydrograph. Since the depth does not show a long term trend in time, neither do the welfare loss and producer costs, however, they are clearly linked to the discharge at a time.

The averaged dredging price calculated with this formula was 3.41€$/m^3$, it is low because the dredged material is often dumped in the same or a nearby rkm. Since the price does not change with time the dredging cost in time are directly related to the dredging volumes, thus decreasing in time and smaller

TABLE 8.1: Total (non-discounted) costs per years per source: expected values and 5th and 95th percentile values.

	MCS		Deterministic		
	Expected	Percentiles	Expected	Percentiles	
Welfare loss	65.3	13.7 - 189.1	34.8	23.0 - 49.9	million €
Producer loss	2.72	0.13 - 11.25	0.45	0.20 - 0.75	million €
Dredging costs	0.20	0.01 - 0.57	0.27	0.04 - 0.53	million €
Total costs	68.3	14.1 - 199.5	35.5	23.4 - 51.1	million €

for MCS than the deterministic simulations. This last difference shows the importance of including uncertainty. Part of the reason for the difference is the hydrograph schematisation, but also extremes are often not included in a deterministic calculation resulting in a less skewed distribution of outcomes.

8.1.4 Risk-based decision making

'How can the risk assessment method support decision makers?'.

Decision makers often work with different alternatives that have to be evaluated. The most used method is a cost-benefit analysis. In this case the costs are the measure costs: dredging and nourishments and the benefits are the decrease in damage or welfare loss for navigation due to low water depths.

The case that was used applied nourishments on the IJssel, Neder-Rijn, Pannerdensch channel, Boven-Rijn and the part of the Waal just past Nijmegen. The measure is applied for mitigation of the bed degradation on the Dutch Rhine. The morphological modelling indeed showed an increase in bed level for the nourished locations and the main trend in the Boven-Rijn was even accretion in the last 5 years. The dredging amounts did not show clear differences. The nourishment case has in general higher depths and an improvement in time therein. The larger depths resulted in the decrease in welfare loss as seen in Table 8.2.

TABLE 8.2: Comparison between the statistical values of the costs of the reference case and the nourishment case divided by cost contributor. In million euro over 15 years.

	Reference case		Nourishment case		Difference	
	Expected	Percentiles	Expected	Percentiles	Expected	Percentiles
Welfare loss	776.6	457.1 - 1146.8	764.1	449.8 - 1132.3	12.5	3.3 - 25.6
Producer loss	31.9	9.2 - 64.6	30.5	9.0 - 60.7	1.48	0.01 - 5.11
Dredging costs	2.46	1.48 - 3.74	2.46	1.48 - 3.70	0	-0.03 - 0.03
Nourishment costs	0	0	12.6	9.9 - 15.3	-12.6	-15.3 - -9.87
Total costs	811	469.1 - 1219.8	809.6	474.6 - 1215.8	1.38	-9.76 - 18.94

Even though the expected difference is positive, there is still a significant chance that the measure makes a loss in 15 years. Generally cases where no profits are made have high discharges because of which the losses are already low and a small increase in depth has less effect. The deterministic simulation resulted in a loss of 26 million euros, due to different morphological behaviour and the discretisation of the hydrograph, this value is not comparable to the MCS.

8.1.5 Extendibility of the risk assessment method

'How can the developed risk assessment and management method be applied in a broader context?'.

The developed method can be applied for multiple uncertainties, however, to keep the same accuracy the amount of simulations that need to be done will quickly increase. Applying the method for multiple river functions is most likely not realistic since for different functions different scale processes are important. For example the river function cables and pipelines requires a more detailed model since small scale processes are of importance. Applying the risk assessment for a different study area is difficult, since of most areas in the world no well maintained or accurate enough models exists. To model morphodynamics well a complex model is needed. Also other phenomenons might play a role such as high flows and river ice.

This method was mainly developed for decision makers, working on large time scales. However, an operational model can for example help shippers in planning trips or give a more accurate measure of how much they should unload. Therefore also short time scale data needs to be available.

In the future possibly other ship types are used and it is difficult to say what the prognosis is for the demand in water transport. If data about these types of questions becomes known it can implemented quite easily in BIVAS.

8.2 Discussion and recommendations

In this the issues with the model and method are discussed and some recommendations are presented.

8.2.1 Risk assessment method

The risk assessment method only looks at uncertainty in discharge and consequences for inland water transport. To get a good overview of the total risk all important river functions and uncertainty variables must be taken into account. Since this might lead to extremely long computational times in case Monte Carlo Simulations with crude sampling is applied to the Delft3D, it is recommended that other models and uncertainty propagation methods are investigated. A relatively simple change of the method is applying Latin Hybercube Sampling, which reduces the number of simulation, however, the accuracy is difficult to predict.

The discharge hydrographs were discretized and in case of the DVR hydrograph even reduced to one peak per year. This resulted in an incomplete spectrum of discharges, especially for the DVR hydrograph in which the minimum discharge is 1203 m^3/s. To estimate the importance of applying uncertainty analysis a more realistic deterministic hydrograph, including more variation and a full range of discharges, must be used.

8.2.2 Morphodynamic modelling

The model preformed reasonably well when compared to measurement data from Sieben (2008). The large phenomenons in this data were found in the model. Still a number of problems occurred with the Delft3D model.

The simulations showed some fast initial changes, such as large amounts of erosion near rkm 890 and 935, while these locations were also found by Sieben (2008) in measurements, the erosion was more than to be expected. For rkm 890 an explanation is that there the boundary between two domain is situated. At this boundary the sediment goes from graded to uniform (in the flow direction) and the transport formula is also different. It is recommended that the effects of this boundary on sediment transport are investigated further.

Moreover, initial period are often noticed in models due to a mismatch between bed level and sediment transport, due to which a new equilibrium bed level is created. For comparative studies this is less important, but for direct calculations of bed levels it can result in significant errors. It is recommended that in order to decrease the difference the model is properly calibrated and validated using recent measurement data. Moreover, the calculated reference plane and the known reference plane differed up to 0.5m, this difference might also be decreased by doing so. Also the morphological boundary at the upstream end of the model needs to be investigated again, since recent measurements show that the Boven-Rijn has stabilised, i.e. stopped eroding.

An instability occured at the Neder-Rijn, where a discharge boundary instead of a water level boundary is applied to simulate a number of weirs controlling the section. The discharge distribution is controlled by the bifurcation as well as this boundary, but there is no equilibrium between the two causing an instability at the downstream boundary. The maximum error was approximately 60 m^3/s. It is recommended that this problem is solved before the model is used in the future.

In Delft3D only for discharges between 1250 and 2250 m^3/s dredging occurs, while in reality the range might differ, depending on the contractor. Also other schematisation issues such as where to dump and when dredging occurs are difficult issues, especially due to the performance contracts used not much data exists.

8.2.3 Consequence modelling

The theory of consumer surplus was used to determine welfare loss. An important variable in this calculation is the elasticity of demand. The currently used value was -0.6. Jonkeren (2009) showed that this value has an standard deviation of 0.27. Increasing or decreasing the elasticity of demand with its standard deviation resulted in a difference in loss of 1 to 6%. This is most likely small compared to other uncertainties, such as the use of a constant supply function. Since the loss is calculated per trip, more knowledge is needed on the behaviour of shippers (producers) and consumers in such a situation.

A database of trips was used to obtain the data needed for the welfare loss calculation. The database is based on actual trips for which the costs are calculated using BIVAS. First of all there is no knowledge on how accurate the costs calculation made by BIVAS is, the average price from investigations by Panteia 8.76 €/tonne though the value found from the calculations was 11.37 €/tonne. It is difficult to say how accurate either is since the investigations contain only a small amount of all trips and it cannot be easily traced back where costs in BIVAS came from. Therefore more research on transport prices is needed. Moreover, an assumption of a price ceiling was done to simulate the maximum price a consumer is willing to pay for transport. This resulted in an average maximum price of 38 €/tonne instead of the 103 €/tonne it would have been if no ceiling was applied. According to Havinga (pers. com.) the 'summer' price is 20 €/tonne, this agrees reasonably well with a maximum of 38€/tonne.

The database used is composed of 3 database from BIVAS, but since these database al include signs of insufficient depth it migth not be a good representation of the trends in demand of transport in a year.

8.2.4 Total risk

The expected costs in 15 years are 811 million euro. When comparing the results with two other studies, the results in this study were higher, however, in the same order of magnitude. The two mention studies were of H. Havinga (personal communication), who based his conclusions on Bosschieter (2005) and some rules of thumb from his experience at RWS and Jonkeren et al. (2007) who investigated the price related to water data in time and than used the method of consumer surplus as done in this study.

It is difficult to say how accurate the entire method is, since there are so many uncertainties involved in all steps that cannot be quantified. The method is too time consuming to be a logical choice for an first estimate and to much is uncertain for it to be used as in a proper cost-benefit analysis. It is especially recommended that more research be done into the cost calculation since much is still unknown.

To simplify the morphodynamic part of the method, which is the most time consuming, a resampling of a historic time series of LAD's is a possibility. However, this could not be used for decision making since it has no change in time, but can help obtain an estimate of the expected loss.

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Appendix A

Delft3D DVR Rhine model

A.1 Measures

The Room for the River measures as included in the model (Sloff et al.; 2014).

#	Omschrijving	Maatregel	Riviertak	Begin- rkm	Eind-rkm
1	Rijnwaarden	oevergeul met langsdam en regelwerk Pannerden	Bovenrijn en Pannerdensch Kanaal	862,5	873.6
2	Millingerwaard	aanleg geulen	Waal	867,6	873
3	Bemmelsche Waarden	uiterwaardvergraving	Waal	878,2	881,4
4	Lent	aanleg geul en dijkteruglegging	Waal	881,5	886,6
5	Kribverlaging Midden-Waal	kribverlaging	Waal	886,8	914,7
6	Afferdensche en Deestsche Waarden	aanleg geul en uiterwaardvergraving	Waal	898,5	903,2
7	Langsdammen Tiel	aanleg dammen binnenbocht en kribverlaging buitenbocht	Waal	911,5	921,5
8	Kribverlaging Waal Fort St. Andries	kribverlaging	Waal	914,7	934,2
9	Kribverlaging Beneden-Waal	kribverlaging	Waal	934,3	953,6
10	Munnikenland	aanleg geulen en dijkteruglegging	Waal	947,6	952,6
11	Avelingen	aanleg geul	Boven Merwede	955,8	957,5
12	Noordwaard	aanleg doorstroomgebied	Nieuwe Merwede	963,0	979,7

The Water Framework Directive measures as included in the model (Ottevanger and Giri; 2015).

Measure	Measure abbrev.	River km	Length [m]	Left/right bank	Foreshore	Erosion rate (m3/year)	
						2015-2034	2035-2054
River IJssel							
Spankerense Weilanden	ij_sw_a1	911,8	1360	L	yes	880	587
Gelderse Toren	ij_gt_a1	913,5	760	L	yes	331	220
De Schans	ij_schans_a1	914,9	300	L	yes	200	133
Leuvenheim	ij_leuv_a1	915,7	530	L	yes	289	193
Bronkhorsterwaarden	ij_bhw_a1	918,8	690	R	yes	411	274
Stroomkanaal	ij_sk_a1	922,1	210	R	yes	94	63
Zutphen-links	ij_zutl_a1	925,4	2180	L	yes	966	644
Rammelwaard	ij_ram_a1	932,9	320	L	no	23	
Eesterwaard	ij_eest_z1	934,0	750	R	yes	669	446
Ravensweerd	ij_ravw_a1	938,0	140	R	yes	132	88
Bolwerksweiden/Bolwerksplas	ij_bolW_a1	942,2	610 + 140	L	yes	98	65
De Worp	ij_worp_z1	944,6	300	L	no	54	
Ossenwaard	ij_ossw_a1	945,6	1640	L	no	457	
Stobbenweerd 1/2	ij_stobw1_a2	947,7	670	R	yes	196	131
Stobbenweerd 2/2	ij_stobw2_a2	948,7	360	R	yes	239	159
Keizerswaarden	ij_keiz_a1	950,0	650	R	yes	163	108
Slichtenbreesweerd	ij_slich_z1	950,8	340	L	no	136	
Katerstede	ij_kats_a1	954,6	490	L	no	250	
Hengforderwaarden	ij_heng_a1	954,8	680	R	yes	160	107
Wijhe-zuid	ij_wijhZ_a1	964,5	500	R	no	129	
Wijhe-noord	ij_wijhN_a1	965,2	480	R	no	150	
Herxer Uiterwaarden	ij_herx_z1	970,5	770	R	no	225	
Tichelgaten Herxen zuid	ij_tichZ_a1	971,3	460	R	no	91	
Tichelgaten Herxen noord	ij_tichN_a1	971,8	390	R	no	159	
River Waal							
Winssensche Waard	wl_winw_z1	895,2	135	L	no	600	
Ochtendse Buitenpolder	wl_ocht_z1	902,4	190	R	no	800	
Dreumel	wl_heerw_z1	981,9	380	L	no	1800	
Heerewaarden	wl_zuil_z1	922,8	410	L	no	600	
Zuilichem	wl_dreu_z1	943,5	140	R	no	1700	

A.2 Van Rijn formula 1984

The following pages show the modified van Rijn formula (1984) as implemented in the Delft3D DVR model. The pages are a direct copy of appendix A of the report 'Voorspelinstrument duurzame vaarweg: calibration of the multi-domain model' (Yossef et al.; 2008). The user defined parameters are shown in Table A.1.

TABLE A.1: Parameters for the sediment transport formula modified van Rijn (1984).

Parameter	Description	Value	Unit
α_1	coefficient	1.0	-
α_{bed}	calibration coefficient for bed load	0.3	-
α_{sus}	calibration coefficient for suspended load	0.2	-
ϵ_c	bottom roughness height	0.3	m
θ_{cr}	critical mobility factor	0.016	-
	factor reduced formula for all $T >= 3$	1.0	-

A Van Rijn (1984)

The formula of Van Rijn (1984) takes the form:

$$S = S_s + S_b \tag{A.1}$$

where:

$$S_b = \begin{cases} 0.053\sqrt{\Delta\, g\, D_{50}^3}\, D_*^{-0.3} T^{2.1} & \text{for } T < 3.0 \\ 0.1\ \sqrt{\Delta\, g\, D_{50}^3}\, D_*^{-0.3} T^{1.5} & \text{for } T \geq 3.0 \end{cases} \tag{A.2}$$

First the bed-load transport expression will be explained. In Eq. A.2 T is a dimensionless bed shear parameter, written as:

$$T = \frac{\mu_c\, \tau_{bc} - \tau_{bcr}}{\tau_{bcr}} \tag{A.3}$$

It is normalised with the critical bed shear stress according to Shields (τ_{bcr}), the term $\mu_c \tau_{bc}$ is the effective shear stress. The formulas of the shear stresses are:

$$\tau_{bc} = \tfrac{1}{8}\, \rho_w\ f_{cb}\, u^2 \tag{A.4}$$

$$f_{cb} = \frac{0.24}{\left[\log_{10}(12h/\xi_c)\right]^2} \tag{A.5}$$

$$\mu_c = \left(\frac{18\log_{10}(12h/\xi_c)}{C'} \right)^2 \tag{A.6}$$

where $C_{g,90}$ is the grain related Chézy coefficient:

$$C' = 18\log_{10}\left(\frac{12h}{3D_{90}} \right) \tag{A.7}$$

The critical shear stress is written according to Shields:

$$\tau_{bcr} = \rho_w\, \Delta\, g\, D_{50}\theta_{cr} \tag{A.8}$$

in which θ_{cr} is the critical Shields parameter for initiation of motion, which is a function of the dimensionless particle parameter D_*:

$$D_* = D_{50}\left(\frac{\Delta g}{\nu^2} \right)^{\frac{1}{3}} \tag{A.9}$$

The suspended transport formulation reads:

$$S_s = f_{cs}\, u\, h\, C_a \tag{A.10}$$

In which C_a is the reference concentration, u depth averaged velocity, h the water depth and f_{cs} is a shape factor of which only an approximate solution exists:

$$f_{cs} = \begin{cases} f_0(z_c) & \text{if } z_c \neq 1.2 \\ f_1(z_c) & \text{if } z_c = 1.2 \end{cases} \tag{A.11}$$

$$f_0(z_c) = \frac{(\xi_c/h)^{z_c} - (\xi_c/h)^{1.2}}{(1-\xi_c/h)^{z_c}(1.2-z_c)} \tag{A.12}$$

$$f_1(z_c) = \left(\frac{\xi_c/h}{1-\xi_c/h}\right)^{1.2} \log_e(\xi_c/h) \tag{A.13}$$

where ξ_c is the reference level or roughness height (can be interpreted as the bed-load layer thickness) and z_c the suspension number:

$$z_c = \min\left(20, \frac{w_s}{\beta \kappa u_*} + \phi\right) \tag{A.14}$$

$$u_* = u\sqrt{\frac{f_{cb}}{8}} \tag{A.15}$$

$$\beta = \min\left(1.5, 1 + 2\left(\frac{w_s}{u_*}\right)^2\right) \tag{A.16}$$

$$\phi = 2.5\left(\frac{w_s}{u_*}\right)^{0.8}\left(\frac{C_a}{0.65}\right)^{0.4} \tag{A.17}$$

The reference concentration is written as:

$$C_a = 0.015\alpha_1 \frac{d_{50}}{\xi_c} \frac{T^{1.5}}{D_*^{0.3}} \tag{A.18}$$

The following formula specific parameters have to be specified as input to the model.

w_s	the settling velocity of the sediment [m/s]
α_1	coefficient (should be *O(1)*)
ξ_c	reference level (bed load layer thickness) or roughness height [m]
d_{90}	D_{90}-particle diameter [m]

It is recommended to introduce the following changes:

1. Reduce Eq. A.2 to

$$S_b = \alpha_{BED} \cdot 0.1\sqrt{\Delta g D_{50}^3} D_*^{-0.3} T^{1.5} \tag{A.19}$$

with α_{BED} calibration parameter for bed load transport component, and for consistency we use α_{SUS} instead of α_1 as a calibration parameter for suspended load transport component. Both calibration parameters are user specified inputs.

2. Use a variable fall velocity (w_s) that is internally calculated based on the sediment size rather than using a user specified input value.

3. Introduce the possibility to specify a user defined critical Shields parameter θ_{cr}. This option is introduced inline with the experience from modelling the Bovenrijn, where a rather low critical Shields parameter is needed to reproduce its morphological behaviour correctly.

A.3 Sediment transport formula Boven-Rijn and Upper Waal

The formula for the Boven-Rijn and upper Waal is based on a general transport formulation (with the structure of MPM):

$$S = \alpha D_{50}\sqrt{\Delta g D_{50}}\theta^{b}(\mu\theta - \epsilon\theta_{cr})^{c} \tag{A.1}$$

where ϵ the hiding and exposure factor specified for the sediment fraction, D_{50} the median grain diameter, Δ the relative density, C the Chézy coefficient and

$$\theta = (\frac{u}{C})^{2}\frac{1}{\Delta D_{50}} \tag{A.2}$$

in which u the flow velocity (for the user specified parameters see Table A.2).

TABLE A.2: Parameters for the sediment transport formula for the Boven-Rijn and upper Waal.

Parameter	Description	Value
α	calibration coefficient	5
b	power	0.0
c	power	1.5
μ	ripple factor or efficiency factor	0.7
θ_{cr}	critical mobility factor	0.025

A.4 Dune height and roughness predictors

Fredsoe (1982) developed a method to calculate dune height from a transport formula, in this case MPM:

$$h_d = \frac{24}{63}\epsilon H max(1 - \frac{\theta_{cr}}{\mu\theta}, 0) \tag{A.3}$$

where, h_d the dune height, ϵ the multiplication factor, H the water depth, μ the efficiency factor defined as $(C/C_{g,90})^{1.5}$ based on the grain related bed roughness of $3D_{90}$ and θ_{cr} the critical Shields parameter. In the model the critical Shields parameter is 0.047 instead of values from the conventional Shields curve. The multiplication factor ϵ is 0.8. In the model dune length (L_d) is defined by van Rijn (1984): $L_d = 7.3H$. To account for the time aspect of the bedforms a constant relaxation time is introduced ($T_H = 57600min$), while the special aspect (dune migration) is neglected.

Moreover, from the bedforms dimensions the roughness height related to dunes is calculated using Van Rijn (1984c):

$$k_{s,d} = 1.1h_d(1 - e^{25h_d/L_d}) \tag{A.4}$$

A.5 Dredge and dump module

The dredge and dump module assesses and enforces the requirement that the minimum depth of the Waal and Boven-Rijn is ALW -2.8 m for a fairway width of 150 m. These minimum dimensions should hold for at least 95% of the time. Therefore a discharge that is exceeded at least 5% of the time has been determined, at the moment its value is $1020^3/s$. The water level that occurs at this discharge is the definition of the reference plane from which the requirement is measured.

The reference level or ALW is adjusted every 5 years for the new situation, as it is practice as well. The initial reference level is the current reference level know from measurements. The first step of year one is calculation of the reference plane by simulating a discharge of $1020m^3/s$, since this calculated reference plane often differs significantly from the measured one, it is not used in the first 5 years, but the known reference plane is used. After 5 years another calculation of the reference plane is made by Delft3D. The difference between the two calculated planes is added to the original plane to obtain the new reference plane; this is repeated every 5 years. The difference between the measured and calculated reference planes can be explained by the importance of the roughness for low discharges; if the roughness is slightly off this can already have large consequences for the water level.

Dredging only takes place when the discharge of the step is between 1250 and 2250 m3/s, because during high and low discharges dredging is much more difficult. In reality the dredging companies work with a performance contract, where every two weeks the requirements are checked. A problem with the current dredge and dump module is that dredging is done in the same time step as the morphological change occurred. Therefore it is possible that in reality more insufficient depths might occur, due to dredging not being done straight away. However, since insufficient depths occur at low discharges where dredging is not possible, it can possible be neglected. During the sensitivity analysis in chapter 4 this assumption is investigated.

Figure A.1 shows the dredging criteria and enforcement strategy which is applied per river km. An extra 0.5 m is dredged as shown in the figure and the material is dumped in the deeper part of the cross section if it's below 4.0 m. If there is no room available in the current cross-section the material will be dumped in the downstream rkm, the second downstream rkm or one rkm upstream, in that order. In case the material can't be dumped in any of these sections it's taken out of the system.

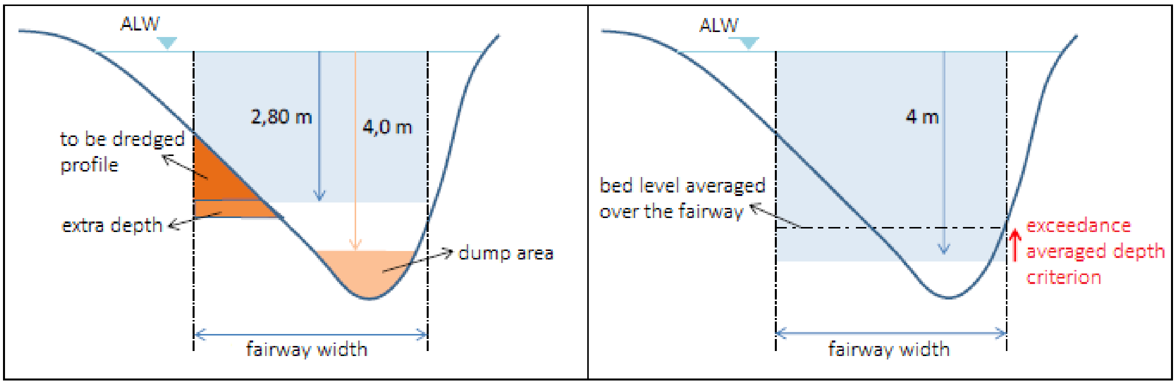

FIGURE A.1: Illustration of the dredge criteria. Left: minimum depth criterion. Right: averaged depth criterion. (Ottevanger and Giri; 2015). Translated from Dutch.

For the determination of the minimum and average depth over the fairway a dune predictor is used. This dune predictor takes into account the flow conditions and natural variability of dune heights. Half of the calculated dune height is added to the bed level. Especially for the Midden-Waal the dune heights are an important factor in dredging amounts.

A.6 Sand mining

In the DVR model sand mining is included per river kilometre and is only done for discharges at the Boven-Rijn of between 1200 and 2250 m^3/s. The allowed volume is 90.000 m^3/y. Delft3D distributes the yearly allowed volume w.r.t the length of the time step. Because dredging does not occur in every time step, using 90.000 m^3/y would result in too little mining.

In the original DVR hydrograph this range of discharges occurred 286 days in the year: 78% (Sloff et al.; 2014). Therefore the amount of dredging that was allowed in the model files is 11,500 $m^3/year$: 0.78*11,500 = 90.000 $m^3/year$. Since not every modelled year has exactly 286 days in the discharge range, in some years the mined volume might be higher or lower than 90.000 $m^3/year$.

A.7 Minimum water depth determination from morphological grid

The width of the fairway in the above analysis is given by the dredge polygons used in the Delft3D DVR systematization. Since the output of the morphological model is given in a grid, the grid points that are included in the analyses do not include the exact fairway boundaries.

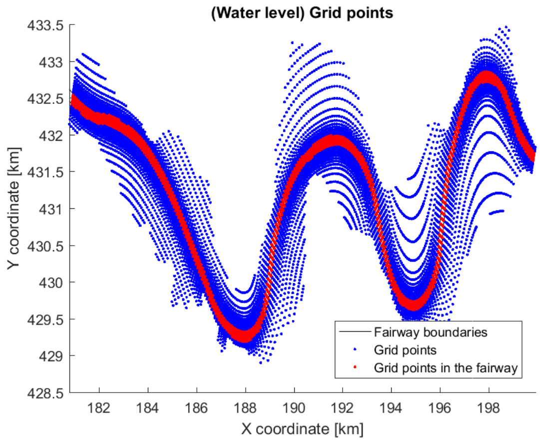

FIGURE A.2: The water level grid points (staggered grid) including the fairway for the Upper-Waal. The pattern is more dense near the fairway and becomes less dense further away.

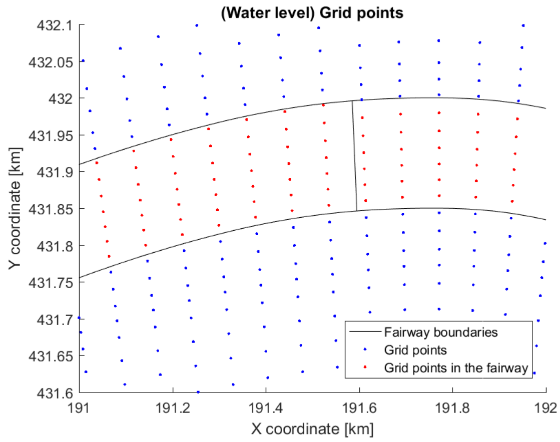

FIGURE A.3: The water level grid points (staggered grid) including the fairway. Zoomed in to show the difference between the location of the grid points and the actual fairway boundaries.

The fairway width is calculated by the coordinates of the most outer points which are still included within the fairway width, since these are also the ones that are used for the minimum depth determination. The error caused by this method might be up to $2\sqrt{\Delta x^2 + \Delta y^2}$, where Δx and Δy the difference between two points in x and y coordinates respectively. The distance between two grid points lies approx. between 20 and 25m, therefore the error can be as large as 50m. Figure A.4 shows the width based in the grid points included and the width based on taking an extra point on both sides of the fairway outside the boundaries. The average difference between taking extra grid points and not doing so is 45m, which is in line with the above argumentation.

TABLE A.3: Statistical information on the two options for fairway width over the river.

	Option 1	Option 2
Average	128.2 m	173.3 m
5th percentile	108.8 m	155.6 m
50th percentile	131.0 m	173.9 m
95th percentile	143.5 m	189.0 m

Moreover, due to the points of minimum depth being most likely near the banks, the determined minimum depths are also influenced by this issue. The actual minimum depth over the fairway most probably is smaller, resulting in an underestimate of the costs. However, in reality when the depth decreases the width fairway is decreased to a minimum of 100m, thus and average width of 128m is quite reasonable and the increased width would give an overestimate of the costs.

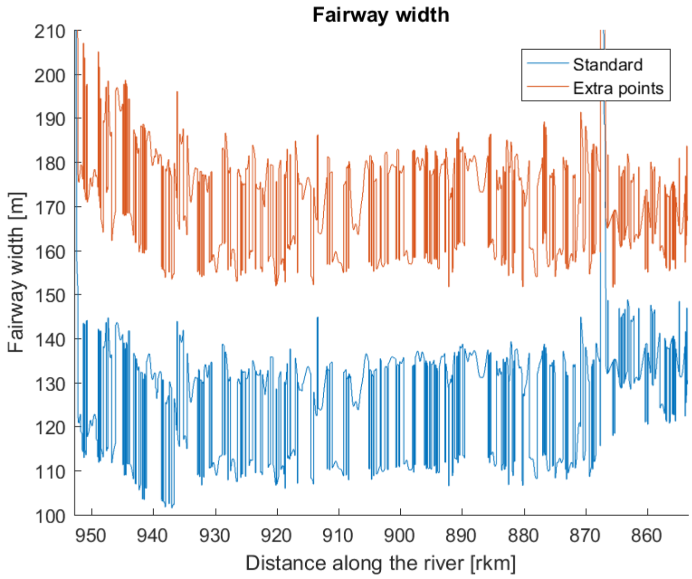

Figure A.4: The width of the fairway for the grid point that were taken into account. Standard: The grid points that are within fairway boundaries. Extra points: on both sides of the fairway an extra point was included that lies outside the boundaries.

Appendix B

Consequence modelling

B.1 The BIVAS model

Network

The network consists of nodes, fairways and structures. It is based on ViN (Vaarwegen in Nederland, Fairways in the Netherlands) and NWB-V (Nationaal Wegen Bestand Vaarwegen). A scenario of the future network (MIRT 2030) based on planned projects is included in the network model, which is not used in this study.

Journey allocation

First the network is filtered to take out the routes a vessel cannot take due to fairway restrictions, such as vessel length, width and height requirements. These restrictions can be relaxed and a fine can be chosen for using fairways with restrictions. There are two options for filtering, namely based on the vessel dimensions from the journey database or on the dimensions of 10%-percentile of the ship class. The second method allows only for slightly larger fairways to be used, since the 10% of the vessels are smaller than these dimensions. These less strict rules are applied to prevent too many infeasible trips due to fairway dimension restrictions.

Travel time

The travel time consists of two elements: 1) travel time on the channel and 2) travel time due to structures. The vessel speed is the minimum of the desirable, theoretical limiting and low-water speed and multiplied with the fairway length gives the travel time. The second part is a bit more complicated since the calculation per type of structure differs and the capacity of the structures and intensity of the fairway plays an important role. For a detailed description the reader is referred to the website of BIVAS (http://bivas.chartasoftware.com/).

B.2 The BIVAS databases

As explained in chapter 5 three trip databases exist: 2011, 2013 and 2014. A database needed to be selected to be used in the method. To prevent water depth restrictions in the year of the database from having influence on the cost calculations it would be optimal to use a database from a year when no depth restrictions occurred Unfortunately such a database does not exist. Therefore a combination of the databases was made where based on the maximum discharge on a day a database was selected for that day. The result of this is a database that does have the demand of goods in it, but is the least affected by restrictions (Figures B.1, B.3 and B.4).

From the initial analysis of the databases it was found that for water depths above 3.8m nearly no restrictions occur and above this value the load factor stays constant. In the new database it can be seen that a depth below 3.8 m only occurs a few times (Figures B.1 and B.3). The load factors decrease with ship size as to be expected, but are almost constant with water depth. Except for CEMT class V, here still a small upward trend with depth can be noticed. The most used ship class is CEMT class V and class I and II are not often used any more.

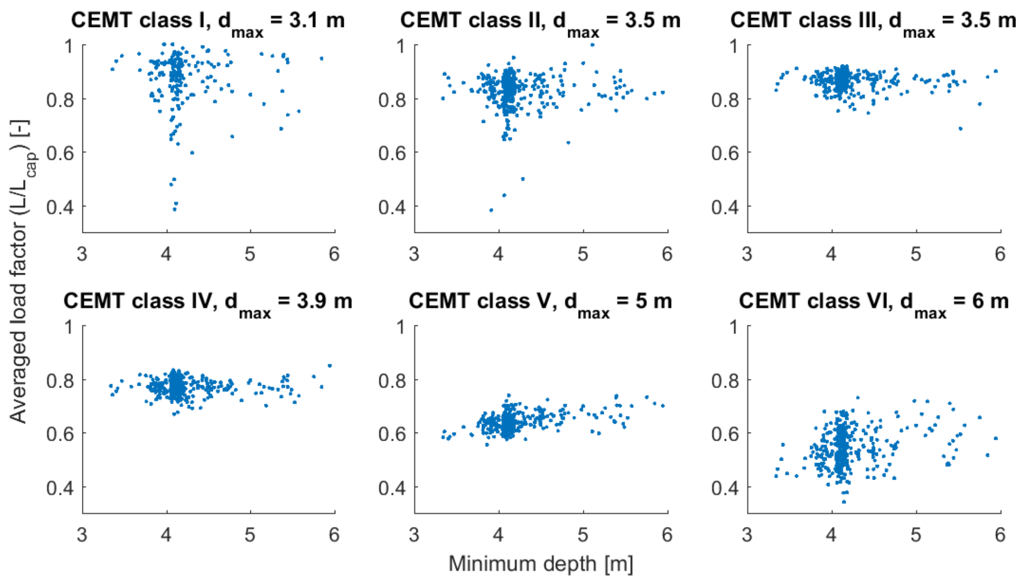

FIGURE B.1: The load factor defined as transported load divided by load capacity plotted against water depth for the CEMT classes for the adjusted database. The water depth was obtained from discharge data in the three years (RWS) and combined with the mean of the relation between discharge and water depth as found from the model

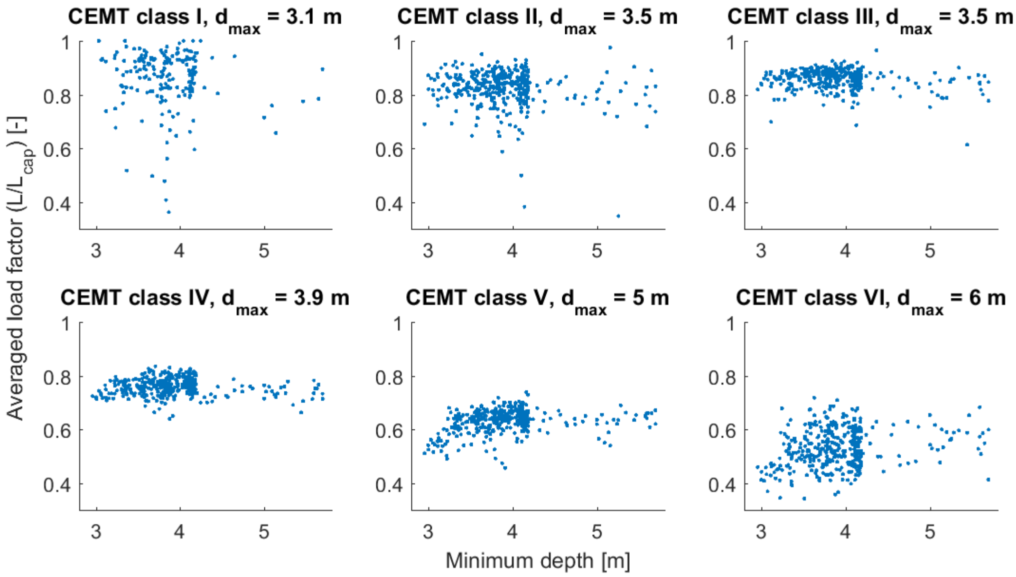

FIGURE B.2: The load factor defined as transported load divided by load capacity plotted against water depth for the CEMT classes for the database of 2013.

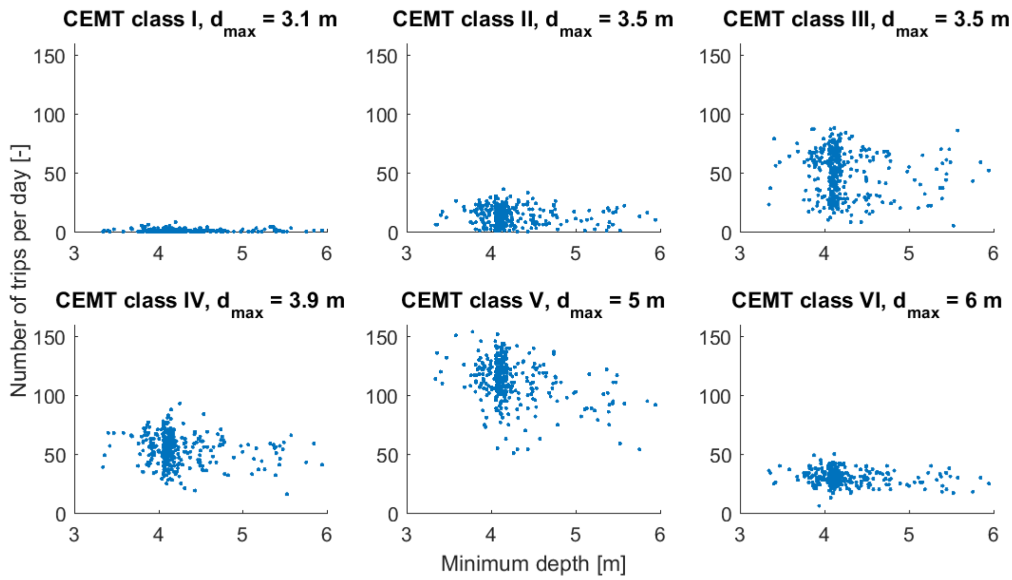

FIGURE B.3: The number of trips per day plotted against water depth for the CEMT classes for the adjusted database. The water depth was obtained from discharge data in the three years (RWS) and combined with the mean of the relation between discharge and water depth as found from the model

A clear trend in demand with time can be noticed, therefore using a constant demand over time as done in the fleet calculations is unrealistic. In Figure B.4 the holidays in the winter: Christmas and new year are shown with a fast decrease in transported load. After the winter break it takes a while to start up again since almost the whole of January is still lower than other months. Moreover, also the summer break can be noticed with a decrease in transported load around July.

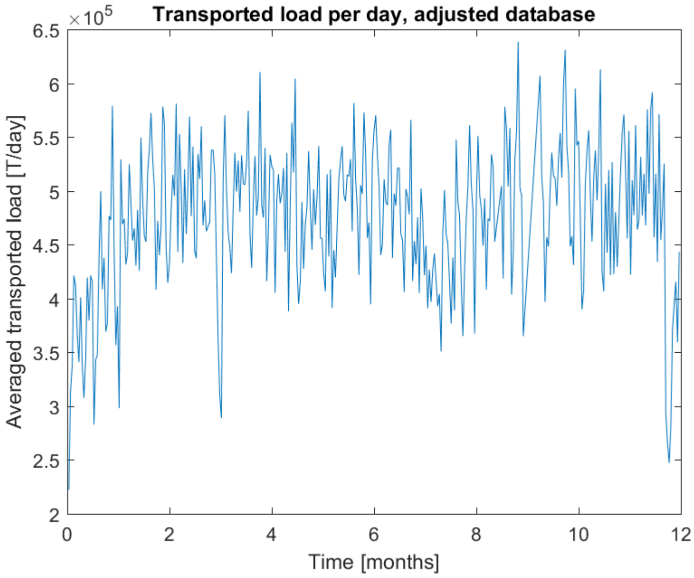

FIGURE B.4: The amount of transported load against time for the adjusted database. All values where the depth was below 3.8m taken out.

B.3 Modal split

The change in modal split is another component in the cost calculation. Krekt et al. (2011) investigated this for the worst case scenario (Wp) of KNMI06 and found that 8% of the inland water transport will be redirected to rail and road transport of which rail transport will take the largest part. Jonkeren et al. (2008) showed for the same climate scenario an loss of tons of 5.4% compared to the reference scenario. For one of the less extreme scenarios it was only 2.3%. Since, the current study does not take into account the climate change, the change in modal split will be much smaller and is therefore neglected.

Appendix C

Adjustments Delft3D DVR model

A number of adjustments have been made to the Delft3D DVR model as used in Ottevanger and Giri (2015). These are explained shortly below.

No morphological change on nr1a domain

As explained in Appendix G it is explained why it was decided to 'turn off' morphological change on the Neder-Rijn. This was applied by setting the layer thickness of the moveable sediment to zero, because it is not possible to turn of bed level change in one domain only. The result of this method is that some erosion occurs at the entrance of the Neder-Rijn.

Initial period

Before calculation morphological behaviour for each discharge level of approx. 10 days an initial period where just the hydraulics are simulated is applied. The user of the model can specify the size of the initial period, in this case 60 min. Applying a larger initial period would not contribute significantly to the decrease in discharge error.

First 5 years

It was decided that, because in the first 5 years of the modelling period the bed level change was significantly large, these years would be taken out of the investigation. Therefore also the nourishments where moved 5 years, as well for the reference case as the nourishment case.

Appendix D

Flood risk assessment

Risk analyses in flood safety is an upcoming subject, especially in the Netherlands. Recently new standards have been developed to include more risk analyses in the decisions on flood safety. Below the Dutch methods for flood risk analyses are explained.

General flood risk approach

The Dutch government uses dike rings to assess the flood safety; dike rings are independent areas surrounded by flood protection structures, so if a dike fails only that dike ring will be flooded. However, the new standards are based on dike sections as well since difference breaches have different consequences and between these scenarios is now distinguished. The standards give a change of flooding per dike section, on which design can be based. The map of the standards looks simple, however, a full analysis of the risks was done before coming to these values.The risk of flooding is defined as the probability of flooding multiplied with the consequences of flooding. The steps of assessing flood risks are explained below.

Probability of flooding

Per dike section the probability of flooding is determined; this can be done by using a fault tree consisting of events and their probability and OR and AND relations. At the bottom of the fault tree for example a hydraulic event such as high water levels or waves with a certain probability are found. One level higher one could find the failure mechanisms; three important ones are overtopping, piping and macro instability. These have certain probabilities for certain hydraulic conditions, which can be expressed in fragility curves (see Figure D.1) .

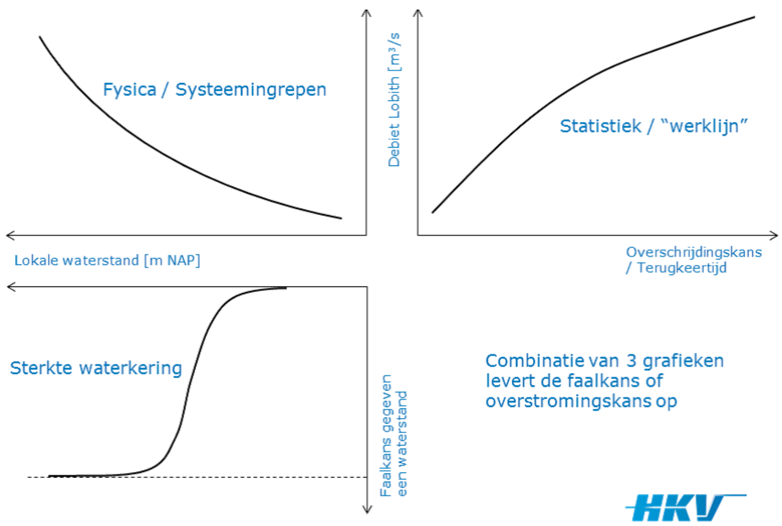

FIGURE D.1: Example of determination of the fragility curve. The two upper curves can be combined to the lower curve. (Saskia van Vuren, 2015)

The failure probability of a dike section can be determined by these fragility curves for the dikes. The fragility curves give a probability of failure per water level (bottom graph). To construct this curve two other relations are needed: the probability of occurrence per discharge (upper right graph) and relationship between discharge and water level (upper left graph). This last relation can be found through models; in these models river morphology is used. The probability of a certain discharge is generally extrapolated from data; for the Rhine approximately 100 years of discharge measurements exist.

Consequences

Consequences in flood risk are generally expressed in scenarios, where for a number of scenarios the consequences are estimated. Since the breach location is of influence on the consequence scenario it was recently decided by the Dutch government to use dike sections instead of rings to determine risks of flooding. The consequences are typically quantified in loss of life or economical values. It would be optimal if these two types of quantities could be compared with the same unit, however, there is a large discussion on how much a life is worth. Therefore, these two quantities are looked at separately in Dutch flood risk analyses.

Risk

The different types of consequences can be recognised in the final risks; the Dutch government distinguishes between economic risks, local individual risks and societal risks. Economic risk is defined as the expected value of the monetary damage per year. The economic risk is generally used to balance the cost of increasing safety and the revenue from decreased damage. The advantage of quantifying economic risks is that it is possible determine an optimum risk level (see next paragraph).

Local individual risk is the probability that a person in a certain location dies that year. In the Netherlands the maximum local individual risk allowed is 10^{-5}. This decision was made based on an earlier decision on the allowable risks for chemical factories. The total risk of loss of life (determined by the government) is 10^{-4} per year in the Netherlands and the decided that the risk of these factories could not contribute to the total risk for more than 1%, hence a risk of 10^{-6}. The reasoning behind the higher risk for floods is that factories are manmade, while floods can be categorised as natural disasters, thus a higher risk could be accepted. Also the costs of a risk of 10-6 per year were simply too large.

The last category is societal risk which is the probability that a certain number of people die per year; this is generally represented in a FN-curve (Frequency of N or more fatalities (F) plotted against number of fatalities (N)).

Figure D.2 shows the three types of risks for Texel. Note that for the local personal risks the number of people living in the area is not taken into account.

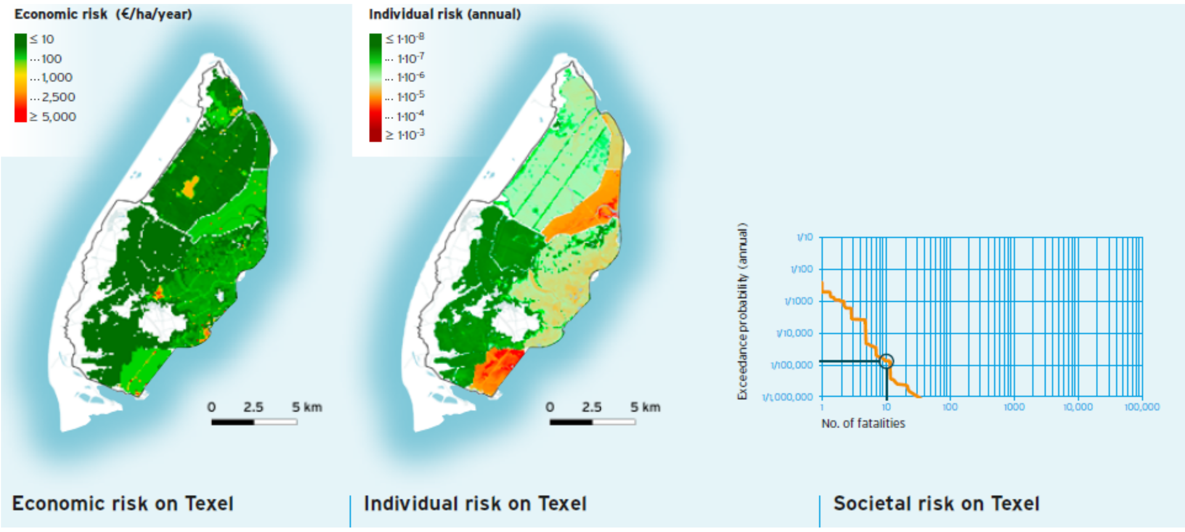

FIGURE D.2: Economic, individual and societal risks for Texel. (VNK, 2014)

Appendix E

High flows and ice

High flows and river ice also form treats to navigation, however, the frequency of occurrence is much lower than for low flows (see Figure E.1).

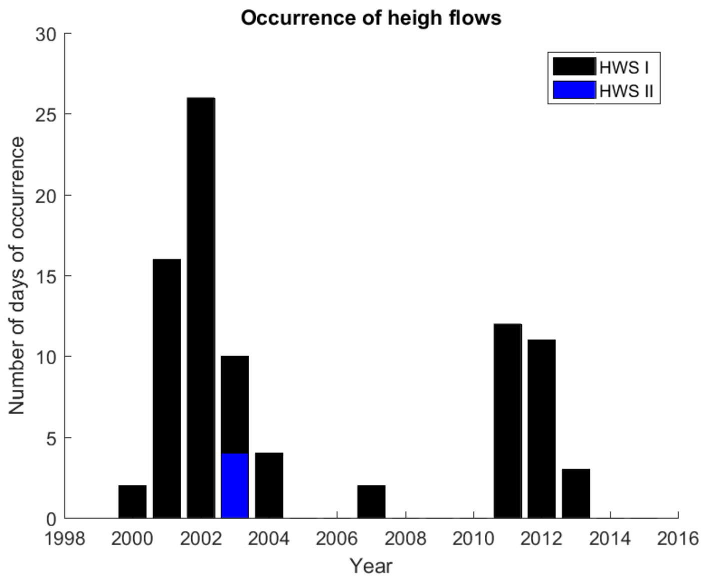

FIGURE E.1: The number of days per year that high flows occur, according to the German criteria (for Emmerich) for the last 15 years.

Due to high flows the air draught of ships is limited, due to for example bridge height. Mainly for container ships needing to unload is a consequence. The clearance above MHW (maatgevende hoogwaterstand) is 9.1m for the Dutch Rhine. In the Netherlands no rules exist for a maximum navigable water level. In Germany two criteria exist named HSW I and HSWII (hchster Schifffahrtswasserstand), the first criterion stops only certain ship types and limits the sailing speed of all ships, the second prohibits all ships from sailing, however, there are some exceptions. Table E.1 [1] gives an impression of the water level and discharges belonging to HSW I and II.

TABLE E.1: Comparision of HSW at Emmerich with normative water levels and discharges at lobith. Corrected for the difference between Emmerich and Lobith. Normative values are based on measurements; data provided by Rijkswaterstaat.

HSW		Normative values		
HSW	Water level [cm]	Return period [years]	Water level [cm]	Discharge [m^3/s]
HSW I	1371	1/1	1380	5800
HSW II	1541	1/10	1585	6800

The higher flow velocities reduce manoeuvrability of vessels and therefore the risk of collision increases. Ship-waves might damage the infrastructure or flood defences. For these reasons the HSW thresholds have been deployed.

A risk method which includes high flows should also account for longer sailing times due to the reduced manoeuvrability as well as cost due to complete stopping or lack of headroom.

[1] https://staticresources.rijkswaterstaat.nl/binaries/Referentiewaarden%20waterstanden_tcm174-326696_tcm21-24223.pdf

Appendix F

Uncertainty in water level

In Warmink et al. (2011) uncertainty parameters were analysed by expert judgment. They concluded that the upstream discharge and the empirical roughness equation of the main channel are the largest source of uncertainties when analysing design water levels. Bathymetry and topography of floodplains were found to contribute the most to the uncertainty of effect studies of changes in the floodplain area (i.e. the cross section). Note that the focus of this study is morphological response, however, in the model used effects on water depths due to other processes are also included.

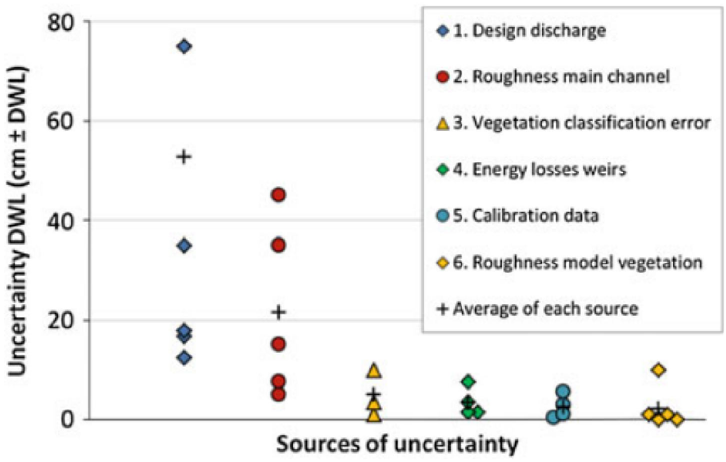

Figure F.1: Results of expert judgment of the uncertainty in design water level for different uncertainty sources. (Warmink et al.; 2011)

Horritt (2006) found that the spatial variation in roughness is a significant contribution parameter to uncertainties. Warmink et al. (2013) investigated the uncertainty in design water levels due to variable roughness parameterisations; they showed that for the river Waal the 95% confidence interval of the water level was 68cm (possible errors in the model were not taken into account, therefore this should be treated as a maximum). Three sources of uncertainty were investigated: bed form roughness of the main channel, classifications errors in flood plain vegetation and the choice of roughness parameterisation. Of these 3 sources the classification errors gave the largest uncertainties and the choice of parameterisation was least significant. An earlier study of Warmink, Booij, et al. (2013) showed that the 95% confidence interval for bed form roughness is a water level difference of 53cm, the given cause for this is the lack of knowledge about the process of bed form evolution. Pappenberger et al. (2008) also found that channel roughness was the most important source of uncertainty when applying their model to the river Alzette in Luxemburg.

Appendix G

Instability of domain nr1a (Neder-Rijn)

The boundary conditions that are applied to this model are a discharge at the upstream end of the Boven-Rijn and water levels at the Merwedes and Ketelmeer. To control the discharge-distribution in the Neder-Rijn weirs have been built. The weirs are not included in the schematisation, but instead a discharge boundary condition is applied to the nr1a domain. During initialisation[1] of the model it was found that the nr1a domain boundary causes instabilities. The instabilities manifest as an incorrect total discharge through the boundary and unrealistic water levels and velocities (see Figures G.1 and G.2).

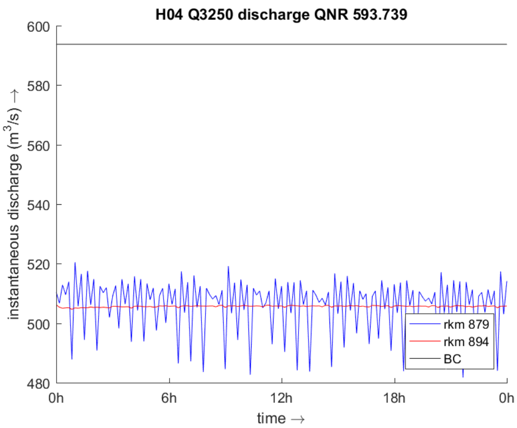

FIGURE G.1: Discharge in time for the upstream (rkm 879) and downstream end (rkm 894) and the boundary condition of the Neder-Rijn schematisation. Given for a constant discharge at Lobith of 3250 m^3/s.

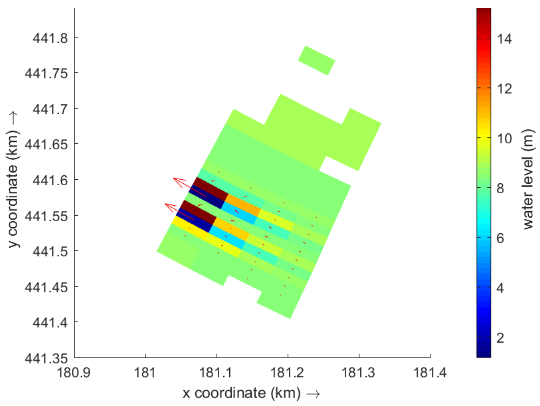

FIGURE G.2: The water level (colours) and flow velocities (arrows) for the last time step of Figure G.1. Given for a constant discharge at Lobith of 3250 m^3/s.

[1]Running the model for the different discharge levels without morphological change

While a large number of tests to improve the situation were done, within the available time it was not possible to completely solve the problem. Decreasing the time step during initialisation reduced the error significantly. Unfortunately it is not feasible to also apply a smaller time step during morphological simulations.

Error in used simulations

Figure G.3 shows the size of the error, while a significant amount of the time it is near zero, there are also peaks around 40 and 25. The error increases over time (figure G.4).

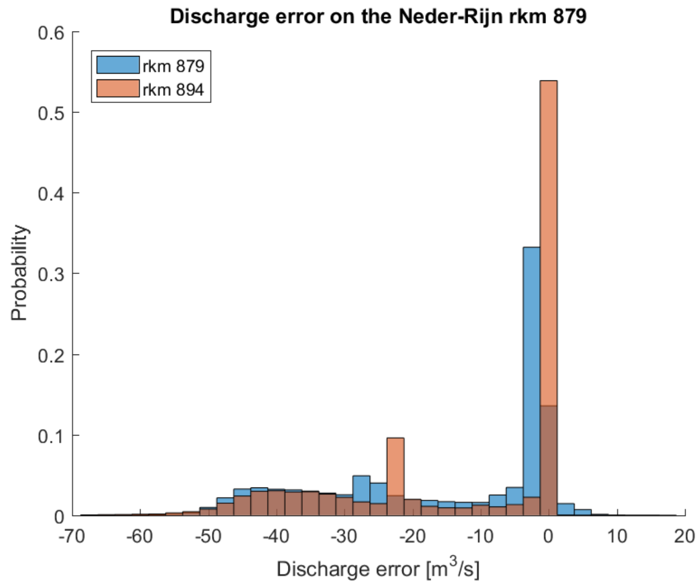

FIGURE G.3: The discharge error near Pannerdersch Kop on the Neder-Rijn.

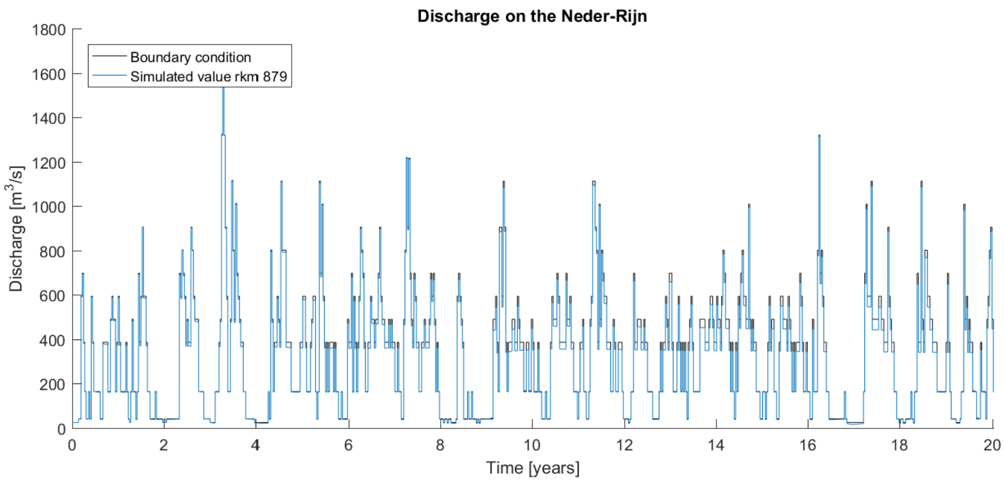

FIGURE G.4: The discharge error near Pannerdersch Kop on the Neder-Rijn for on simulation.

Appendix H

Simple method for consequence modelling

There are two methods for determination of economic loss: the simple and advanced one. The advance method uses a database created by BIVAS including initial price, load and ship dimension which are again used to calculate the load factor and restricted price. The advanced method was explained in the main text of this thesis. Since this method is quite time consuming it is compared with a more simple method, which might be more feasible for some initial studies.

The simple method is based on the work of Bosschieter (2005). It uses information about the active fleet, historic price data and a ship return period as input for the welfare loss determination based on the concept of economic surplus. The return period is a value for how often a ship passes through the study area. The inputs needed for the welfare loss calculation are:

Initial transported load	q_0
Initial price per tonne	p_0
Restricted price	p_1
Elasticity of demand	ϵ

In the following sections it is explained how these values are obtained in the simple method.

H.0.1 Input - Initial load

To obtain the initial transported load the fleet and a return period of ships is used. The fleet information is obtained from the IVR Schepen Informatie Systeem. The return period of ships was estimated to be 5 days by Bosschieter (2005), but this study contains a smaller study area. Moreover, data is available on how often ships pass the study area (BIVAS database). The BIVAS database contains 95,231 trips and the IVR database contains 4,429 ships thus the yearly return period is 21.5. The return period is applied as a factor of 0.6 (21.5/36) per decade. The costs in a decade for al ships are multiplied with this factor.

H.0.2 Input - Initial price

An estimate of the 'unrestricted' price (for adequate depth) is made from price data provided by Panteia (Figure H.1). It is found that for a discharge of 1800 m^3/s at Lobith the median price no longer decreases with increasing discharge. The median price for which this happens is €5.74/tonne and the bandwidth size (difference between 95th and 5th percentile) is approx. €12.1/tonne. The large spread is explained by differences in ship types, travelled distance, transported load etc.. Linear fitting of the unrestricted price data [1] results in a price of 6.70€/tonne in January 2003 and 8.76 €/tonne in January 2017: an increase of almost 31%. The large difference between the earlier named median unrestricted price is explained by some very large outliers in price (in the order of hundreds of euros per tonne).

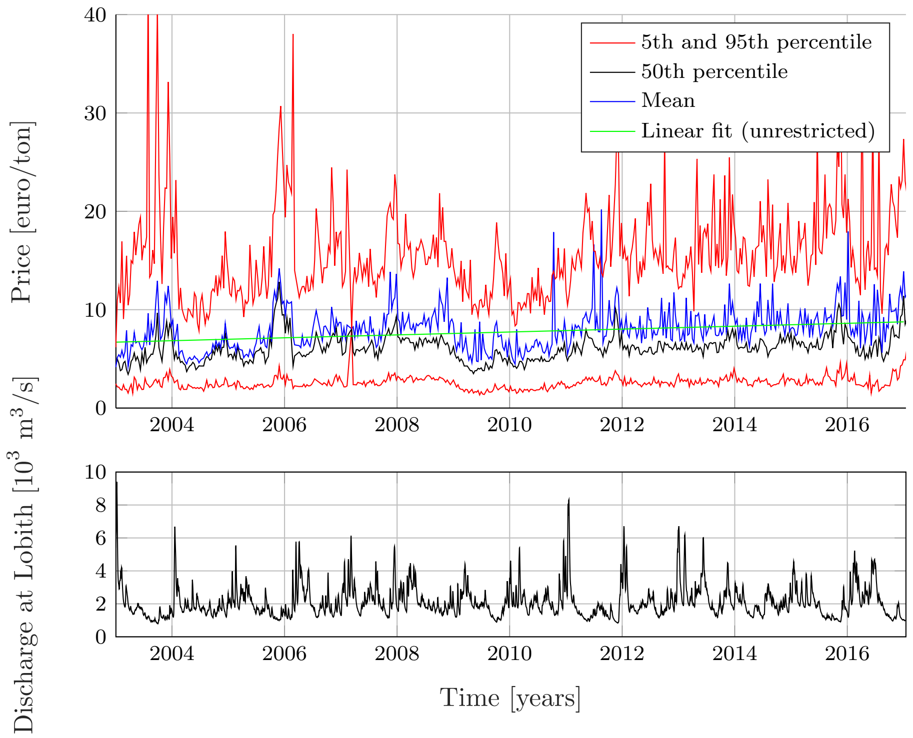

FIGURE H.1: Price data provided by Panteia compared with the discharge at Lobith.

[1] Days were the discharge was below 2000 m^3/s taken out of the data set.

H.0.3 Input - Restricted price

For the calculation of the restricted price the same method is used as described in chapter 6. The price ceiling is also set for a load factor of 0.3: 29.2 €/tonne. The estimate of the fixed costs are taken from the BIVAS database where 51% of the initial costs were fixed.

H.0.4 Results

Figure H.2 shows the probability distribution of the total discounted costs in 15 years. It is significantly less skewed than for the advanced method.

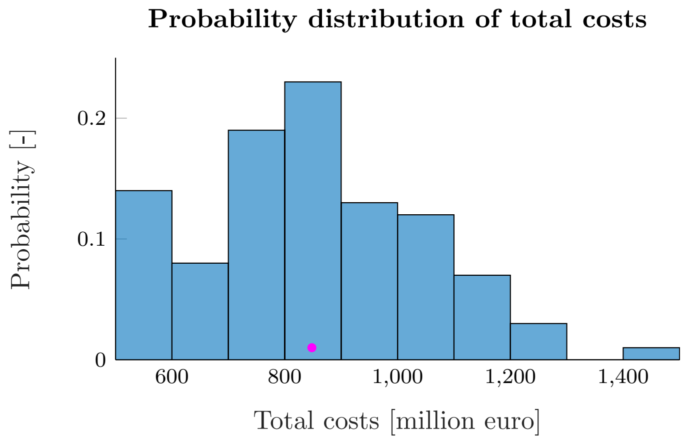

Figure H.2: Probability distribution of the total discounted costs in 15 years for the simple method. In magenta the expected value.

The results divided per source are given in Table H.1, the dredging costs are the same for both methods and are therefore left out of the table. The total costs are lower for the simple method and also the spread is much smaller. There are a couple of explanations, but it is not possible to trace it back. The explanations are: one initial price, not timing issue as with the BIVAS database, the return period is used as a factor and the estimate for the fixed costs.

Table H.1: Discounted costs in 15 years (excl. Dredging costs). The expected values and the 5th and 95 percentiles.

	Simple method		Advanced method		
	Expected	Percentiles	Expected	Percentiles	
Welfare loss	776.6	457.1 - 1146.8	690.7	437.6 - 964.0	million €
Producer loss	31.94	9.15 - 64.62	3.73	2.04 - 5.57	million €
Total costs	808.5	467.1 - 1217.8	694.4	439.7 - 969.5	million €