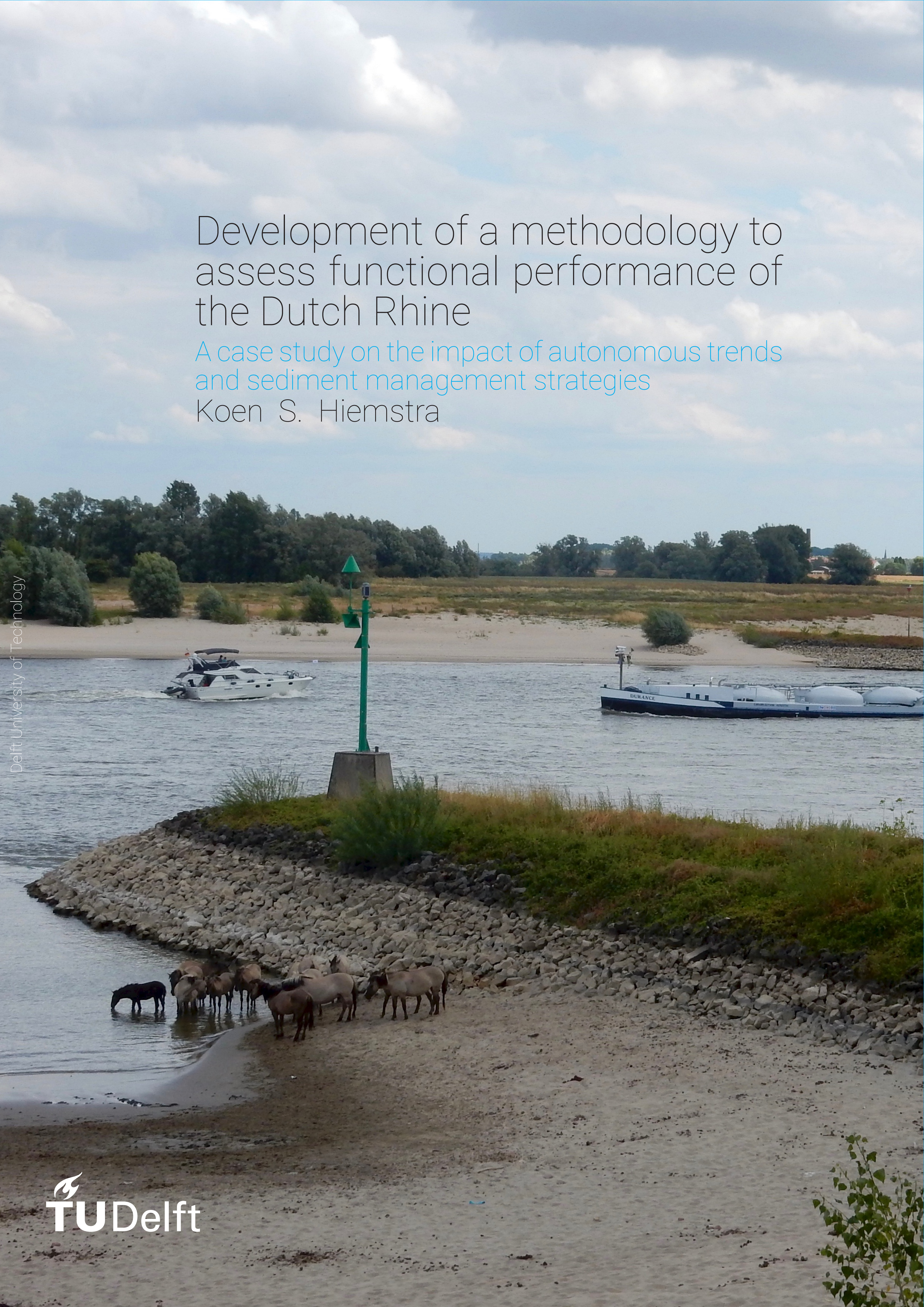
Development of a methodology to assess functional performance of the Dutch Rhine

A case study on the impact of autonomous trends and sediment management strategies

Koen S. Hiemstra

Delft University of Technology



Development of a methodology to assess functional performance of the Dutch Rhine

A case study on the impact of autonomous trends and sediment management strategies

by

Koen S. Hiemstra

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Thesis committee:	Prof. dr. ir. Matthijs Kok	Delft University of Technology
	Ir. Richard Jorissen	Delft University of Technology
	Dr. ir. Saskia van Vuren	Delft University of Technology & Rijkswaterstaat
	Ir. Frederik Vinke	Delft University of Technology

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Cover image: Groyne field of the Waal at Ooij. Picture made by Koen Hiemstra.

TU Delft Delft University of Technology

Rijkswaterstaat
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Preface

This graduation thesis is my final work, to complete the master Hydraulic Engineering at Delft University of Technology. The research has been carried out in close collaboration with Rijkswaterstaat. As the research has been setup very broadly, I was able to learn about many aspects of the Dutch river management. I have become fascinated about the value a river can give in both economic, societal and ecological manner and how depended we (the Netherlands) have become to the Rhine and all its functions. This thesis has encouraged my enthusiasm about rivers and to cycle along them, as I have cycled along all Dutch Rhine branches during my thesis period. I am very thankful for the extremely dry summer of 2018, that indicated the impact of climate change and bed degradation to a wider public. It enabled me to explain my thesis topic in an easier way and people were more aware of the importance of a research like this. Untill the end of my thesis I was hoping for extreme (flood) discharges, as this would have shown both sides of the spectrum in which river functions operate.

As the research has been conducted in strong collaboration with the Programme Integrated River Management (IRM) of Rijkswaterstaat, it gave me the opportunity to interact with those involved and to work together on interesting projects that will improve integrated river management in the near future. Furthermore, it has put me in the position to present my research at the group of experts of the ENW (Expertise Network Flood Protection). I am very grateful for these experiences, as I consider them as important experiences for my career.

I am very thankful to all people who helped me during this thesis by making time for meetings or providing me with relevant reports. I would like to name a few people whom were closely involved in this thesis subject and contributed to the final result of this thesis. First of all, many thanks to my daily supervisor Saskia van Vuren, for the supervision throughout the thesis period. I am really grateful for the chances she gave me during the thesis period (IRM meetings and ENW) and for connecting me to all those people in the large organization of Rijkswaterstaat. I also want to thank her for her valuable input during our meetings, it really helped me during moments I was lost in my thesis. Furthermore, I would like to thank Richard Jorissen for his energetic brainstorm-sessions and outside the box thinking. I would like to thank Frederik Vinke for the pleasant discussions at the TU Delft. As my thesis topic shows a lot of similarities with his PhD work, it was really helpful to exchange thoughts. Finally, I would like to thank Matthijs Kok, for his adequate comments from another perspective and for putting me in touch with Saskia. I always enjoyed our thesis committee meetings, and I wish to thank all members of this committee, thank you!

Furthermore, I would like to thank my close family, friends and girlfriend for the support during the final stage of my study. Specially, thanks to my parents which I have bothered for reading and commenting on my report. This really improved my writing skills. I am looking forward to the next step in my life after finishing my master thesis.

Koen S. Hiemstra
Rotterdam, December 2018

Abstract

Introduction

The summer and autumn of 2018 showed the negative effects of both low-flow conditions and bed degradation over the last century on the river functions of the Dutch Rhine. These resulted in record-breaking water levels, extreme low navigation depth and subsequently nautical problems. The Rhine's long-term bed degradation is the response to river training of the last centuries focused on improvement of navigation and flood protection. Over the past hundred years the river bed of the Upper Dutch Rhine branches degraded 1 to 1.5 m, while a current trend of 1 to 2 cm per year is observed (Blom, 2016). The ongoing bed degradation is problematic since it induces (i) a reduction of navigation depth due to the presence of non-erodible layers, (ii) lowering of ground water levels and dehydration of nature, (iii) less interaction between floodplains and main channel, (iv) lowering of coverage rates of infrastructure (e.g. cables in subsoil, bridges and groynes) and (v) a gradual shift in discharge distribution at the bifurcation points. As climate change will increase the inter-annual variability of the Rhine's discharge pattern, low-flow conditions are likely to occur more often, reinforcing the abovementioned impacts on nature and navigation (Sperna Weiland et al., 2015). Rijkswaterstaat is preparing for an integrated approach to mitigate the impact of the bed degradation on the river functions. Sediment management is considered as a sustainable way to counteract the bed degradation. Nourishment partly restores the deficit of sediment, while it also (temporarily) elevates the river bed increasing water levels.

As the impact of climate change and bed degradation is strongly connected, an integrated solution has to be found to guarantee a future multi-functional river system. To justify an investment in large scale nourishments, the impact of a nourishment on the river functions has to be compared with the result of a reference situation without nourishments. Also the cost-effectiveness of nourishments counteracting the bed degradation should be assessed. This study aims to develop a methodology that evaluates the future performance of rivers functions accounting for the autonomous trends (bed level and climate changes) and provides insights in the functional performance and cost effectiveness of river interventions. By means of a simplified 1D hydrodynamic model, the impact of these trends and the effectiveness of nourishment strategies on the performance of three functions (navigability, nature and flood protection) has been assessed. This was done for a river section of the Waal.

Methodology

This methodology consists of a two main blocks: (i) a model describing river conditions incorporating the autonomous trends and (ii) a model that quantifies the functional performance based on river conditions. Figure 1 illustrates the conceptual model of the methodology including a feedback loop between the functional performance and river intervention measures to enable the assessment of measures on the performance of the river system. The future hydrodynamic conditions is analysed by applying an 1D semi-analytical model that describes the system behaviour based on simplified equations. Bed trends are imposed on the river profile with a fixed rate. The bed erosion rate was varied per river section depending on observed trends. In addition, two extra scenarios for bed degradation rate were considered: a minimum (stabilization) and a maximum (doubled observed trends). This enables the assessment of uncertainty in the bed trends. Daily discharge records are imposed at the inflow boundary. Discharge time series adapted by Mens and Kramer (2016) are used to account for future climate scenarios (($W_{H,dry}$ (dry) and G_L (wet), KNMI'14). The climate scenarios have been linearly interpolated between the present and 2050 to define a bandwidth accounting for climate change uncertainty.

The assessment of the river's performance follows from sub-models per function. For the assessment of the **navigability** we used the model of Jonkeren, O. (2009) relating navigation depth restrictions with the welfare loss for navigation. This model requires the navigation depth as input in order to determine the average loading factor of the fleet composition. To obtain the navigation depth and the corresponding loading factor, the water level information was combined with the width-averaged bed level that was corrected for transverse slope effects in river bends and bed forms. In addition, the agreed low water level (ALW) is obtained by

simulating the future agreed low discharge for two climate scenarios. A navigation channel of 2.80 m deep by 150 m wide should be guaranteed. If these dimensions are not met, dredging is required. Dredging is only undertaken in alluvial parts of the river bed. The river function **nature** is assessed by analysing the inundation frequency of ‘objects’ contributing to nature, such as side channels or floodplains. For the ’performance of nature’ the amount and frequency of inundation is important since it says something about (i) the interaction between the river channel and the tranquil water bodies in the floodplain, and (ii) dehydration. The impact on nature has not been translated in economic values within this thesis. Finally, the impact of autonomous processes and nourishment strategies on **flood levels** during design discharge conditions has been assessed. We did not account for a potential change in discharge distribution. As an increase in flood levels requires heightening of the flood defence, the impact on dike heightening costs has been assessed.

Results

In a river system without intervention all three functions are affected by autonomous bed level degradation and climate change. Navigation depths drop. During ALW conditions, the water depth is assumed to be most critical at the fixed layer of Nijmegen and responds almost one-to-one to the bed degradation. Due to bed degradation, the depth requirements during ALW already are not met in 2040 without accounting for climate change. When considering the most extreme bed degradation scenario in combination with a dry clime change scenario this could be even the fact already in 2024. The autonomous changes also reduce the loading capacity of vessels with subsequent impact on the transportation costs. As water levels drop, the inundation frequency of side channels decreases from 300 days per year (start condition) up to 260 days per year in 2050. The combined effect of $W_{H,dry}$ and a doubling of the bed degradation rate results in an inundation frequency of only 150 days per year in 2050. Next, to the average impact on navigation and nature, the statistical character of dryer and wetter years has been evaluated, indicating the impact of autonomous trends on the extremes. Considering flood protection it is shown that future flood levels during design conditions drop with a rate of 3 mm per year due to bed degradation (11 cm in 2050), whereas more extreme flood discharges in climate scenarios result in an increase in flood levels with approx. 25-30 cm in 2050.

The impact of two nourishment strategies has been assessed: a strategy restoring and maintaining the bed level to its state in 2010 and in 1997. The navigability and the inundation frequency of floodplains and side channels improve for each strategy. The flood levels increase. An indicative cost-benefit analysis shows that the costs of nourishing and flood level compensation are lower than the future navigational losses due to reduced loading capacity. This implies that a nourishment strategy could be considered cost-effective.

Conclusions

Preliminary results show that the approach is a promising way to obtain insight in (i) the impact of bed level degradation and climate change on the river’s performance, and (ii) the effectiveness of nourishment strategies. It is recommended to extent the methodology to the remaining part of the Rhine system, verify the results with more complex models and also evaluate other measures than nourishments. The results reveal the importance of an accurate prediction of the bed degradation.

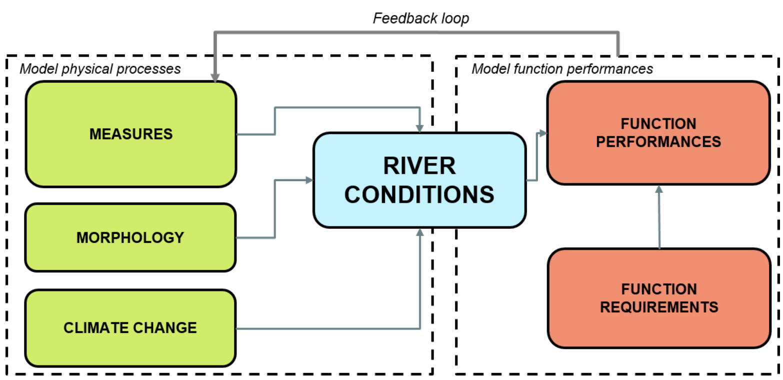

Figure 1: Overview of the conceptual model of the developed methodology

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List of Symbols

Symbol	Description	[Units]
A	cross sectional area	m^2
B	width river section	m
b	coefficient of non-linearity of the sediment formula	-
B_N	normal width river cross-section	m
B_S	safety distance from the river bank	m
Ch	Chezy coefficient representing hydraulic roughness	m$^{1/3}$/s
C_L	load capacity vessel	ton
C	current costs	€
C_d	discounted costs	€
c	velocity of propagation of disturbances in the longitudinal bed profile	m/s
c_H	velocity of propagation of dune height variations	m/s
d	water depth	m
D_{50}	median grain size of the bed material	m
d_c	critical water depth	m
d_{cor}	transverse slope bed correction	m
d_e	extra depth on top of dredging depth (0.3)	m
D_f	fully loaded vessel draught	m
d_{min}	minimum depth along river cross-section	m
d_n	normal flow water depth	m
Fr	Froude number	-
g	gravitational acceleration	m/s^2
h	water level	m + NAP
h_n	normal water level	m + NAP
H	dune height	m
H_e	equilibrium dune height	m
I	costs of dike heightening per kilometer	€/km
i	bed slope	-
i_t	transverse slope	-
k	keel clearance (0.3)	m
L	length of river section	m
$L_{1/2}$	half length of backwatercurve	m
L_D	dune length	m
n	Manning's coefficient representing the hydraulic roughness	s/m$^{1/3}$

$p0$	unrestricted price of transported goods	€/ton
$p1$	restricted price of transported goods	€/ton
Q	discharge	m3/s
$q0$	unrestricted quantity of transported goods	ton
Q_f	discharge through floodplains	m^3/s
Q_m	discharge through main channel	m^3/s
Q_T	total discharge in river section	m^3/s
R	hydraulic radius	m
r	discount rate	%
R_b	radius of curvature	m
S	sediment transport capacity	m^3/s
s	distance	m
t	time	s
T_H	time scale op bedform adaptation	s
u	flow velocity	m/s
u	height of dike elevation	m
V_D	dredging volume	m^3
z_b	bottom level	m + NAP
z_{bf}	bed level of floodplain	m + NAP
z_{bm}	bed level of main channel	m + NAP
α	angle of internal friction after dredging	-
ϵ	elasticity of demand	-
κ	Von Karmann coefficient (0.4)	m/s -
θ	Shields parameter	-
θ_{cr}	critical Shield's parameter	–

1 Introduction

1.1. Background

According to river engineers the primary function of a river is the conveyance of water, ice and sediment (Janssen et al., 1979). Since the dawn of civilization rivers became more and more important serving different other functions. The Dutch responsible river management authority, Rijkswaterstaat (Directorate-General for Infrastructure and Water Management) is responsible for the design, construction, management and maintenance of the main waterway network and water systems in the Netherlands. As a result their work includes the supervision and control of the rivers Rhine and Meuse and their various functions, and as such Rijkswaterstaat's core business is defined as responsibility for flood risk management, sufficient water supply, clean and healthy water, smooth and safe transport by water and a sustainable living environment. Different user functions can be defined, which basically are the societal and economic uses of the river. For some of these user functions Rijkswaterstaat is responsible by law (e.g. nature and drinking water) and for others Rijkswaterstaat can be considered as the host providing the permits (e.g. fishery and cables and pipelines) (Rijkswaterstaat, 2015).

After Rijkswaterstaat's major river management projects over the last twenty years of Room for the River (RfR, Maaswerken (Meuse Works) and projects related to the EU Water Framework Directive (WFD), the Netherlands is now preparing for the next river management challenge of a future proof river system complying with multiple functions of the river. The summer and autumn of 2018 illustrated the impact of low-flow through the Dutch Rhine inducing problems considering navigation, salt intrusion and dehydration of the riverine ecosystem and agriculture. Autonomous developments in the river system will most likely induce more pressure on the various river functions, requiring a new river management approach.

Current river management challenges

This study focusses on the upper Dutch Rhine, that flows from Germany into the Netherlands at Lobith and almost directly bifurcates into the Waal, IJssel and the Lower-Rhine. Since the onset of modern civilization man-kind does influence river systems to enable living in deltas. In the 19th century Rijkswaterstaat proposed a unified normalisation to reduce flooding and improve navigability in periods of low flow (Bosch and van der Ham, 1998). These so-called normalisation measures resulted in a considerable loss of surface area of the river Rhine. Hence, the measures induced by the normalisation significantly affected the present river system resulting in improved flood protection and navigation. However, the normalisation also triggered negative changes, since bottom and water levels are slowly changing (Sieben, 2009). Due to these normalisation measures taken in the last two centuries, a reduced sediment supply from Germany and coarsening of the sediment composition the river system is aiming for a milder slope following the principles of Janssen et al. (1979), resulting in ongoing bed erosion (hereafter reffered as degradation) in the Upper Dutch Rhine. As a result of the degrading bed, summer water levels dropped about 1.5 m in the last hundred years at Lobith as well as at the Waal and 1 m in the IJssel. Studies showed that the river bed of the upper Dutch Rhine is degrading at a rate of 1 to 3 cm per year (Sieben, 2009, e.g.). Another ongoing process is the slow elevation of the floodplains due to sedimentation during inundation, which subsequently results in less interaction between the main channel and the floodplains since they grow apart from each other. As investigated by Ministry of

Infrastructure and Water Management (2018), the bed degradation leads to conflicts with several functions at locations of bed degradation, such as

- **A reduced water depth for navigation**. At non-erodible layers (e.g. structures or rock protection in river bed) the bed does not degrade resulting in doorsteps of the river bed and a reduced water depth for navigation.
- **Insufficient coverage of cables and pipelines**. Due to bed erosion the coverage becomes insufficient (less than 1.5 m).
- **Insecure fresh water regulation**. The bed degradation might affect the ability to regulate the discharge distribution at the bifurcation points resulting in an undesired discharge distribution over the Rhine branches. Furthermore, lowering water levels requires adaptation of the inlets that provide fresh water from the river to the hinterland.
- **Nature and agriculture are negatively affected** due to dehydration of floodplains and reduced flow through side channels as the result of lowering water levels.
- **Uncertain effects on flood safety** - initially reduced water levels result in increased flood safety since water levels drop. However, it could also affect the discharge distribution at the bifurcation points resulting in increased water levels in certain Rhine branches. Furthermore, degradation of the bed could damage flood protection structures.

Next to these morphological trends, climate change will most probably change the incoming discharge statistics. Based on the newly developed KNMI'14 climate scenarios, Sperna Weiland et al. (2015) predicted that both the Rhine and the Meuse river will likely experience an increasing inter-annual variability with more frequent extreme flow events (both low-flow and flood discharges) in 2050, which will likely increase the problems induced by the bed degradation considering low-flow conditions.

To ensure flood protection and fresh water supply with ongoing climate change, a Delta Commission was formed in 2007 by the Ministry of Infrastructure and Water Management . This commission advised to prepare major measures in the fluvial and coastal zones and to make the legal base for flood safety and fresh water supply stricter (Veerman, 2008). This resulted in a long-term Delta Programme, which consists of different plans to ensure a safe and robust delta in 2050. For the Rhine this includes the strategy of an interplay between dike reinforcement and measures aiming at an increase of the flood conveyance capacity (Kuijken, 2017). The Delta Programme also included the implementation of new risk based flood defence standards, which are by law required to be complied by 2050. As a result of the new flood defence standards the Flood Protection Programme (HWBP) initiated over 500 km dike reinforcement for the first tranche till 2023.

Future approach

The developments mentioned in the previous section require an integrated approach, which serves all river functions together. The droughts of the summer and autumn of 2018, with record-breaking low water levels, show the need for an integrated approach since both riverine nature and the river transport sector experienced substantial problems resulting from the drought. In earlier river management programmes, the focus was on local projects and where possible connection with other functions was made. Considering the future challenges, there is a need for a close collaboration between the HWBP (dike reinforcements and possible adaptation of the river bed), measures related to serve all national functions (e.g. flood protection, navigation and water quality) and the regional ambitions (e.g. economy and recreation). If measures related to HWBP and the other future challenges are assessed in isolation, opportunities for interactions and synergy could be missed or unwanted side effects may occur due to unintended, adverse interactions.

Currently Rijkswaterstaat is working on another river management approach, namely the Programme Integral River Management (IRM) (Kuijken, 2018). This approach envisions a future-proof, well-functioning river system that is not based on standalone measures, but is characterized by an integrated approach taking the several river functions in account (Van Vuren, 2018, e.g.). In the Delta Programme of 2019 the Minister of Infrastructure and Water Management has reserved a budget of 375 million euro for this IRM (Kuijken, 2018). Creating more room for the river will be part of the programme since redesign of the river geometry can serve multiple functions:

- By means of increasing the flood conveyance, flood levels are lowered which reduces the probability of flooding and/or can be used as compensation for other flood level increasing activities, such as nourishments or dike reinforcement in the river domain.
- Redesign of the river geometry can also mitigate the discussed bed trends by reducing the sediment discharge capacity of the river.

It is expected that sediment management will play a role in mitigating the problems of the bed degradation. A pilot nourishment has been placed at Lobith and different research programs are running analysing river processes to provide better insights for possible measures. At first, IRM will focus on the current assignments related to the river functions and will strive for synergies in the execution, while an integrated solution will be developed to mitigate the impact of processes impairing river functions.

1.2. Problem definition

As discussed in the introduction Rijkswaterstaat is working on the Programme Integrated River Management (IRM), which has been incorporated in the Delta Programme of 2019. As the major river manager authority, Rijkswaterstaat is responsible for all measures in the river system, which are related to the bottom, river bed geometry, roughness, and in the end the water level. All measures have to fit into a management strategy, which implies a sustainable river that properly serves all its social and economic functions. To evaluate the impact of different measures targeting the river system, an evaluation methodology is required that assesses the way the adapted river system serves its various functions now and in the future. As these processes affect the river system over time, the future performance of river functions will be affected as well. Rijkswaterstaat has been analysing the impact of the bed degradation on the river functions for many years (Verweij, 2016; Ministry of Infrastructure and Water Management, 2018, e.g.). However, simple methods to incorporate the findings of these extensive studies into a function performance evaluation has been lacking. A simple tool could also provide insights in the effect of measures, such as redesign of the river channel or sediment management.

Quantitative information on how river functions perform will enable IRM to assess the value of the different measures for the river functions compared to a do-nothing scenario. Since some of the river processes might have a larger impact on river functions than others, the relative importance of river processes could provide valuable insights. As we know bed degradation or climate change are uncertain processes, which again results in uncertain outcomes for the performance of river functions. The band-width in river function performance obtained from uncertain river processes might influence decision-making within IRM. The evaluation methodology should be able to assess both the proportional impact of river processes and the impact of uncertainty on the performance of river functions, providing valuable information for assessment of integrated river management.

1.3. Objectives

The aim of the research presented in this thesis is to develop a method that can provide insights in the future performance of a river system focused on various river functions. This method should enable a river manager to assess the impact of various river management measures and strategies in an integrated way. To provide useful insights for decision-making, the relationship between river processes and the performance of river functions needs to be analysed in a quantitative fashion. The research objective of this thesis is therefore defined as:

To develop a methodology that evaluates the impact of autonomous river processes on the future performance of river functions and provides insights into the assessment of (integrated) river measures.

This research objective is approached by addressing the following research questions:

1. What are the autonomous developments in the Rhine and how are these affecting the river functions?
2. Which river conditions are relevant for the assessment of the performance of a river function?
3. How can the performance of river functions be quantified and provide useful information for an assessment of integrated river management?
4. Which aspects of the assessment process are important for decision-making?

1.4. Methodology

To achieve these objectives, a generic methodology will be developed and applied to a part of the Waal (subbranch of the Dutch Rhine). The methodology should relate the river processes with the function performances in a quantitative way. This thesis combines two research fields, namely the physical behaviour of the river and the quantification of the river functions. Both research fields are captured by a conceptual model by means of:

1. A numerical model to predict river conditions
2. Different methods to quantify the performances of river functions.

To understand both research fields, an extensive analysis will provide the background information for the development of a methodology. The framework of the methodology has to link the river processes with river conditions and subsequently with the river function. Different tools will be developed to quantify the performance of the river functions. By means of application of the framework to the future rivers system without interventions, the impact of river processes on river functions can be studied. Furthermore, the proportionality and (uncertainty) bandwidth of the impact of river processes on river functions can be evaluated. The framework will also be applied on different sediment strategies, which will be tried to assess by means of a cost-benefit based on the river performance and the costs of the strategies.

Limitations to the scope of the study

The focus of the research will be on the process of assessing integrated river management measures based on performances of river functions. Based on feasibility considerations, the scope is limited and the numerical model simulating river conditions is simplified. Some of the limitations are listed below:

1. Only three river functions will be considered: flood risk, navigation and nature;
2. As described in the Introduction different measures are possible to mitigate the impact of bed degradation. However, within this thesis only sediment management will be studied.
3. Despite the fact that we are dealing with morphological challenges, no morphological model will be used due to its complexity. A simple hydrodynamic one-dimensional model will be used for the simulation of the river conditions, which is forced with an degrading bed by a fixed autonomous bed rate. To account for two-dimensional shallowness corrections are applied for bedforms and transverse slope in river bends.

1.5. Thesis outline

The section provides an overview of the structure of this report, which is composed of seven chapters. Chapter 2 contains the theoretical background to set-up the methodology of the assessment process in Chapter 3. Chapters 4 through 6 discuss the application of the assessment process, while chapter 7 summarizes the conclusions and recommendations. The outline of the chapter can be summarized as follows:

- In **Chapter 2** the Dutch Rhine system is introduced as the study area, introducing the relevant autonomous developments. An analysis in the processes behind the bed degradation and floodplain sedimentation is performed, and the state-of-the-art KNMI'14 climate scenarios are discussed. Next, the Rhine's functions are described and if possible linked with the autonomous development. Within this chapter, the theoretical background for the research questions is provided.

- **Chapter 3** elaborates on the case study conducted in this research. Within this chapter the Waal river section is defined and the various sediment management strategies are described.

- Subsequently, **Chapter 4** describes the methodology that will predict and assess the future performance of river functions. By means of choosing an appropriate hydraulic indicator, research question 2 is answered. The methodology touches upon two research fields, the description of the river system behaviour and the function performance assessment. This chapter will describe tools and methods that simulate the physical processes and to quantify function performances.

- **Chapter 5** discusses the application of the methods to a future river system without stabilization, revealing the impact of processes on river functioning. Also a financial analysis will be performed to analyse the financial impact of no interventions in the period 2018-2050. Next, various sediment management strategies will be assessed based on a cost-benefit analysis and various methods to quantify the functional performance. By means of the application of the methodology research question one, three and four are addressed.

- Finally, the conclusions and recommendations are given in **Chapter 6**.

- The appendices (A, B, C, D, F, G, H, I, J and K) are attached behind the bibliography and contain background information and additional plots and figures.

2

The Dutch Rhine river system and its functions

2.1. Introduction

The objective of this research is to relate the autonomous processes to the performance of various river functions. This chapter will elaborate on the theoretical background of the river processes and the impact on river functions. As the Rhine river system is affected by various autonomous trends, these trends are identified and discussed. River works might have fixed the course of the river by stabilizing the banks, but the river floor is in constant movement (morphology). After flood discharges a different river bed is observed as after low-flow periods. This chapter will discuss the different morphodynamical processes and will elaborate on the relevant ones for this study. Furthermore, this chapter elaborates on the consequences of climate changes for the non-tidal part of the Rhine. Finally, the functions of the Dutch Rhine system are discussed and the relationship with hydraulic properties is elaborated. The findings of this chapter will be used as input for the assessment framework discussed in Chapter 4. This chapter addresses the following research question:

What are the autonomous developments in the Rhine and how are these affecting the river functions?

Before addressing the processes and river functions, the Rhine river system has to be introduced. The Rhine is a large river in Western Europe and has a total length of 1320 km. It springs in the Swiss Alps as a snowmelt-fed mountain river and eventually flows as a rain- and snowmelt-fed lowland river in the North Sea. Over the course of centuries the Dutch have regulated their rivers for inland navigation and flood management. Due to these large scale river training works (Appendix A.1) the Rhine has become the present day river (Figure 2.1): fixed river banks, non-permeable groynes, single main channel intensively used for navigation, low levees ('summer dikes') to protect the floodplains from frequent flooding, flat open floodplains and high dikes acting as primary flood defence.

These works allowed the Rhine river to become an important transportation corridor. The Rhine is the most intensively used inland waterway of Europe serving as major corridor for the transportation of goods from the North Sea to the Western Europe hinterland and vice versa. Hence, industries were established along the river banks and the area around the Rhine became densely populated requiring high safety against flooding. Despite the river training works recent flood events in 1993 and 1995 raised again attention for flood protection and initiated large-scale dike construction (Deltaplan Large Rivers) and river works in the Netherlands (RfR program).

The Dutch Rhine can be subdivided into six branches: Upper-Rhine (Bovenrijn), Waal, Pannerdensch Kanaal, IJssel, Lower-Rhine (Nederrijn) and Lek (see Figure 2.1). Almost directly crossing the Dutch-German border the Rhine bifurcates at the Pannerdensche Kop, where after the second bifurcation follows the IJsselkop. Due to measures taken in the past, the bifurcation points are more or less stable with discharge distribution of 1/3 (Pannerdensch Kanaal), 2/3 (Waal), 1/9 (IJssel) and 2/9 (Lower-Rhine/Lek). During low-flow the discharge distribution changes due to the presence of the weirs in the Lower-Rhine. From those bifurcation points the

Figure 2.1: Dutch Rhine branches. Reproduced from Van Vuren et al. (2015)

branches flow through the Dutch delta into the North Sea (or the IJssel first in the IJssel Lake).

2.2. River morphology

Erosion and sedimentation processes induces changes in river planforms and cross-sectional shape, this phenomenon is called river morphology. The dynamic interaction between water and loose-sediment motion triggers these processes. Changing river processes result in an immediate hydrodynamic response and in a delayed morphodynamical response. These morphological changes take place at different time and length scales for which Vriend (1999) introduced a qualitative scale cascade that is generally applicable. Table 2.1 reviews considered scales within the Dutch Rhine. Within this section only the macro scale will be discussed due to its effect on the long-term river conditions. The trends observed in the macro-scale are the longitudinal (width-averaged) bed degradation in the river channel and the sedimentation in the floodplain.

Scale level	Morphological phenomena
micro	- bedforms (e.g. ripples or dunes) - vertical segregation of sediment fractions
meso	alternate bar formation and cross-sectional profile evolution such as: - transverse slope development and pointbar/pool combinations in bends - local scour - bank erosion
macro	- longitudinal profile evolution - channel planform evolution such as meandering or braiding

Table 2.1: Morphological processes in the Rhine distinguished by scale (Van Vuren, 2005).

2.2.1. Degradation of the river channel bed

On the basis of daily records of water levels and annual sounding of bed levels, changes in the Niederrhein and the Dutch Rhine branches are reviewed in Figure 2.2 and 2.3. As a result of an eroding bed, water lev-

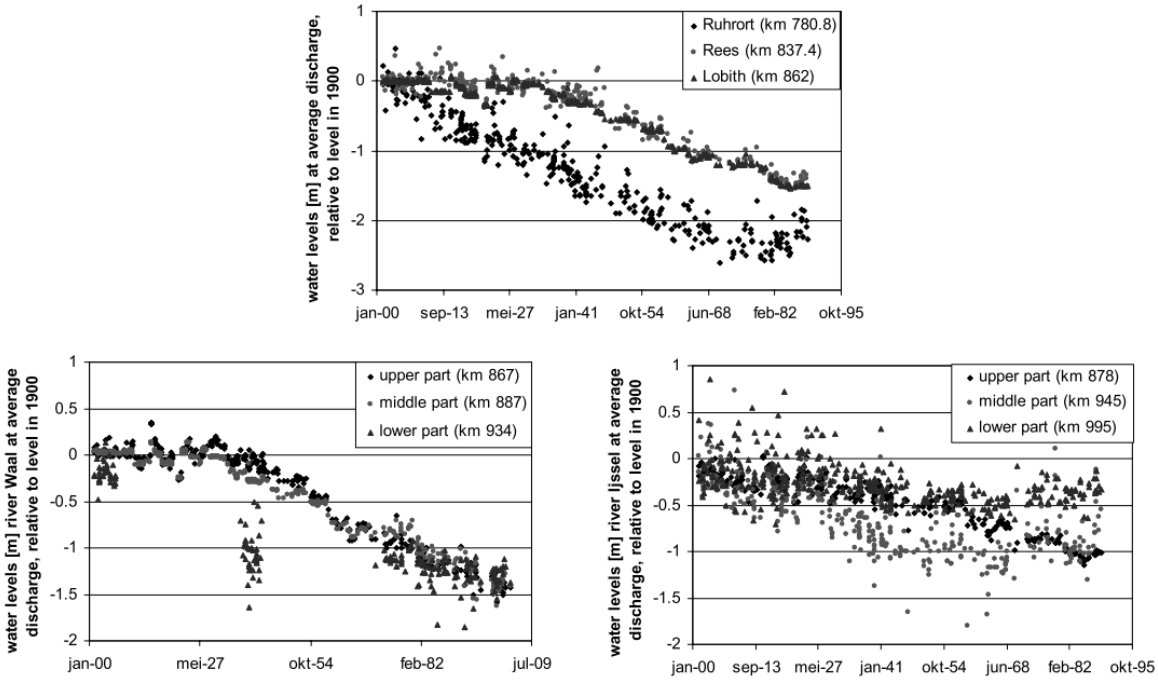


Figure 2.2: Water levels at Upper-Waal discharge of 2200 m^3/s relative to level in 1900, in Niederrhein, Bovenrijn, Waal and IJssel. Reproduced from Sieben (2009).

els dropped about 1.5 m in the last hundred years at Lobith as well as at the Waal and 1 m in the IJssel as (Figure 2.2). In Figure 2.3 the bed level changes are observed along the Upper-Rhine and the Waal. The bed degradation also results in a reduced water depth at certain locations due to an uneven degradation rate. For example at entrances of locks and sites of bed protections, the bed is non-erodible and therefore it does not degrade. Due to the backwater effect shallowness appears at the downstream edge of the bed protection. As already discussed in the introduction, the bed degradation results in many unwanted effects such as limited navigable depth, coverage of cables and pipelines and floodplains dehydrate.

It is proposed by Sieben (2009) that human interventions to the river bed have influenced morphology, which results in the present bed level changes in the main channel. A review of human interventions is discussed in Appendix A.1. To interpret the trends in bed level changes of Figure 2.3 due the normalisation measures, the changes in terms of sediment transport are analysed. With the theoretical impact of the large-scale reduction in width as a results of the normalisation, the sediment capacity of the Rhine branches increased with the proportionality (Janssen et al., 1979)

$$\frac{S_{new}}{S_{old}} \approx (\frac{B_{new}}{B_{old}})^{1-b/3} \tag{2.1}$$

with S [m^3/s] the average sediment transport capacity, with the subscripts *old* and *new* referring to the situation before and after the normalisation (Sieben, 2009). The coefficient b [-] is the coefficient of nonlinearity of the sediment formula , with b=5 conforming to Engelund and Hansen (1967). The degree of sedimentation or erosion depends on the proportion of sediment supply and sediment transport capacity, as erosion will take place when the sediment transport capacity is larger than the sediment supply. Considering sediment supply it is uncertain whether it has changed over the years, because little to no measurements of sediment transport are available (Frings et al., 2014b). However, there are reasons to assume that the sediment availability and has changed. As Frings et al. (2014a) states human interventions like nourishments, dam regulation and mining activities are likely to contribute to a lower sediment availability in the Rhine.

To stabilize the ongoing bed level degradation, the Germans started sediment feeding in the Niederrhein around 1980 (Appendix C). The German nourishments not only compensated the bed erosion, but also coars-

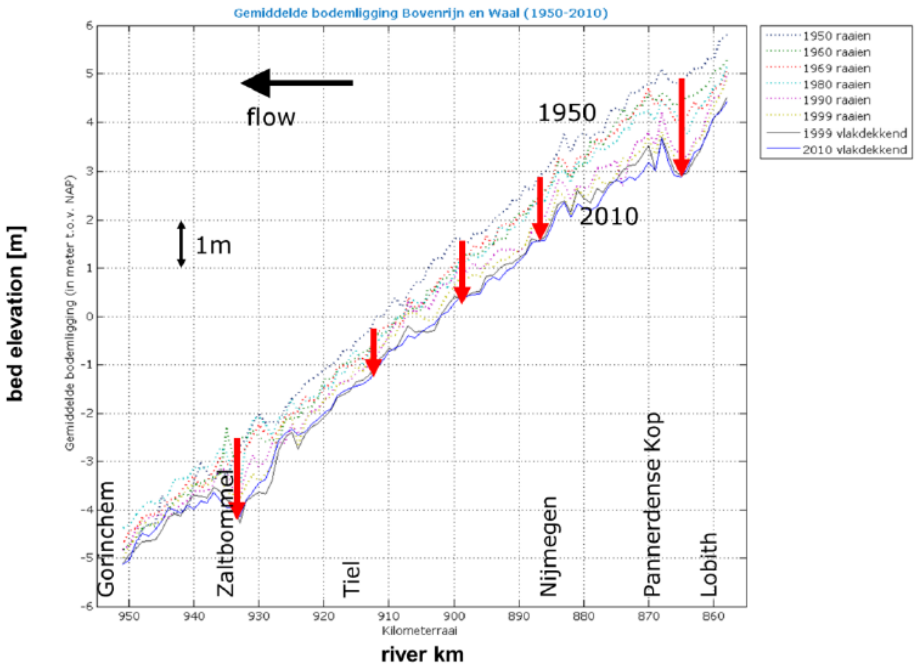


Figure 2.3: Bed degradation in the Upper Rhine and Waal over a period of 60 years based on data of Rijkswaterstaat Nederland-Oost. The red arrows show the development of bottlenecks due to non-uniform degradation. From 1999 bed elevation was recorded through multibeam surveys ('vlakdekkend'); previously single beam surveys were conducted ('raaien'). Reproduced from Blom (2016)

ened the river bed protecting the underlying sediments from being eroded (Frings et al., 2014b). Blom (2016) revealed also two other reasons for the coarsening, namely the bed degradation results in (i) coarsening of the river bed as fine sediments are more mobile and the coarser particles remain on the the river bed and (ii) the river bed is degrading into coarser Pleistocene fluvial deposits. This coarsening of sediment supply in the German and Upper-Rhine is a concern, as it affects downstream degradation rates. Due to the finer sediments downstream of Germany and the Upper-Rhine, a gradient in mobility remains, with a subsequent gradient in transport capacity and an anticipated further degradation (Sieben, 2009). As the river bed of the Upper-Rhine becomes coarser, this effect could also be observed in the downstream Rhine reaches in the future.

The increased sediment transport capacity and the change in sediment supply (composition), the river is adjusting to a new equilibrium. According to Janssen et al. (1979), an increased sediment capacity results in a steeper channel slope in a non-tidal river:

$$\frac{S_{new}}{S_{old}} = \left(\frac{i_{new}}{i_{old}}\right)^{b/3} \tag{2.2}$$

with i_{old} and i_{new} [-] the current and future bed slope. However, at the mouth of the river, the sea level forces the water level affecting the sediment transport capacity. This means the bed is stabilized near the sea, while upstream the bed is degrading. This results in a new state governed by a channel slope that is smaller than the original situation (opposite as expected for a non-tidal river). Hence, a decrease in channel slope is accompanied by a tilting of the river around a downstream hinge point as illustrated in Figure 2.4 (Blom, 2016). Van Vuren (2005) reveals the hinge point is located near Tiel (rkm 915), downstream of which long-term sedimentation takes place. As a result of the tilting river, the bed level change rates in the Waal depend on the locations. As Manning (1891) demonstrates, the equilibrium depth also depends on the bottom slope. Manning's theory reveals an increased depth for a milder slope counteracting the observed trends of degrading water levels and reduced water depth. Eventually an equilibrium state is reached, as Equation 2.2 shows a milder slope will decrease the future sediment transport capacity till it meets the sediment supply again. In Appendix A.2 this phenomenon is explained by means of an estimate for the Waal based on the previous formulas.

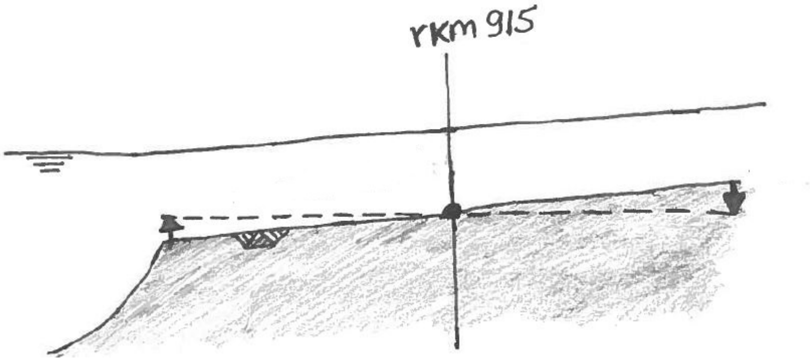


Figure 2.4: Indicative overview of the process of the river aiming for a milder slope with the hinge point at Tiel (rkm 915)

Studies of Sieben (2015), Sieben (2009) and Hendrikssen (2018) show the trend of the bed degradation rates in different periods and sections of the Rhine. However, little is known about the future bed degradation in the Rhine. Studies are conducted to analyse the past bed degradation of the Rhine, as illustrated in Table 2.2. Following Blom (2016) the reduced bed degradation in the Upper-Rhine could be the result of coarsening of the river bed. Such trends could travel downstream and affect future bed degradation rates at different locations. Sloff et al. (2014) analysed the morphological changes in the Rhine incorporating the recent constructed RfR projects by use of Delft3D and predicts that the bed levels in the Upper Waal will continue to degrade at the same rate (1.5 cm) for the following years. However, Sloff et al. (2014) expect an increased bed degradation (approx. 2 cm/yr) downstream of Nijmegen. Regarding the future bed degradation, Sieben et al. (2012) revealed similar findings. However, the future bed developments remain highly uncertain.

Location	Sieben (2009)	Sieben et al. (2015)	Hendrikssen (2018)
Measurement period	1950-2000	2000-2012	2000-2015
Bovenrijn (858-867)	-3.0 cm/year	-1.0 cm/year	-0.4 cm/year
Upper-Waal (867-890)	-3.0 cm/year	-1.5 cm/year	-1.5 cm/year
Middle-Waal (891-915)	-1.0 cm/year	-0.7 cm/year	-0.9 cm/year
Lower-Waal (916-951)	-2.6 cm/year	+0.4 cm/year	+0.4 cm/year

Table 2.2: Development rate of the branch average bed degradation in the main channel. Reproduced from Hendrikssen (2018)

Sediment management as mitigation measure

According to the research project Sustainable Fairway Rhine-delta by Verweij (2016) combining constructive (or hard) measures and sediment management (soft measures) is most promising to mitigate the long-term bed degradation. As stated above (Figure 2.4) the river bed is striving for a milder slope. By either increasing the sediment supply or decreasing the sediment transport capacity a steeper equilibrium slope could be reached, which reduces or even stops the bed degradation. When considering hard measures Verweij (2016) proposes longitudinal dams or side channels both extracting water and sediment from the main channel resulting in a lower sediment transport capacity. However, the research scope is limited to only an assessment of sediment management as a measure. Therefore, the physical processes induced by sediment management that eventually mitigate the impact of bed degradation are discussed in Chapter 3.3.

In general, sediment management can be referred as sediment nourishments in this case. However, the term sediment management is more comprehensive, by also pursuing on the sediment cycle in time. This incorporates the source of the sediment and the frequency of nourishing. A sediment nourishment (Dutch: suppletie) means artificially feeding the river with sediment. Apart from the effect on the bed of the nourishment at the nourishment location itself, it can also affect reaches further downstream and upstream. The sediment can be a source for reaches downstream and it could influence hydraulic conditions upstream, which subsequently could have morphological consequences as well. By increasing the sediment capacity, the river is aiming for a steeper equilibrium slope than the original equilibrium slope, which eventually results in a reduction in bed level degradation. As the migration of sediment is rather slow, it could take years to decades before the downstream reaches are affected. The hydrodynamic response is more obvious since the elevated bed is elevating the water level at locations of the nourishment and reaches upstream. In this way some urgent problems of the bed degradation could be mitigated by the hydrodynamic response. So it could be said that the sediment nourishment solves the bed degradation problems in two ways: at first the hydrodynamic response (i.e. increase of water depth and elevated water level) and secondly a morphodynamic response. The hydrodynamic response is rather straightforward. However, the morphodynamic response is more complex since multiple factors such as the location, frequency, volume, grain size of the nourishment are determining the downstream migration of sediment. Therefore, the design of the nourishment is crucial for the effect of the bed degradation downstream. In Germany sediment management has already been an accepted river management intervention, which is reviewed in Appendix C.

Driessen et al. (2018) is currently developing an instrument to maintain norms for the dynamic behaviour of the river bed, namely Basis Rivierbodemligging. A bandwidth is proposed in between which the river bed is allowed to develop. The bandwidth is proposed to incorporate the effects on river functions, such as navigation, flood protection, ecology and coverage of cables and pipelines. Considering this study, it might be necessary to elevate the bed level to a certain level by means of a sediment nourishment, where after it can dynamically be stabilized by smaller frequent nourishments. However, the instrument is still being developed and the bandwidth has not been developed yet.

2.2.2. Sedimentation in the floodplains

Floodplain deposition is an important process in storage and cycling of sediments, nutrients and contaminants in river basins (Thonon et al., 2007). When the bankful discharge is exceeded, sediment can be transferred from the main channel to the floodplain by different mechanisms. Coarse sediment may be transported by traction as bed load and will be deposited close to the channel. Finer sediments is usually transported in suspension and is deposited further away from the channel. Middelkoop and Asselman (1998) estimated 0.24 Mton for the total accumulation of overbank fines (not including sand sheets) on the entire river Waal floodplain for the high magnitude flood of December 1993, which is 19 percent of the total suspended sediment load transported through the river Waal during the flood. Middelkoop (2002) performed a reconstruction of floodplain sedimentation from heavy metal profiles and found sedimentation rates for different flood plains along the river Waal. Sedimentation decreases with increasing distance from the river channel, but may be higher in local depressions. Despite the spatial variability Middelkoop (2002) showed that over the 100 years, average sedimentation rates of overbank fines on the embanked floodplains typically ranged from 0.2 to about 10 mm/year. Considering our study area Middelkoop (2002) found sedimentation rates for the Klompenerwaard (867 km) between 7.2-11.6 mm/yr and at the floodplains around Slijk-Ewijk (892 km) between 2.5-5.0 mm/yr. Middelkoop (2002) concluded that the highest sedimentation rates along the Waal are present in the low floodplains (>10 mm/yr). Sedimentation behind minor dikes are considerably lower with rates of 1-3 mm/year and for sites on natural levees or close to the river channel the sedimentation rate varies between 3-7 mm/yr.

Location	Sedimentation rate [mm/year]
Floodplains behind minor dikes	1-3
sites on natural levees or close to the river channel	3-7
Low floodplains	>10

Table 2.3: Floodplain sedimentation rates for different types of floodplain based on research of Middelkoop (2002)

2.3. Climate change affecting future discharges

Weather patterns over the world have changed considerably over the past million years. So did the Earth's average surface temperature as well: from 1906 to 2005 it rose by 0.74°C (IPCC, 2007). The Intergovernmental Panel on Climate Change (IPCC) concluded that it is extremely likely that human influence has been the dominant cause of the observed warming since the mid-20th century (Stocker et al., 2015). Climate models predict the global warming is going to continue the coming century. To reduce damage due to the so called global warming, almost all countries agreed that the increase in average global temperature should not exceed 2°C compared to the pre-industrial levels. However, climate change will still have a massive impact on river conditions in the Rhine, such as:

- Sea level rise is likely to elevate water levels near the river mouth. Within this study the effects of sea level rise will be neglected and the focus will be on the changing discharges.
- Changes in precipitation in the catchment area of the Rhine influence the discharges.
- Melting of glaciers influences the amount of snowfed discharge throughout the year.

Sperna Weiland et al. (2015) investigated the potential changes in discharge for the Rhine by simulating discharges at Lobith by means of the hydrological rainfall and run-off models (HBV). Hereto the KNMI'14 and Coupled Model comparison Project (CMIP5) climate scenario sets were down-scaled to the sub-catchments of the hydrological model. To investigate the discharge extremes, the Generator of Rainfall and Discharges Extremes (GRADE) was used by resampling the historical time-series of precipitation and temperature to synthetic time-series of 50.000 years using the KNMI weather generators for the Rhine. Finally, the propagation of the flood wave through the Rhine was simulated by the hydraulic Sobek model.

The KNMI generated the four KNMI'14 climate scenarios, which are scenarios that differ due to a difference in temperature increase and shifts in air currents (Klein Tank et al., 2015). However, it is uncertain which scenario is most likely to occur since it depends on the burden of global warming and the unknown complex process of air currents. The four scenarios are denoted as G (Medium) and W (Warm) and by the subscript l (low) and h (high) indicating the influence of change in air currents, resulting in the scenarios presented in Figure 2.5. Finally, Sperna Weiland et al. (2015) included a fifth alternative scenario representing the potential of relatively strong drying in summer over the Rhine basin as a result of the CMIP5 model runs denoted as $W_{H,dry}$ (Lenderink and Beersma, 2015). Considering the flood discharges, Sperna Weiland et al. (2015) also accounted for the effect of upstream flooding. The upstream flooding causes dampening of the flood waves, resulting in lower peak discharges.Klein Tank et al. (2015) simulated synthetic discharge series for the five scenarios, which envisage an increase in winter discharge and a decrease in late summer and autumn discharge increasing the inter-annual variability compared to the reference climate as can be seen in Figure 2.5.

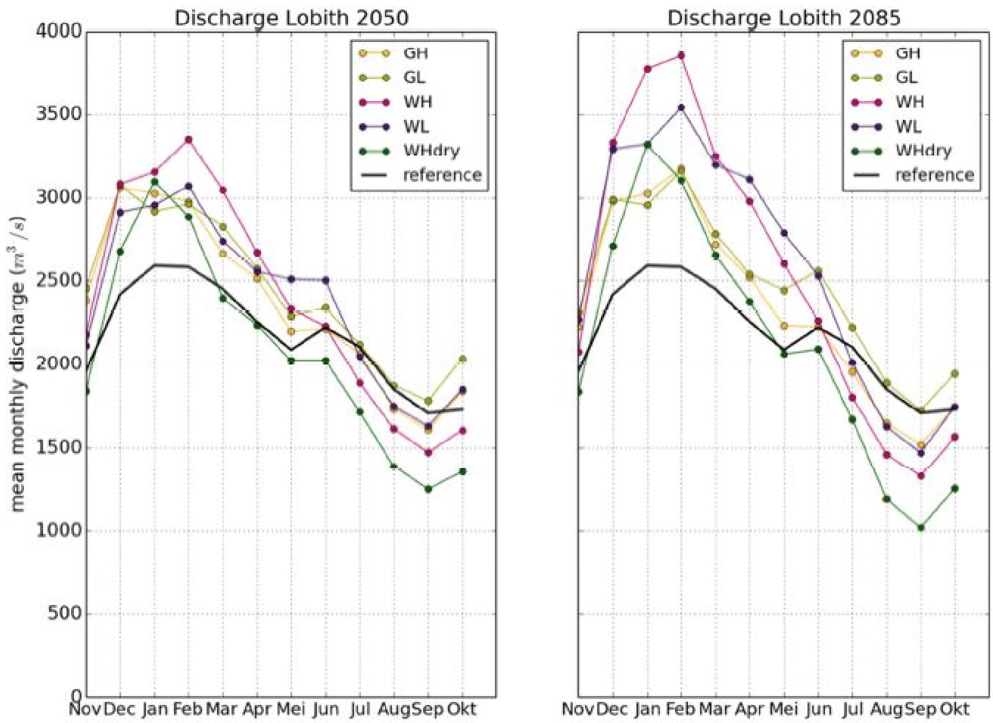


Figure 2.5: Average monthly discharge regime for Lobith for the five KNMI'14 scenarios in comparison to the reference situation. Reproduced from Sperna Weiland et al. (2015).

The more frequent and more severe droughts could possibly hinder navigation, nature and drinking water (increased salt intrusion). When discharge drop below 1000 m^3/s at Lobith, the load capacity of a large part of the fleet are reduced due to limited water depth (i.e. already at 1400 m^3/s for the largest vessels). Hence, the average number of days per year with discharges below 1000 m^3/s is copied from Sperna Weiland et al. (2015) in Table 2.4 revealing that three of the four climate scenarios predict more frequent low flow-conditions. On the other hand, the extreme discharges during winter are expected to show up more frequent and more severe as is shown for different return period in Table 2.5. Despite the increased precipitation in the KNMI'14 scenarios it seems unlikely that flood waves will exceed discharges of 18 000 m^3/s as a result of upstream flooding. To conclude, it is expected that climate change is going to affect (i) flood protection (and also to some extend navigation and nature) due to the more frequent and extremer flood discharges and (ii) the more frequent and stronger low-flow events will induce problems for nature and navigation.

		2050				
	Current statistics (1951-2006)	G_L	G_H	W_L	W_H	$W_{H,dry}$
# of days (Q_{Lob}<1000 m^3/s)	23	14	18	19	23	46

Table 2.4: Average number of days with a discharge at Lobith lower than 1000 m^3/s

		2050			
Return Period [years]	Current discharge statistics	G_L	G_H	W_L	W_H
10	9 130	10 880	10 590	10 760	11 130
100	12 580	14 090	13 930	14 020	14 060
1000	14 290	15 680	15 170	15 370	15 100
3000	14 800	16 740	16 240	16 410	16 300
10000	15 270	17 180	16 980	17 050	16 990
30000	15 700	17 330	17 160	17,220	17 250

Table 2.5: Overview of discharges in m^3/s (Sobek) for specific return periods for all scenarios except the $W_{H,dry}$ including the effect of upstream flooding based on Sperna Weiland et al. (2015)

2.4. The Dutch Rhine's functions

According to Janssen et al. (1979) the river's main function is the conveyance of water, sediment and ice to the sea. When considering this function for the society this could be translated as protection against flooding. However, the Dutch Rhine has to provide various other functions, such as safe and efficient navigation, sufficient and good quality drinking water, agriculture, nature and recreation. To limit the scope of the thesis, not all functions are considered in this research. According to Rijkswaterstaat (2015), Rijkswaterstaat's main objective is responsibility for flood risk management, sufficient water supply, clean and healthy water, smooth and safe transport by water and a sustainable living environment. From this the functions flood protection, navigation and nature will be considered in this thesis. This section provides insights on the way river conditions are related to these functions.

2.4.1. Navigation

The Rhine as transport axis is of enormous importance for the Netherlands. This originates from the geographic advantageous location of the Netherlands in the Rhine delta and the inland waterway access to the European hinterland (especially Germany). In 2015, 18 % of the Dutch domestic cargo (measured by weight) and 31 % of the Dutch exported cargo is transported by Inland Water Transport (IWT) (CBS, 2016). The attractiveness of IWT is due to the low transportation costs and lower pollution load per transported ton compared to other transportation types as aviation, transport by rail or by road. Also because of congestion on European roads it is desired to transport more goods by IWT. Due to the important transportation function, safe, efficient and profitable inland shipping is desired. This requires a reliable infrastructure, which has a high capacity and reliable navigation. Rijkswaterstaat is the responsible authority for facilitating inland shipping in the Dutch rivers. In the past various interventions to the river bed has increased the transport capacity of the Rhine (e.g. construction of weirs, groynes and normalisation of the river). Due to the international character of inland shipping, international rules have been defined. The European Conference of Ministers of Transport has defined classes for vessels with corresponding dimensions. In this way, European waterways are categorized in classes, which determines for which vessels the waterway is safe navigable, namely CEMT-classes (Rijkswaterstaat, 2017b). Due to economic growth the IWT fleet is upscaling their load capacity resulting in larger vessels (Groen and van Meijeren, 2010). The water depth of the fairway can be considered as an important factor, as a larger load capacity requires larger and especially deeper vessels. Hence, the use of these larger vessels might become hindered by the more frequent and severe droughts caused by climate change.

To allow these classes in the Dutch waterways, guidelines have been composed which also provide guidance for the dimensions of the waterways, called Richtlijn Vaarwegen (Rijkswaterstaat, 2017b). Also other institutions blend into the Rhine policy, like the Central Commission for the navigation on the Rhine (CCR), which is a cross-bording institution that aims to improve transportation over the Rhine. In this way requirements for the minimum depth, width and vertical clearance (bridge) have been defined for navigation on the Rhine. A larger width and depth of the fairway increases the potential load capacity of the fairway, since more and larger vessels can use the river. Water depth is of great importance for IWT since the vessel's load capacity depends on the draught (vertical distance between waterline and bottom of the hull), which is adapted based on the available water depth. Considering the Upper Rhine region, the water depth has become a limiting factor in the fairway dimensions. Considering the height dimensions of the fairway the expected trends will probably positively influence the vertical clearance since lower water levels are expected in the upper Dutch Rhine. However, sea level rise and the introduction of high cube containers could introduce problems at the lower Dutch Rhine.

The CCR prescribes a minimum depth for different sections of the Rhine at an Agreed Low Water level (ALW or in Dutch: Overeengekomen Lage Rivierstand). The ALW is defined as the water level not exceeded 20 days (without ice) a year, so basically a water level that 5 % of the time returns. The ALW is once in a while (the last time in 2012) adapted to the new river conditions and is linked to an agreed low-flow, called ALD (Agreed Low Discharge) which is currently 1020 m^3/s at Lobith equivalent to 816 m^3/s in the Waal. The minimum depth prescribed for the Waal at ALW is 2.80 m and the minimum width 150 m.
In this way, the Dutch designed the river channel to fulfil the agreed dimensions (Figure 2.6). Groyne fields fix the river banks and more water into the navigation channel creating a larger depth. In between the groyne fields a main channel has been designed in such a way least effort is required (i.e. outer bends are in general deeper than inner bends). However, due to morphodynamics the minimum water depth cannot always be

Fairway property	Requirement at ALW
Channel width	150 m
Depth	2.80 m

Table 2.6: Fairway dimensions of the Waal at ALW.

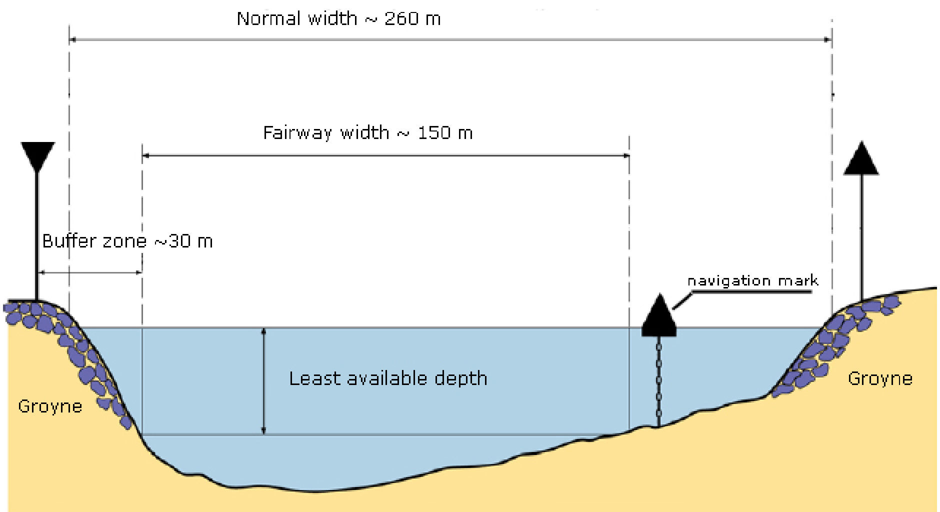


Figure 2.6: Cross-section of a typical Waal river channel. Adapted from Rijkswaterstaat (2007).

guaranteed. Hence, maintenance dredging is required once in a while. In general, after the flood season (1st of May) when the discharge drops below 3000 m^3/s the navigation channel is prepared for a low-water season by dredging. However, at certain locations dredging is not possible due to cables and pipelines underneath the river bed or due to structures such as fixed layers often resulting in the Least Available Draught (LAD).

The ALW is a requirement developed by policy-makers to enhance IWT. However, even at larger water depths vessels are obliged to sail with a reduced loading capacity due to draught up to 4.5 m. During those periods more vessels has to be recruited to transport all the goods increasing the transportation costs and the emissions. When discharges even drop below the ALD, a water depth of 2.80 cannot be guaranteed. However, in most cases the water depth in the Waal should not be the limiting factor for transportation to Germany. Hence, lots of vessels determine their loading capacity based on water levels at Kaub or Duisburg-Ruhrort (Germany).

Bed degradation results in navigational bottlenecks at locations where a bed protection has been placed and at lock entrances. The bed protection are placed in river bends to stabilize the dynamic erodible river bed. Since the bed at the location of the bed protection does not erode, humps in the river bed become present. Due to the backwater effect shallowness occurs at the downstream edge of the bed protection as can be seen in Figure 2.7. Due to this effect the water depth at the fixed layer of Nijmegen (rkm 883-885) has become a urgent bottleneck. The water level during ALD (ALW) at Nijmegen has dropped with 48 cm between 1992-2012 (Table 2.7), resulting in a reduced water depth as well (Verweij, 2016). A reduced depth during low-flow will force vessels to sail with a reduced loading capacity, which will have economic consequences. Ministry of Infrastructure and Water Management (2018) conducted a preliminary assessment of the economic consequences for the reduced loading capacity due to more frequent draught restrictions, which estimates the economic damage in 2028 between 9.3-15.1 million euros.

2.4.2. Nature

The river ecosystem is a dynamic system that accommodates many animals and plants. A large part of the upper Rhine delta is part of the Natura 2000 areas and various flood plains belongs to the Dutch Forestry Commission (Dutch: Staatsbosbeheer). However, the ecosystem is also vulnerable due the fact that the ecosystem boundaries are permeable with respect to energy and materials flux; and is therefore easily influenced by ex-

Location	ALW 1992	ALW 2002	ALW 2012
Pannerdensche Kop	7.52	7.33	7.13
Nijmegen	5.71	5.45	5.23
Tiel	2.70	2.62	2.58

Table 2.7: Historical ALW data at Nijmegen with the water level denoted in m + NAP. Obtained from Verweij (2016)

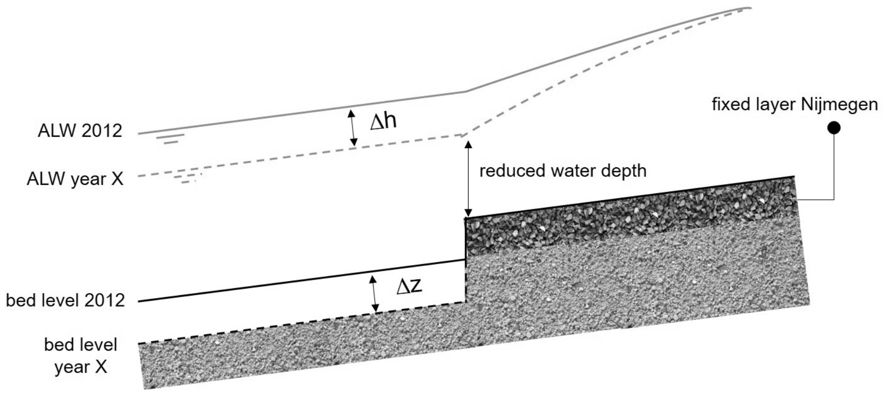


Figure 2.7: Effect of bed degradation illustrated by means of a longitudinal profile at the bed protection at Nijmegen.

ternal events such as global climate change, pollution, national and global economies (Stanford et al., 2017). Civilization reduced the natural value considerably in the past, because of the attention was mainly focussed on other functions, such as navigation, agriculture and flood protection. This resulted in a heavily navigated main channel, which hardly interacts with the floodplains. Natural side channels and other water-rich nature in the floodplains disappeared to force all discharge through the fairway in case of low-flow. Furthermore, pollution from industries upstream affected the water quality downstream. When biodiversity decreases in the water, the water quality will worsen as well, which touches upon other functions such drinking water, swimming water and agriculture. Also due to increased awareness of the people for natural value, since the 90s it is attempted to improve the ecological quality of the Rhine by ecological rehabilitation programs, such as:

- Water Framework Directive or WFD (Dutch: Kader Richtlijn Water) is a program initiated by the European Union to improve the water quality in both chemical as ecological point of view (Rijkswaterstaat, 2017a).

- Room for the River (RfR) had a dual goal by improving protection against flooding and improving the living environment, which incorporated also improving natural value of the river.

- NURG (Dutch: Nadere Uitwerking Rivierengebied) is a program initiated by the Dutch Ministry of Infrastructure and Water Management and the Ministry of Agriculture, Nature and Fishery to develop 7 000 ha of new nature along the Rhine and Meuse.

Characteristic elements of ecological rehabilitation projects are: riverine forests, secondary channels and fish traps at structures. Also slopes without revetment are important for more interaction between the low water bed and the flood plains. The improvement of the natural value of the river requires a different lay-out of the river, which is in most cases in opposite to the lay-out of the other interests (e.g. shallower channels, vegetation, frequent inundation of floodplains). The biotopes (specific ecological area-arrangements) must be integrated into the other landscape elements in such a way that the remaining functions are not harmed too much (de Vriend et al., 2011). This is one of the major challenges of IRM. In practice this results in complex structures like man-made side channels or wetlands connected to the river system at certain discharges, regulated by an inlet. By nature, side channels are dynamic and will eventually silt up. A lot of maintenance is required, which on its turn disturb nature. During low-flow periods the water bodies might be disconnected to the river channel not allowing exchange between the river system and water bodies.

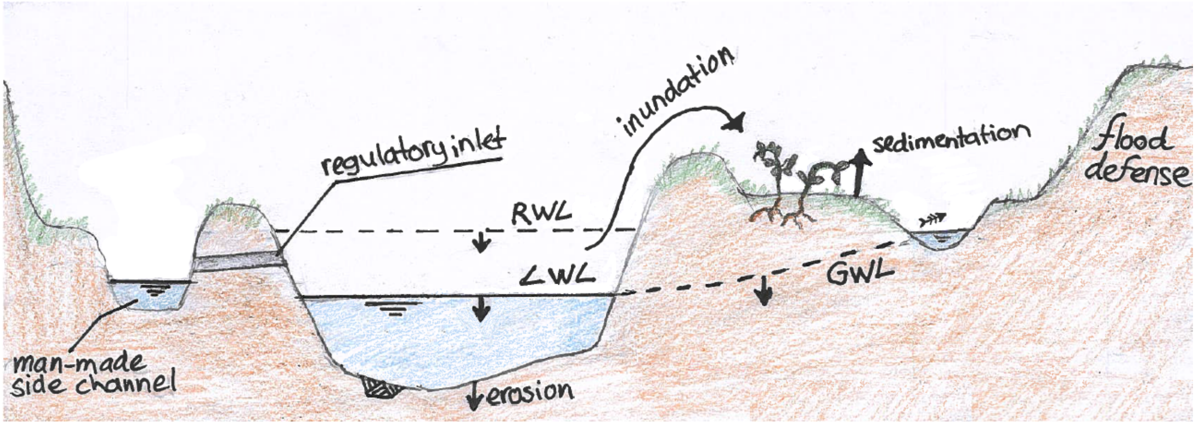


Figure 2.8: Cross-sectional overview (not on scale) of the effect of large-scale bed trends and climate change for nature in floodplains with lowering low water levels (LWL), regular water levels (RWL) and ground water levels (GWL). On the left side a man-made side channel connected to the river by a regulatory inlet and right a typical floodplain separated from the river channel by a levee.

Verweij (2016) predicts a negative effect with ongoing bed degradation and most certainly climate change will affect the riverine ecology as well. More frequent and more severe low-flow conditions will result in the following phenomena:

- **Dehydration of the floodplains**. Groundwater in the floodplain is connected to the water level in the river, so during low-flow periods the groundwater level will sink (i.e. rate depends on the permeability of the soil) as is shown in Figure 2.8. Another way to reload groundwater level is inundation when water can infiltrate into the soil reloading the groundwater level. However, bed degradation results in in lower water levels and climate change scenarios predict more frequent and more severe periods of low-flow. Lower ground water levels will affect flora in the floodplain eventually affecting the whole ecosystem. Water bodies within the floodplain might evaporate when not connected to the river channel, thus destroying aquatic life. Also other functions like agriculture are negatively influenced by this phenomena.
- **Reduced interaction between river channel and floodplains**. As earlier discussed, the riverine ecosystem is a highly dynamic ecosystem due to river dynamics. However, flora and fauna related to this ecosystem are disappearing when the interaction between the river reduces. Once in a while, the riverine ecosystem requires a flood to reset and to allow pioneer species to develop. Considering the typical river cross-section with a river channel and floodplains separated by levees, sedimentation of the floodplains and degradation of the main channel results in less frequent inundation as illustrated in in Figure 2.8. When man-made wetlands and side channels are regulated by an inlet, the period throughout the year during which those water bodies are in connection with the river channel is most likely also reduced (see Figure 2.8 left side).
- **More navigational nuisance in the river channel**. During low-flow the river channel is reduced in size due to a limited amount of water. Furthermore, more vessels are expected to transport the same amount of goods (i.e. reduced loading due to draught restrictions). Those two effects during low-flow disrupt aquatic life in the channel.

Measurements of the quality or performance of nature are commonly related to chemical and biological methods, such as measuring water quality and counting species. Those methods are outside the scope of this study requiring another indicator to quantify the performance of nature. Another approach is to quantify the performance of 'objects' contributing to the function nature. These 'objects' are affected by the bed degradation and measures to the river bed, such as:

- **Side channels** contribute to the ecology by accommodating flora and fauna requiring shallower water, slowly flowing water that is not disturbed by navigation. Those water bodies are important for fish to mate and reproduce, when not connected to the river system those fish are trapped. When water levels lower, the side channels become singular or even not connected at all as can be seen in Figure 2.9. This result in trapping of the organisms in the side channel, which eventually could result in mortality

Figure 2.9: Dehydration of the side channel Klompenwaard with left regular conditions and two-sided connected and right low-flow conditions. Satellite images obtained from Netherlands Space Office

of those organisms. The side channels are commonly designed with a water inlet or threshold, so it will not extract too much water from the main channel. The design of the side channel is based on a certain function requirement which is amongst other things expressed in a minimum frequency of flow through the channel. Other autonomic processes like sedimentation of the side channel also affects the frequency and intensity of flow through the channel.

- **Floodplains** accommodate many types of flora and fauna and are moreover important to accommodate agriculture. Those floodplains are mostly separated from the river channel by means of levees resulting in inundation when the river reaches a certain water level. Other plains might be in direct connection with the river channel or by means of a regulated inlet. As already earlier discussed both climate change and bed degradation are dehydrating the floodplains (i.e. lowering water bodies and groundwater levels). When the floodplains dehydrate, this could cause mortality of organisms.

- **Fish stairs** are constructed to allow fish to overcome the height difference caused by a weir or sluice structure. However, when water levels sink the opening or exit of the fish trap will not be accessible for fish. When water levels drop downstream, the step could be to high to overcome for fish. When water level drops upstream the highest step could become dry.

Those objects are commonly designed with a certain design criteria, while after construction no formal policy consist to maintain the 'objects'. As large parts of the Rhine is a part of Natura 2000 and many protected animals live along the rivers (e.g. the beaver and the salmon), the Netherlands is legally responsible of protection of this ecosystem and those habitats (Program director of Natura2000, 2014). In this manner, it could be argued that a function requirement of nature is no change of the ecosystem.

Researchers throughout the world did not succeed yet to invent a way of expressing natural value in an economic value, while this might be possible for other river functions, such as agriculture or navigation. Therefore, no economic comparison is possible between the natural value and the performance of other river functions. However, Rijkswaterstaat is investing money in ecological rehabilitation programs in the Netherlands, which can serve as indication of money that can be wasted when objects do not function as designed (e.g. side channels). Within WFD €282 million has been invested in the first phase (2010-2015) (Rijkswaterstaat, 2017a).

2.4.3. Flood protection

As stated in the introduction the primary function of a river is transport of water, ice and sediment from source to sea. Over the course of centuries communities settled along the banks of the Dutch Rhine. Since rivers are dynamic systems constantly adapting their geometry, those communities were on a regular basis plagued by floods. The large-scale normalisation works, such as dikes and bends cut-offs trained the river in its course resulting in an increased flood protection. Nowadays, Rijkswaterstaat is responsible for safe transport of water, ice and sediment through the Dutch Rhine. Therefore the river provide to sufficient discharge capacity, which can retain the probability of a flooding within socially accepted boundaries. Increasing flood protection could be managed by either increasing the discharge capacity (lower flood levels) or reinforcing the flood defences.

New flood protection standards
For several decades the safety standards for flood defences in the Netherlands have been based on the probability of exceedance of hydraulic loads. Flood defences would need to be designed to safely withstand hydraulic loads with a certain probability of exceedance. These standards strongly focussed on the failure modes overflow and overtopping and were derived in the 1960's and 1970's (Jonkman et al., 2017). The design flood discharge was based on the 1/125,00 exceedance probability with a typical discharge at Lobith of 16 000 m^3/s. In last 100 years of daily discharge records, such an extreme flood discharges never happened, which raise the need for numerical models to derive flood levels along the river. By means of the numerical model WAQUA the flood levels have been simulated along the Rhine branches. These levels were called the design water levels (Dutch: maatgevende hoogwaterstand) and were used to design flood protection. However, in 2014 the Dutch government, as part of the Delta Program, has introduced new safety standards for flood defences. Reason for that is the growth of the economy and population in the protected areas and the enlarged risk due to climate change (Veerman, 2008). Also, over the last decades more advanced methods for risks assessment have been developed. Hence, it was decided to derive new safety standards based on an advanced risk assessment of individual, societal and economic risks. The new standards are based on an accepted failure probability (also called target reliability) of the flood defences instead of an exceedance probability of a flood level. The various failure modes contribute in a certain way to the total target reliability. Within the new standards stricter rules are applied for the failure mode piping and macro-stability. For this reasons lots of dikes have to be reinforced by means of width, berm stability or dike cover.

The strength of the dike is assessed based on the target reliability of the various failure modes and a corresponding design load. The traditional approach for flood defences is that this design load is generally given as a water level with a certain probability of exceedance related to the target reliability (Jonkman et al., 2017). As a design load is used in reliability analysis for all failure modes, a distinction can be made between the analysis for failure modes that are dominant by water levels and wave actions (e.g. overflow, revetments and overtopping) and for the geotechnical failure modes (piping, stability). For the latter, a design load is generally given as the water level associated with the target reliability of the flood defence system, while for failure modes dominated by water and wave action a nearly full probabilistic assessment is carried out by means of HYDRA-NL (Appendix B. However, for overflow the water level is the only stochastic variable and and if that water level exceeds the minimum crest level within the dike trajectory the flood protection structure fails. In other words, a design water level associated with the target reliability of the dike trajectory is assumed.

The new standards have been adopted in the Water Act in 2017 and all flood defences will need to comply by these standards by 2050. All tools and design criteria to assess a flood defence based on the new standards have been incorporated in an instrument, called WBI2017 (Dutch: Wettelijk Beoordelings Instrumentarium). This instrument prescribes how design water levels could be derived. The implementation of adoption to the new safety standards is being executed by the HWBP (Rijkswaterstaat, 2015).

The river engineering guidelines of Rijkswaterstaat (Dutch: Rivierkundig Beoordelingskader) state that no elevation of the design flood level is allowed due to interventions on the river bed (Kroekenstoel, 2017). An elevation of the design flood level has to be compensated by means of another measure which reduces the water column again below the design flood level. However, in the new flood defence standards there is not a single design water level but multiple, and these have not been adapted in the river engineering guidelines yet. Furthermore, Kroekenstoel (2017) prescribes that the change in discharge distribution at the Pannerdensche Kop for 16 000 m^3/s Lobith discharge should be smaller than 5 m^3/s due to measures to the river bed. Despite the fact that design water levels are outdated, the guidelines remain valid until adapted. Which means measures negatively affecting flood protection have to be compensated.

Effect of river processes
The bed degradation has a direct influence on the hydraulic load of the flood defences and therefore influences the function flood protection. Another aspect of the bed degradation is the fact that it could destabilize flood defences connected to the main channel and other structures in the river bed, such as bridge piers. Within this thesis the research is limited to the effect on the hydraulic load. Bed degradation results in a

lowering of the design flood levels. The WBI2017 uses a bathymetry profile of 2013 (Driessen et al., 2018), which means that the main channel bed is situated higher than the actual case due to ongoing bed degradation. Flood defence managers have to assess the flood defences every 12 years based on the methods of WBI. It seems reasonable to assume that the bathymetry is also updated every 12 years resulting in design lower flood levels as is illustrated in Figure 2.10. This means that following the river engineering guidelines measures increasing the flood levels (e.g. nourishments) have to be compensated again. In other words, when the bed level is brought back to the level it was a few years ago, compensation is required by the client who is executing the nourishment (while this compensation was not required a few years ago).

Another point of interest is the future change of discharge distribution that could be influenced by the bed degradation. Differences in bed degradation between the different Rhine branches could change the future discharge distribution (i.e. more severe degradation in the Waal attracts more water into the Waal), which could lead to increased discharges in a certain Rhine branch resulting in an increase of the flood levels (Verweij, 2016).

As already denoted in Section 2.3, flood discharges corresponding with certain return periods are increased due to more frequent and extremer flood discharges (see Table 2.5). Sequential flood levels rise during higher discharges. In summary, bed degradation is lowering flood levels while climate change is rising the flood levels again. When assessing the effect of river processes on the function flood protection either the new probability of failure has to be compared with the old one, or the design conditions (flood level and wave characteristics) have to be compared at the same probability of failure.

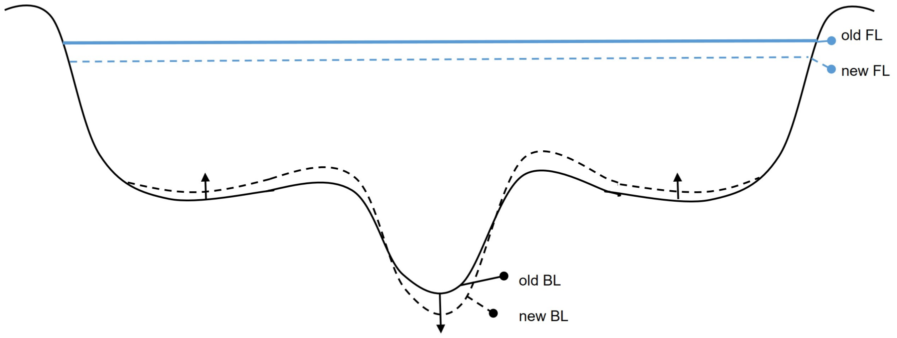


Figure 2.10: Cross-sectional overview of result of bed degradation with bed level (BL) and flood level (FL).

3 Case study

3.1. Introduction

A methodology that enables assessment of the impact of autonomous trends and river interventions on the future performance of river functions will be developed and evaluated within this thesis. In order to develop these methods in the time span of a Master thesis, a case study will be conducted with a narrowed scope. As the previous chapter already discussed the considered autonomous trends and river function, this chapter will define the case study within a certain study area and the considered sediment management strategies. In general, sediment management can be referred to sediment nourishments, but the term sediment management actually is more than sediment nourishment alone as it also includes the sediment cycle in time. As different nourishment designs, stabilization frequencies and source of the sediment could be considered, various strategies will be defined. The methodology described in the next chapter will be applied on the case study evaluating the impact of bed trends, climate change and sediment management strategies on the functional performance of shipping, nature and flood protection from 2018 to 2050.

3.2. Study area

The initial scope of IRM is the non-tidal part of the Rhine and Meuse (Van Vuren, 2018). According to Sieben (2009), Blom (2016) and Hendrikssen (2018) the main channel of the Rhine is exposed to a significant bed degradation resulting in several (future and current) conflicts with river functions. Following Ministry of Infrastructure and Water Management (2018) this leads to navigation bottlenecks at the Nijmegen river bend (rkm 883-885) in the Rhine branch the Waal. Therefore it seems valid to consider this part of the Waal as case study. To assess the performance of the river function 'nature', the study area should include 'objects' enhancing nature (e.g. floodplains and side channels). In this manner a 25 km long section of the Waal from rkm 867 till rkm 892 is considered as case study. By applying the approach on this section the research questions can be answered. The precise location is indicated in Figure 3.1 and stretches from the bifurcation point Pannerdense Kop (rkm 867) a few kilometres downstream of Nijmegen (rkm 892) including, the river bends at Nijmegen (rkm 883-886), the bendway weirs in the river bend at Erlecom (rkm 873-876), the floodplains of the Millingerwaard (rkm 868-874), the side channel of Klompenwaard (rkm 869-870) and the flood channel at Lent (rkm 883-886). As the river conditions have to be determined in the study area, the model will reach further downstream than rkm 892. Also the nourishment will be extended downstream, as is has to impact the water levels in the study area.

3.3. Selected sediment management strategies

In addition to the application of a future river system without stabilization interventions, multiple sediment management strategies will be applied to evaluate whether the assessment methodology provides insights into decision-making of integrated river measures. A sediment management strategy consists of various aspects, such as the nourishment design, sediment material, sediment source, stabilization frequency and construction methods. Within this thesis the nourishment design, sediment material and stabilization frequencies will be considered. The morphodynamics imposed by the sediment strategies are neglected in this research, which enables the assumption that the bed degradation will continue with the same expected degra-

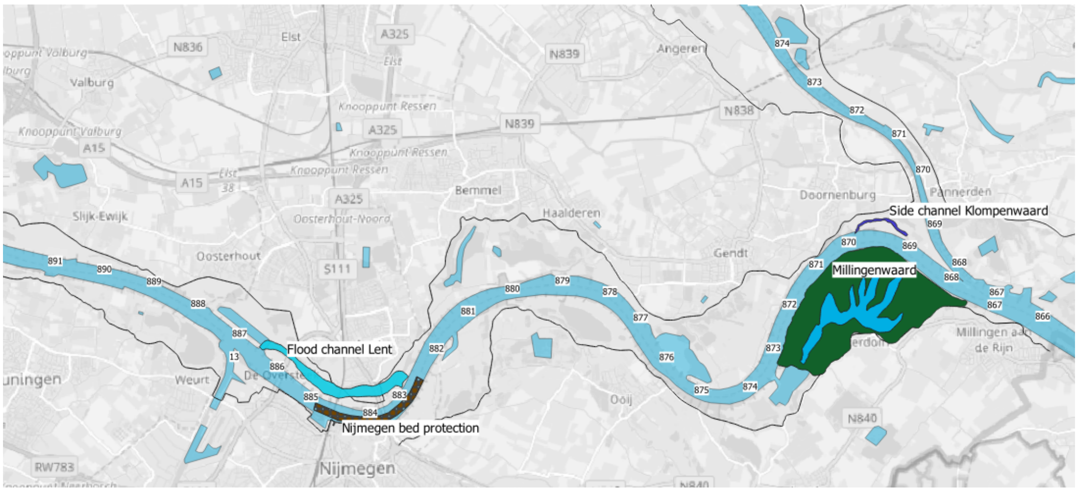


Figure 3.1: Overview of study area with points of interest and river kilometers indicated.

dation rate as a river without nourishment. Furthermore, this research will not concern the feasibility of the strategies, such as the use of sediment from the floodplain. This means that within this thesis, strategies are assessed based on the hydrodynamic impact on the river functions. The considered strategies will differ, as follows:

- **The nourishment dimension**, which is basically the one-dimensional height (width-averaged) of the initial nourishment and the distribution over the longitudinal profile.
- **The source of the nourishment sediment**. Either sediment comes from outside the study area not affecting the hydrodynamics (e.g. external sand pit or further down-, or upstream in the river) or it comes from inside the study area, such as from the floodplain. As the floodplains are elevating, synergy might be possible to use sediment from the floodplain as nourishment material. This option also compensates for the increasing flood levels of the nourishment.
- **The stabilization frequency**, which is the frequency of nourishing. Without stabilization the river bed will be on the same level as today within a few years (depending on nourishment dimension), which makes the nourishment just a short-term solution.

3.3.1. Nourishment dimensions

A nourishment can be designed in several ways, but due to the fact that the numerical model is not able to account for 2D effects, the nourishment design in the cross-section is not considered. This means that a width-averaged height of the nourishment has to be chosen, which can vary over the longitudinal profile. For simplification, a uniform layer is assumed on top of the current river bed, except for the fixed layer, as this is currently the bottleneck that requires improvement. The nourishment is extended downstream outside the study area till Tiel (rkm 915), to lift up the water level at Nijmegen. A pragmatic choice is made to restore and maintain the river bed to its state in a certain year in which the river should have functioned well. During the RfR projects measures have been designed to optimize the river functioning with respect to the functions flood protection and in less extend nature. By means of these assumptions, two designs of the nourishment are proposed:

1. **’Nourishment 2010’**, which is the river bed of 2010 when more or less the majority of the RfR projects had been completed. When assuming a uniform bed degradation of 1.5 cm/year over the entire river section (except for the fixed layer at Nijmegen), this means a nourishment of 12 cm high.
2. **’Nourishment 1997’**, representing the river bed of 1997 when the majority of the RfR projects was designed. When following the same approach as for ’nourishment 2010’, this situation is equivalent to an elevation of the river bed of 31.5 cm.

Figure 3.2 shows an overview of both the cross-sectional and the longitudinal profile. Yet, these profiles can most probably be optimized by means of adapting the design in the longitudinal profile. This will be examined with one variant of the ’situation 2010’, in which the bumpy profile of the river bed is smoothed by a non-uniform layer, which is on average the same volume as 'situation 2010'. This 'smoothed 2010' variant is illustrated together with situation 2010 in Figure 3.3. Furthermore, a stabilization of the river bed of 2018 is assessed. This strategy does not imply an initial nourishment, as the above mentioned strategies.

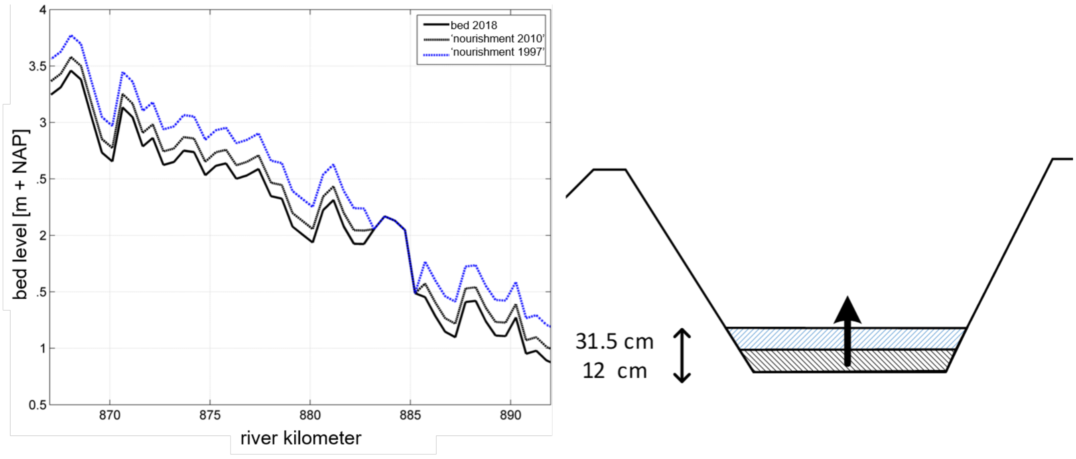


Figure 3.2: Longitudinal and cross-sectional profile of the main channel illustrating the nourishment situation 2010 and situation 1997.

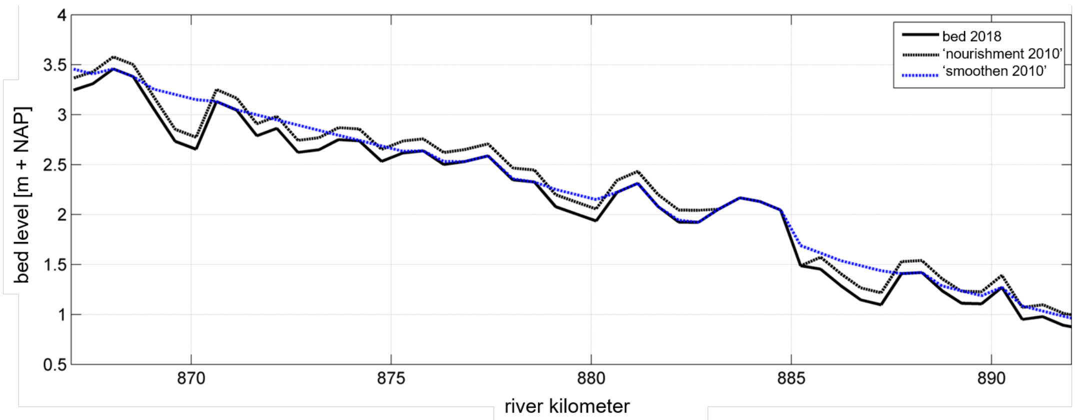


Figure 3.3: Longitudinal profile of the main channel with the smoothed nourishment and the uniformly elevated nourishment

Trajectory	Strategy	Nourishment height	Mean normal width	Required volume [m^3]
Waal (rkm 867-892)	'Nourishment 1997'	31.5	256	2 016 000
Waal (rkm 867-892)	'Nourishment 2010'	12	256	768 000

Table 3.1: Calcuations of required volumes for different nourishment dimensions.

3.3.2. Sediment from floodplains

Within the previous strategies it is assumed that sediment has been extracted from outside the study area, while also other possibilities might be possible. The two considered options will be elaborated:

- **Sediment from outside the study area** can be extracted from river reaches further downstream or upstream, while it is not affecting the river conditions in the study area. As the eroded sediment from the study area settles downstream, a cyclic flow of sediment can be initiated when sediment is extracted from the downstream stretch of the river and nourished in the study area. However, an initial nourishment will induce increased flood levels, which has to be compensated in a certain way (dike heightening or measures to decrease flood level again). Advantageous is the relatively low costs, while the induced side effects in the area of sediment extraction and the increased navigation on the Waal (as many vessels has to sail upstream) can be considered as disadvantageous.

- **Sediment extraction from the adjacent floodplains**, can be used to nourish the main channel. As the floodplains are elevating by means of sedimentation, synergy might be possible to use sediment from the floodplain as nourishment material. Whether the material is suitable to apply as nourishment material will not be considered in this thesis. When floodplains are lowered, the design flood levels will lower, reducing the costs of heightening the adjacent flood defences. Furthermore, World Wide Fund for nature (WWF) is proposing a new river design for the Rhine with side-channels along long stretches of the Rhine (Beekers et al., 2017). Currently, the impact of side-channels on the bed degradation is analysed, as it possibly reduces the sediment transport capacity. The construction of side-channels in the floodplains, could be combined with sediment extraction for nourishments. However, also disadvantageous could be expected, such as the higher costs, the suitability of the available material and disruption of current nature in floodplains.

This strategy will be applied on both the 'nourishment 2010' and 'nourishment 1997', as illustrated in Figure 3.4. As Figure 3.4 shows, the extraction of sediment from the floodplains is schematized in the model as a uniform lowering of the floodplain. Table 3.1 shows the required nourishment volume from the floodplains (on average 677 m), which requires a uniform floodplain lowering of respectively 4.5 and 11.9 cm. In practice, sediment extraction can also be performed by means of excavating deeper channels increasing the conveyance capacity, which results in a decrease of the flood levels. However, different designs of the floodplain sediment extraction will not be considered in this thesis.

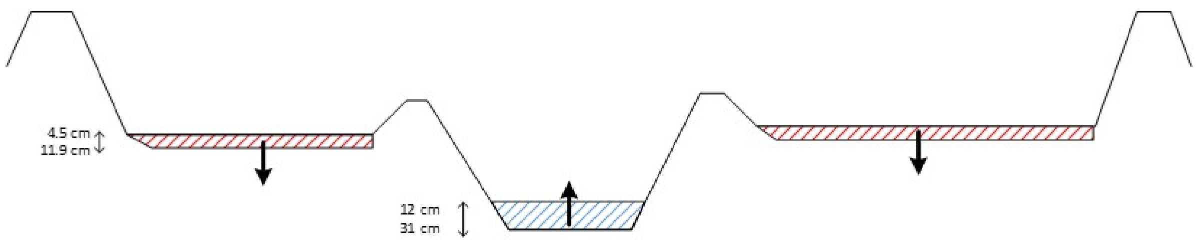


Figure 3.4: A cross-sectional profile of the strategies with sediment from the floodplain.

3.3.3. Stabilization frequency

As the previous sections only elaborate on the situation directly after realisation of the nourishment, this section accounts for the processes in time. As discussed in section C, nourishment might increase the sediment supply affecting the bed degradation. However, it is assumed that the bed degradation will continue with a similar rate as before the nourishment placement. By nourishing on a regular basis, a certain bed level can be stabilized. Within this thesis three cases will be considered:

- Yearly stabilization
- Once per five years stabilization
- Once per twenty year stabilization

When stabilizing situation 2010, this means the bed level of situation 2010 should not be exceeded, resulting in a layer above the bed level of 2010. The required volumes can be obtained by multiplying the frequency by the annual volume trend, which will be presented in Section 4.3.1.

4

Methodology

4.1. Introduction

This thesis combines two main blocks, namely the physical behaviour of the river and the quantification of different river functions. Background information on both research fields is discussed in the previous chapter. This chapter includes the methodology of the research by discussing the choice of the methods, models and tools that will be used in the remainder of this thesis. This research aims to develop a methodology that can assess the impact of (i) river processes and of (ii) river bed measures on the performance of the river functions: flood protection, navigation and nature. Using this methodology, the relationship between the two main blocks is extensively analysed and visualized by the conceptual model illustrated in Figure 4.1. The relationship between river processes and river functions could be considered in two steps:

1. The relationship between river processes and river conditions. For example the relationship between bed degradation and water depth in time and space.

2. The relationship between river conditions and river function performance, such as the relationship between reduced depth and navigability. In some cases function requirements are (legally) established, which can be simply scored as satisfied or not satisfied. Also other ways of quantifying the river performance are developed showing a direct gradual relation between river condition and river performance.

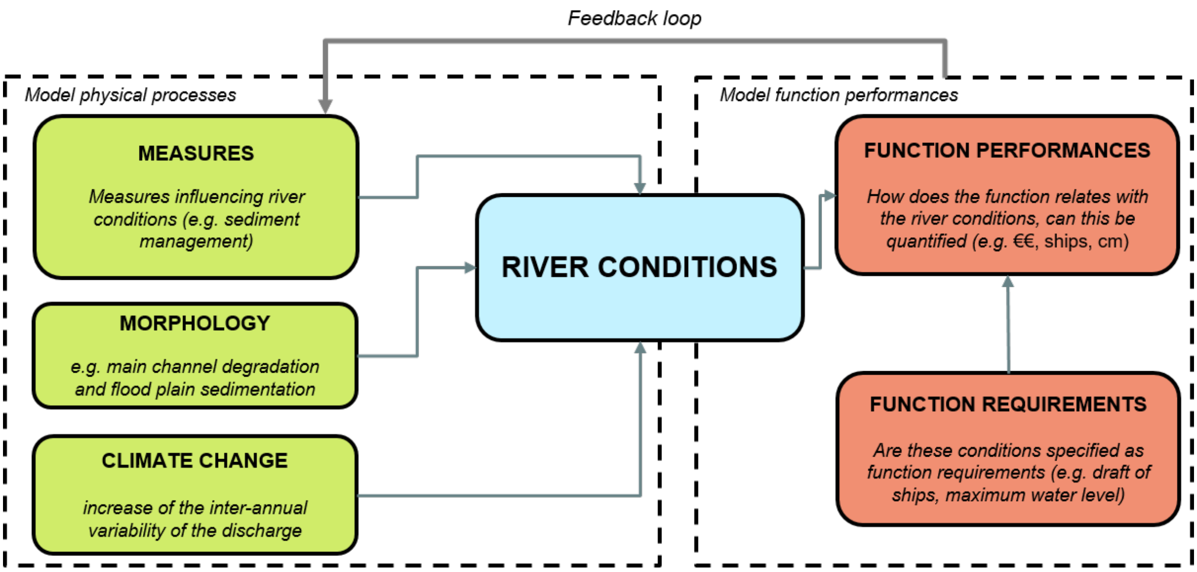


Figure 4.1: The conceptual model of the research approach

Before proceeding with the methodology, the following research question is addressed in Section 4.2:

Which river conditions are relevant for the assessment of the performance of a river function?

Section 4.3 describes the first step of simulating the physical processes by means of a numerical model. Subsequently, Section 4.4 discusses a number of methods to express the impact on the function performance in a quantified way, while Section 4.5 elaborates on methods to express the functional performance in an economic value. Based on this approach the river manager can assess a measure following the integrated river management principles (i.e. greater good for the river system and its functions). Section 4.6 summarizes the methods described in this chapter and will illustrate the methodology setup.

4.2. Choice of the assessment indicators

In Section 2.4 the relationship between river processes and river functions is already shortly discussed. The degree of functioning has to be expressed in indicators, which can be related to the river processes. To develop a methodology, decisions have to made on which hydraulic indicators have to be simulated. However, the assessment of function performance is not always possible by an hydraulic indicator. As also discussed in the introduction of this chapter, the relationship between river processes and river functions could be considered in two steps. Hence, both river conditions and function performance can be expressed by separate indicators:

1. Indicators to assess the performance of river functions have to be defined. The indicators can be subdivided in: (i) primary indicators and (ii) secondary indicators, which could be expressed by river conditions. In some cases those indicators are legally established, such as failure probabilities or fairway dimensions. This thesis will develop methods to assess the secondary indicators, while in some cases also the primary indicators can be assessed with the tools provided in this research.

2. The river processes are influencing the river conditions, such as water level, flow velocities or wave characteristics. The determining river condition(s) is appointed as the hydraulic indicator(s) of the assessment process. In most cases, time is an important aspect of the quantification of the river function performance, such as how long does a river condition persist or how often does it return.

Before choosing the appropriate indicators for the assessment, the definition of a good functioning river function should be clarified. It is possible that the definition chosen in this thesis deviates from other definitions. A well functioning river system for the considered river functions is defined as follows:

Navigation	A well functioning navigable river is defined as stated in Rijkswaterstaat (2015): "providing safe and efficient transportation".
Nature	The perfect river considering the function nature does not exist, since various types of ecosystems could be possible. Rijkswaterstaat and the Dutch Forest Commission formulated visions on the desired ecosystem including corresponding habitat in this area. In this way, a good functioning river system is defined as a system fulfilling the goals of those authorities.
Flood protection	A river system is offering good protection against flooding when the probability of failure is meeting the flood protection requirements. Kuijken (2015) (Delta Programme 2016) proposes preferred strategy of strong collaboration between dike reinforcement and creating more room for the river to meet the flood protection objectives in the Rhine.

In this way both primary indicators and various secondary indicators creating the circumstances to allow good functioning could be defined as presented in Figure 4.2. Those secondary indicators could be translated in hydraulic indicators.

Hydraulic indicator

As indicated in Figure 4.2 some of the secondary indicators could be directly linked to an hydraulic indicator. Within this section the feasibility and relevance of different river conditions is discussed. The considered autonomous trends are not expected to affect all river conditions in the same degree. The feasibility of a river condition has to do with the scope of the research and whether it is feasible to simulate the river conditions.

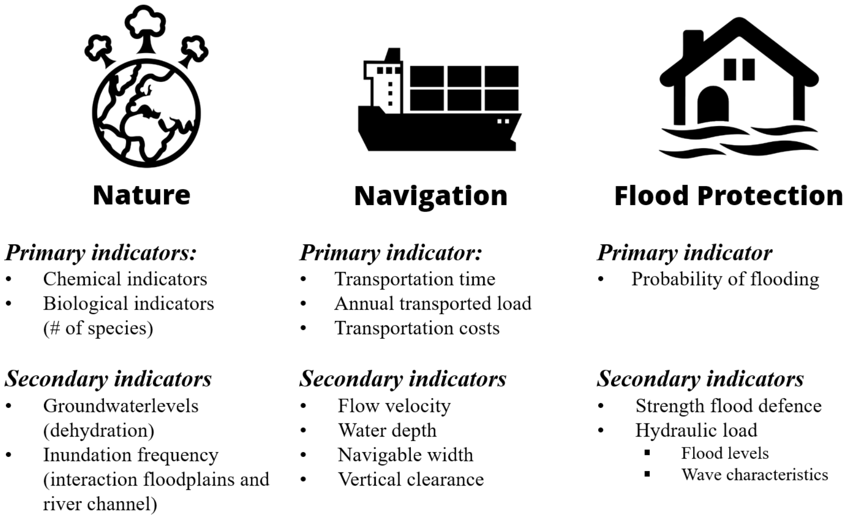


Figure 4.2: Overview of possible assessment indicators for the different river functions

Referring to the research questions the objective is to provide insights in the assessment process. To be able to go through the assessment process within the timespan of this thesis, there is no time for complex modelling. According to the indicators shown in Figure 4.2 the following river conditions are important:

- **River planform** is the shape of the river including river bends and river width. Due to the normalisation measures the planform is considered to be stable also accounting the considered river processes.
- **Flow velocity** can be considered as a local phenomenon or as a depth-averaged flow velocity. Local flow velocity could hinder navigation or influence the riverine ecology. Depth average flow velocity is of less importance for the different river functions. Since the autonomous processes influence the river geometry and future discharges, flow velocity is most probably also affected by the river processes.
- **Water level** is defined as the absolute height of the water (without waves) on the river axis expressed as a distance from the Dutch reference level NAP. The autonomous bed trends do directly influence the water level since a higher bed level results in a higher bed level for normal flow (definition is discussed in Section 4.3.1). As well, climate change affects the discharge pattern resulting in more lower and more low water levels and higher and more frequent flood levels. Water level is considered as the hydraulic load for flood defences. In other words water levels are an important indicator for the function flood protection. Furthermore, nature is affected by water levels since dehydration and interaction between floodplains and river channel depend on water levels. Simple hydrodynamic or hydro-morphodynamic models are available to simulate water levels.
- **Water depth** is the relative height of the water column between the water level and the bed level. In essence, when water level and bed level lower at the same rate, which is the case for normal flow the water depth remains unchanged. However, due to an non-uniform bed degradation in the longitudinal direction the water depth is reduced or increased as a result of the backwater effect (also discussed in Section 4.3.1). Especially the river transport sector has an interest in the river condition water depth. When bathymetry data is available or can be simulated (e.g. morphodynamic models) the water depth is easily simulated when combing bathymetry data and results of the simulated water levels.
- **Wave height** is the distance between the through and crest of the wind waves. Waves depend on various properties, such as wind velocity, fetch length and water depth. Despite the fact that flood protection is affected by waves, it is not expected that the river processes will affect wave conditions to a major extent.

Depth-averaged flow, waves and fairway width are not expected to exert major consequences on the river functions. Since local flow velocity requires to complex modelling within the scope of this thesis, future water levels can be predicted rather easily by hydrodynamic or hydro-morphodynamic models. As the water depth and water level can be prediced by the same models, those river conditions are chosen as hydraulic indicators.

Most hydro-morphodynamic models are forced by a discharge (time series) to simulate water levels. To obtain normative water levels, such as DWL and ALW, a single discharge has to be simulated. Other elements of the functional performance can best be expressed by analysing the function throughout the year, requiring water level time series simulations. Hence, the discharge, serving as input for the water level simulations, differs for every function as follows:

Navigation The formal CCR water depth requirement during ALW (1020 m^3/s at Lobith) and in a minor manner the navigability throughout the year requiring simulated water level time series.

Nature No formal function requirements for certain discharges has been formulated for nature. So it seems valid to analyse the function nature by simulating water levels throughout the year.

Flood protection A probability of failure has been assigned to flood defence sections. The probability of failure has been composed from failure probabilities of all failure mechanisms. In this manner a design return period can be formulated for the failure mechanisms overtopping and overflow. This return period corresponds to a flood discharge for which the flood levels can be simulated.

Methodology setup
As concluded in the previous paragraph this research requires a numerical model to simulate water levels based on discharge time series. As will be discussed in the next section, a simple 1D hydrodynamic model has been selected to simulate water levels. The simulated water levels will serve as primary input for the function assessment. An extensive elaboration of the methods applied to assess the function performance is provided in Section 4.4. This section includes methods to quantify function performance in several ways, while financial aspects of the impact on river processes will be discussed in Section 4.5. Furthermore, corrections methods will be discussed to obtain 2D depth simulations from a 1D (width-averaged) bathymetry dataset to assess the impact on the water depth (i.e. draught for navigation).

4.3. Numerical model simulating water levels

The physical behaviour of the river can be captured in a numerical model, which predicts the river conditions at various locations. Different numerical models to predict hydrodynamic and morphodynamic conditions are available (e.g. WAQUA, SOBEK and Delft-3D). However, due to their complexity and computational effort required for analysis, a simple analytical 1D model is used.

4.3.1. Model setup

The model used to predict the water levels in a river section is a 1D hydrodynamic model, which is compiled and validated by Strijker (2018) in his master thesis. The model is further developed in this thesis by using the principle for a longer river section and adaptation of the river geometry in time as a result of the autonomous bed trends. The principles of the hydrodynamic model are discussed in the following section.

Hydrodynamics
The water levels in the river depend on the flow capacity of a river channel and adjacent floodplains, which is subsequently affected by the cross-sectional geometry, the bottom slope and the hydraulic roughness. Manning (1891) captured this all together in the Manning formula that describes the discharge of steady uniform flow in a single channel. The cornerstone of the formula is the balance between the forces of gravity and bottom friction. The Manning formula is defined as follows:

$$Q = \sqrt{i_b * \frac{1}{n} AR^{2/3}} \tag{4.1}$$

where i_b is the bottom slope [-], n is Manning' s coefficient representing the hydraulic roughness [$s/m^{1/3}$], A the cross sectional surface area [m^2] and R the hydraulic radius [m]. The $AR^{2/3}$ term in the Manning formula

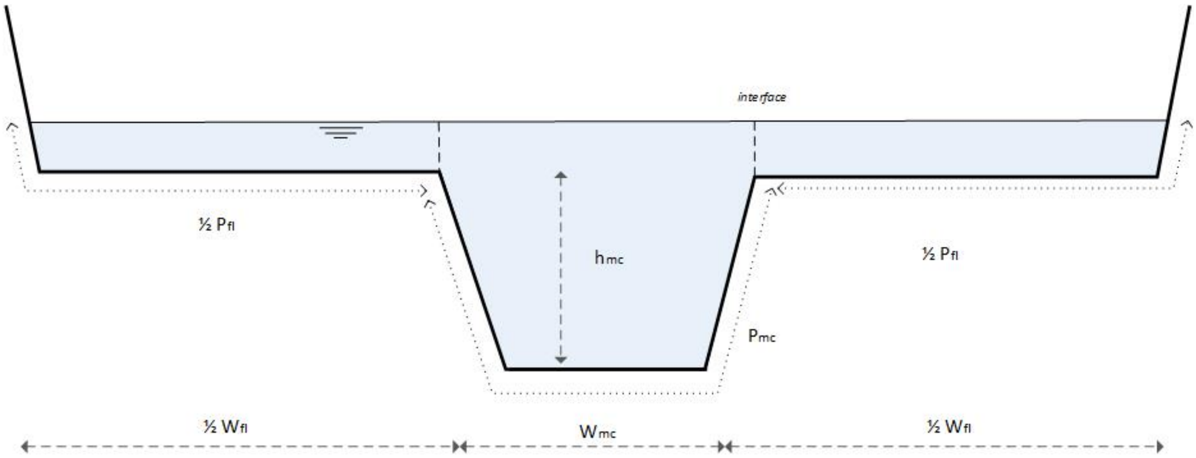


Figure 4.3: Overview of compound channel with corresponding characteristics.

is also called the conveyance part and can be considered as a measure of the discharge capacity of a river section.

The geometry of the river plain is described as a main channel with on both sides symmetrical floodplains (see Figure 4.3), also called a compound channel. During flood events these floodplains will increase the conveyance capacity of the river and the total discharge Q_T in the river will be the sum of the discharges of all the compartments. Within those floodplains the water depth is lower and the roughness due to vegetation higher than in the main channel. To account for the different characteristics in the floodplain, the Divided Channel Method (DCM) proposes a division through vertical lines, where the total discharge is given by:

$$Q_T = \sum_{n=1}^{i} \left(\frac{A_i R_i^{2/3}}{n_i} \right) * \sqrt{i_b} \tag{4.2}$$

where the index i indicates each subsection. According to Weber and Menéndez (2004), the DCM produces reasonable overall discharge predictions but it systematically overestimates the flow velocity within the channel and underestimates flow velocities in the floodplains due to the neglect of lateral momentum exchange. The presence of groynes in the main channel perpendicular to the flow creates even more complex flow fields. However, these 2D effects are not covered in this model.

The Manning's formula assumes uniform and steady flow. Flow can be considered steady when at any point in the river pressure and velocity do not change in time. It seems reasonable to assume that flow in the Waal is quasi steady, which means the flow through any cross-section has fully adapted to the discharge, as if the discharge does not change in time. A river is called uniform when there is no change in water depth or flow velocity over the longitudinal axis of the river, which basically means that the water level is equal to the water level predicted by the DCM. Assuming the river geometry does not change significantly the DCM seems reasonable.

However, observing the bathymetry data of the Waal River, the cross-section does vary significantly in longitudinal direction and therefore non-uniform flow will occur. The non-uniformity is observed in the difference in normal depth, which results in a gradually varying water level in longitudinal direction, a so-called backwater curve. This phenomenon is best described by the Bélanger equation:

$$\frac{\mathrm{d}d}{\mathrm{d}s} = i_b * \frac{d^3 - d_e^3}{d^3 - d_c^3} \tag{4.3}$$

where d_c is the critical flow depth (depth at which the flow is critical given a certain discharge) [m] and d_n the normal flow depth [m]. Since the flow in lowland rivers like the Waal are sub-critical ($Fr = u/\sqrt{gh} << 1$) this can be simplified as:

$$\frac{\mathrm{d}d}{\mathrm{d}s} = i_b \left(1 - \frac{d_n^3}{d^3}\right) \tag{4.4}$$

Bélanger describes the backwater curve by using the absolute parameter the water depth. The water level (h) and bottom level (z_b) are absolute values above a reference level, in this case Amsterdam Ordnance Datum (NAP), while water depth (d) is the absolute difference between the water level and the bottom level. Bélanger assumes a straight rectangular channel, while a compound channel is modelled in this study with a different water level in the main channel and the floodplain. To account for this a weighted discharge depth is computed by:

$$d_e = \frac{Q_m(h - z_{bm}) + Q_f(h - z_{bf})}{Q_T} \tag{4.5}$$

where Q_m, Q_f and Q_T are respectively the discharge through the main channel, floodplains and the total discharge [m^3/s]and z_b is the bottom level as denoted with z_{bm} in the main channel and z_{bf} in the floodplain [m + NAP].
To simplify the calculation of Equation 4.4, the analytical equation derived by Bresse is used:

$$h = h_n + (h_0 - h_n) * 2^{\frac{x-x_0}{L_{1/2}}} \tag{4.6}$$

where h_0 is the water level at the downstream edge of the river section ($x = 0$) [m + NAP], h_n is the normal flow water level [m + NAP] and the so-called 'halflength' $L_{1/2}$ [m] is defined as:

$$L_{1/2} = 0.24\frac{d_n}{i_b}\left(\frac{d_0}{d_n}\right)^{4/3} \tag{4.7}$$

which describes the length at which the backwater curve has reached half of the elevation difference between the normal flow depth and the water depth at the downstream edge.

Area information
To put these principles into practice the actual geometry and characteristics of the Waal are required. The geometry of the main channel can be assumed to be rather constant due to its importance for the navigation. Other properties of the river do vary significantly within the Waal (Figure 3.1), such as wide and narrow floodplains, and variation exists in levee height, vegetation cover, storage and conveyance capacity of floodplains. Table 4.1 gives a good indication of those characteristics. However, more specific geometric data is required to obtain reasonable predictions of water levels along the river section. Therefore the geometry data (i.e. bed level and width of main channel and winter bed) is obtained from Rijkswaterstaat 1D SOBEK model of 2013. No variation in roughness is assumed and the presented Manning roughness coefficients from Table 4.1 are used in the computations. Although the bed slope locally shows a large spatial spreading, less spreading is expected in the slope of the water line, which allows choosing a constant characteristic bed slope as presented in Table 4.1.

Detailed geometry of the floodplain is averaged and the presence of summer dikes and storage capacity in the floodplains is eliminated in this study, which could give aberrant water levels around bankfull discharge. In the dataset obtained from Sobek the winter bed includes floodplains, levees, river banks and groyne fields, which are schematized as uniform floodplain within our model (see Figure 4.3). It could be argued that the presence of groynes is included as part of the winter bed due to its negligible conveyance contribution during low flow. In this way the conveyance capacity of the river is computed every 500 m with specific cross-sectional geometry data.

Dimension and character	Symbol	Unit	Waal
Bed Slope	S_b	-	$1.2 * 10^{-4}$
Width main channel	W_m	m	260
Width floodplains	W_{fl}	m	550
Manning coefficient main channel	n_m	$s/m^{1/3}$	0.03
Manning coefficient floodplain	n_{fl}	$s/m^{1/3}$	0.0367

Table 4.1: Indication of dimensions and characteristics of Waal section near Nijmegen

Downstream at the Dodewaard floodplain (rkm 901.3) a Qh-relation is imposed as downstream boundary condition. The Qh-relation is constructed based on the stage relation curve of 2016 (Dutch: betrekkingslijn),

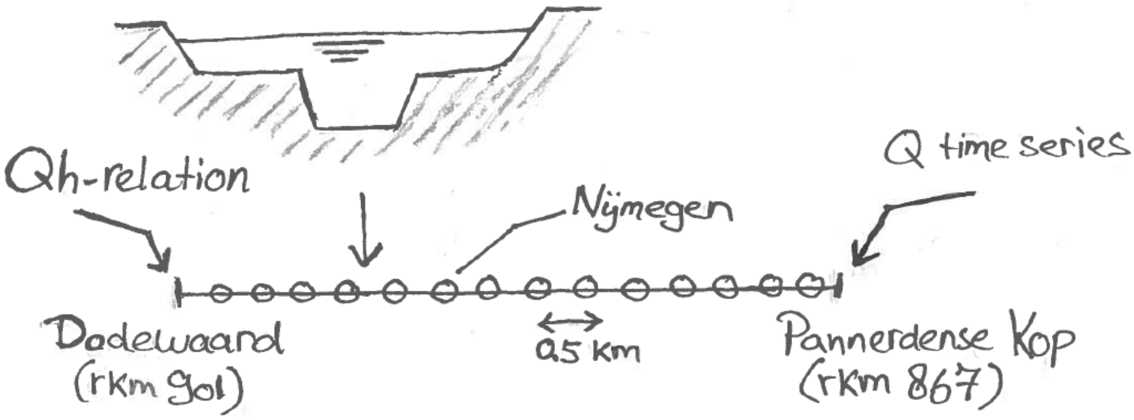


Figure 4.4: Overview of computational grid with the downstream boundary condition a Qh-relation and the upstream boundary condition discharge time series illustrated.

which relates discharge and water level at Lobith with water levels downstream. This relationship is captured by a logarithmic formulae (formulas are presented in Appendix D.1), which computes water levels at the Dodewaard based on a given Q. The results of the Qh-relation and the measured water levels in 2018 are presented in Figure 4.5. In this way a backwater curve is constructed by calculating water level conditions at every 500 m from downstream to upstream as illustrated in Figure 4.4.

Autonomous bed trends

The autonomous bed degradation of the main channel has to be incorporated in the model by means of fixed bed degradation rate. As discussed in Section 2.2.1 the future bed degradation is highly uncertain. However, based on the work done by Sloff et al. (2014) and Sieben et al. (2012), an expected bed degradation for the coming years has been estimated 1.5 cm/year for the Upper Waal (rkm 867-885), except for the bed protection in the river bend at Nijmegen. The non-erodible bed protection is only present in the outer bend allowing the inner bed to degrade resulting in best estimate width averaged degradation rate 0.5 cm/year. Directly downstream of Nijmegen a more severe bed degradation of 1.5 cm/year is assumed, which is interpolated to zero bed degradation at the hinge point of Tiel (rkm 913). The degradation rates per river kilometer are presented and discussed in more detail in Appendix D.2. To account for the degrading water levels at the downstream boundary (Qh-relation), the water levels are lowered by 1 cm/year during low-flow conditoins (slightly less during regular and extreme discharges, as the floodplain will convey discharge as will). This schematization of the bed degradation is just an one-dimensional simplification and a best estimate; to account for uncertainties a best estimated bandwidth is defined based on a milder and extremer scenario with respectively zero bed degradation and a doubled bed degradation rate (3 cm/year for Upper Waal, see Appendix D.2).

Considering the sedimentation in the floodplain, Middelkoop (2002) distinguishes different types of floodplains (low floodplains, behind minor dikes and on natural levees) with typical sedimentation rates (respectively >10 mm/year, 1-3 mm/year and 3-7 mm/year). Besides this distinction, a uniform sedimentation rate of 5 mm/year is assumed uniformly along the floodplains, since the variation within the floodplains itself is expected to has a minor impact on the water levels. However, within the model the floodplain is schematized including objects such as groyne fields, river banks and levees. By analysing Google Earth it is concluded that these objects generally cover 200 m per river bank. When the average winter bed width following Sobek along rkm 867-892 is reduced by 400 m, the average floodplain width is 677 m. In this way the sedimentation along the entire winter bed is assumed to be 3 mm/year.

As sediment is eroded in the main channel of the study area; while sediment is accumulating in the floodplains, a rough sediment balance could provide insights in the volume trends of the river section. When assuming the expected degradation rates as presented in Appendix D.2, approximately 110 000 m^3 per year is eroded from the 25 km long river section's main channel. For an extreme bed degradation scenario the volume trend is doubled to -220 000 m^3 on an annual basis. In a similar way, the accumulation of sediment in the floodplain can be quantified, as the sedimentation rate of the on average 677 m wide floodplains is approximated at 0.5 cm/year, resulting in an annual sedimentation of 84 625 m^3. Hence, on a yearly basis more sediment is leaving the study area than incoming, but the magnitudes are in a similar order as can be seen in Table 4.2. However, whether the sediment material is suitable to use for a nourishments remains questionable.

	Volume trend [m^3/year]
Expected bed degradation (approx. 1.5 cm/year)	- 110 486
Extreme bed degradation (approx. 3 cm/year)	- 220 972
Floodplain sedimentation	+ 84 625

Table 4.2: Volume trends calculations for different bed degradation scenarios.

Schematize sediment management strategies

Section 3.3 describes the various sediment management strategies selected in this thesis. As the numerical model is an one-dimensional model, the nourishment height is considered as an elevation of the actual width averaged river bed level. Figure 3.2 and 3.3 show the bed level after nourishing and before nourishment. As the nourishment continues downstream outside the study area till Tiel, the Qh-relation is adapted for the different sediment management strategies. Also scenarios with sediment extraction from the floodplain are considered in this thesis. The floodplain lowering is slightly different schematizes as presented in Figure 3.4, as the model schematizes the floodplain including river banks and groyne fields. This means the floodplains in the model are 400 m bigger, resulting in an average lowering of 2.8 and 7.5 cm of the model's floodplain instead of 4.5 and 11.9 cm.

4.3.2. Refinement and validation of the model

Objective of the model is a prediction of the water levels for different discharges corresponding with measured water levels of 2018 in the order of 10 cm. This accuracy is required to identify trends in the river conditions and indicate river function bottlenecks. To validate the model, the model results have to be compared with the measured discharge and water levels along the Waal. However, the SOBEK geometry data schematizes the situation of 2013 and does not involve the ongoing bed degradation in the main channel and sedimentation of the floodplains. Furthermore, the RfR projects of the flood channel at Lent (rkm 883-885) and the floodplain lowering of the Millingerwaard (rkm 868-874) were not finished in 2013. To account for these developments, the model is refined as follows:

- The bed level of the main channel is lowered with the annual bed degradation rates as described in the previous section.
- The floodplain is elevated by 2.5 cm to account for sedimentation as discussed in the previous section.
- The flood discharge is reduced at the location of the flood channel at Lent based on the design properties (de Jong et al., 2010). Appendix D.3 elaborates on the model schematization and effect of the flood channel.
- The floodplains of the Millingerwaard are lowered by 80 cm on average as a result of the RfR project.

After this conversion to the actual river profile, the water levels measured by Rijkswaterstaat can be compared with the model results. Rijkswaterstaat measures the water level at different locations throughout the Netherlands. For our area of interest the water level at the Pannerdense Kop (rkm 867) and at the Nijmegen Waal harbour (rkm 885) is measured with a float switch (see Figure 4.5). The discharge at the measured water level through the Waal is estimated by Rijkswaterstaat based on a Qh-relation (i.e. computed relation between water level and discharge at a certain location) (Behrens, 2008). Rhine flood discharges above 8000 m^3/s (equivalent to 5000 m^3/s in the Waal) cannot be calibrated by measurements, since they have not taken place since 2011. Hence, validation of flood levels can be done by comparison of own simulations and simulations of the numerical model WAQUA (the results of the WAQUA simulations are obtained from the Hydra-NL

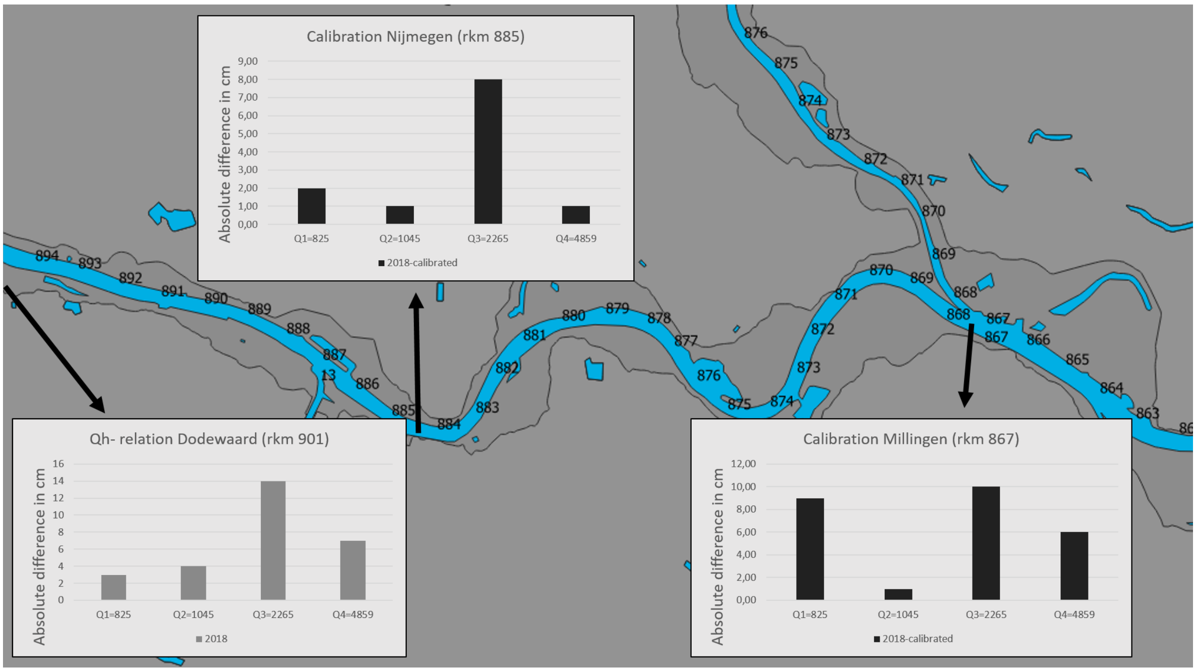


Figure 4.5: Overview of validation of the model by means of the absolute difference between the results of the measurements and the calibrated model 2018 at Dodewaard (rkm 901), Nijmegen (rkm 885) and Millingen (867).

		Millingen		Nijmegen	
date	Q [m^3/s]	h_m [+ m NAP]	h_p [+ m NAP]	h_m [+ m NAP]	h_p [+ m NAP]
11-01-2018 12:00	4446	13.62	13.54	11.62	11.57
02-04-2018 14:00	1698	9.30	9.24	7.50	7.55
12-05-2018 08:00	1276	8.28	8.31	6.55	6.61

Table 4.3: Validation of replication dataset with h_m the measured water level and h_p the predicted water level by the calibrated 2018 model

database).

To validate different discharges stages, two low-flow, one regular and one flood discharge from 2018, are compared to the model results. To account for the fact that the water level at Nijmegen does not respond instantaneous to the discharge at Millingen, a delay period of approximately 4 hours is introduced assuming a flow velocity of 1 m/s. Without calibrating the model, the model overestimates the water level. To improve the accuracy of the model, the model has been calibrated by tuning the model parameters such that the desired accuracy of the simulated results is reached. In hydraulic modelling the roughness of the main channel is often used as calibration parameter. By doing so this parameter is also used to include physical processes that are not captured by the model due to errors or simplifications. The roughness is reduced during low and regular flow (Manning coefficient of 0.026 [s/m$^{1/3}$] equivalent to a Chezy value of 45 [m$^{1/2}$/s] during ALW) , which results in the desired accuracy of 10 cm as presented in Figure 4.5. Also for the flood discharges the accuracy has been met by slightly reducing the roughness of the main channel. An overview of the flood discharge calibration can be seen in Appendix E.

Finally, the settings are validated again by comparing the model results to a new dataset from measurements of other moments in 2018. The replicated dataset also meets the proposed accuracy of the model as can be seen in Table 4.3.

4.3.3. Model input

Discharge time series are imposed at the inflow boundary of the model determining the water levels in the river section. Various methods will be discussed in Section 4.4 to assess functional performance, depending on different inflow boundary conditions. To perform statistical analysis of river conditions realistic discharge time series are required. For the reference situation, it is assumed that the future discharges are not going to change compared to the last century, which allows us to impose the 117-year daily discharge records at Lobith. However, other methods rely on a fixed discharge, such as the ALD or design flood discharge, which will be discussed in Section 4.4.

As all discharges are given for Lobith, a part of this discharge flows into the Pannerdensch Kanaal and will not flow through the Waal. This requires an analyses of the discharge distribution at the Pannerdense Kop. The discharge distribution of the Rhine at the bifurcation point Pannerdense Kop is regulated with weirs in the Lower-Rhine and Lek at lower discharges. During flood discharges an regulatory work in the floodplain is supposed to regulate the discharge distribution. However, the discharge distribution remains uncertain and depends on the water level at the bifurcation and in the branches. Ogink (2006) concluded that the most important sources of uncertainty are morphodynamics and the roughness of the main channel and the floodplain. Ogink (2006) estimated the uncertainty of the discharge distribution at the Pannerdense Kop at a standard deviation of 130-180 m^3/s.

To translate discharges at Lobith to discharges in the Waal, the most recent fixed discharge distributions for various discharges are obtained from RWS ON and interpolated between the different discharges (Appendix F). In this way the discharges for the Waal between 1901-2017 can be simulated based on a fixed discharge distribution and the daily discharge records at Lobith. When comparing the simulated Waal discharges with the daily discharge records at Waal from 1961, the uncertainty is illustrated in Figure 4.6. A more in-depth analysis is conducted in Appendix F. Despite the uncertainty in discharge distribution, a fixed discharge distribution is assumed to translate Lobith discharges to Waal discharges.

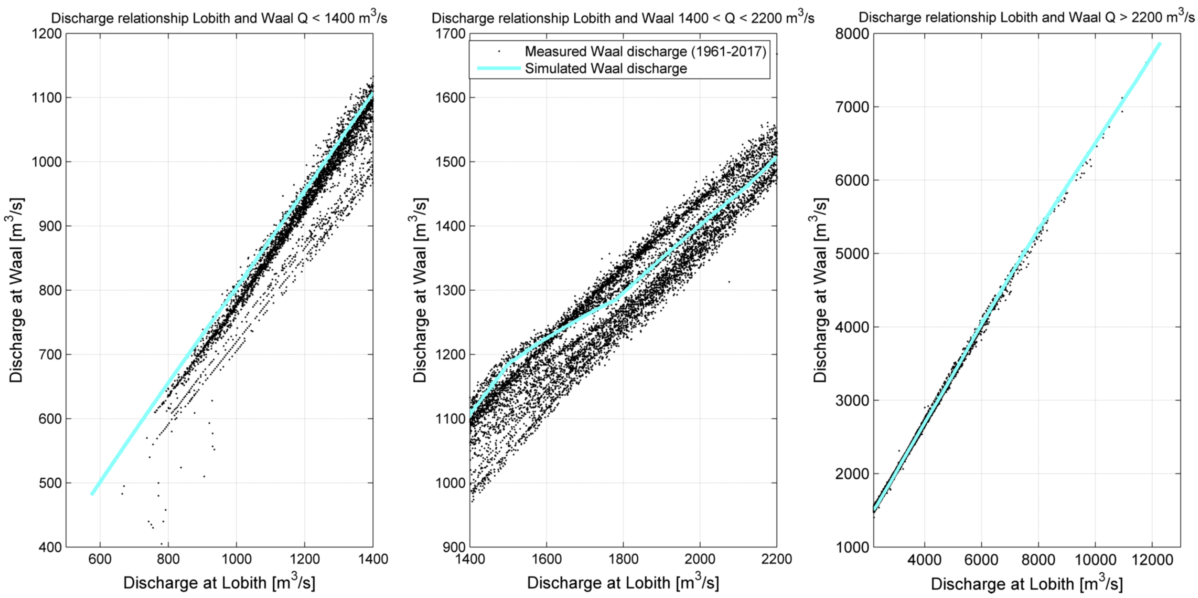


Figure 4.6: The relationship between discharge at Lobith and Waal based on daily records of the Waal (1961-2017) and simulated Waal discharges for low flow (left), regular flow (middle) and flood discharges (right).

Climate change

To assess the effects of climate change, discharge time series representing the future are required to incorporate effects of climate change. As discussed in Section 2.3 Sperna Weiland et al. (2015) calculated high and low flow conditions for the KNMI'14 scenario by generating synthetic percipation, which are subsequently translated into discharge time series. This transformation has been based on the HBV model for which input (e.g. precipitation and temperature) is available for the Rhine between 1951-2015. This input has been tuned for different climate scenarios, which allow the HBV model to simulate discharge time series. These analysis revealed an increase in winter discharge and a decrease in late summer and autumn discharge, increasing the inter-annual variability compared to the reference climate. The synthetic precipitation and temperature are generated by resampling the recordings which only date from 1951-2015, while for certain applications a 100-yearly discharge time series is desired. Hence, Mens and Kramer (2016) developed a method to generate a 100-year synthetic discharge time series for low-flow analysis of climate scenarios G_L and $W_{H,dry}$ by applying a transformation method to the missing HBV input of 1901-1950. The transformation method relies on the relation between the HBV simulated reference discharge (Q_{HBVref}) and climate discharge ($Q_{HBVclimate}$), which will be applied on the recorded discharge time series (1901-1950) to determine the climate discharge time series for the missing years (1901-1950).

Yet, a 114-year synthetic discharge time series is obtained from Mens and Kramer (2016) and will be used as will be used as input to simulate the G_L and $W_{H,dry}$ 2050 climate scenarios. Only the G_L and $W_{H,dry}$ will be assessed during low-flow conditions as these are the most relevant scenarios, respectively the driest and the wettest. However, considering (extremely) high discharges the forecasts of the climate scenarios are more similar, for which all scenarios predict higher design water levels in the future. Within this thesis the effect of climate change on flood protection is assessed based on the discharge statistics of the climate scenario G_L and G_H, as these are respectively the most extreme and least extreme scenarios (Table 2.5). These synthetic discharge time series are discharges at Lobith discharge and have to be transformed to discharges of the Waal following the same discharge distribution as discussed in the previous section.

4.4. Assessment of functional performance

While in the previous section is discussed how future water level time series along the Waal are predicted, this section elaborates on methods to assess functions performance based on these water level datasets. Chapter 2.4 provides the theoretical background of the translation of river conditions in function performance. In Section 4.2 the hydraulic indicator is chosen based on certain criteria (e.g. complexity and relevance), which already eliminates some of the indicators to assess function performance (secondary indicators) of Figure 4.2. This section will put those indicators into practice and will provide methods to quantify function performance based on the water level time series.

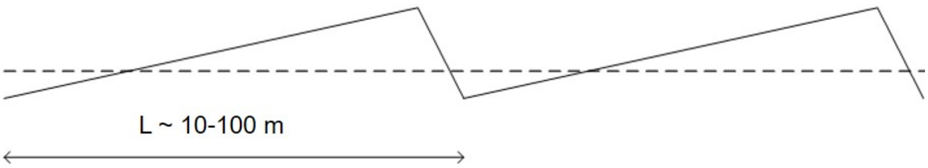
4.4.1. Navigation

Both the vertical clearance (bridge height) and water depth can be assessed. Based on expert judgement it is stated that within the selected river section vertical clearance is less of a problem than depth restriction. Hence, the assessment is limited on the water depth. As discussed in Section 2.4.1 the fairway is maintained by means of dredging to meet the required CCR fairway dimensions (2.80 x 150 m). However, at locations with structures or bed protections in the river bed, dredging becomes impossible. Within this thesis it is assumed that the required fairway profile can be dredged throughout the river section apart from the bed structures at Erlecom (rkm 873-876) and Nijmegen (rkm 883-885). Due to the non-erodible layer and the backwater effect the water depth at the bed protection of Nijmegen has been reduced in the past and is expected to be reduced in the future (Ministry of Infrastructure and Water Management, 2018). Nijmegen will likely become the bottleneck considering the water depth, which requires assessment of the CCR depth requirement and the navigability at this location (downstream of the bed protection, rkm 884.88). Since the river processes might affect the required dredging volumes in the fairway, this will be assessed along the entire Waal section.

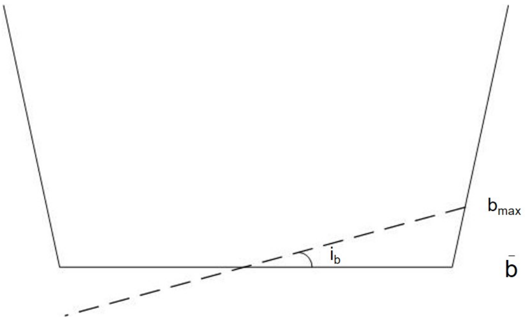
Water depth determination

The Waal model is purely one-dimensional meaning the model has predicted water levels based on width averaged bed levels of the main channel (dataset obtained from SOBEK 2013), which easily could be converted to the absolute parameter width averaged water depth ($\overline{d}$). However, width averaged water depth might be of interest for navigation, but the more locally focused LAD is of real importance for the load capacity of vessels. Also the CCR requirement of 2.80 draught at ALW is considered as the LAD instead of the width averaged depth. To differentiate bed levels in the averaged longitudinal and cross-sectional direction, corrections are required. These spatial and temporal changes in bed level are the results of morphology for which different processes can be formulated as presented Table 2.1 in Section 2.2. Within the numerical model, the large-scale width averaged bed degradation is already processed. However, to account for spatial variation, corrections are applied for transverse slopes, bedforms and non-erodible layers in the fairway (Figure 4.7).

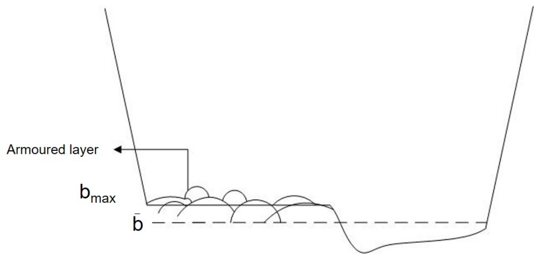
1. The **transverse slope** in river bends results in shallow inner bends and deeper outer bends possibly hindering navigation during low flow conditions (Figure 4.7b). As Van Vuren (2005) demonstrated, it is possible to account for 2D-transverse slope effects by post-processing the numerical results. The approximation for lateral bed slopes given by Struiksma et al. (1985), which will be used in this thesis



(a) A longitudinal overview of dunes on the river bed



(b) A cross-sectional overview of a transverse slope in a river bend



(c) A cross-sectional overview of the fixed layer at Nijmegen (rkm 885)

Figure 4.7: Different reasons for the fact the water depth can not be assumed to be width averaged.

relying on bed characteristics and the characteristic morphological discharge (Q_{Lobith} = 2700 m^3/s) as shown in Appendix H. The location of the fairway relative to river axis (mostly the fairway is located in the outer bed), the minimum depth can be corrected as follows (Figure 4.8):

$$d_{min}(x,t) = \overline{d}(x,t) - d_{cor}(x) = \overline{d}(x,t) + \Big(\frac{B(x)}{2} - B_S(x) - B_N(x)\Big) i_t(x) \tag{4.8}$$

in which d_{min} is the critical depth in the cross-section of the fairway [m], $d_c or$ is the transverse slope correction [m], B is the fairway width [m], B_S is the safety distance from the river bank (typically 25 m) [m], B_N the fairway width [m] and i_t the transverse slope [-].

2. **Bedforms**, such as ripples and dunes affect the longitudinal bed profile and could cause shallowness in the fairway (Figure 4.7a). Within the one dimensional approximation of the model, the bed level and so the bedforms (typical dune lenght between 10 and 100 m) are averaged over a longitudinal sections of 500 m. Yalin (1964) and van Rijn (1984) propose that the height (and also the length) of these bedforms is highly depended on the water depth. However, researchers did not succeed to define a universe relationship. Based on expert judgment two approximated relationship defined by Bradley and Venditti (2017) depending on water depth regimes (deep or shallow), will be used in this thesis:

$$\text{for water depth} > 2.5: \qquad H = 0.13h^{0.94} \tag{4.9}$$

$$\text{for water depth} < 2.5: \qquad H = 0.23h^{0.91} \tag{4.10}$$

in which H is the dune height [m] and h the water depth [m]. During flood discharges large river dunes grow enlarging the roughness of the river (van Rijn, 1984). However, these bedforms might also cause hinder during regular or low flow conditions since bedforms have a lagging character not instantaneously adapting to the water depth. This relaxation behaviour is implemented in the Delft-3D-3DMOR model based on Allen (1976), which is described as

$$H(t) = H_e - (H_e - H_0)e^{\frac{t}{T_H}} \tag{4.11}$$

in which H is the time dune height at location x and time step t [m], H_e the equilibrium dune height predicted by Bradley and Venditti (2017) [m], H_0 the dune height time step earlier [m], T_H the time scale of bedform adaptation which is described in more detail in Appendix H [s] and t the time between the previous dune height computation [s]. By means of a discharge time series the equilibrium dune height and the actual (lagged) dune height can be computed throughout the year (see Figure H.1 in Appendix H).

3. At last, the height of the **non-erodible structures or bed-layers** in the fairway is assumed to be fixed in time . Franssen (1995) reveals that the bed protection is placed in the outer bend between river kilometer 883-885 (as shown in Figure 4.7c and the original design drawings in Appendix H). However, larger vessels are not able to sail the inner bed, so the water depth in the outer bed is crucial for navigation. The top of the bed level at the downstream edge of the bed protection is elevated + 2.15 m NAP, which is the leading in determining the LAD.

Water depth at ALW
The critical water depth during ALW conditions is assumed to be located at the downstream edge of the bed protection in the Nijmegen river bend (rkm 884.88). As discussed in the previous section the downstream edge of the bed protection is elevated + 2.15 m NAP, while the bed averaged bed level is located + 2.05 m NAP. To compute the future ALW the model has to be forced by the ALD, which is defined as the discharge that is not exceeded 20 days a year (5 % exceedance probability) . This discharge is for Lobith agreed on 1020 m^3/s, which results in 816 m^3/s for the Waal conform the fixed discharge distribution.

The present ALD is computed based on discharge statistics of the previous years. Climate change is likely to change the discharge statistics resulting in a different ALD when policy-makers decide to adhere to the 5 % not exceedance criteria. When applying a data-analysis to determine the 5 % exceedance value within the historical discharge dataset (1901-2017), at Lobith a value of 1015 m^3/s is obtained (not accounted for ice days). For the two extreme climate scenarios $W_{H,dry}$ and G_L the obtained 5 % exceedance value are respectively 825 m^3/s and 1102 m^3/s (Table 4.4). The data-analysis can be reviewed in Appendix G

Case	ALW [m^3/s]
Reference	1020
$W_{H,dry}$	825
G_L	1102

Table 4.4: Considered ALW value for the reference case and the extreme climate scenarios

Navigability
The 2.80 depth requirement at ALW is just a function requirement agreed to allow efficient navigation. However, it does not say anything about the degree of navigability throughout the year. Different methods could be used to quantify the degree of navigation, such as the number of days that a normative ship has to sail with reduced loading or the load factor, which is defined in this thesis as the load capacity without restrictions divided by the actual load capacity. Both methods can be evaluated for different normative draughts. For safety reasons shippers account for a certain keel clearance between the keel and the bottom. This keel clearance is estimated to be as high as 30 cm due to the vessel's consequences of hitting the rocks of the bed protection and the fact that shippers experience a drop when sailing over the armoured layer. Based on interviews within Rijkswaterstaat a reduced loading of 120 ton per 10 cm water depth reduction is assumed. In this way the load factor of a ship can be computed as follows:

$$\text{if } D_f + k < d : \text{LF} = \frac{C_L}{(d - D_f - k) * 120} \tag{4.12}$$

$$\text{if } D_f + k > d : \text{LF} = 1 \tag{4.13}$$

in which C_L is the load capacity without restrictions [ton], d is the actual water depth [m], D_f the draught when fully loaded [m] and k the keel clearance [m]. Considering the load factor for a typical barge transporting coal or iron ore to Germany, the normative draught is 4 m and the load capacity is 2750 ton per barge (up to 6 barges). The financial consequences of reduced loading will be discussed in Section 4.5.2.

Maintenance dredging

In the previous sections the navigability at the fixed layer in the Nijmegen river bend is assessed. However, except for the fixed layer Nijmegen (rkm 883-885) and the Erlecom bendway weirs (rkm 873-876) it is assumed shallowness can be removed by dredging. Regardless, dredging activities will cause hinder for navigation and there are costs related with dredging activities (Section 4.5.3), which makes it interesting to assess the development of dredging volumes in the near future. Rijkswaterstaat maintains the fairway on a depth of 2.80 m and a width of 150 m for ALW conditions by means of dredging. The numerical model describes the bed one-dimensional, while to obtain dredging volumes a three-dimensional bathymetry is required. Therefore corrections for bedforms and transverse slope have to be applied as discussed in the Section 4.4.1. This section discusses how these corrections eventually result in an annual dredging volume. It is expected that bed degradation will change the water depth during ALW conditions over the Waal section due to relative changes between the ALW and the river bed. Climate change is most probably changing the ALD value resulting in a different ALW that has to be maintained.

In general, after the flood season (1st of May) when the discharge drops below 3000 m^3/s at Lobith, maintance dredging is undertaken to fulfill the required fairway dimensions for the low-flow season. However, morphological processes might cause new obstacles during the low-flow season requiring additional dredging activities. Van Vuren (2005) applied an analytical approach to compute the bed recovery of a transverse slope and concluded that the bed adapt in approximately 150-200 days towards the situation before dredging. As the dry season ends in November, it could be possible that new nautical bottlenecks will be formed. Bedforms might recover even faser. As dredging will be executed with an extra depth of 30 cm (Dutch: overdiepte), bed recovery is expected to cause obstacles a bit slower. However, the phenomena of bedform recovery during a low-flow season will not be dealt with in this thesis due to its complexity, while full bed recovery is assumed after a flood season again. It is assumed that the dredged sediment is not deposited again in the river (in the deeper parts), while in practice this is the case. Van Vuren (2005) revealed, deposition of dredged sediment in the Waal does not have a major impact on the dredging volumes. For every cross-section the annual dredging surface can be considered in two steps:

1. Firstly, the fairway dimensions are assessed when applying a depth correction for the transverse slope. Equation 4.8 describes how the minimum depth within the cross-section can be obtained. The amount of dredging required when fairway dimensions are not fulfilled is illustrated in grey in Figure 4.8 and described with Rooij (2005)

$$V_D(x) = \frac{1}{2}(d_1(x,t))^2 * L(x) * (\tan(i_t(x))^{-1} + \tan(\alpha - i_t(x))^{-1} \tag{4.14}$$

   in which d_1 equals $d_0 * \cos(i_b)$ [m], α the angle of internal friction of the bed material after dredging (assumed to equal 1:3) [-], L the length of the river section [m] and d_0 is defined as

$$d_0(x,t) = d_N(x) - d_{min}(x,t) + d_e \tag{4.15}$$

   in which d_N is depth requirement (2.80 m) and d_e an extra depth of 0.3 m on top of the dredging depth.

2. Next, bedforms will develop on top of the transverse slope, which might cause an exceedance of the minimum navigation channel dimensions. Due to the relaxation behaviour of the bedforms, the dune height at 1st of May varies over years depending on the discharge throughout the year. Based on the discharge time series, the dune height on the moment of dredging can be computed. This dune height is assumed to be uniform in width and is projected on the transverse slope. As the discharge time series varies per year, both a statistical and an average analysis can be conducted for the dune height and subsequently the dredging volume.

As the transverse slope could be considered rather uniform in longitudinal direction, the bedforms show a triangular pattern as shown in Figure 4.7a. As only half of the triangle is above the reference level and the surface of the triangle is equal to $A = \frac{1}{2}H_D * L_D$ with L_D the dune length, the required dredging volume is schematized as

$$V_D = \frac{1}{4} * L_D * A_D(x) * L(x)/L_D \tag{4.16}$$

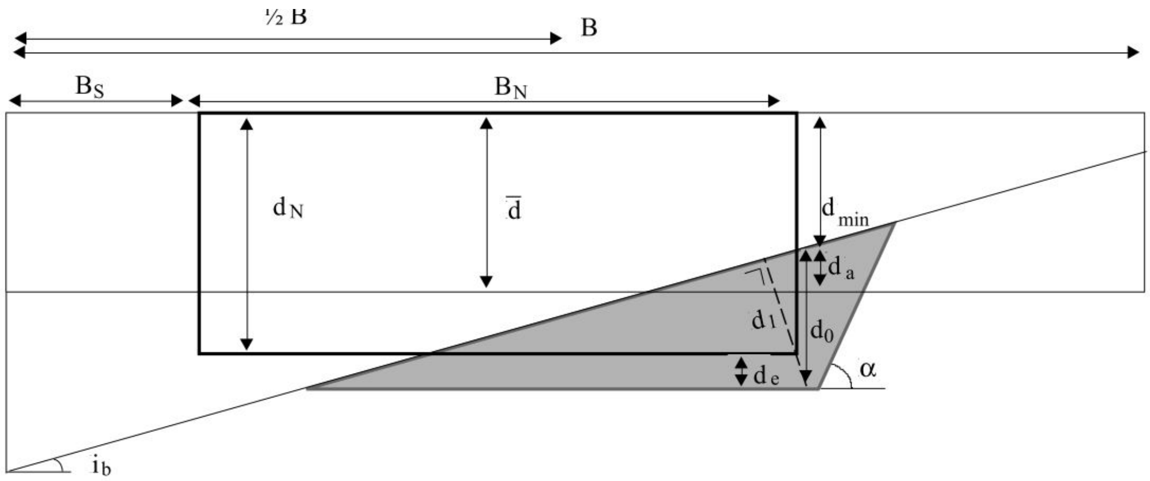


Figure 4.8: Graphical representation of the fairway with corresponding parameters and the affect of the transverse slope illustrated with in grey the dredged volumes in the fairway. Copied from Van Vuren (2005).

in which A_D is the cross-section surface area [m^2]. As can be seen the dune length disappears from this formula. This method can be repeated for every computational node (Figure 4.4), except for the sections with a fixed river bed, as it is assumed those structures stabilize the river bed.

4.4.2. Nature

As discussed in Section 2.4 flow through side channels and inundation of floodplains is important to counteract dehydration and to enhance interaction between the river channel and nature in the floodplain. Although groundwater is an important parameter for dehydration, this will not be evaluated in this research due to the complexity of modelling groundwater (soil characteristics and precipitation influence) and the lack of function requirement. Therefore, the focus of the quantification of the function nature will be based on the inundation frequency of 'objects' contributing to the nature value. The inundation frequency of these 'objects' are not covering the entire functioning of the 'objects', as the design of a side-channel or floodplain might be relevant as well (sufficient variation and presence of quiet shallow zones). Within the study area relevant objects are the side channel Klompenwaard and the recently adapted water-, and nature-rich floodplain Millingerwaard. Also the assessment of fish stairs was discussed in Section 2.4, but the functioning of the only fish stair in the study area at the sluice of the Hollands-Duits canal, does not depend on the water level in the Waal.

Frequency of flow through side channel Klompenwaard

Within our study area, Rijkswaterstaat has constructed a side channel in 1999 within the floodplain of the Klompenwaard (rkm 869) near the Pannerdensche Kop (see satellite image of Figure 2.9). Considering navigability of the main channel only a limited amount of water can be extracted by the side channel, which is at regular flow of 2000 m^3/s almost 1 % (de Jong, 2016). To regulate this, an inlet structure has been placed at the upstream edge of the side channel. Originally the inlet consisted of a threshold elevated + 12 m NAP, which allowed hardly any flow through the side channel throughout the year (approx. 30 days/yr). In 2016 the inlet has been adapted to a rectangular funnel structure (16 m^3) elevated + 7.50 m NAP, which allows the side channel to meet the objectives of the WFD. The permitted design requirement for the adaptation of the floodplain is a frequency of flow through the side channel of on average 10 months per year, equivalent to 300 days/year (Platteeuw and Lammens, 2015). So by evaluating the water level statistics at the upstream branch of the side channel, the inundation frequency can be obtained.

Inundation frequency of floodplain Millingerwaard

Recently the floodplain reconstruction of the Millingerwaard has been completed as part of the RfR Programme. Levees have been removed or lowered enabling more frequent inundation of the nature behind the original levees. Also a part of the floodplain has been adapted to allow more discharge through the floodplain during flood conditions. As the situation in 2018 shows in Figure 4.9, the floodplain consists of lots of water bodies enabling various types of organisms also requiring interaction with the river channel. On the downstream side of the floodplain (rkm 873.1) the inlet (point of inundation) has been constructed at the former access road of the brick factory the Beijer. To prevent the water bodies from dehydration the inlet is elevated +8.2 m NAP to keep water inside during low-flow conditions (Berkhof et al., 2013). As water levels tend to drop in the future, the inundation frequency of the inlet will probably be reduced. However, no function requirement has been formulated for the Millingerwaard, while it could be argued that no decline is allowed to protect the ecosystem (Natura 2000) and protected animals. To provide insights in another phenomenon the number of days without inundation is assessed.

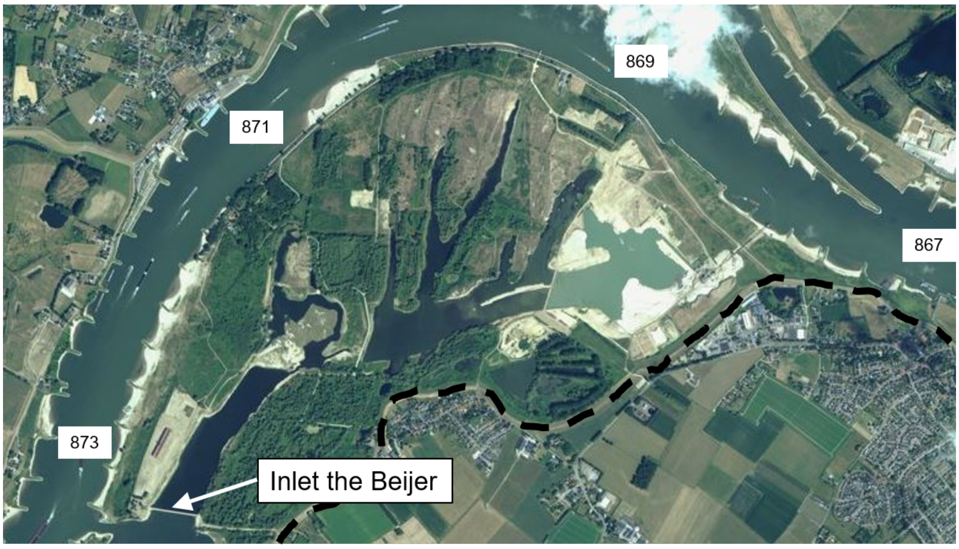


Figure 4.9: Satellite image of the Millingerwaard in August 2018 with the downstream inlet indicated (not connected to the river system at this situation) and primary dike (dotted black line). Satellite image obtained from Netherlands Space Office.

4.4.3. Flood protection

Whereas extensive studies have been conducted on the impact of climate change on flood protection, less is known about the impact of the bed trends. During low-flow conditions climate change and bed trends will most probably reinforce the impact on functions. This is not the case considering flood protection since bed degradation is expected to lower flood levels and climate change will most probable increase the flood levels. The ratio between the impact of those processes will be topic of the flood protection evaluation. Also considering the bed trends it is expected that they have a counteracting impact on the flood levels since bed degradation in the river channel lowers water levels, while sedimentation in the floodplain increases flood levels. The river processes might also have an impact on the discharge distribution. Uneven bed degradation in different Rhine branches, results in attraction of more water into a certain branch (i.e. faster degrading branch) affecting flood levels. This might result in higher design flood levels than the original design flood levels. However, this phenomenon will not be considered in this study, which will assume a fixed future discharge distribution.

Flood protection is a function of the strength of a failure mechanism and hydraulic load. As discussed in Section 2.4, for every dike trajectory a probability of flooding (target reliability) is assigned, which is distributed again over different failure mechanisms. The dominant target reliability of the dike sections in the study area is 1/10 000, while the southern dike trajectory from Nijmegen to Millingen has a target reliability of 1/3000. As Section 2.4.3 described for overflow and geotechnical failure the design water level is rather straightforward, as it the water level associated with the target reliability of the dike trajectory (see Appendix **??** for derivation design water level for overtopping). As 1/10 000 year is the dominant target reliability in the study area, the associated flood level will be evaluated in this research. The associated flood level is hereafter referred to - in Dutch - as MHW (Maatgevend Hoogwater or - in English - as Design Water Level (Dutch: Maatgevend Hoogwater) (DWL). A discharge with an exceedance probability that equals the target reliability (also called design flood discharge) has to be simulated. As the design flood discharge is commonly given at Lobith, a fixed discharge distribution is assumed.

There are different ways to derive the design flood discharge varying in derivation approach and other assumptions (e.g. will German dikes breach or will sandbagging be applied). Within Rijkswaterstaat the discharge statistics for designing flood defences are established in the OI (flood defence design instrument of Rijkswaterstaat). However, the KNMI'14 climate scenarios are not yet implemented in the OI, which means the OI currently operates with the KNMI'06 climate scenarios. Since the most recent KNMI'14 scenarios are used for the low-flow conditions, these scenarios are also preferred for the flood conditions. For this reason the discharge statistics from Sperna Weiland et al. (2015) are used within this study. However, the absolute values of the discharge do not really matter since the absolute values of the flood levels will not be analysed within this thesis, while the differences between now and the future (including bed trends and climate change) will provide insights. As presented in Table 2.5 the design flood levels are expected to increase due to an increase in design flood discharge with climate scenario G_L as most extreme scenario and G_H as mildest scenario. The considered design flood discharges are presented in Table 4.5.

Discharge statistics	Design flood discharge [m^3/s]
Current or reference	15 270
G_L 2050	17 180
G_H 2050	17 980

Table 4.5: The considered design flood discharge associated with the dominant target reliability of the study area. Obtained from Sperna Weiland et al. (2015).

4.5. Financial analysis

4.5.1. Introduction

Figure 4.1 shows the conceptual model of this research, also indicating a feedback loop from the impact on river functions to the measures that can be taken by the river manager. The methods discussed in the previous section provide quantitative information on the functional performance of individual functions. To obtain a universal parameter, money can be used to express functional performance of river functions in a market value (i.e. the net market value over a certain time period). However, the river function nature might be difficult to quantify in money, while other river functions as navigation or flood protection have a clear link with economics. For this reason a financial assessment will be developed without incorporating the affect of nature. Different methods will be applied to quantify aspects of the river functions flood protection and navigation in an economic value. Also the costs of a nourishment will be estimated to asses whether a nourishment is profitable based on the incorporated costs.

4.5.2. Economic consequences reduced loading

Efficiency is reduced when ships have to sail with reduced loading resulting in higher transportation costs. Although, shipping companies might profit from reduced loading since they have to sail more trips resulting in a higher turnover, the higher transportation costs are eventually charged on society due to the increased cost price of goods. Many methods could be applied to calculate the economic consequences of reduced loading. Appendix J.1.1 shows the application of a method developed by Taekema (2017). In this thesis the approach of Jonkeren et al. (2007), modified by Flierman (2017), will be followed. Jonkeren et al. (2007) developed a method to calculate the price and quantity changes due to reduced water depth. This method is based on the principles of the economic concept of surplus assuming a fully saturated market where the price for which transport is supplied is equal to the production or operation cost (C_0). Jonkeren et al. (2007) uses supply and demand functions describing the price per unit as a function of the transported load. When the load capacities of vessels are reduced due to water depth restrictions, the new cost increases. The demand function (D) gives the maximum amount of products q the consumer wants to transport for a price p as shown in Figure 4.10. The slope of the demand function is called the elasticity of demand ϵ, which describes consumer behaviour as a results of change in the price.

When the equilibrium price is lower than the price the consumer is wiling to pay, the consumer benefits which is called the consumer surplus. Looking at the supply function the producer surplus is found, which is equal to zero in this case since the supply-function is assumed to be constant. The wellfare loss (WL) is the difference in surplus between the unrestricted and the restricted situation described as

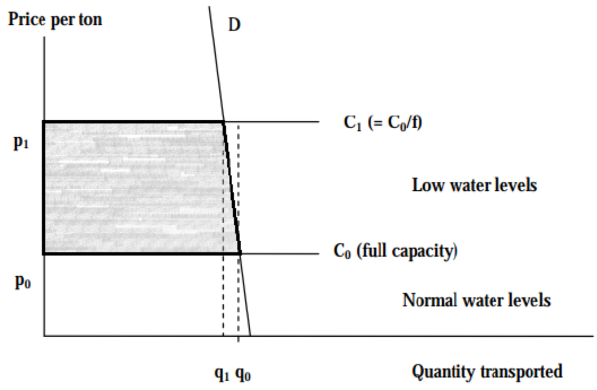


Figure 4.10: Demand function of transported load. Adapted from Jonkeren et al. (2008)

$$WL = (p_1 - p_0)q_0\Big(1 + \frac{\epsilon(p_1 - p_0)}{2q_0}\Big) \tag{4.17}$$

in which p_0 is the unrestricted price [€/ton], p_1 the restricted price [€/ton] and q_0 the unrestricted quantity of transported goods [ton]. Jonkeren, O. (2009) found an average elasticity of demand of -0.6 with a standard deviation of 0.27 by applying a regression analysis with data from the German Rhine near Kaub. It is assumed that the findings of Jonkeren, O. (2009) are also applicable for the Waal near Nijmegen due to the fact that it concerns the same geographical area (Rhine).

Jonkeren conducted several studies on the wellfare loss due to restricted water depth at Kaub (Germany) (Jonkeren et al., 2007; Jonkeren, O., 2009, e.g.). However, no studies have been applied for the Waal at Nijmegen. Flierman (2017) applied the principles of Jonkeren et al. (2007) for Nijmegen to compute for the entire fleet passing Nijmegen the load factor and subsequently the restricted price by means of

$$p_1 = \frac{p_0}{LF} \tag{4.18}$$

When the load factor drops too much, transport over the Rhine becomes infeasible for consumers (i.e. industries) due to the increasing price. However, to prevent consumers from choosing another way of transport, shipping companies offer a lower price to have their ships sailing. This means the price becomes unchanged when the load factor drops below a certain value, this is estimated to be the case at a load factor of 0.3. Flierman (2017) calculated the extra costs of the restricted water depth for different vessels with the shipping model BIVAS (see Figure 4.11) for which she applied an exponential fit with the following equation

$$\frac{p_1}{p_0} = 22.89\exp^{-1.47d} + 0.80\exp^{0.041d} \quad \text{and} \quad r^2 \quad \text{is} \quad 0.98 \tag{4.19}$$

Based on Equation 4.19 and the daily simulated water depth the price change can be computed. Flierman (2017) estimated the initial price from historical data to be €8.76/ton and calculated the wellfare loss for individual vessels passing Nijmegen. However, such an extensive database of vessels passing Nijmegen has not been available within this study and due to the limited scope pragmatic assumptions of the transported load are made. The annual transported load by IWT at Lobith is approximately 39 % of the total transported load (Volker and Volker, 2015). Taekema (2017) revealed, based on a vessel journey-database (IVS-90), that the average load passing Nijmegen equals 90 % of the load passing Lobith. For simplification, it is assumed that the entire fleet is loading their vessels based on the water depth at Nijmegen. To compare the results of the wellfare loss accounting for bed degradation and climate change, no growth of the transported load is assumed. When evaluating the latest data of transported load by IWT in the Netherlands of 2017, (CBS, 2018) it is assumed that 128 million ton passes Nijmegen. The model does not account for future up-scaling of the fleet's load capacity, possible changes in annual transported load over the Waal nor changes in the transportation price per ton.

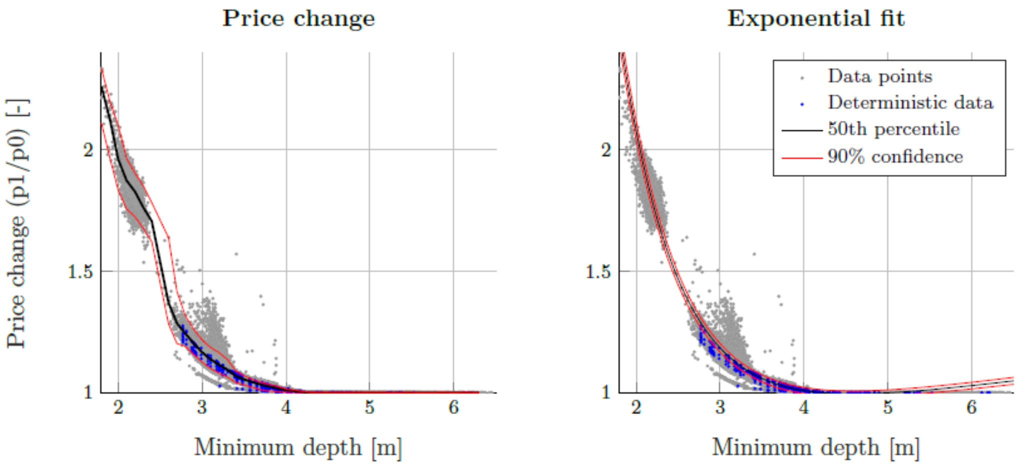


Figure 4.11: The average price change related to the minimum depth in the study area (left). The same data exponentially fitted (right). Copied from Flierman (2017)

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- The exponential fit might not hold when fleet composition, destinations and transported load is changing or when shallowness in other branches becomes crucial.
- 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.
- When this method is applied to justify a measure to the river bed to improve navigability, this method computes the 'international' wellfare loss instead of only the Dutch.
- It is assumed that the transported load is uniformly distributed throughout the year.

Jonkeren et al. (2008) found a yearly expected wellfare loss for reduced water depth at Kaub (Germany) of €28 million, while the result acquired in this research using Flierman (2017) exponential fit with the simulated water level is equal to €24 million.

4.5.3. Maintenance dredging costs

In the Netherlands maintenance dredging is executed by dredging companies hired by Rijkswaterstaat. Mostly, these companies work with a performance contract, which means the company must guarantee the required depth. Due to this type of contract, the costs per cubic meter are not defned formally. Flierman (2017) explains the actual dredging costs depend on volume and transported distance, for which she found an average of €3.41/m^3 in situ (bandwidth €2-€20/m^3). As the dredging volumes are quantified in Section 4.4.1, this can also be translated in costs.

4.5.4. Costs flood protection

As the design flood levels are elevated, a heightening of the flood defence is required by means of the current Guidelines of River Engineering (Rivierkundig Beoordelingskader) (Kroekenstoel, 2017). Currently, many flood defences are reinforced due to the new flood defence standard or/and are heightened as a result of the climate change assignment (Jorissen and Kraan, 2017). Many studies has been conducted on the required heightening of flood defences in the HWBP, which will not be incorporated in the cost-benefit analysis of this thesis. de Bel (2014) analysed the cost efficiency of different measures that lowers flood levels (i.e. increases flood conveyance). The water level reduction is quantified in m^2, which varies for the Waal between €10 000/m^2 and €100 000/m^2. The costs for dike heightening considering the assignment of climate change is estimated on €55 000/m^2. However, the HWBP assignments require construction on the flood defences anyway, which allows linking of the activities and reducing the costs. Eijgenraam (2005) estimated those costs in general as a function of

$$I(u) = (c + bu)\exp^{\lambda u} \tag{4.20}$$

with I the total costs of the dike heightening per kilometer [€], u the height of the elevation [cm] and λ, b and c are constants varying per dike trajectory. However, it is assumed that only the linear part will be incorporated in this thesis, since fixed costs can be connected with the HWBP. For the dike trajectories of the study area these costs per cm (b) are equivalent to €20 000/cm/km.

4.5.5. Nourishment costs

Kabout and Deltrap (2017) developed a cost estimations for different types of nourishments, which is used for the Research Sustainable River Bed Rhine branches (Ministry of Infrastructure and Water Management, 2018). The cost estimation of the construction costs consists of different aspects, such as material costs, placement costs, indirect costs and additional risk costs. Table 4.6 illustrates the costs per cubic meter for different materials, In thesis, it is assumed that sand will suffice as nourishment material.

Material	Costs per cubic meter [€]
Sand	32
Gravel	71
Tout-Venant	47

Table 4.6: Nourishment construcion costs estimation of different materials based on a nourishment volume of 30 000 m^3. Data obtained from Kabout and Deltrap (2017).

However, these are the costs estimated for a nourishment volume of 30 000 m^3, which is just a fraction of the volumes required for the selected nourishment strategies. As discussed in the previous section, the required volumes for 'nourishment 2010' and 'nourishment 1997' within the river section are respectively 0.77 million m^3 and 2.02 million m^3 (Table 3.1). However, it is assumed that the downstream boundary conditions is also elevated, requiring a nourishment further downstream. It is assumed that a nourishment till Tiel (rkm 915) is required, which eventually results in an increased volume of respectively 1.4 million m^3 and 3.6 million m^3 (elaboration in Appendix J.4). Based on interviews with Rijkswaterstaat cost experts, a correction factor of 0.55-0.60 can be applied to calculate the costs of a nourishment of 1.4 million m^3.

As stabilization is required, a smaller volume has to be nourished once in a while. Table 4.2 estimates the erosion within the river section, while also downstream of the river section erosion will be present. As bed disturbances travel downstream (Mosselman et al., 2008, e.g.), morphological experts of Rijkswaterstaat assumed that the nourishment will travel downstream at a rate of 1 km/year. This volume has to be refilled at the upstream edge of the nourishment. Eventually this results in a yearly stabilization volume of approximately 140 000 m^3 (Appendix J.4). A correction factor of 0.8-0.85 will be applied to calculate the nourishment price from Table 4.6. The costs per cubic meter for different nourishment volumes are estimated in Table 4.7.

The costs of a nourishment with material from the floodplains is more difficult to estimate. Based on an interview with costs experts within Rijkswaterstaat, a costs range is estimated. The location and size of the extraction operation is an important variable of the costs, as a more compact deeper sand pit will be cheaper than extraction from the entire floodplain. When assuming a deep gully in the floodplain of the consider volumes of Table 4.7, the costs of extraction and supply of the material for a nourishment is estimated between €20-30/m^3 (including removal of 1 m useless soil, transport, excavation, construction estate). This are only the costs of the material, while Kabout and Deltrap (2017) defined a cost estimation model in which the material costs is just a share of the total costs. The material costs of sand from outside the study area is estimated €10/m^3 (Kabout and Deltrap, 2017), while on the other costs of the cost model a correction factor has been applied due to the scale of the nourishment.

Another strategy was to extract more than the required sediment from the floodplain to directly compensate for the increased flood levels. This material could be sold on the market. However, sand is a demand-driven product, which makes it difficult to sell large quantities for a fixed price. A best estimate of the selling price of a cubic meter sand is €0.50-2.50. Based on the assumptions of this paragraph, a price per cubic meter is estimated for the different nourishment strategies in Table 4.7.

Strategy	Volume [mln m^3]	Nourishment costs [€/m^3]
'Nourishment 2010'	1.4	17.6 - 19.2
'Nourishment 1997'	3.7	16.0 - 17.6
Sediment from floodplain	1.4 / 3.7	44 - 62
Yearly stabilization	0.14	27.2 - 28.8
1/5 year stabilization	0.69	22.4 - 24.0
1/20 year stabilization	2.77	17 - 18

Table 4.7: Nourishment cost estimation based on assumptions listed in Section 4.5.5.

4.5.6. Discounted value

In most civil engineering project a discount rate is used to account for the loss of value of money over time. As a result of the scarcity of means postponing or advancing a payment has a value (Vrijling and Verlaan, 2015). This is indicated by the time value of the money. The discounted value depend on an discount rate according the formula:

$$C_d(t) = C(t)(1+r)^{t_0-t} \tag{4.21}$$

where C_d is the discounted costs [€], C the actual costs [€], t_0 the moment of recognition and r the discount rate [%]. A similar approach is used for benefits. This discount rate is an important parameter in the determining of the future value of money. An increased discount rate means the value of money decreases faster in time. Flierman (2017) denoted the discount rate of 2016 was 3%, which will also be used in this thesis.

The net market value method sums the cash flows over the life time cycle of the project with due regard for the time value of the money. The net market value of a future river system without interventions can be compared with a river system with measures, such as nourishments. The net market value is described as follows:

$$NPV = \sum_{t=t_0}^{t_0} C(t)(1+r)^{t_0-t} \tag{4.22}$$

4.6. Methodology overview

The methodology consists of many tools and methods that together form the assessment methodology of future functional performance. This section will summarize the workflow of the methodology and position the described tools and methods. Figure 4.12 illustrates the methodology setup with the interconnections between models and an overview of the model in-, and output. The imposed inflow boundary vary per analysis, as for the ALW and DWL only the corresponding discharge will be simulated, while for statistical analysis, such as inflow of side channels or load factor of vessels, a discharge time series is imposed. Either discharge value(s) of the actual (reference) discharge statistics or one of the KNMI'14 climate scenarios will be imposed as an inflow boundary. As Section 4.3.1 elaborates a minimum and maximum scenario of the bed degradation is defined, which can be imposed as input resulting in different results.

The model generates results every 5 years with the reference statistics. However, as the adapted climate scenario discharge time series, ALD or design flood discharge are only representative for the year 2050, only an analysis in 2050 will be conducted. Subsequently, an interpolation between 2018 with reference discharge value(s) and 2050 with adapted climate discharge value(s).

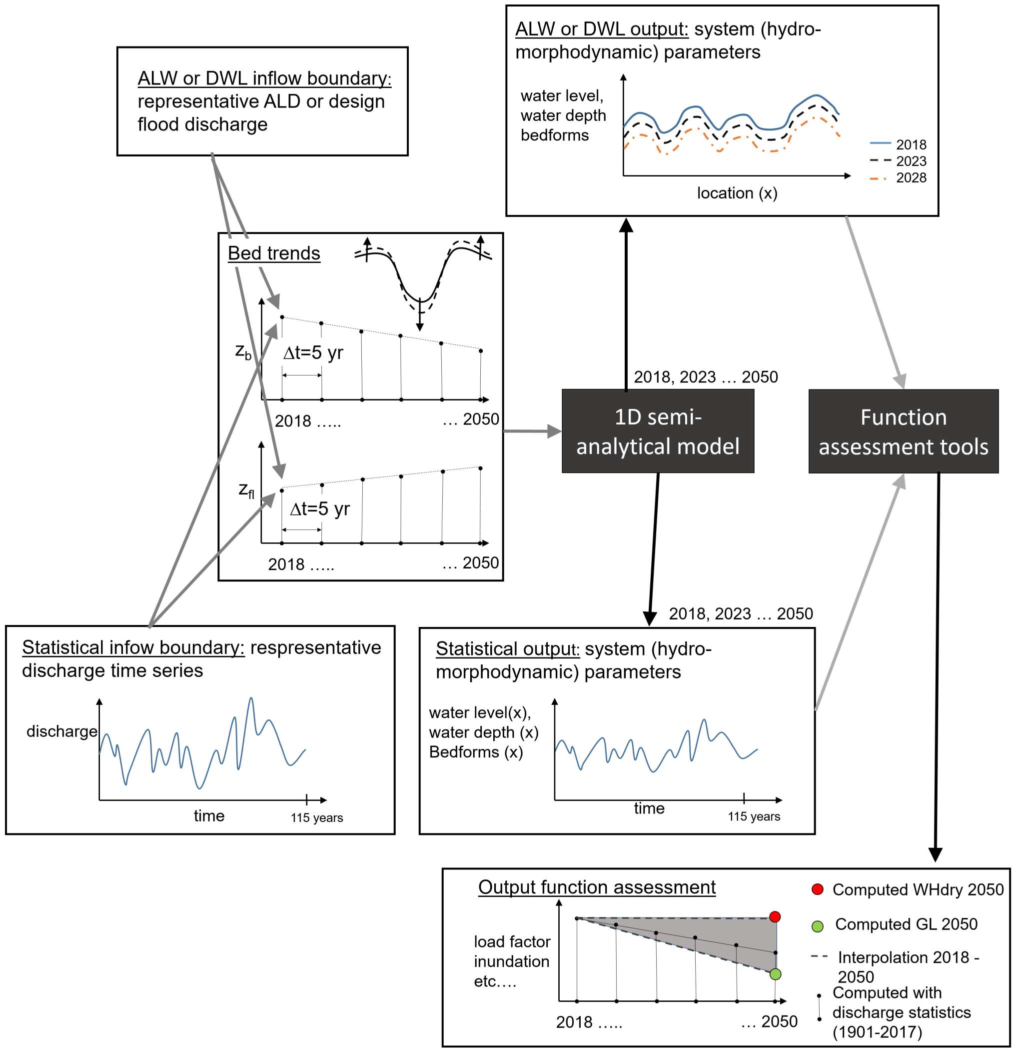


Figure 4.12: Overview of the methodology setup illustrating the input and output.

5
Application of the methodology

5.1. Introduction

This chapter will elaborate on the application of the assessment methodology to the functional performance of the considered Waal river section accounting for autonomous trends. At first, Section 5.2 will discuss a river system without intervention measures nor stabilization of the river bed, which enables evaluation of the urgency and importance of mitigation of processes, such as bed degradation. Furthermore, the relative importance of bed degradation and climate change impacting the river functions will be assessed. Also a cost-benefit analysis of a future river system without stabilization will be conducted. Subsequently, Section 5.3 elaborates on the impact of various sediment management strategies, also evaluating the cost-effectiveness of such a measure. By means of application of the methodology the following research questions will be addressed in this chapter:

What are the autonomous developments in the Rhine and how are these affecting the river functions?

How can the performance of river functions be quantified and provide useful information for an assessment of integrated river management?

Which aspects of the assessment process are important for decision-making?

As described Chapter 4, the bed degradation is imposed on the one-dimensional bed levels, affecting the simulated water levels with an unchanged discharge. This procedure is illustrated in Figure 5.1, showing as an example the degrading bed levels and lowering ALW, while the same approach can be applied for other discharges.

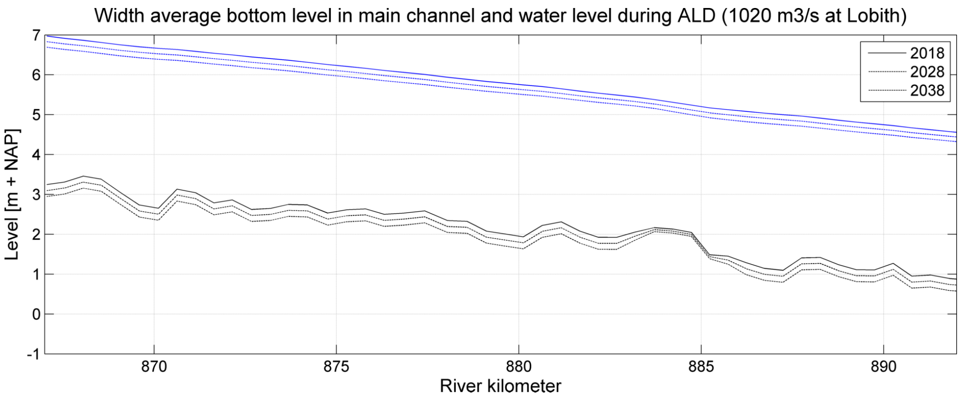


Figure 5.1: The model results of the actual ALD (1020 m^3/s) the 1D (averaged) main channel bed level (black) and water levels (blue) of 2018, 2028 and 2038.

5.2. Future river system without stabilization

This section will analyse the results of the assessment methodology applied to a future river system without any stabilization interventions from 2018 untill 2050. In this assessment, the impact of river processes on river functions is analysed and quantified following the tools discussed in Section 4.4. As already denoted in Chapter 2, the autonomous trends are highly uncertain. For this reason a base case scenario is defined and an estimated bandwidth is composed from the estimated minimum and maximum of the impact of bed degradation, climate change and the combination of both processes:

- **The base case** is the actual representative discharge statistics based on discharge recordings between 1901-2017, hereafter referred as reference discharge statistics, combined with extrapolation of the observed bed erosion, which vary over the longitudinal axis from 0.5 to 1.5 cm/year (Section 4.3.1).
- **Estimated bandwidth bed degradation** - extrapolation of the observed trends is assumed to be the expected bed degradation. However, Sieben (2009) revealed a degradation rate of 3 cm/year between 1950-2000 in the Upper Waal has been taken place, while the erosion trend could also stop as is the case in the Upper-Rhine. For this reason an estimated bandwidth is defined between no erosion and doubling of the bed degradation rates with the reference discharge statistics.
- **Estimated bandwidth climate change** - Sperna Weiland et al. (2015) has analysed the impact of climate change on the future discharge statistics. For low-flow conditions it appears that $W_{H,dry}$ (driest) and G_L (wettest) are the most extreme climate scenarios, while for the flood season G_L (wettest) and G_H (driest) are the extremest scenarios. The boundaries of the estimated bandwidth of the impact of climate change is defined as the extremest (driest and wettest) climate scenarios with the expected (base case) bed degradation.
- **Estimated bandwidth combined effect** is the driest climate scenario with the strongest bed degradation scenario and the wettest climate scenario combined with stabilisation of the current river bed.

Based on these scenarios the impact of the processes is analysed within this section. Despite the floodplain sedimentation rate is also highly uncertain, this uncertainty will not be considered in this research and a uniform floodplain sedimentation of 0.5 cm per year is imposed.

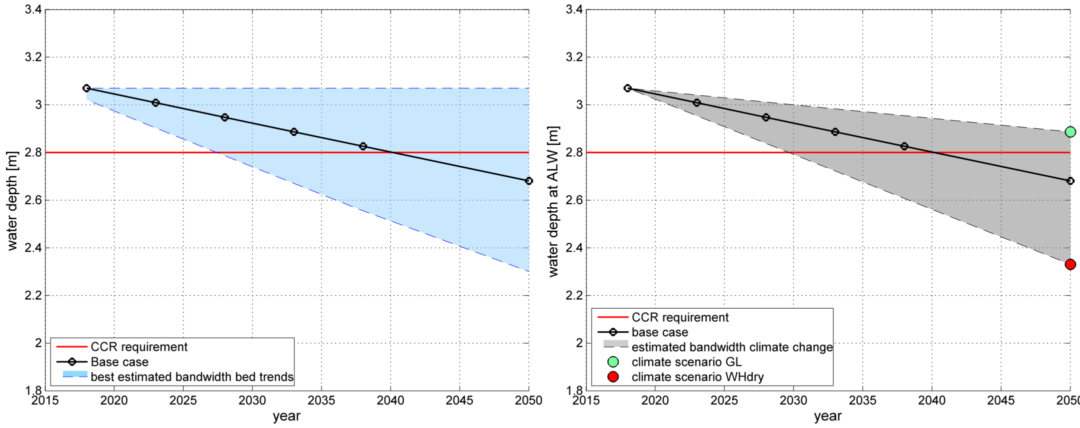
5.2.1. Consequences for navigation

Water depth during ALW

Before evaluating the navigability of a typical Rhine vessel throughout the year, the water depth during ALW at the fixed layer of Nijmegen is evaluated. Figure 5.2a illustrates the future impact of bed degradation on the water depth at Nijmegen till 2050. The model predicts, that the CCR fairway dimensions are not fulfilled at Nijmegen in 2040 with the expected bed degradation rate, while for a more severe bed degradation (approx. 3 cm/year) the requirement is already exceeded in 2027. This shows that the magnitude of the bed degradation is of great importance for low-flow water depths, as the water depth more or less responds one-to-one to the bed degradation at Nijmegen. The floodplain sedimentation does not play a role during low-flow conditions, since it does not contribute to the conveyance capacity.

When considering the effect of climate change, two climate scenarios at the time horizon 2050 are considered $W_{H,dry}$ and G_L, respectively the driest and the wettest scenario. As described in Section 4.4.1, the ALD corresponding with the climate scenarios in 2050 are determined by means of a data-analysis. The changed ALD in combination with the base case bed degradation results in a bandwidth in water depth between the driest ($W_{H,dry}$) and the wettest (G_L) scenario as illustrated in Figure 5.2b. The bandwidth of the climate scenarios is linearly interpolated between 2018 and 2050, as indicated in grey in Figure 5.3, revealing that the water depth due to a dry climate scenario and the expected bed degradation will not suffice in 2030. It appears that the $W_{H,dry}$ scenario reinforces the effect of bed degradation and even less water depth can be guaranteed during ALW (less than 2.4 m in 2050). However, the wettest scenario G_L will counteract the bed degradation and guarantees a water depth 2.8 m in 2050.

Figure 5.3 shows the combined effect of climate change and the bed degradation scenarios. Appealing is also the fact that a dry climate scenario ($W_{H,dry}$) with the base case erosion trend and a case with extreme bed degradation show a similar water depth in time horizon 2050, approximately 2.3 m. This reveals that the impact of the bed degradation is similar to the impact of a dry climate scenario.



(a) Base case and the estimated bandwidth bed degradation.

(b) Base case and the best estimated bandwidths climate change.

Figure 5.2: Water depth development at Nijmegen during ALD.

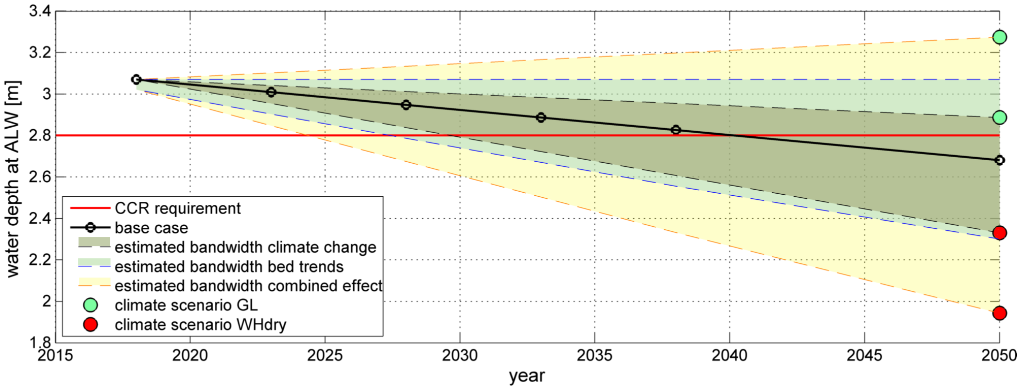


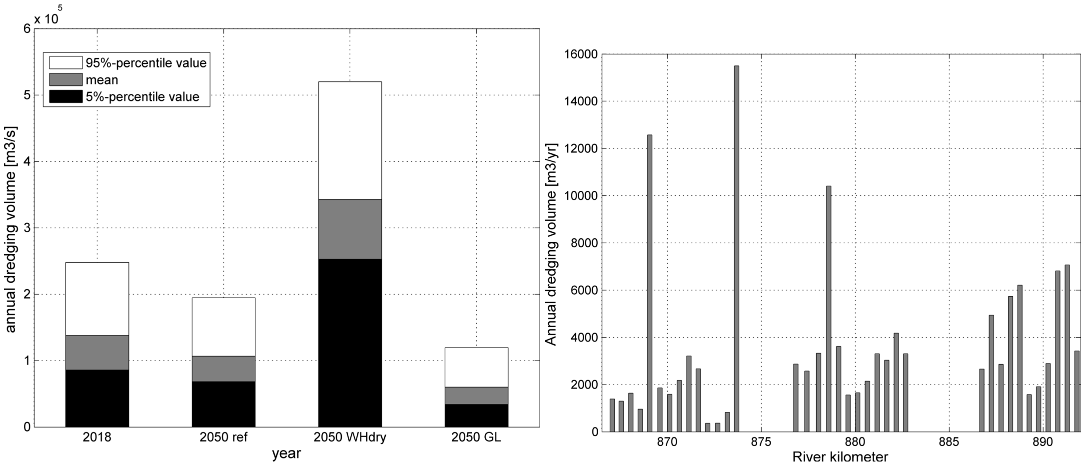
Figure 5.3: Base case and the best estimated bandwidths of bed degradation, climate change and the combined effect.

Annual maintenance dredging volume

To fulfill the required navigation channel dimensions, annual maintenance dredging is required. As the navigation channel dimensions are changing in time due to bed degradation, as is observed in Figure 5.2a, the dredging volumes might change as well. At the fixed layer at Nijmegen a reduced depth is observed, while this is not the case in the remainder of the river section. When the decrease of bed degradation and water level is taking place at the same rate, the depth remains unchanged. As the river bed of the fixed layer is degrading slower than the bed upstream of the fixed layer, water levels upstream of the fixed layer are degrading slower than the river bed due to the backwater effect. This phenomenon results in an increased water depth upstream of the fixed layer, requiring less dredging activities.

Based on the method described in Section 4.4.1, the required annual dredging volumes over the considered rivers section can be assessed. The alluvial bed is corrected for the 2D phenomena: (i) transverse slope which is developed based on a characteristic morphological discharge, while the (ii) bedforms are developed based on the discharge forcing throughout a year. For instance, flood discharges before the low-flow season result in high dredging volumes. The statistical spreading induced by the annual variety (within the discharge time series) is presented in Figure 5.4a, showing a 90%-confidence interval of the required annual maintenance dredging between 80 000 and 250 000 m^3 with a river bed of 2018. As already discussed in Section 4.4.1, this approach is based on several assumptions, which could deviate from the actual measured dredging volumes. The dredging volume measurements between 2014-2017 for the section Pannerdensche Kop - Nijmegen (rkm 867-883) vary between 60 000 and 150 000 m^3. However, these values can not be compared one-to-one, as our river section stretches 8 km longer and in practice river dunes might be pushed away (like a bulldozer) instead of dredged (and not measured). Figure 5.4b shows the spatial spreading of the dredged volumes by presenting the averaged annual dredged volume per 500 m. At the fixed bed structures at Nijmegen (rkm 883-885) and Erlecom (873-876) no dredging is taken place, as is observed in Figure 5.4b. Upstream of the river bend of Erlecom and in the river bend near the Millingerwaard two peaks are observed, which contribute considerably to the total dredging volume.

To analyse the trend and to reveal the relative importance of the autonomous trends the development of the mean annual dredging volume is presented in Figure 5.5. A decrease of the dredging activities due to bed degradation is observed (mean dredging volume of 140 000 m^3 per year in 2018 compared to 110 000 m^3 per year in 2050), while a dry climate scenario shows an enormous increase of the required dredging effort counteracting the decrease due to bed degradation. In case of a dryer climate scenario, an increasing effort is required to fulfill fairway dimensions, as the ALD (and ALW) is expected to decrease considerably (Section 5.2.1). The mean dredging volume is increased to 350 000 m^3 per year in 2050, while also the 95^{th} percentile value is even increased above 500 000 m^3 per year. In case of the milder scenario G_L the required dredging effort will be reduced to only a halve of the dredging effort of 2018 (mean 60 000 m^3 per year). to the reference ALD due to the elevated ALW .



(a) Statistical properties for all simulated years for situation 2018 and 2050 accounting for base case erosion trend and climate change.

(b) Annual dredging volume per 500 m for river bed of 2018.

Figure 5.4: Annual maintenance dredging volume in the Waal river section

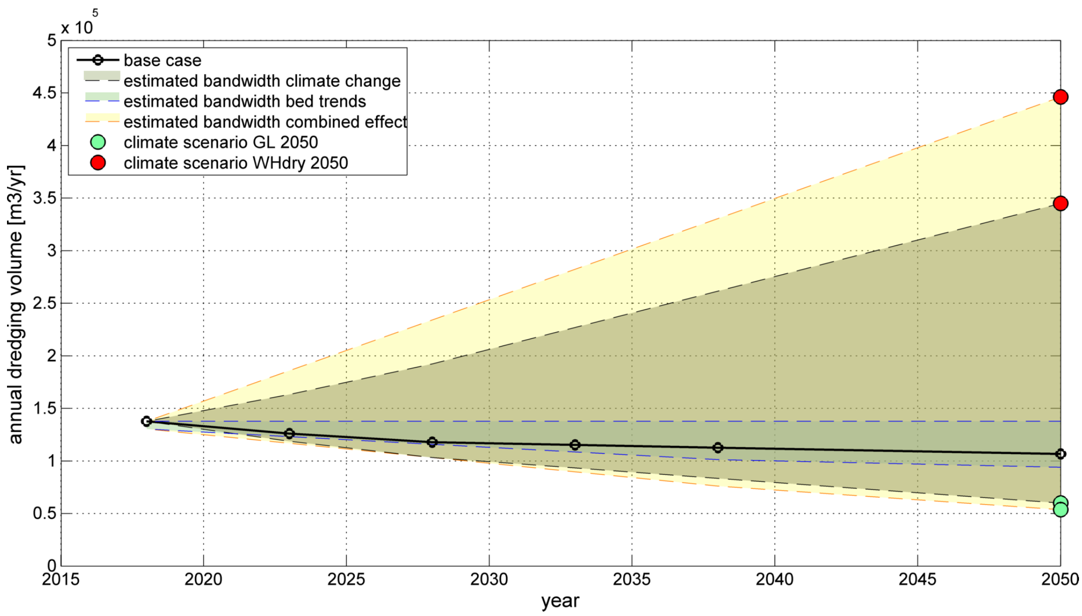


Figure 5.5: Development of the annual averaged maintenance dredging volume based on corrections for bedforms and transverse slope

Maintenance dredging costs

Based on the results of Figure 5.5, the mean annual maintenance dredging costs can easily be calculated by multiplying the annual volumes with €3.47 /m^3 (Section 4.5.3). Table 5.1 (column two), shows the results of the annual dredging costs in 2050 of the base case and the various scenarios. To allow a cost-benefit analysis of various aspects the net market value of the dredging activities has to be calculated. This will be done for the period 2018-2050 compared to the reference situation of 2018. This means that a reduction of the dredging costs is accounted as a benefit (positive net market value)in 2050, while an increase in dredging costs is accounted as a cost (negative net market value) in 2050 (see Table 5.1). The net market value is evaluated following:

$$C_d = \sum \frac{C_t - C_{2018}}{(1+r)^{t-2018}} \tag{5.1}$$

in which C_d is the discounted net market [€], C_t the value in year t [€] and C_{2018} the value in 2018 [€]. The third column of 5.2 shows the market value of all scenarios.

Scenario	Dredging costs in 2050 [mln €/yr]	net market value of all costs within 2018-2050 [mln €]
Base case	0.37	1.3
No erosion	0.47	0
Doubled erosion	0.33	1.5
$W_{H,dry}$ and base case erosion trend	1.20	-6.3
G_L and base case erosion trend	0.21	2.4
$W_{H,dry}$ and no erosion	1.55	9.4
G_L and doubled erosion trend	0.18	2.6

Table 5.1: Mean annual maintenance dredging costs in 2050 and net market value of all costs between 2018-2050 compared to situation in 2018.

Navigability typical Rhine vessel with varying discharge circumstances

The determination of ALW water depth only describes the fairway dimensions during ALD, while navigation is taken place under varying discharge circumstances. The evaluation of the navigability of a typical barge transporting coal and iron ore to Germany, will be assessed based on discharge time series. The effectiveness of navigation of a typical barge will be expressed in a load factor, which is the factor expressing ratio between the actual loading and the loading capacity without depth restrictions (1 is fully loaded). The load factor is determined based on the water depth at Nijmegen. The load capacity is a parameter that varies throughout the year depending on the discharge (Appendix I.1.1). Based on the results of the daily discharge recordings of 115-years, the statistical properties of the navigability at various load factors can subsequently be determined. The cumulative distribution of load factors based on the discharge time series is presented in Figure 5.6, revealing a shift in the load factor statistics with more often low load factors.

Figure 5.7a illustrates the navigable percentage for three years for various load factors. From this figure, the statistical characteristics of the percentage of the navigable time per year can be constructed from the simulation of the base case in 2018, as presented in Figure 5.7b. This indicates that the average percentage of the navigable time for ships fully loaded is 70%, while the figure also shows there is 90% probability that the percentage of navigable time of a ship fully loaded lies between 28% and 96%. This is readily computed from the difference between the 5^{th} and the 95^{th} percentile values. However, to analyse the changes due to climate change and bed degradation the statistical properties have to be compared. Figure 5.8a assesses the statistical character of the annual percentage of the navigable time with a load factor that equals 1 (fully loaded). From this figure it can be observed, that the average time a ship navigates fully loaded, will drop from 70% to 53% and the 90%-confidence interval will lie 15% between 85%, while the predictions for a dry climate are even a stronger decrease in navigable time. Figure 5.8b evaluates the sailing time with a load factor greater than 0.4, resulting in higher percentages (mean 97% in 2018). However, as low-flow conditions become more frequent and more marked due to bed degradation and climate change, the navigable time with a load factor of minimal 0.4 will also drop considerably in 2050 to 92% without climate change and 83% for $W_{H,dry}$. This means a vessel described in this section will experience more and more problems in 2050 when the impact of bed degradation and climate change is reinforced. This could eventually result in higher transportation costs

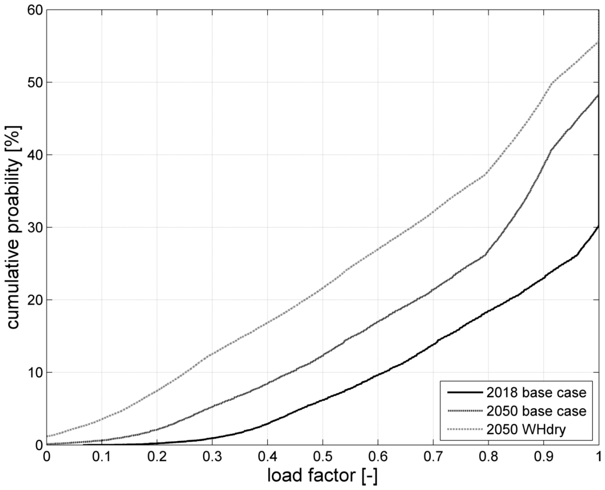
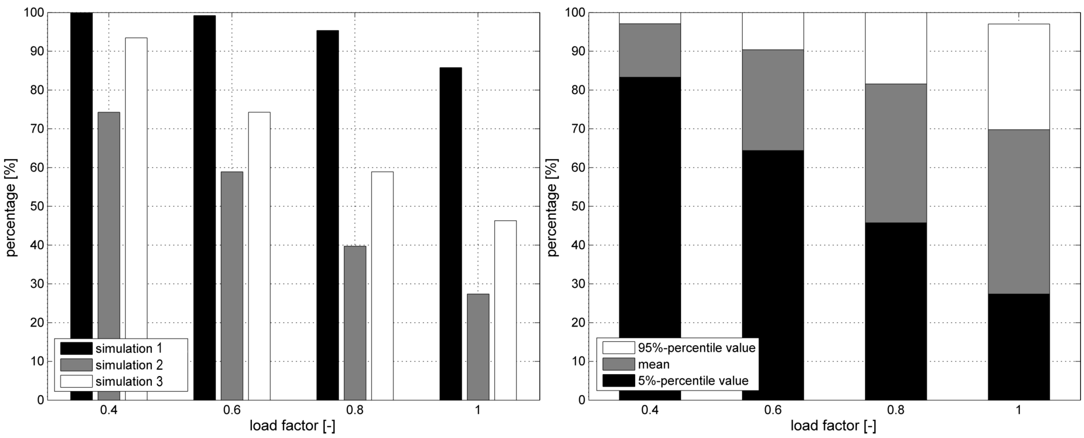


Figure 5.6: Cumulative probability distribution of the load factor of all simulated years

and more greenhouse gas emission.



(a) Simulation of discharge time series of three years.

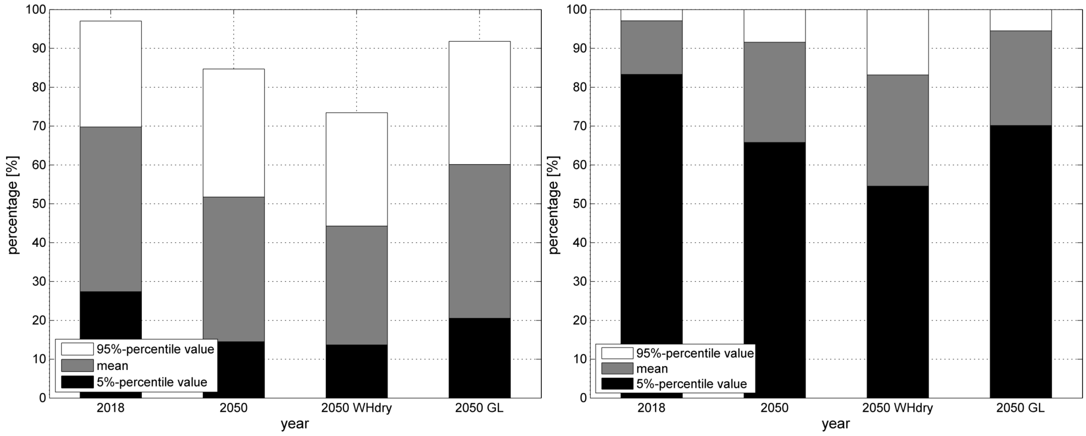
(b) Statistical characteristics for all simulated years.

Figure 5.7: Percentage of navigable time as a function of the load factor of a typical Rhine vessel in 2018 based on the depth restrictions at Nijmegen.

The annual average loading capacity might not tell everything about the navigation efficiency, it allows to show the trend of both climate change and bed degradation. Figure 5.9a shows the average load factor development between 2018-2050 when assuming the expected bed degradation (base case). As can be seen the load capacity is reduced from 0.91 to 0.84 due to bed degradation. It should be noted that these values are the average of historical discharge records, while this might vary in dryer and wetter years as shown in Figure 5.9b. This figure shows a stronger drop of the 5^{th} percentile value (0.73 to 0.46) for the 2050 $W_{H,dry}$ than the average value (0.91 to 0.76), revealing the impact of low-flow conditions is reinforced in 2050.

The load factor development can not be assumed to be linear over time, in contrast to the anticipated linearity of the bed degradation and the depth reduction (Figure 5.2a). This is attributed to the fact that the dataset is not uniformly distributed as is explained in the Intermezzo and Figure 5.10a. An interpolation between 2018 and 2050 to account for the climate scenarios has been conducted, showing the effect of climate change. Even a more non-linear behaviour is observed for the $W_{H,dry}$ climate scenario with a severe drop in load factor.

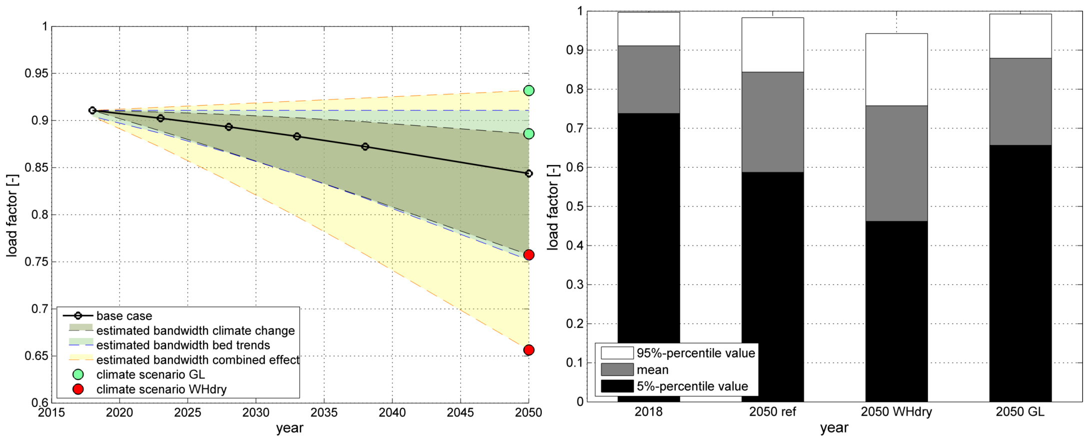
To put the impact of a reduce load factor into concrete terms, the load factor reduction can be translated into a reduction of annual transported load within a year. When assuming a 6-barge vessel transporting approx-



(a) load factor = 1

(b) load factor > 0.4

Figure 5.8: Percentage of navigable time as a function of the load factor of a typical Rhine vessel for different situations based on the depth restrictions at Nijmegen.



(a) Development of mean load factor for base case and the estimated bandwidths.

(b) Statistical properties for all simulated years for situation 2018 and 2050 accounting for base case erosion trend and climate change.

Figure 5.9: Development of the averaged annual load factor of a typical Rhine vessel based on water depth restrictions at Nijmegen.

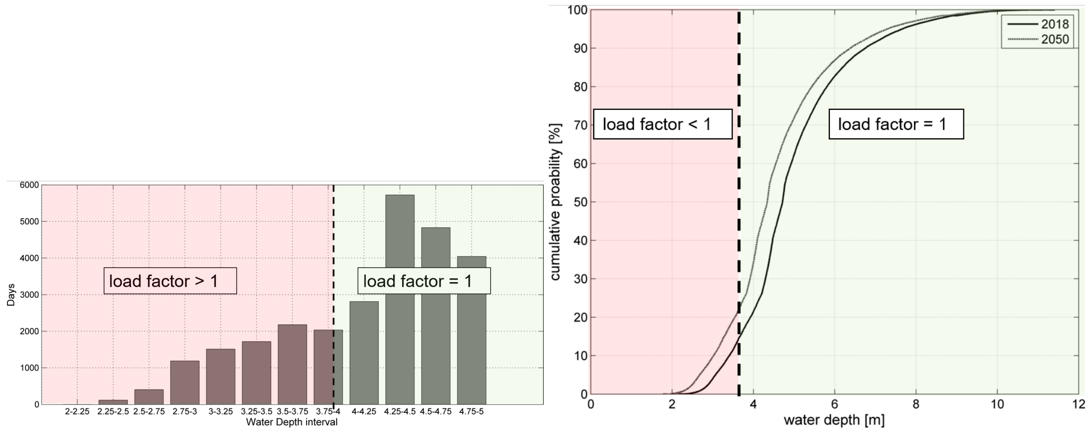
imately 16 000 ton that is returning every 5 days fully loaded ($0.91 * 365/5 * 16000 = 1.06$ million ton), the reduction in average load factor of 2050 (0.98 million ton) compared to 2018 means a load capacity reduction of nearly 80 000 ton/year or in other words 5 extra trips. When applying the same assumption for the climate scenario $W_{H,dry}$ in 2050, this means a reduction in load capacity of 180 000 ton equivalent to extra 11 trips.

Intermezzo: non-uniform distribution of water depths

As can be expected, the observed water depths during regular flow range will occur more frequent than extreme low-flow water depth. These phenomena is illustrated in Figure 5.10a and 5.10b. Figure 5.10a how many days a water depth is simulated based on historical discharge records between 1901 and 2017. As can be seen, water depths around 4.5 m will be present more frequent than water depths around 2.5 m. This explains that a linear reducing water depth due to bed degradation will cause a non-linear shift in the frequency of occurrence of a water depth. The cumulative distribution of Figure 5.10b shows how the statistics of the water depth change in 2050 compared to 2018. From this figure it can be understand that a typical Rhine vessel will experience more often depth restrictions.

Wellfare loss reduced loading

While the other aspects of the cost-benefit analysis are more or less straightforward, the wellfare loss due to reduced loading is more complex to obtain. As the wellfare loss is a function of the fleet composition and its load factor, it shows a similar pattern as for the load factor of a Rhine coal carrier. The wellfare loss can be calculated per day, from which the mean annual wellfare loss can be calculated and statistical properties. Figure



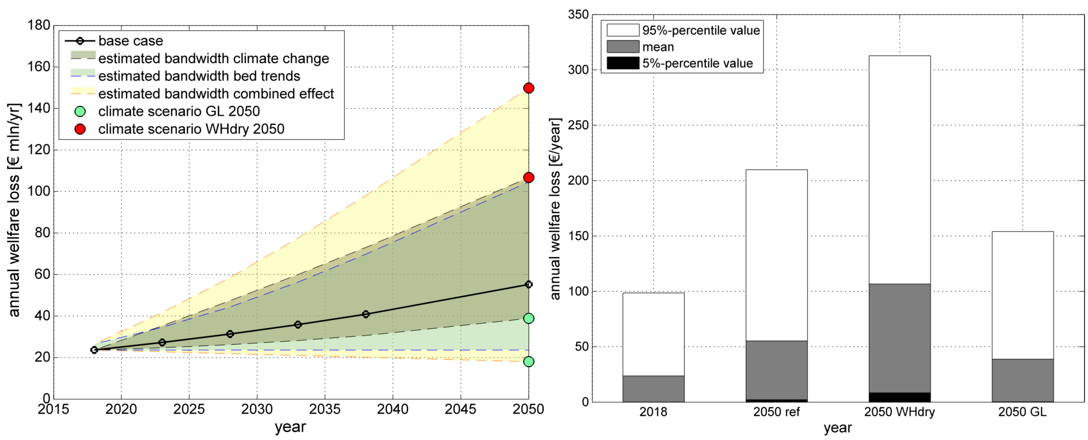
(a) Frequency of occurrence of different water depth intervals for state 2018.

(b) Cumulative distribution for 2018 and base case 2050.

Figure 5.10: Water depth statistics at Nijmegen simulated with discharge time series 1901-2017.

5.11 shows the impact of bed degradation and climate change on the mean annual wellfare loss. Also with the current river profile and present discharge statistics, water depth restrictions result in a wellfare loss. However, it is expected that these wellfare costs will rise considerably the following years. Due to the expected bed degradation, the annual wellfare costs are predicted to almost double from €24 million to €55 million, while the annual wellfare costs for a dry climate scenario might increase to €107 million in 2050.

Figure 5.11b shows the impact on the 5% driest years and the 5% wettest years, while the mean can be considered as a regular year. It shows that the wellfare costs can be as high as €310 million per year in 2050 considering $W_{H,dry}$. The development of the wellfare costs of 5% wettest years remain relatively low, also for the driest climate scenario (< €10 million per year).



(a) Development of the mean annual wellfare for the base case and the estimated bandwidths.

(b) Statistical properties for all simulated years for situation 2018 and 2050 accounting for base case erosion trend and climate change.

Figure 5.11: Results of development of the annual wellfare loss due to water depth restrictions at Nijmegen.

In a similar way the total annual costs of all transported goods through the Nijmegen (Waal) can be evaluated by multiplying the average price per ton (depending on depth restrictions) with the total transport goods (128 million ton). It appears that annual costs of the transported goods are predicted to be approximately €1.15 billion per year in 2018 (Figure 5.12), while due to the observed erosion trend these costs will increase by 3%. Figure 5.12 also evaluates the impact on the total transported costs due to climate change. It appears that considering the dry climate scenario $W_{H,dry}$ an increase of 8% is expected compared to 2050, while for a wet climate scenario an increase of 1% is expected.

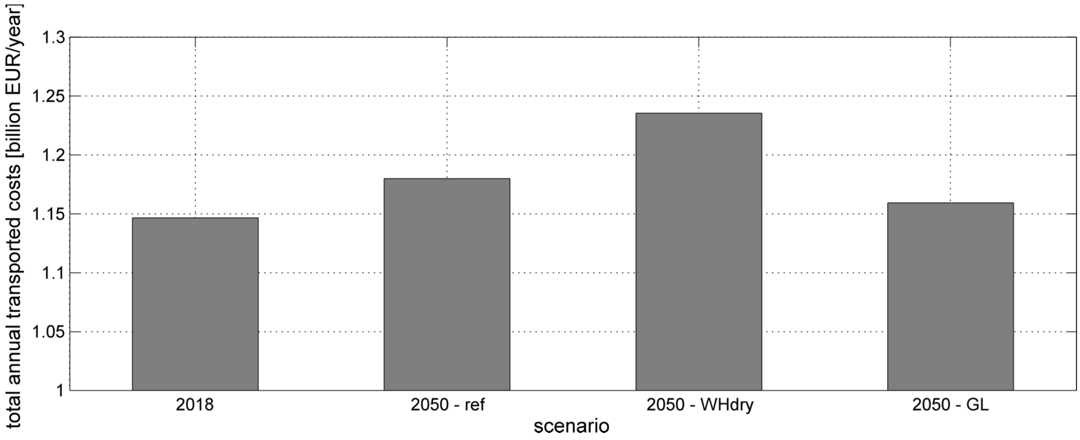


Figure 5.12: The development of the total annual transportation costs of goods transported over the Waal through Nijmegen accounting for the base case erosion trend and the two selected climate scenarios.

When conducting a cost-benefit analysis over the period 2018 to 2050 compared to the wellfare costs in 2050, a future investment to mitigate the low-flow problems can be assessed. As is observed from Figure 5.11, the wellfare costs of all scenarios except for the situation with no bed erosion and climate scenario G_L, are predicted to rise. A rising wellfare loss results in relative costs (negative net market wellfare value) in 2050, while a reduction of the wellfare loss compared to the situation in 2018 is accounted as a relative benefit (positive net market value) in 2050. The net market value follows from the same approach as Equation 5.1. Table 5.2 shows the market value of all scenarios, revealing that the net market value related to wellfare loss are an order of magnitude larger than the net market value related to maintenance dredging (Table 5.1). The combined net market value (sum of Table 5.2 and 5.1) is elaborated in Appendix J.3.

Scenarios	Net market value [mln €]
Base case	-248.9
No erosion	0
Doubled erosion	-722.2
$W_{H,dry}$ and base case erosion trend	- 741.6
G_L and base case erosion trend	- 135.8
$W_{H,dry}$ and doubled erosion trend bed	- 1126.2
G_L and no erosion	+ 50

Table 5.2: Net market value of the relative navigational wellfare loss compared to the situation of 2018.

This model assumes no change in fleet composition, while in practice the fleet dimensions are scaling-up (Groen and van Meijeren, 2010). When this is the case, the wellfare costs will most likely rise, as larger vessels will experience more depth restrictions. Another component that is not taken into account is the change in the modal split, as it is predicted that for a dry climate scenario 5.4% of the IWT transported load is redirected to rail and road transport (Jonkeren et al., 2007). These effects will most likely also affect the fleet composition, prices and annual transported load numbers. Furthermore, it should be noted that the calculated wellfare costs will not be direct costs for Rijkswaterstaat, which is the case for maintenance dredging. The wellfare less can be considered as the societal costs carried by the consumers of the transported goods. This can be in the Netherlands, but also abroad. This could put a question mark on the justification of an investment by Rijkswaterstaat in an intervention to improve navigation.

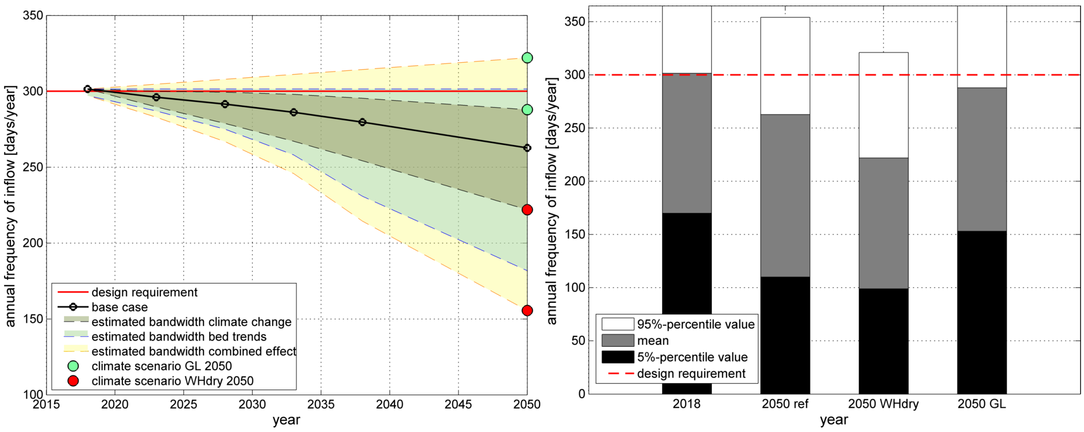
5.2.2. Future nature trends

Frequency of flow through side channel Klompenwaard

Next, the effect of future trends on the recently adapted side channel in the Klompenwaard are assessed. As discussed in Section 4.4.2 at least 300 days per year flow through the side channel is required following the design criteria. This means that during dryer years the frequency can be smaller, since on average it should be 300 days. When simulating the historical discharge records, it is calculated that currently the side channel does just suffice (301 days/year). In Figure 5.13a it is observed that with the expected bed degradation the frequency becomes 263 days per year in 2050. The Figure shows that the estimated bandwidth of climate change and bed degradation in the same manner as Figure 5.3, showing a strong decrease in frequency for the driest climate scenario. Again the driest climate scenario and bed degradation have a reinforcing impact on the frequency of flow through this side channel, while for this location the estimated bandwidth of the bed degradation dominates the driest climate scenario in 2050. As already discussed the evaluation of the river function nature will not be expressed in an economic value, and should be evaluated on an individual basis (not incorporated in the cost-benefit analysis).

The design criteria is based on the mean frequency over several years, while Figure 5.13b illustrates the statistical character of the entire discharge time series (117-years). This figure shows that for the situation in 2018 the 5% driest year has an inflow frequency of 170 days per year, while in the wettest years the side channel will inflow the entire year (365 days per year). The statistics are going to change when time proceeds and climate change will also affect the discharge statistics with lower inflow frequencies.

The inflow frequency of the side channel Klompenwaard is assumed to depend on the water level and the inlet height. However, in practice morphological processes could induce sedimentation in the side channel, which could clog the inlet (as it is a funnel shape inlet) or create a dam in the side channel not allowing flow through the entire side channel. This process is not taken into account and it is assumed that the inlet height will not change.



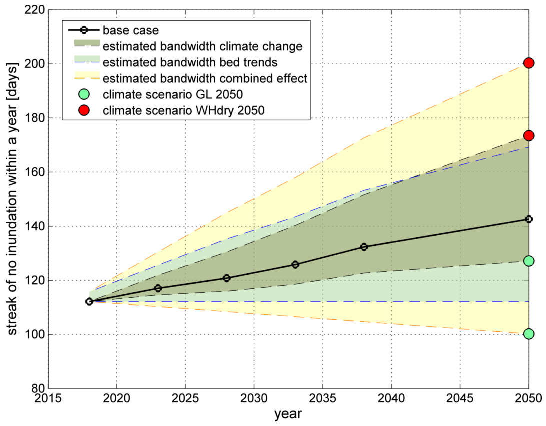
(a) Development of the mean annual inflow frequency for the base case and the estimated bandwidths.

(b) Statistical properties for all simulated years for situation 2018 and 2050 accounting for base case erosion trend and climate change.

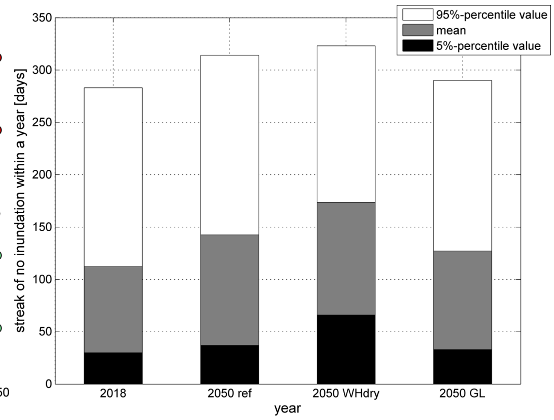
Figure 5.13: Annual inflow frequency of the side channel Klompenwaard.

Consecutive days of no-inundation of Millingerwaard

Similar analysis have been conducted for the floodplains of Millingerwaard a few kilometers downstream. Yet, this analysis is focussing on the streak of non-inundation of the water bodies in the floodplain, which is important for both dehydration and interaction between the riverine nature and the river channel. Nowadays, it is predicted that on average the water bodies in the Millingerwaard are disconnected for 110 consecutive days per year, while considering the expected bed degradation this is predicted to be 140 consecutive days in 2050 (Figure 5.14a). When evaluating the statistical properties of the non-inundation streak, it appears that during the 5% driest years for the situation in 2018, the waterbodies are disconnected 283 days per year, while the 5% wettest years the water bodies are only disconnected for 30 days. The bed degradation in combination with a dry climate scenario will increase these numbers resulting in more than 300 days per year disconnection for the driest 5% years. During those days organisms are not able to interact with the river channel and water levels within the water bodies tend to drop with approximately 1 cm per day. When the inundation statistics change, this induces changes in the ecosystem, which could results in a (undesired) change in habitat. However, when a wetter climate scenario will come true in combination with bed degradation, the inundation circumstances remain more or less equal to the current situation, which most likely means that the current habitat will remain unchanged.



(a) Development of the mean annual consecutive days without inundation for the base case and the estimated bandwidths.



(b) Statistical properties for all simulated years for situation 2018 and 2050 accounting for base case erosion trend and climate change.

Figure 5.14: Consecutive days without inundation of the Millingerwaard within a year

5.2.3. Impact on future flood levels

While in the assessment of the previous assessment the low-flow conditions were determining, flood discharges become normative considering flood protection. For low-flow the impact of bed degradation and climate change is expected to become more marked, whereas for flood conditions the opposite is expected. During low-flow conditions the sedimentation of the floodplains is not affecting flow conditions, since the floodplains are not conveying discharge. However, during flood conditions the floodplain elevation might increase flood levels, while bed degradation is lowering flood levels. Firstly, the impact of the different bed trends is analysed for a discharge with an exceedance probability of 1/10 000 (Q_{Lobith}, without incorporating climate change this discharge is 15 270 m^2/s). The DWL to decrease due to bed degradation, while they are expected to increase due to floodplain sedimentation. The annual change in DWL has been simulated by the model for both processes separated and combined as is shown in Figure 5.15. Despite the impact of floodplain elevation, the bed degradation is dominating the process due to more severe bed trends (1.5 cm/year versus 0.5 cm/year) and the larger conveyance capacity of the main channel compared to the (rough) floodplains. The combined affect is predicted as 0.34 cm/year, which is simulated till 2050 resulting in a decreased flood level of 10.8 cm compared to 2018.

Figure 5.16 shows the development of the DWL in time incorporating both climate change and bed trends. The trend without climate change is a drop of the DWL. When bed degradation becomes more severe (3 cm/year), the flood level are further decreased till 25 cm in 2050. However, as Table 2.5 already denoted, all climate scenarios are predicted to cause higher flood levels in 2050. In this case the KNMI'14 scenario G_H is the mildest scenario considering extreme high discharges, while G_H is the most extreme scenario. Figure 5.16 shows the combination of the base case bed trends in combination with both climate scenarios and revealing that climate change dominates in 2050, as the net effect on the DWL is an elevation. The mean of both climate change is illustrates with the black line, showing that the DWL will be increased with approximately 30 cm in 2050. When the erosion trend will stop in the near future, this will results in more than 40 cm rise of the DWL. The analysis presented in Figure 5.16 corresponds with the flood levels at Nijmegen, while in Appendix I.1.2 also the change in flood levels for other locations is assessed.

5.2.4. Conclusions

This section predicts the impact of river processes on the different river functions by applying the assessment methodology of Chapter 4. Within this section the assessment is summarized, while from the findings from the results will be discussed. The future impact of the river conditions on the river functions, is predicted as follows:

- The autonomous processes put pressure on both navigation and nature, as it appears that the impact of low-flow conditions becomes more marked in 2050. This results in navigational efficiency losses with corresponding higher transportation costs, respectively 3% increase in 2050 due to bed degradation and 8% in 2050 for bed degradation and a dry climate scenario. The net discounted value over the pe-

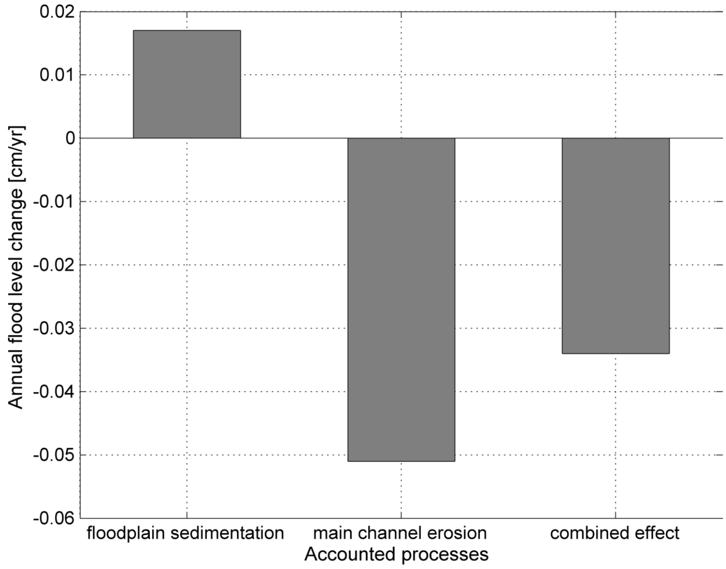


Figure 5.15: The contribution of the expected impact of bed trends (main channel -1.5 cm/yr and floodplain +0.5 cm/yr) on the changing normative water levels at Nijmegen.

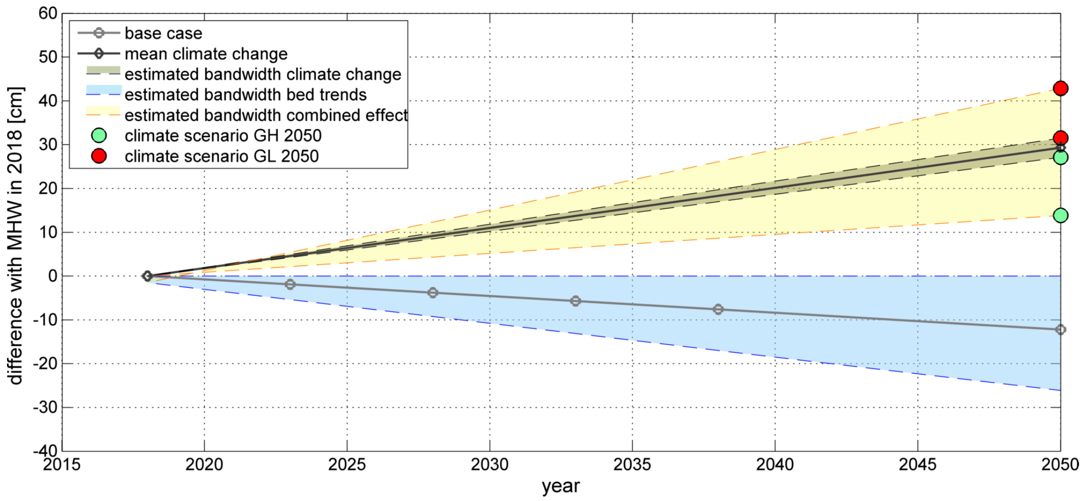


Figure 5.16: The development of design water levels at Nijmegen including the estimated bandwidth of bed degradation in the main channel and changing discharge statistics due to climate change.

riod 2018-2050 ranges from €-1126 to +50 million depending on the bed degradation rates and climate scenario. The increased transportation costs will possibly induce changes of the modal split, when IWT will be redirected to rail and road transport. Bed degradation might reduce the required maintenance dredging to guarantee fairway dimensions, a dry climate scenario will require an strong increased effort to maintain the fairway, as the ALD is likely to drop. However, associated dredging costs are marginal compared to the navigational efficiency losses. Considering nature the autonomous developments will most likely result in less frequent inundation inducing corresponding (undesired) habitat changes. Flood levels are predicted to increase due to the autonomous trends (especially climate change).

- During low-flow conditions it appears that the impact of climate change and bed degradation is more or less equivalent. However, during flood conditions, the impact of climate changes dominates the impact of bed degradation in 2050 (respectively + 40 cm versus - 10 cm) resulting in an increased ALW.
- The estimated bandwidths reveal the importance of accurate predictions of bed degradation and climate change, as the range of efficiency losses is currently over a billion euro. Without an accurate prediction of the bed degradation rates, it becomes difficult to justify an investment to mitigate bed degradation.

5.3. Assessment of sediment management strategies

5.3.1. Introduction

Within this section the methodology is applied on various sediment management strategies, which are described in Chapter 3. The impact of the proposed strategies will be evaluated based on the tools described in Section 4.4. As the morphodynamics imposed by the sediment strategies are neglected, it is assumed that the bed degradation will continue with the same expected degradation rate as if no nourishment has been paced. This research will not concern the feasibility of the strategies, such as the use of sediment from the floodplain. The selected strategies will differ, as follows:

- **The nourishment dimension**, which is basically the initial elevation of the river bed. In addition to the stabilization of the actual river bed (state in 2018), the 'nourishment 2010' and 'nourishment 1997' will be evaluated, which can be considered as restoring and maintaining the river bed to its state in 2010 and 1997. Also an optimized nourishment 'smoothen 2010' will be evaluated. These strategies are evaluated, as if they are continuously stabilized.

- **The source of the nourishment sediment**. Either sediment comes from outside the study area not affecting the hydrodynamics (e.g. external sand pit or further down-, or upstream in the river) or it comes from inside the study area, such as from the floodplain.

- **The stabilization frequency**, nourishment dimension it is assumed that the nourishment will be stabilized continuously, while also a sensitivity analysis will be conducted on less frequent stabilization. Less frequent stabilization requires a larger initial nourishment, as the river bed should not degrade under a certain level (in this case 'nourishment 2010').

A nourishment might improve the performance of navigation, while a future river without stabilization will cause a reduced navigational efficiency. Based on this improvement, this could be argued as a potential benefit that justifies a certain investment in the nourishment and its side-effects, such as dike heightening. The functional performance of the various river functions in 2050 will be assessed and compared with the situation in 2050 without stabilization, while this section will end with a comparison of the cost-benefit analysis of the sediment management strategies and a future river system without stabilization intervention.

This section will assess the impact of a nourishment in most cases without incorporating climate change, to keep the figures uncomplicated. However, Appendix I will assess the nourishment in combination with climate change. Furthermore, it is expected that the bed degradation rate will only effect the stabilization volume, as this will increase with a stronger erosion trend. Subsequently, this results in higher nourishment costs. Hence, different erosion scenarios will not be evaluated in the results.

5.3.2. Impact on navigation

It is expected that a nourishment will elevate the water level, resulting in a larger water depth at the fixed layer of Nijmegen. Figure 5.17 shows the impact of the stabilized nourishments and the situation without stabilization in 2050 (expected bed degradation rate). The results of the current ALD (reference) are projected in grey. As can be seen the water depth at Nijmegen is approximately increased with the same rate as the height of the nourishment. The 'smoothen 2010' strategy guarantees a slightly larger water depth as the uniformly elevated 'nourishment 2010' (difference is equal to 1.5 cm). The difference is dedicated to the fact that a relatively large volume of the nourishment has been placed just downstream of the fixed layer at Nijmegen. To illustrate that a nourishment will also suffice concerning climate change, the combination of a nourishment and a dry climate scenario is shown in white.

Similar analysis as in the previous paragraph are conducted to compare the different stabilization frequencies. Figure 5.18 shows what the stabilization frequency means for the ALW water depth accounting for the expected bed degradation rate. Figure 5.18a shows the maximum and minimum depth for different stabilization frequencies, while Figure 5.18b shows the development over time. Less frequent stabilization, requires higher initial nourishment, guaranteeing a larger water depth the majority of the time. In practice, directly after placement of the 1/20 year stabilization nourishment, the bed level is approximately at the same level as for 'nourishment 1997'.

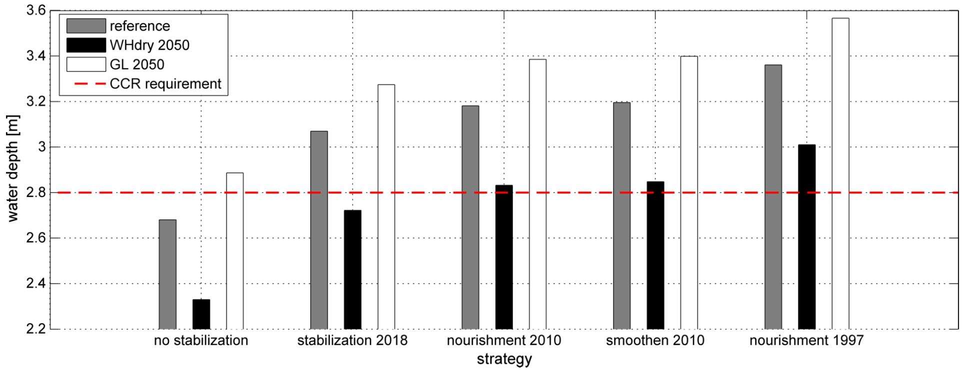
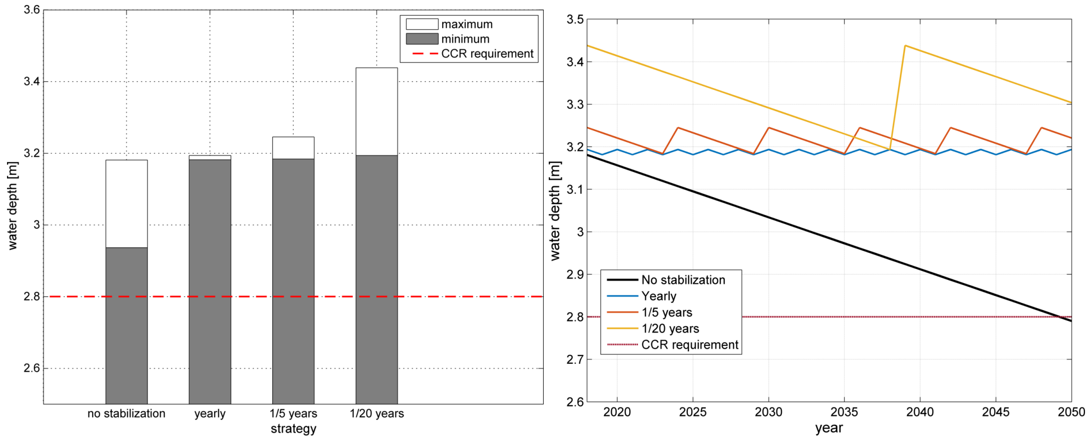


Figure 5.17: Predicted water depth at ALW for different sediment management strategies in 2050. The different bars indicate the effect of climate change (black: GL 2050 and white: $W_{H,dry}$), while grey shows the reference ALD (1020 m^3s).



(a) The minimum and maximum throughout the period 2018-2050.

(b) The development in time.

Figure 5.18: Water depth development at Nijmegen during ALD for different stabilization frequencies of the 'nourishment 2010'.

Maintenance dredging

In Figure 5.19 the required maintenance is evaluated, revealing that the dredging volumes are slightly increased due to a uniform elevation of the river bed. The mean dredging volume will increase from 138 000 m^3 per year in 2018 to 139 000 m^3 per year for 'nourishment 2010', while a larger nourishment ('nourishment 1997') will increase the dredging effort to 146 000 m^3 per year. As the bed level upstream of the nourishment is elevated, while the fixed layer remains at the same level, the backwater effect results in relatively smaller water depth upstream of the fixed layer. Smaller water depths are equivalent to more dredging effort. The more strategic placed nourishment 'smoothen 2010' will result in smaller dredging volumes (122 000 m^3 per year), as within this strategy the majority of the sediment is placed in the deeper trajectories resulting in an increased water level while the shallow locations are not elevated. The corresponding annual dredging costs are indicated in Table 5.3. Appendix I.2.1 elaborates on the combined impact of climate change and a nourishment on the future dredging effort. Less frequent stabilization requires a larger initial nourishment, which will increase the maintenance effort (Appendix I.2.1).

Strategy	Dredging costs in 2050 [mln €/yr]
No stabilization	0.37
'Stabilization 2018'	0.47
'Nourishment 2010'	0.49
'Smoothen 2010	0.42
'Nourishment 1997'	0.51

Table 5.3: Maintenance dredging costs

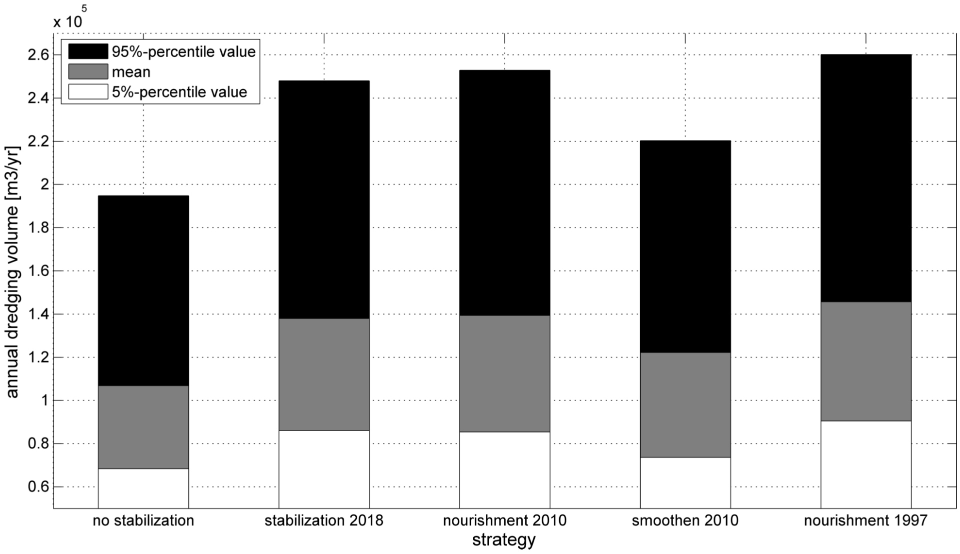
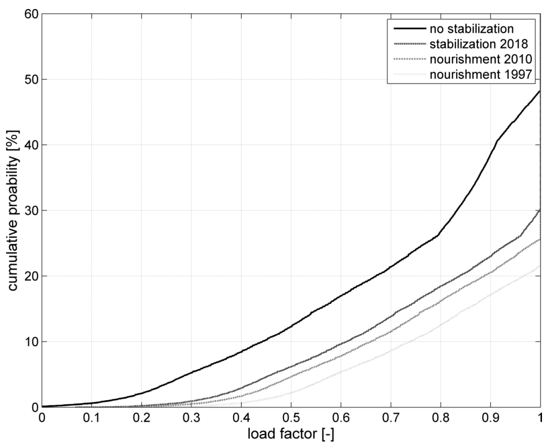
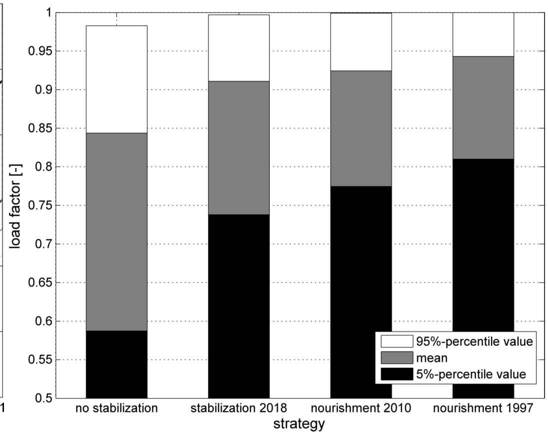


Figure 5.19: The statistical properties of the annual maintenance dredging volumes for different sediment management strategies in 2050 (without accounting for climate change).

Navigational efficiency
As the water depth is elevated by nourishments, the navigability is expected to be enhanced as well. As for the water depth, strategy 'nourishment 2010' and 'smoothen 2010' are almost identical, for this reason 'smoothen 2010' is not evaluated in these figures. The cumulative distribution of navigable time with a certain load factor is shown in Figure 5.20a. This shows that stabilization of the bed enables more navigable time fully loaded, as for the strategies 'no stabilization', 'stabilization 2018', 'nourishment 2010' and 'nourishment 1997', respectively 52%, 70%, 75% and 72% of the time navigation fully loaded is possible. Appendix I.2.2 elaborates in more detail on the percentage of the navigable time with various load factors. Figure 5.20b shows the model result of the annual load factor of a typical Rhine vessel with the corresponding statistical properties. When considering the mean annual load factor (grey), it appears that 'nourishment 1997' increases the mean annual load factor with 3% compared to the stabilization of the river bed of 2018. As Figure 5.18 illustrates, less frequent stabilization results in a larger water depth at t=0, this will also improve the navigation efficiency (Appendix I.2.2). The effect of climate change and nourishments in 2050 is evaluated in Appendix I.2.2.



(a) Cumulative probability distribution of the load factor of all simulated years.



(b) The statistical properties of the annual averaged load factor.

Figure 5.20: Annual load factor of a typical Rhine vessel in 2050 for different sediment management strategies and no stabilization (without accounting for climate change).

In a similar way the annual wellfare loss for the different cases, that will be considered in the cost-benefit analysis, can be predicted (Figure 5.21). The estimated bandwidth due to bed degradation and climate change of the no stabilization intervention is illustrated in Figure 5.11a. The impact of climate change on the stabilized sediment management strategies is assessed in Appendix J.1.2, while a different erosion rate only effect the required stabilization volume. Similar analysis have been conducted considering the stabilization frequency, resulting in a fluctuating annual wellfare loss. Table J.3 in Appendix J.1.2 shows the range of the annual wellfare loss fluctuations due to the selected stabilization frequencies.

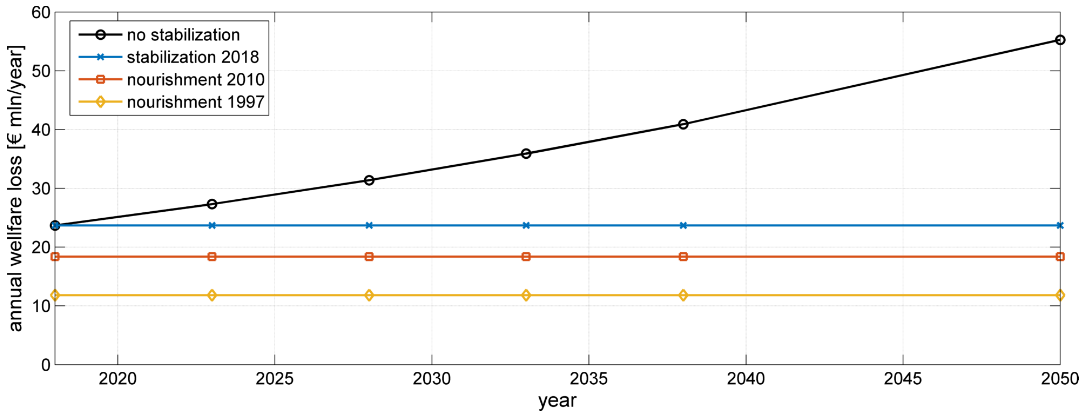
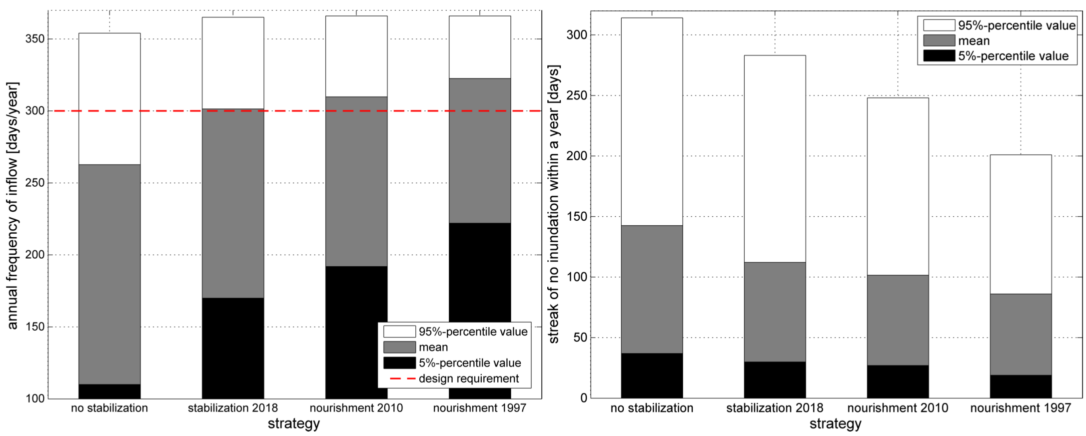


Figure 5.21: Development of the annual wellfare loss due to water depth restrictions at Nijmegen for different sediment management strategies. The presented values are the mean of the entire reference discharge time series incorporating the base case bed degradation.

5.3.3. Impact on nature

As the nourishment elevates water levels along the river section, it is expected that the impact of a nourishment is more frequent inflow of side channels and floodplains. Figure 5.22a presents the impact on the annual inflow frequency of the side channel Klompenwaard. It appear that with stabilizing the river bed of 2018, the design criteria can be fulfilled in 2050 (mean inflow frequency of 301 days per year), while the stabilization of either 'nourishment 2010' and 'nourishment 1997' increases the inflow more (respectively 310 and 323 days per year) and reduces the impact of dryer years (black bar in Figure 5.22a). 'Smoothen 2010' is not been considered, as it shows identical results as 'nourishment 2010' (mean inflow frequency of 310 days per year). Similar trends are observed for the drought streak of the Millingerwaard as is shown in Figure 5.14a, as the drought streak is reduced from 112 days in 2018 to respectively 102 and 86 days for 'nourishment 2010' and 'nourishment 1997'. Appendix I.2.3 elaborates on the combined impact of a nourishment and climate change.



(a) Annual averaged frequency of flow through the side channel Klompenwaard.

(b) Consecutive days of no-inundation of the Millingerwaard.

Figure 5.22: Statistical properties 'nature objects' for different sediment management strategies in 2050 (without accounting for climate change).

5.3.4. Effect strategies on flood levels

As a nourishment is predicted to have a positive impact on the functional performance depended on low-flow conditions, the impact on the flood conditions is expected to be negative as design water levels are predicted to rise. As Figure 5.16 shows the impact of various sediment management strategies on the DWL compared to the current DWL. As expected the nourishments will elevate the DWL, requiring a compensation or dike heightening. A larger nourishments results in a larger increase of the DWL. The relative DWL increase for respectively 'nourishment 2010' and 'nourishment 1997' is equal to 4.4 cm 11.3 cm. It appears that the strategies 'nourishment 2010' and 'smoothen 2010' are identical. Yet, it becomes relevant to assess the impact of sediment extraction from the floodplain, as flood levels are predicted to lower due to a lower floodplain. By means of sediment extraction from the floodplain the normative flood level is reduced from 4.4 cm to 2.6 cm for 'nourishment 2010' and 11.3 cm to 6.6 cm for 'nourishment 1997'. Despite sediment extraction from the floodplain can be seen as redistributing of sediment over the river plain, the net effect on the flood levels is an increase, as the main channel contributes relatively more to the total conveyance capacity of the river than the floodplains.

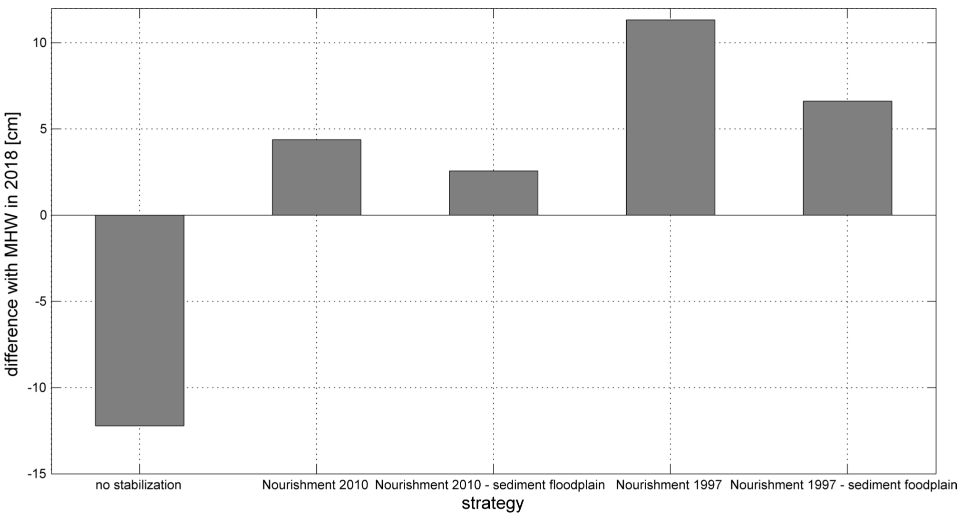


Figure 5.23: Model results of relative increase of the design water levels compared to the state in 2018 for different sediment management strategies (without accounting for climate change).

Section 4.5.4 describes two methods of estimating the compensation costs of the increased design water levels. However, it is assumed that the dike heightening will be coupled to reinforcement works of the HWBP, which allows to only account for the linear costs of €20 000/cm/km. As the no stabilization and stabilization of the river bed of 2018 does not imply increased design water levels, no compensation costs are related with those strategies. The costs can be calculated by means of multiplication by the length of the section (two times 25 km) and the averaged increased normative flood level. The corresponding costs of the different scenarios are shown in the second column of Table 5.4. When also has to be accounted for the fixed costs by means of the method proposed by de Bel (2014), the costs rapidly increase to more than a tenfold (third column of Table 5.4). The costs for compensation of effects induced by climate change, are not considered in this thesis.

When less frequent stabilization is desired, the larger initial nourishment results in higher flood levels. As shown in Table 5.4 the relative increase in DWL compared to the state in 2018 is 4.6 cm. Table 5.5 shows the impact of various stabilization frequencies of 'nourishment 2010' and the corresponding translation to costs. Less frequent stabilization requires a larger nourishment, which will elevate the DWL.

5.3.5. Cost-benefit analysis sediment management strategies

In Section 5.2.1 the financial impact of the various autonomous processes on navigational efficiency and maintenance dredging has been assessed. It appeared that due to both climate change and bed degradation, the wellfare loss due to navigation depth restrictions will increase compared to 2018. The costs related to maintenance dredging appeared to be marginal compared to efficiency loss of navigation. The net market

Strategy	Averaged increase in DWL [cm]	Variabe costs [mln €] (Eijgenraam, 2005)	Costs [mln €] based on de Bel (2014)
'Nourishment 2010'	4.6	4.6	63.3
'Nourishment 1997'	11.9	11.9	164
'Nourishment 2010' - sediment from floodplains	2.9	2.9	40
'Nourishment 1997' - sediment from floodplains	7.5	7.5	103

Table 5.4: Cost to compensate for increased design water levels for various sediment management strategies.

Stabilization frequency	Averaged increase in DWL [cm]	Variabe costs [mln (Eijgenraam, 2005)
Yearly	5.0	5.0
1/5 years	7.0	7.0
1/20 years -	13.1	13.1

Table 5.5: Cost to compensate for increased design water levels for various stabilization frequencies of the nourishment 2010.

value of a river system without stabilization will be compared to the cost-effectiveness of a nourishment. The Sections 5.3.2 and 5.3.4 already elaborated on the impact of a nourishment on the costs of navigational efficiency, maintenance dredging and the required costs to heighten the flood defences. Firstly, the financial assessment will focus on the nourishment dimensions, incorporating the following strategies:

1. **No stabilization**, where the bed degradation will continue for the coming years following the expected bed degradation.

2. **Stabilization 2018**, which stabilizes the river bed of 2018.

3. **'Nourishment 2010'**, which is restores and maintains the river bed of 2010.

4. Lastly, **'nourishment 1997'**, which is an initial elevation to the river bed of 1997, after which this state will be continuously stabilized.

Strategy 'smoothen 2010' is not considered due to its similarity with 'nourishment 2010'. Within these cases, it is assumed that sediment is obtained from outside the study area, while Section 5.3.4 shows the effect of extraction of sediment from the floodplain. Therefore, two modifications of the above cases will be assessed:

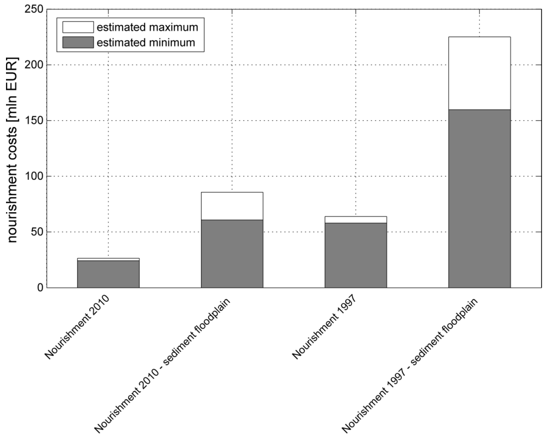
1. **Required extraction sediment from floodplains** - within this case the required volumes for the initial nourishment (not for stabilization) will be extracted from the floodplain.

2. **Extra sediment extraction from floodplains** - within this case more than only the required volume for the initial nourishment will be excavated from the floodplain. This case will extract sufficient sediment required to compensate for the increased design water levels of the nourishment.

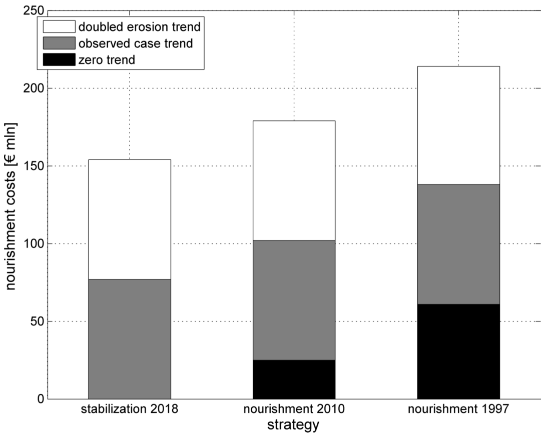
Finally, different stabilization frequencies (yearly, 1/5 years and 1/20 years) will be assessed, stabilizing (not exceeding) the nourished bed level of 'nourishment 2010'.

Nourishment costs
Section 4.5.5 has already discussed the required volumes with corresponding nourishment prices per cubic meter. When multiplying the nourishment volume with the price per cubic meter (Table 4.7), the costs of the nourishment can be calculated. The nourishment costs are indicated as a range due to its uncertainty in construction costs, as Figure 5.24a illustrates the initial costs for different sediment management strategies. In a similar way, the stabilization costs can be calculated for different stabilization frequencies (Appendix J.4).

As discussed in Section 4.5.1, also a strategy, that extracts more than the required nourishment volume from the floodplain, will be assessed. This strategy compensates the entire DWL increase, which requires a larger volume of sediment extraction from the floodplain. With the current model schematization, 2.7 times the nourishment volume has to be extracted from the floodplain to compensate for the increased flood levels.



(a) The estimated range of the initial nourishment costs for different sediment management strategies (due uncertainties in price determination).



(b) The estimated range of the initial and stabilization nourishment costs with the uncertainty of the erosion trend indicated.

Figure 5.24: The uncertainty of the nourishment costs.

As the costs of the material vary between €20-30/m^3 and the selling price ranges from €0.50-2.50/m^3, the net price for the extra volume can be considered as €17.50-29.50/m^3.

Another uncertainty are the stabilization costs, which depend again on the bed degradation rate. As Appendix J.4 reveals, the volume trend of the study area and the nourishment area is approximately - 140 000 m^3 per year equivalent to €3.77 million per year. When considering the estimated bandwidth for bed degradation scenarios, this value could be reduced to zero erosion or be doubled resulting in respectively €0 and €7.5 million annual stabilization costs. When these costs are discounted in the period of 2018-2050, this result in an relative increase of the net market value of €75 million compared to the base case erosion as shown in Figure 5.24b. Figure 5.24b shows that 'nourishment 2010' and 'nourishment 1997' also include nourishment costs when erosion will stop, as a nourishment is required to restore the river bed to its state in respectively 2010 and 1997.

Net market value
In the cost-benefit analysis, the outcomes of the previous sections will be combined and discounted. Both the wellfare costs and the dredging costs will be compared to the reference situation, which is the river state in 2018 without nourishment. The wellfare loss and costs related to maintenance dredging in the project duration period will be discounted to 2018 following Equation 5.1 to eventually evaluate the cost effectiveness of a nourishment. The nourishment costs have been estimated in a certain range with a minimum and a maximum value due to the uncertain construction costs. Also the other costs elements do include a certain degree of uncertainty, such as the predictions of the water depth, discharge uncertainty, bed degradation rates, model uncertainty and uncertainties of corrections for bedforms and transverse slope. However, to retain the cost-benefit analysis (and the thesis) manageable, the uncertainties will be neglected and an average value will serve as an estimation of the costs. Also the nourishment costs will be averaged. However, the impact of climate change and a different bed degradation rate will be assessed and illustrated by means of range in the net market value.

Eventually, the profitability of a nourishment can be calculated by discounting and combining all costs and benefits within the project duration. For civil engineering project, a project duration last typically between 20-50 years. Table 5.6 and Figure 5.25 illustrate the elements of the cost-benefit analysis of different sediment management strategies for a project till 2050 (32 years). This are the costs without considering climate change and with the expected bed degradation. Appendix J.5.5 will elaborate on the effect of different project durations and discount rates. Table 5.6 shows that 'nourishment 1997' ranks best in the cost-benefit analysis due to its improved navigation. Furthermore, it appears from the cost effectiveness analysis that a river without interventions is on the long-term not profitable.

Appendix J.5.1 assesses the cost-effectiveness of a nourishment considering climate change. The impact of

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No stabilization	0	0	1.3	- 249	- 248
Stabilization 2018	0	- 77	0	0	- 77
'Nourishment 2010'	- 5	- 102	- 0.3	108	1
'Nourishment 1997'	- 12	- 138	- 0.7	242	91

Table 5.6: The net market value in million euros for different strategies without incorporating climate change. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements in the period 2018-2050.

climate change on the cost elements of the cost-benefit analyses is illustrated in Figure 5.25. The net market value of various sediment management strategies are presented in Figure 5.26 showing the impact of climate change and the reference discharge statistics. It appears that for a both a dry and wet climate scenario, a nourishment is predicted to be cost-effective with the current assumptions. When evaluating a doubled erosion trend, only the stabilization costs of the 'nourishment 1997' will be increased (i.e. €75 million extra compared to base case stabilization costs). However, the strategy of no stabilization intervention becomes also less attractive as wellfare losses increase due to a doubled erosion trend (Table 5.2). When degradation rates will slow down till 2050, a nourishment becomes less attractive as the wellfare losses related to the strategy without stabilization reduce (Table 5.2). Appendix J.5.2 assesses the scenarios with reduced and increased bed degradation rates compared to the observed trends. A similar figure as Figure 5.25 is presented in Appendix J.5.2, illustrating the nourishment costs will increase for a stronger erosion rate.

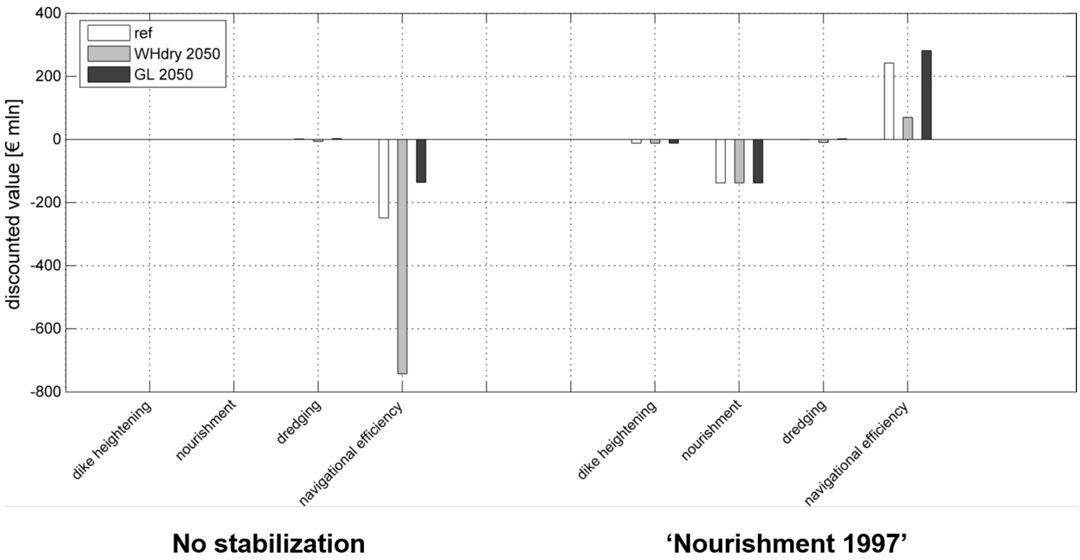


Figure 5.25: All cost and benefit elements between 2018-2050 the cost-benefit analysis of the strategies 'no stabilization' and 'nourishment 1997' compared to the reference year 2018. For all cases the base case erosion trend is applied.

In a similar way the profitability of different strategies using sediment from the floodplain is assessed, as can be reviewed in Appendix J.5.4. The advantageous of the strategy with extra sediment extraction from the floodplain, is that costs are costs are saved considering dike heightening. However, as shown in Figure 5.27, both cases appear to be less profitable than using external sediment, as the nourishment costs are higher, while the costs of dike heightening are relatively low. Appendix J.5.4 elaborates on scenarios, that the dike heightening cannot be linked with HWBP, resulting in a strong costs increase.

This approach is also followed for different stabilization frequencies, as is elaborated in Appendix J.5.3. Figure 5.27 illustrates the different nourishment strategies, guaranteeing (not exceeding) the bed level of 'nourishment 2010'. The nourishing costs of less frequent stabilization are higher due to two reasons: (i) more costs will be made in the future, while the value of money is lower (Section 4.5.6) and (ii) more sediment will be placed within the project duration since in 2050 the bed is above 'nourishment 2010'. However, less frequent stabilization means on average a higher bed level, resulting improved navigability, which increases the bene-

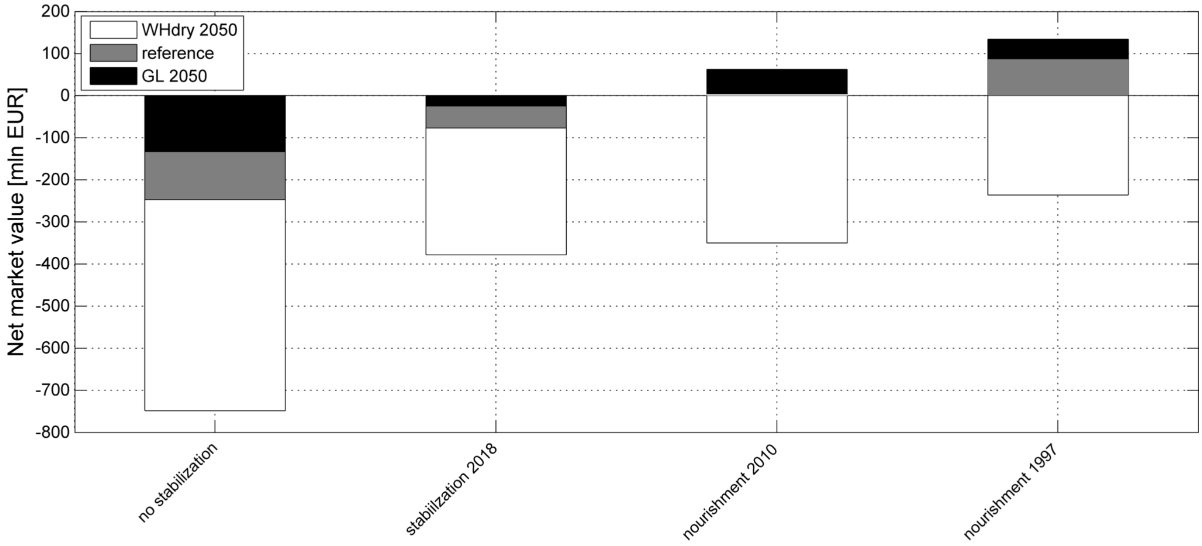


Figure 5.26: The net market value of all costs and benefits in the period of 2018-2050 considering climate change strategies compared to the reference year 2018.

fits. In the end, a stabilization frequency of 1/20 year is most profitable considering this cost estimation.

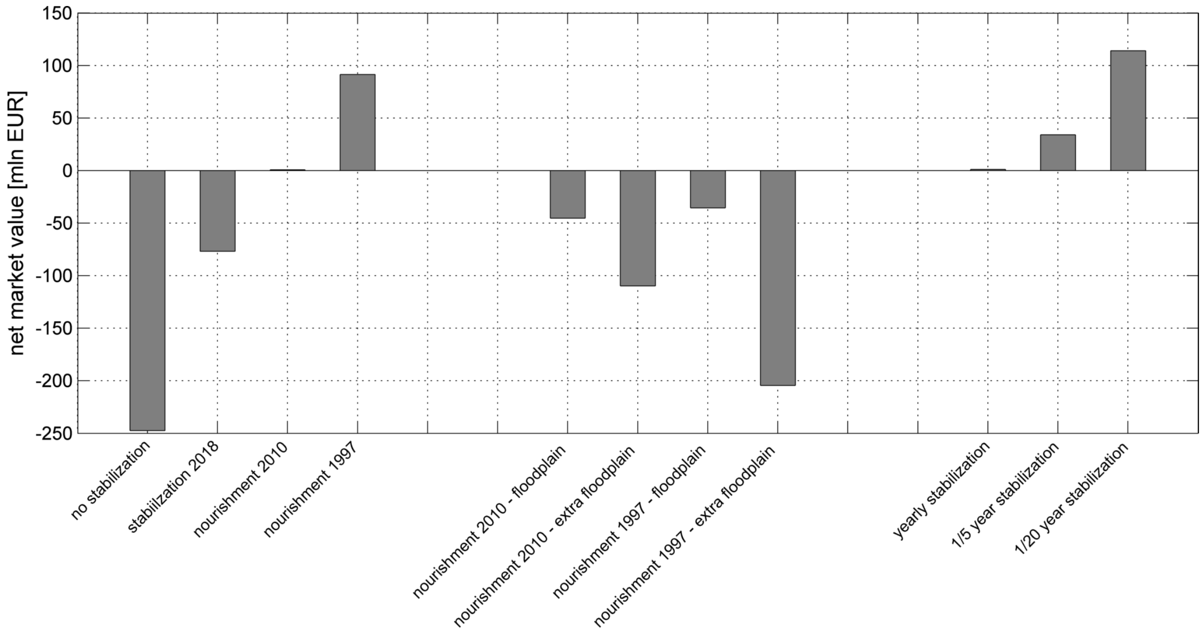


Figure 5.27: The net market value in the period of 2018-2050 for different sediment management strategies compared to the reference year 2018.

5.3.6. Review impact sediment management strategies

The results of the application of the assessment methodology to different sediment strategies has to be compared with the scenarios with no stabilization of the river bed. Furthermore, the various sediment management scenarios differing in nourishment dimension, sediment source and stabilization frequency can be assessed by interpretation of the results of this section. Based on the analysis performed in this chapter, the followings conclusions can be drawn:

- Both 'nourishment 2010' as 'nourishment 1997' increase the water depth at Nijmegen, which guarantees the required 2.80 m for the actual ALW and the future ALWs concerning climate change. However, the higher the nourishment the more depth is created at the downstream edge of the fixed layer, resulting in more efficient navigation (e.g. higher load factor). In practice (especially when considering three-dimensionality), a larger nourishment will induce new navigational bottlenecks. Hence, it is recommended to study this in a 2D or 3D model.

- One of the major differences between a uniform elevated nourishment ('nourishment 2010') and a smoothed nourishment ('smoothen 2010'), is the future maintenance dredging volumes. As 'smoothen 2010' is not placed on higher bed sections ('obstacles'), and the sediment is more tactically placed, the required dredging volumes decline. Furthermore, 'smoothen 2010' ranks slightly better on the water depth at Nijmegen, due to a larger proportion of the nourishment has been placed just downstream of the fixed layer. On other elements, the 'smoothen 2010' and 'nourishment 2010' strategies behave similar.

- Nourishments increase both the frequency of inundation of the Millingerwaard and Klompenwaard, resulting in more interaction and less strong dehydration of the floodplains.

- The effect of a nourishment on the design water levels can be reduced by extracting sediment from the floodplains. However, when this extraction is applied uniformly over the floodplain, the elevation of the design water levels is not totally compensated. This is the result of less conveyance capacity of the floodplains compared to the main channel.

- When less frequent stabilization is applied, the individual nourishment volumes are increased, resulting in more water depth at Nijmegen, more frequent inundation of floodplains and higher design water levels. The choice between the stabilization frequency can be considered as an optimization case.

These findings could provide quantitative input for discussions between decision-makers, but does not provide a universal parameter to assess the best sediment strategies, such as an economic value does. When reviewing the results of Figure 5.27, the cost effectiveness of the different strategies can be discussed. However, the results should be interpret as a first indication, since the costs estimations are highly uncertain. The choice of the project duration or discount rate does strongly affect the outcome of the cost-benefit analysis, as is elaborated in Appendix J.5.5. Despite the indicative character of the financial assessment, it provides useful information for an assessment of different measures. Based on the results of the cost-benefit analysis, the strategies can be assessed resulting in the following findings:

- Stabilization of the current degrading river bed is on the long-term more profitable than no intervention at all. However, elevating the river bed to a situation as in 1997 or 2010 allows even more efficient navigation, resulting in an even more profitable case. It appears that with the current assumptions, a nourishment height of 31.5 cm ('nourishment 1997') performs better than a nourishment of 12 cm ('nourishment 2010'). The impact of a dry climate scenario on low-flow conditions, can make a nourishment even more cost effective, as shown in Figure 5.26.

- Based on the assumptions of this thesis, it appears not economically feasible to extract sediment from the floodplains in order to nourish the main channel (or even more sediment for compensation of the increased flood levels). However, the impact of the sediment extraction on the design water levels might be underestimated. The sediment extraction from the floodplain has been schematized in the model by means of a uniform decrease of the floodplain height, while in practice a better conveying flood channel will be created. This would decrease the flood levels, which increases the profitability of those cases. Also the current assumption that the dike heightening can be linked with the dike reinforcement works of HWBP might influence the business case (Appendix J.5.4).

- Following the outcomes of this chapter, a less frequent stabilization is predicted to be more cost effective. Less frequent stabilization requires a larger initial nourishment to guarantee the 'nourishment 2010' bed level, which subsequently results in an improvement of navigation. Due to the improved navigation the wellfare loss due to depth restrictions is reduced, which makes less frequent stabilization more cost effective. However, it should be noted that a larger nourishment could cause new bottlenecks and could be more difficult to place (it should be elongated further downstream to avoid new bottlenecks).

5.4. Limitations of the methodology

To interpret the results it is important to be aware of the limitation of the model that predicts the future water levels. As denoted in Chapter 4, the hydrodynamic model describes the system behaviour based on simplified physical equations. This section will elaborate on the limitations of the model and will predict possible implications of the processes that are not accounted for. Furthermore, the interpretation of the cost-benefit analysis will be discussed with the associated limitations.

As the model is hydrodynamic, it does only account for the large-scale morphological changes of the observed bed degradation and floodplain sedimentation, which are imposed in a simplified manner on the river geometry. However, it is expected that sea level rise will also induce large-scale morphological changes in the Rhine system, which are not incorporated in the model. As the downstream boundary is elevated, sedimentation can be expected in the downstream delta. This will induce morphological changes upstream. The sedimentation in the downstream delta will induce higher water levels upstream, counteracting the impact of bed degradation and low-flow conditions inducing new morphological processes. Following Blom (2016) the composition of the Upper Waa its bed material is likely to become coarser in the future, resulting in a gradient in sediment transport in the longitudinal direction. The coarsening might reduce or even stop the bed degradation. Within the estimated bandwidth there has been accounted for a scenario with no bed erosion at all. However, within those scenarios it is expected that the bed erosion will be stopped immediately and uniformly, while in practice the change in bed composition will travel downstream resulting in spatial and temporal changes in the bed degradation rate. Another morphological feature that is not accounted for, is the impact of a nourishment on morphodynamics. In the current model setup, similar bed degradation rates are imposed after the nourishment as before the nourishment, while in practice nourishment will affect the morphological processes. As water levels are elevated upstream of the nourishment, it is expected that sedimentation will take place upstream and erosion downstream. This could result in a shorter lifetime of the nourishment and new shallowness at other locations.

The inflow boundary of the numerical model is the discharge in the Waal, which relies on the discharge at Lobith and the fixed discharge distribution at the bifurcation point Pannerdensche Kop. However, the discharge distribution cannot be considered as fixed, as is observed in Appendix F. Furthermore, when certain Rhine branches degrade faster than others, this will affect the discharge distribution as well. When branches degrade faster than others, it is likely that those branches attract more water. However, when a nourishment is placed in only one Rhine branch, water levels will be increased in this branch resulting in less inflow of the nourished branch at the bifurcation point. This will result in an associated drop of the water levels in this branch. Hence, it could be argued that the impact of a nourishment on the water levels is overestimated, as the discharge distribution will counteract the elevation of the water.

The physical equations rely on a simplified schematization of the river cross-section (Figure 4.3) with an almost rectangular river channel and symmetric adjacent floodplains on both sides. However, the floodplains are not symmetric and neither uniform in height and could contain flood conveying parts, blocking parts and water storing elements. This will affect the water levels when floodplains will inundate. Also other parametrizations are highly simplified, such as the assumption of a uniform roughness parameter and 1D flow pattern. These wrong model parametrizations are all compensated by calibration of the hydraulic roughness of the main channel. The extrapolation of the hydraulic roughness is unreliable, as it consists of a physical component and a model error component. The propagation of these two elements could differ, but are now considered together (Strijker, 2018). This could also induce unreliable results for the impact of bed degradation as the model has been calibrated without bed degradation.

The model has been calibrated on an accuracy of 10 cm. This can result in inaccuracy, as is observed at the ALW at Nijmegen. The model overestimated the ALW by 5 cm when comparing the ALD of December 2018. This overestimation results in an overestimation of the current water depth resulting in a better performing navigation than is actually the case. This means that the annual wellfare loss is slightly underestimated with the current calibration parameters.

The cost-benefit analysis is considered to be incomplete, as it only accounts for the costs and benefits related to navigation and flood protection. Other costs related to nature, fresh water supply and crossing infrastructure are not included in the analysis. Furthermore, it could be argued that the wellfare loss due to depth restrictions is not a loss for Rijkswaterstaat, which cannot be used to justify an investment of a nourishment. As the wellfare loss can be considered as a loss for the consumers of the transported goods, Rijkswaterstaat is not directly affected. As the Dutch inhabitants benefit from a nourishment by lower transportation costs, it could be argued that Rijkswaterstaat, as the national governmental responsible river authority, can justify an investment. However, it remains questionable whether an investment can be justified in this way, also because other countries will benefit from lower transportation costs. Another limitation of the cost-benefit analysis is the fact that changes in fleet composition and economic growth are not considered in the wellfare loss model, which could possibly affect the net market value in 2050.

5.5. Conclusions

As this thesis focuses on providing quantitative information on the future performance of a river system, the applied assessment methodology is evaluated by means of addressing the relevant research question. Different research question will be addressed based on the findings of this chapter:

What are the autonomous developments in the Rhine and how are these affecting the river functions?

The application of the instrument reveals the impact of the considered autonomous developments: bed degradation, floodplain sedimentation and climate change. The results obtained in this chapter demonstrate that during low-flow conditions degradation of the river (channel) bed further decreases the water depth and water level increasing the impact on navigation and nature. As the majority of the KNMI'14 scenarios predicts more frequent and more severe low-flow conditions in 2050, it is expected that these will have a further and likely more marked impact on nature and navigation. The simulations of the combined effect of a dry climate scenario and bed degradation, shows that those two will have an equivalent contribution to the reduced depth at ALW in 2050. During flood discharges the bed trends (floodplain and main channel) counteract each other, the analysis predict a net decreasing normative flood level of 3.4 mm/year. However, all climate scenarios predict an increased DWL in 2050, which appears to be dominant compared to the bed trends, as the separate effect of climate change is +40 cm (without bed degradation), while the impact of only bed degradation is -10 cm compared to the DWL in 2018.

How can the performance of river functions be quantified and provide useful information for an assessment of integrated river management?

The tools described in Section 4.4 provide insights into the impact of river processes on river functions. The river functions flood protection and navigation provide function requirements with a legal base (water depth ALW and design levels), while a more qualitative interpretation of the assessment of the function nature is required. Different tools facilitate assessment of separate aspects of functions (e.g. dredged volume, navigational efficiency or consecutive days without inundations), which relies on the interpretation of individuals (i.e. which function is important to you), but could provide interesting insights for a discussion between decision-makers, while the cost-benefit analysis provide an assessment parameter. This allows quantification of the feedback loop between functional performance and the intervention measures by the river manager. However, it should be noted that the current financial assessment is incomplete, since various aspects aspects are not included (e.g. function nature). Therefore, the financial analysis should never be assessed in isolation, but together with the results of the individual assessment of function elements.

Which aspects of the assessment process are important for decision-making?

Based on the results presented in this chapter, the relative impact of a process can be evaluated. When designing a measure to mitigate a process, it might be of interest to understand the relative impact on river functions. Considering climate change, it can be concluded that DWLs will rise, despite bed degradation. Furthermore, multiple scenarios can be considered due to the estimated bandwidth. The bandwidth spreading shown in the various figures in this chapter, indicates the relative importance of the bed degradation rate in the future river function performance. The net market value of the navigational efficiency loss in the period 2018-2050 varies between €0 and 722 million due to bed degradation without incorporating climate change. This shows the importance of an accurate estimation of the future bed degradation.

6

Conclusion and recommendations

In this chapter the conclusions from the research are summarised and a reflection on the research objective is provided. Next, recommendations for further research are proposed and advices for Rijkswaterstaat are listed.

6.1. Conclusions

The objective of this thesis was to develop an assessment methodology that predicts the future performance of various river functions. The future Rhine river system and its relation with the river functions was analysed by applying a semi-analytical model that describes the system behaviour and autonomous developments based on simplified physical equations. To evaluate the future river system, various tools assessing the river function performance were developed and applied to different scenarios.

6.1.1. Summarizing conclusions

Autonomous developments in the river system

The conditions during the summer of 2018 demonstrated that during low-flow conditions degradation of the river (channel) bed further decreases the water depth and water level. This autonomous process is expected to continue in the near future (uncertain in which degree), negatively affecting the river functions nature and navigation. As the majority of the KNMI'14 scenarios predicts more frequent and more severe low-flow conditions in 2050, it is expected that these will have a further and likely more marked impact on nature and navigation. In this thesis, the combined effect of a dry climate scenario and bed degradation was analysed by applying the semi-analytical model, showing that those two will have an equivalent contribution to the reduced depth at ALW in 2050. The effect of climate change on flood protection has already been extensively studied, while the combined effect of main channel bed degradation and sedimentation of the floodplain is relatively unknown. The analysis presented in this thesis, show that the combined effect of the bed trends (floodplain and main channel) is a decreasing DWL without accounting for climate change. However, for all climate scenarios an increased DWL is predicted. Hence, those developments have counteracting effects, which is predicted to be dominated by climate change resulting in a net increasing DWL.

Methodology to quantify the performance of river functions

Various methods have been developed to quantify the performance of river functions, which rely on a simplified numerical model and rough assumptions. These methods provide indicative insights into the impact of autonomous developments and measures applied by the river manager, such as a nourishment. Since for navigation and flood protection, formal function requirements have been defined (fairway dimensions ALW and DWL), interpretation of the results becomes easier. In contrast, for other functions, the outcome of the quantification methods serves as insights for (political) discussions. By simulating various scenarios of the (uncertain) autonomous trends, the sensitivity of the impact of those trends becomes clear. The methods also provide the ability to analyse the statistical character of dryer and wetter years, indicating the impact of autonomous trends on extremes. the average impact on navigation and nature, the statistical character of dryer and wetter years has been evaluated, indicating the impact of autonomous trends on the extremes. To assess the impact of a process or measure on the complete multifunctional river system in a quantitative manner, the impact has to be summarized in one parameter. By means of a cost-benefit analysis, the functions flood protection and navigation have been linked to and expressed as their economic value. Therefore, this analysis provides an indication of the cost effectiveness of a measure applied by the river manager. However, since an integrated approach is desired, the financial analysis should never be assessed in isolation, but together with the other methods of performance quantification. The combination of the financial assessment and the quantification of different river functions provides useful information for an assessment of integrated river management. With the experience acquired in this thesis no limitations are foreseen to use this approach for a larger river section and with more complex models and tools, such as Delft-3D.

Important elements for decision-making
This thesis has revealed the importance of an accurate estimation of the autonomous developments. An estimated bandwidth has been used to define the minimum and maximum impact of both climate change and bed degradation. The accuracy of the estimation is an important aspect in the assessment of integrated river management, as for example the annual wellfare loss due to draught restrictions in 2050 can range from €24 million to €104 million as a result of different bed degradation rate. Furthermore, the proportional impact of a river process has been analysed, and revealed a similar impact of a dry climate scenario and bed degradation in 2050. These findings could be important for decision-making regarding measures that mitigate a certain process. Finally, the cost-benefit analysis reveals that a nourishment is cost effective is, while this statement is based on rough assumptions. Appendix J shows the result of the cost-benefit analysis with different input, such as nourishment costs, discount rates and project duration.

6.1.2. Reflecting on the research objectives

The objective of this research is recalled from Chapter 1:

To develop a methodology that evaluates the impact of autonomous river processes on the future performance of river functions and provides insights into the assessment of (integrated) river measures.

The application of the methodology to both the future river without interventions and a nourished river system, shows that this approach suffices to provide useful insights into the impact of autonomous developments and measures applied by the river manager. Although, the simplified model to describe the river conditions, clear future trends are observed when evaluating the river functions. It shows that without interventions the combination of climate change and bed degradation will induce serious problems in the near future during low-flow conditions. It appears that a nourishment improves the river functioning during low-flow conditions and the impact on the design water levels can be reduced by smart sediment management. By means of the combination of a cost-benefit analysis and the function assessment, quantitative information is provided to support an assessment of different sediment strategies. Based on the experience with a small river section, a limited number of functions and a simplified one-dimensional model in this thesis, no problems are foreseen for use of a similar approach for a larger river section, more functions or/and more complex tools and models. Nevertheless, it is noted that the methodology is not a complete tool for full assessment of measures, but it can provide quantitative information for an assessment, which can be useful for decision-making in integrated river intervention measures.

6.2. Recommendations

The recommendations are subdivided in two different categories, namely the recommendations for further academic research and more pragmatic advices for Rijkswaterstaat.

6.2.1. Further research

Study impact autonomous river processes on the riverine ecosystem
Multiple studies have been performed on the impact of climate change on navigation and flood protection in the Rhine, while the Ministry of Infrastructure and Water Management (2018) also touches upon the effect of bed degradation. However, the impact on nature is more difficult to quantify, since little clear function requirements have been defined. This thesis focused mainly on the interaction frequency between main channel and floodplains and side channels, while the effect on dehydration has not been extensively analysed. When combining ecological research fields with the future hydrodynamic conditions, the interpretation of

the inundation frequency or dehydration can be improved. As a cost-benefit analysis has been performed without accounting for the impact on nature, it would also be interesting to express the nature value in a certain economic value.

Study on nourishment sediment from floodplain
Within this research, the use of sediment from the floodplain for nourishment has been analysed. As the Rhine's floodplains are elevating and the main channel is degrading, it sounds as a perfect match to use sediment from the floodplain. However, based on the cost-benefit analysis conducted with rough assumption, it appeared not profitable to use sediment from the floodplain for a nourishment. The impact of sediment extraction from the floodplain on the design water levels is most likely underestimated by the semi-analytical model. The model schematizes the extraction as a uniform lowering of the floodplain, while in practice a deep flood channel can be constructed increasing flood conveyance. Furthermore, the suitability of the material in the floodplains has to be assessed, as mostly finer particles accumulate in the floodplains. However, in deeper layers might be suitable material available. When sediment is extracted from the floodplain, side channels or other water bodies can be constructed, contributing to nature, but also affecting other river functions.

Effect nourishment and bed trends on discharge distribution
Within this thesis, it is assumed that the future discharge distribution at the Pannerdensche Kop remains fixed. However, due to unequal degradation rates in the Rhine branches, a branch could possibly attract more discharge. This has severe implications on the hydrodynamics during low-flow and flood conditions, affecting all river conditions. When a nourishment will be placed near the bifurcation point, which is assumable as Nijmegen is considered as a navigational bottleneck, it will most likely affect the discharge distribution as well. When this is not preferable, measures has to be taken to counteract the affect of a nourishment in the Waal, such as a nourishment in the Pannerdensch Kanaal or adaptation of the regulatory works.

6.2.2. Advice Rijkswaterstaat
This research has been conducted in close collaboration with Rijkswaterstaat linked with the recent launched Programme Integrated River Management (IRM). Hence, the lessons learnt in this thesis has been translated in some concrete advices.

Validate outcomes with more complex models and apply methodology on larger river section
As the future river system has been simulated with a one-dimensional hydrodynamic model that describes the system behaviour and autonomous developments based on simplified physical equations, the results should be interpreted as an indications of the future trends. A fixed bed degradation has been imposed on the river geometry, while not is accounted for morphodynamical processes. Furthermore, the effect of bed trends and nourishment on the discharge distribution has been neglected, while in practice bed changes will affect the discharge distribution changing the hydrodynamics in the Rhine branches. Hence, it is recommended to extended the model and incorporate the estuaries and the bifurcation points of the Rhine. By means of a more complex hydro-morphodynamic model, such as Delft-3D or Sobek, a future river system without stabilization and various sediment management strategies can be simulated and result in more accurate future river conditions. Subsequently, this can be used as input for the quantification methods of the function assessment, resulting in more accurate number that can be used in decision-making of IRM. By using a two-dimensional model, it can be assessed whether there is room for a nourishment height of 12 or 30 cm over the cross-section. In a one-dimensional perspective (depth-averaged) there might be space for 0.3 m nourishment, while over the cross-section this becomes difficult. Furthermore, the morphological changes induced by a nourishment can cause new shallowness. All these phenomena have to be studied in a more complex two-dimensional hydro-morphodynamical model. Furthermore, it is recommended to also evaluate other measures than nourishments, as lowering of the fixed layer could mitigate the depth restrictions at Nijmegen.

Improve estimations of future bed degradation
As the various figures in Chapter 5.2 show, the estimated bandwidth of the bed degradation rate results in a large uncertainty in the assessment of the future river system. A bed degradation of 0 cm/year versus 3 cm/year, results respectively in €24 mln/year and €104 mln/year wellfare loss due to draught restrictions in 2050. Hence, improving the prediction of the future bed degradation, will provide a better foundation to assess mitigation measures.

Develop policy on degrading design water levels
The River Engineering guidelines of Rijkswaterstaat are still in practice (Kroekenstoel, 2017), prescribing that every increase in DWL induced by a measure has to be compensated. With the current policy in place, a nourishment has to be compensated, as it increases the design water levels compared to the situation before the nourishment. As bed degradation has lowered the design water levels the past decades, a return to the past design water levels is not possible without compensation following the River Engineering guidelines of Rijkswaterstaat. Within WBI2017 (the assessment instrument of flood defences safety standards), the bed level of 2013 is used to compute the hydraulic load, not accounting for bed degradation. In 2023 a new WBI instrument will be launched, most likely updating the river bed geometry incorporating the bed degradation. However, no policy is in place to account for future decrease of design water levels due to bed degradation, while this will influence the future reliability of the Dutch flood defences. With the current policy in place it is also expensive to mitigate bed degradation by nourishing, as this will increase flood levels requiring an expensive compensation or dike heightening by the responsible party (Rijkswaterstaat).

Define clear function requirements considering nature
IRM aims to incorporate all river functions in the approach to realize a future proof multifunctional river system. As flood protection concerns the safety of majority of the Netherlands, a legal base has been established to guarantee sufficient safety (Veerman, 2008), resulting in clear function requirements (the safety standards of flood defences). Also navigation has a strong position in the political debate, as the Rhine is an important transport corridor. This results in clear fairway dimension requirements, that recently have been enlarged. Different programs are running to enhance nature along the Rhine (e.g. (NURG and WFD) and a large part of the Rhine is part of Natura2000. However, little to no clear function requirements related to river conditions are specified, which can be used to clarify the requirements of the riverine ecosystem along the Rhine. When no clear function requirements are provided in a methodology as presented in this thesis, it becomes difficult to interpret the outcomes of different measures.

Relation with HWBP
As currently a large part of the flood defences along the Dutch Rhine is reinforced, there is an opportunity to account for flood level elevation due to nourishments or other interventions to mitigate the problems during low-flow conditions. Appendix J.5.4 shows how the cost-effectiveness of those measures is reduced, when the dike heightening cannot be linked with the HWBP works and also has to be accounted for the fixed costs of dike reinforcement. Hence, it would make sense to make a decision of the required extra dike height (required to mitigate low-flow problems) on the short-term, to be able to connect with the works executed in the HWBP.

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A

Interpretation of bed level changes

A.1. Review of human interventions

Till 1000 years ago the Rhine system man kind did not considerable influence the river system, and it was mainly formed by natural processes. During the Medieval Period the human influenced increased rapidly due to increasing cultivation, draining and embanking of peat, the river area decreased. In the lower areas peat extraction led to subsidence of the surface, which made them vulnerable to flooding. Due to the increased water levels in embanked river channels, dike breaches were common. This led to the water management organisation structure (Dutch: Waterschappen) as is still used nowadays. In the 19th century Rijkswaterstaat proposed a unified normalisation to reduce flooding and improve navigability in periods of low flow (Bosch and van der Ham, 1998). Large scale investments were made in

- cutting off sharp bends
- fixation of eroding bends
- reduction of width of the low-flow bed
- concentrating flow in one main channel
- dredging of shallows

At the same time large scale river training activities were executed in the Niederrhein by the Germans. They also experimented with groynes perpendicular to the flow. Rijkswaterstaat followed their work at regular interval along both banks of the Rhine branches. These so-called normalisation measures resulted in a considerable loss of surface area of the river Rhine. The surface area of the Rhine river plains was reduced to less than half of its size since 1850 (Klein et al., 2017). Hence, the measures induced by the normalisation significantly affected the present river system. As result of all those measures the safety level of flood defences has improved and navigation was enabled following the Mannheim Act (1850). However, the normalisation also triggered negative changes, since bottom and water level are slowly changing (Sieben, 2009).

After the North Sea Flood of 1953, Rijkswaterstaat constructed the Delta Works, which changed the tidal behaviour of the Waal and made the IJssel a non-tidal river. In the middle of 20th century weirs were constructed in the Lower-Rhine to allow navigation during low-flow. The same happened to the upper Rhine and tributaries. Despite the normalisation measures the navigability was still hindered in the late 20th century, therefore different measures were introduced. Due to local effects (spiral flow) in river bends measures were introduced to maintain a fixed width and depth. In the river bends of Nijmegen and st. Andries an armoured layer was placed and at Erlecom bendway weirs were constructed. Other navigation bottlenecks are mitigated by dredging activities (Havinga, 2016).

After the consecutive floods of 1993 and 1995, new river intervention works were built within the Room for the River (RfR) Program to accomondat for higher flood discharges in the future. Within this program over 30 projects were realized increasing the flood conveyance capacity as well as the biodiversity (e.g. increasing

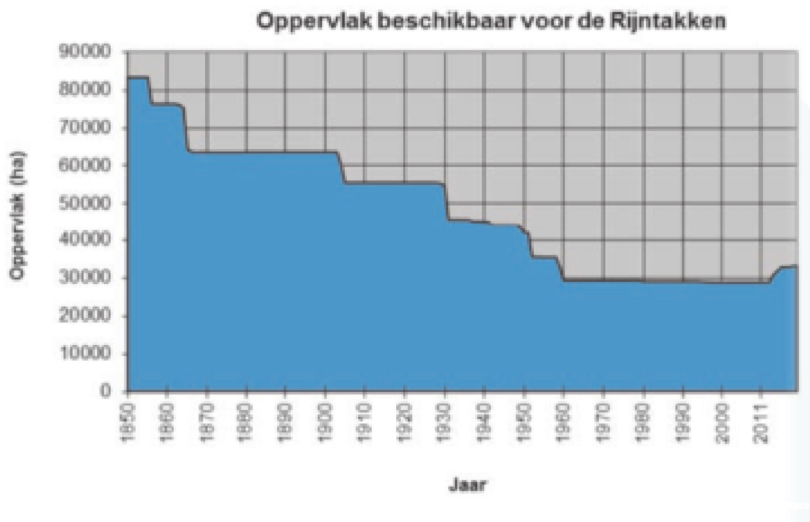


Figure A.1: Surface area of the river Rhine in ha. Reproduced from Klein et al. (2017).

flood plain area, new gullies and channels, removal of obstacles etc.). The RfR Program is designed to increase the flood conveyance capacity to prevent flood levels to increase. At the same time other large scale projects started to a) restore nature in the floodplains, b) to secure water quality (European Water Framework Directive, or WFD). Typical measures are: lowering flood plains (750 ha), reconnecting lakes (15), groyne lowering (40 km), side gullies (40 km), longitudual dams (10 km), free banks (80 km) and oxbow lakes (35 km). As Van Vuren et al. (2015) showed the measures of RfR and WFD may also increase river bed dynamics and results in an increase in maintenance dredging of approximately 10 % compared to the present day situation.

Also a stricter policy considering vegetation in the flood plains had developed. Due to vegetation succession (i.e. the evolution of plant communities at a site over time from pioneer species to climax vegetation) the roughness of the river increased during high flow, resulting in higher water levels. In 2014 an instrument was launched to maintain norms for the vegetation succession in the flood plains (Vegetatielegger, 2014). To ensure sufficient conveyance in the flood plains a Programme has been initiated (2015-2018) to quickly reduce the roughness of the flood plains (Rijkswaterstaat, 2014). On the other hand NGOs desire a more dynamic vegetation management approach on a larger scale to allow cyclic processes of vegetation succession (Beekers et al., 2017).

A.2. Theoretical background of bed level changes

To interpret the trends in bed level changes of Figure 2.3 due the normalisation measures, the changes in terms of sediment transport are analysed. Due to the large-scale reduction in width due to the normalisation, the sediment capacity of the Rhine branches increased with the proportionality

$$\frac{S_{new}}{S_{old}} = (\frac{B_{new}}{B_{old}})^{1-b/3} \tag{A.1}$$

with $S\,[m^3/s]$ the average sediment transport, with the subscripts *old* and *new* referring to the situation before and after the normalisation Sieben (2009). The coefficient b is the power of velocity in a sediment transport predictor, with b=5 conforming to Engelund and Hansen (1967). Hence, when simplifying the Waal branch to a uniform branch with constant discharge, an average width of the active river bed of 360 or 460 to 260 will result in an increase of transport capacity of 20 to 40 %. According to Janssen et al. (1979), an incision of the bed levels induces a reduction of the sediment transport capacity of

$$\frac{S_{new}}{S_{old}} = (\frac{i_{new}}{i_{old}})^{b/3} \tag{A.2}$$

When again consider the Waal branch, following Figure 2.3 the slope reduced from 0.11 m/km to 0.10 m/km between 1900 and 2000. This incision of bed level corresponds to transport capacity reduction of 20 % following Eq. (2.2). This means that the sediment transport capacity has already been decreased over the past decade, while when this process continues the sediment supply and sediment capacity will match in the future resulting in a new equilibrium situation.

Also other processes were triggered during the 20th century. Because bed levels degradated more rapidly in the Pannerden Canal, the discharge tended to shift to the the Pannerden Canal. Obviously, this affect water levels in both the Waal branch as the IJssel and the Lower-Rhine branches. Janssen et al. (1979) revealed the effect of changing discharges on sediment transport capacity with

$$\frac{S_{new}}{S_{old}} = (\frac{Q_{new}}{Q_{old}})^{b/3} \tag{A.3}$$

With an estimated decrease of 10% in the River Waal by Schropp (2002), Eq. (A.3) predicts a 15% decrease in transport capacity. Anyhow those assumption are also influenced by the sediment distribution at the bifurcation.

Other bed level changes are expected due the more more confined flow between the groynes, which is the result of falling water levels.

B

Reliability analysis of overtopping

The failure probability of overtopping has to withstand the hydraulic load consisting of both a water level and waves respectively forced by a discharge and wind conditions. This appendix will elaborate on the derivation of the design flood discharge related to the failure mode overtopping. Figure B.1 illustrates the study area with the corresponding target reliability of the dike trajectories. A contribution of this target reliability is dedicated to the failure mode overtopping (25 %). Based on this the target reliability of the failure mode overtopping is assumed to be more or less equal to 1/40 000. The hydraulic load associated with overtopping, consist of both wave and water action. The water levels solely depend on the upstream discharges since both wind conditions and the influence of the sea do not affect flood levels in this part of the Waal. The water and wave action are assumed to be independent in this part of the Waal, while a combination of both events should not exceed the target reliability of the failure mode overtopping. The normative hydraulic load situation is in most cases in the order of a 1/10 000 discharge condition and a 1/4 wind conditions, but could differ due to different aspects, such as a longer fetch length (i.e. higher failure probability due to waves). So in general, this cascade of statistics results in a similar target reliability of the dike trajectory as an exceedance probability of discharges.

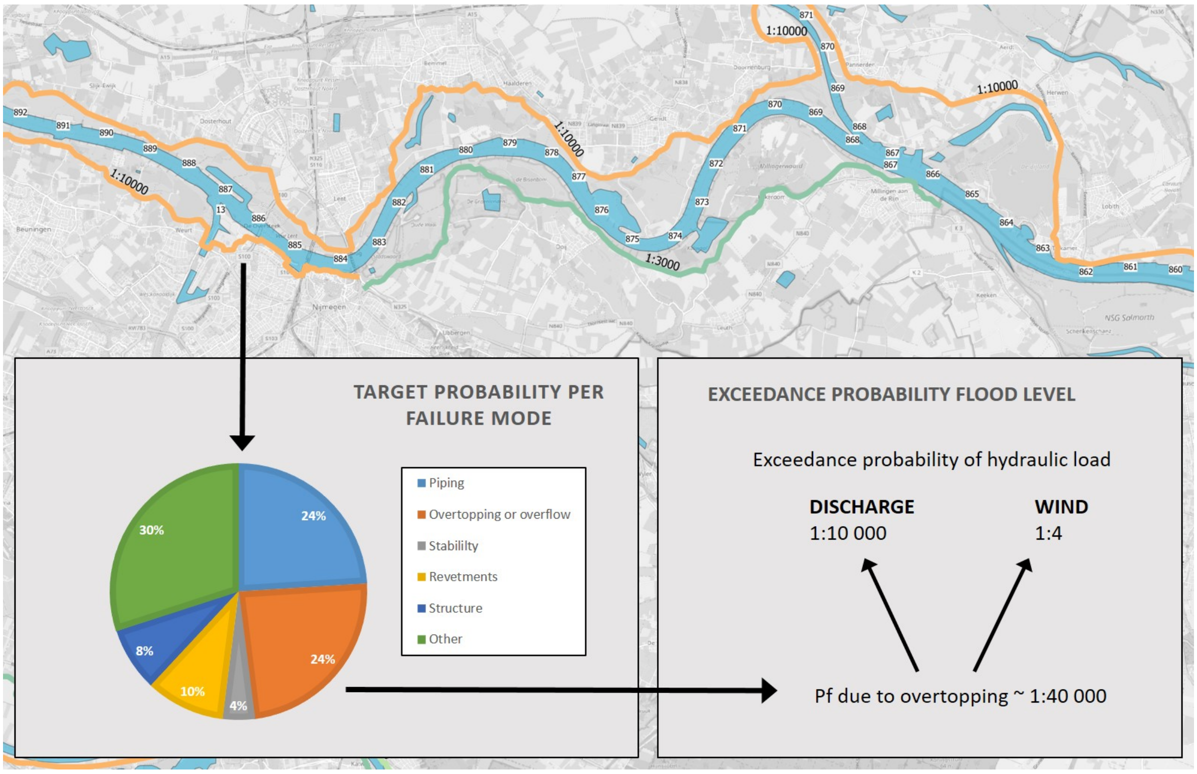


Figure B.1: The norm frequency of the dike trajectories with in orange 1:10 000 and in green 1:3000 of the study area translated in a contribution per failure mode and eventually in hydraulic boundary conditions

C

Review of nourishments in the (German) Rhine

In Germany sediment nourishment have already been an accepted method within the management of German rivers. After construction of the first weirs at Iffezheim in 1978, sediment nourishment were for the first time initiated to reduce the bed level degradation (Rudolph, 2018). Ever since, approximately 181,000 m^3/year of sand-gravel mixture have been deposited near Iffezheim (rkm 336-338) (Hillebrand and Frings, 2017). Also closer to the Dutch border in the Niederrhein sediment has been nourished (Table C.1), which has successfully decreased the bed degradation (Rudolph, 2018). As mentioned earlier, this nourishment travels downstream and will eventually affect the Dutch Rhine. Sieben (2009) stated that the nourishment in the Niederrhein may contribute to the stabilization of the Dutch Rhine bed. However, as Blom (2016) pointed out they might first cause erosion due to the fact the river is not able to transport the courser gravel in the Niederrhein, which results in less sediment supply in the Dutch part of the Rhine. In other words, the nourishment in the Niederrhein may already contribute to the current bed level degradation in the Dutch Rhine branches.

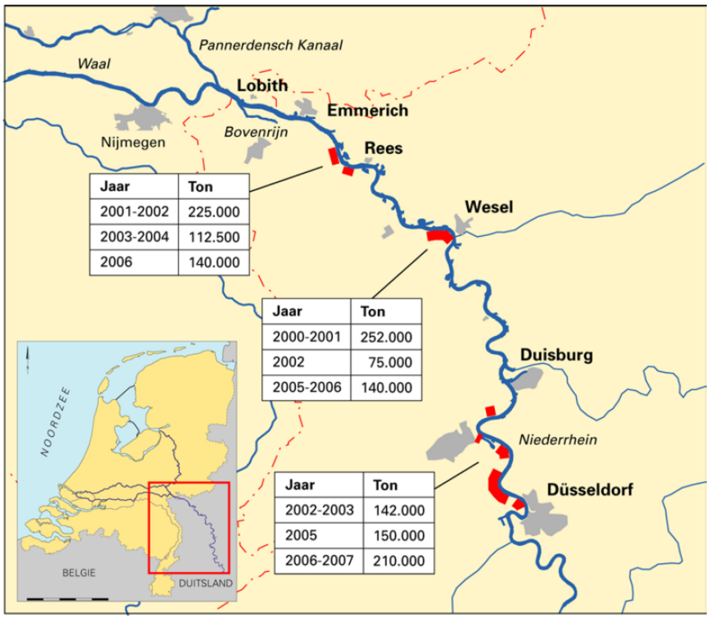


Figure C.1: Locations and quantities of nourishments in Niederrhein. Reproduced from Rijkswaterstaat (2007)

In the Netherlands Rijkswaterstaat is experimenting with the nourishment technique as a possible mitigation measure for the bed degradation. Near Lobith (rkm 862-864.2) 70,000 m^3 of a gravel-granite-mixture has been nourished. In total a layer of approximately 30 cm has been deposited over a length of 2.3 km, in between shallower points to avoid local shallowness. To assess the effectiveness of those nourishments, the response of the bed is currently monitored and studied in more detail using numerical models.

D
Model set-up

D.1. Qh-relation

At the downstream boundary a Qh-relation is imposed as boundary condition. The Qh-relation is constructed based on the stage relation curve of 2016 (Dutch: betrekkingslijn), which relates discharge and water level at Lobith with water levels downstream. As the stage relation curve is not a continuous dataset covering water levels for every discharge, it has been fitted to a logarithmic function. As one universal logarithmic function did not give the desired accuracy for the entire discharge range. Hence, multiple logarithmic function has been fitted for different (Waal) discharge stages, as follows:

$$Q < 1000\text{m}^3/\text{s}: h = 2.33 \log Q - 11.863 \tag{D.1}$$

$$1000 \text{ m}^3/\text{s} < Q < 1400 \text{ m}^3/\text{s}: h = 3.9585 \log Q - 23.128 \tag{D.2}$$

$$1400 \text{ m}^3/\text{s} < Q < 4000 \text{ m}^3/\text{s}: h = 3.7355 \log Q - 21.671 \tag{D.3}$$

$$4000 \text{ m}^3/\text{s} < Q < 7071 \text{ m}^3/\text{s}: h = 3.7364 \log Q - 21.51 \tag{D.4}$$

$$Q > 7071 \text{ m}^3/\text{s}: h = 2.3701 \log Q - 9.4322 \tag{D.5}$$

The logarithmic functions are lowered with a certain rate to account for the bed degradation. As the bed is expected to degrade 1 cm/year, the responds of the equilibrium water level is analysed for different discharges to account for an effect on the logarithmic functions.

D.2. Bed degradation rates

As discussed in Section 4.3.1, the future bed degradation is predicted based on trend observations (Sieben, 2009, e.g.) and morphological studies (Sloff et al., 2014, e.g.). To account for the local, such as effects of non erodible layers, the degradation rate has been differentiated over the considered study area. The bed degradation in the upper Waal is estimated to continue at a rate of 1.5 cm/year, while this is not the case at the river bend of Nijmegen (fixed layer, rkm 883-885). The non-erodible bed protection is only present in the outer bend allowing the inner bed to degrade resulting in best estimate width averaged degradation rate 0.5 cm/year. Directly downstream of Nijmegen a more severe bed degradation of 1.5 cm/year is assumed, which is interpolated to zero bed degradation at the hinge point of Tiel (rkm 913). Figure D.1 show the imposed expected future bed degradation (base case), while Table D.1 elaborates the volume trends within the river section. The degradation rates explained above, can be considered as the estimated degradation rate for the coming 20 years. However, Sieben (2009) revealed a degradation rate of 3 cm/year between 1950-2000 in the Upper-Waal, while also stabilization could be possible as is the case in the Upper Rhine. Therefore, a best estimated bandwidth of the degradation rate is defined with a minimum and a maximum bed degradation rate, which are respectively zero over the entire river section and a doubling of the base case degradation rates.

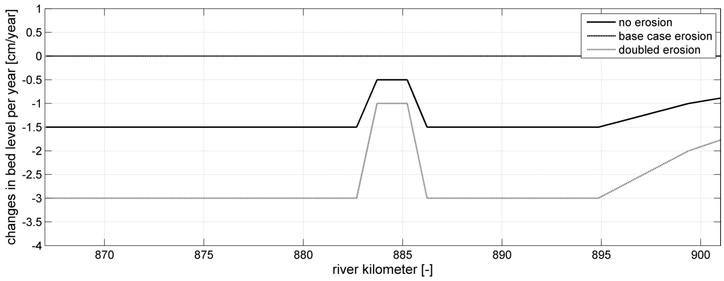


Figure D.1: The bed degradation rates scenarios in the river section used as input for the model,

Trajectory	Normal width [m]	Mean bed trend [cm/year]	Volume trend [m^3/year]
Upper Waal (rkm 867-882)	257	- 1.5	- 57 825
Fixed layer Nijmegen (rkm 883-885)	257	- 0.5	- 386
Middle Waal (rkm 886-895)	255	- 1.5	- 34 425
Middle Waal (km 895-902)	255	- 1.0	- 17 850
Total	-	-	- 110 486

Table D.1: Volume trends in different sections of the study area

D.3. Flood channel Lent

In 2015 the flood channel at Lent (Nijmegen) has been finished as part of the RfR program, to lower flood levels. However, the numerical model of Chapter 4 uses a river geometry of 2013 not including the flood channel, while the impact on flood levels is up to 35 cm (de Jong et al., 2010). The discharge through the flood channel is regulated by an inlet at rkm 883, which start to overflow at 4500 m^3/s (Lobith). At discharge below 4500 m^3/s the channel is not contributing to the flow, while during extreme discharge of 16 000 m^3/s 34% of the discharge of the Waal is flowing through the flood channel (de Jong et al., 2010). As the flood channel is not been included in the model, it will be schematized as a discharge extraction from the main river between rkm 883-886. The discharge extraction depends on the the discharge at Lobith, as explained above. The effect of the implementation of the flood channel Lent in the model is shown in Figure D.2.

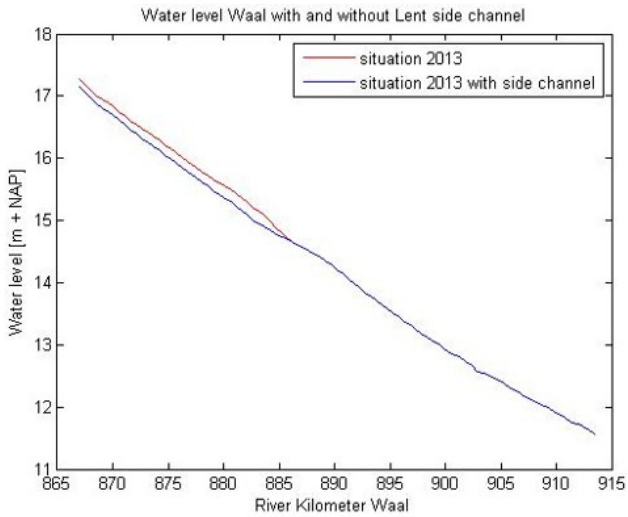


Figure D.2: Indication of effect of the flood channel at Lent for high discharges

E
Model calibration

Section 4.3.2 discusses the calibration process of the numerical model. The model has been calibrated based on discharge and water level recordings of January, May and June 2018. Firstly, the geometry obtained from Sobek 2013 has been used with the model principles discussed in Section 4.3.1 and the hydraulic roughness and slope from Table 4.1. The results of the first model simulations for various discharges are compared with the measurements (Figure E.1). As the previous appendix discusses, the situation of 2013 has to be transposed to the actual situation accounting for bed trends and the finished RfR projects. The relative difference between the model results and the measurements can be evaluated in Figure E.1. However, the desired accuracy (approx. 10 cm) has not been achieved, which requires calibration by means of adjusting the hydraulic roughness (Manning coefficient) of the main channel. Various Manning coefficients has been analysed, while the best results are obtained for dynamic Manning coefficient depending on the discharge regime (rougher river bed during flood conditions). In the end, the Manning coefficient is varies between 0.26 and 0.29 equivalent to a Chezy value of 45 during ALD with the results presented in Figure E.1.

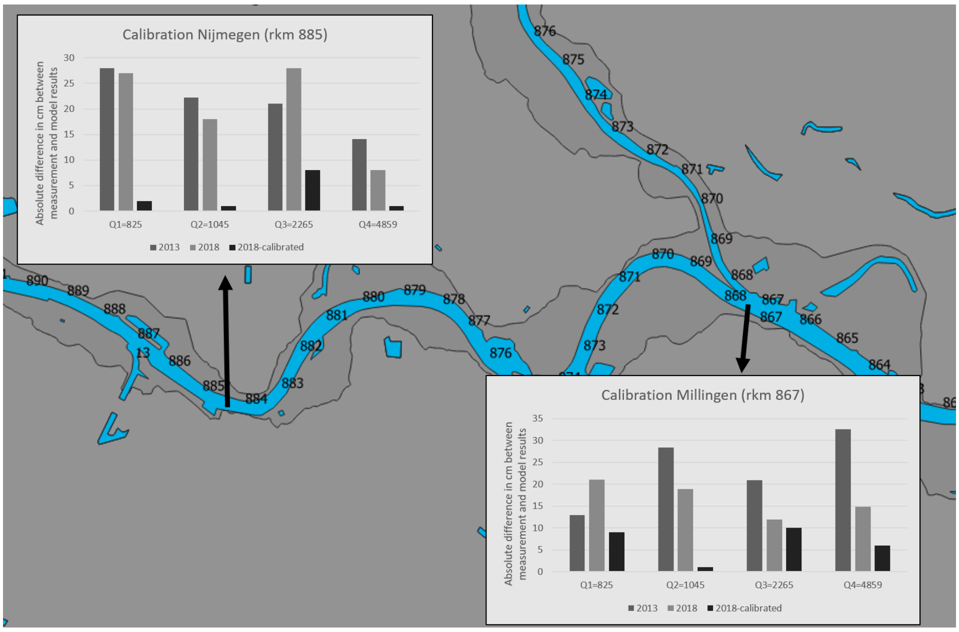


Figure E.1: Overview of the calibration process and validation of the model by means of the absolute difference between the results of measurements and the first model results '2013', the adjustments to transpose to the situation of 2018 and the calibrated 2018 model.

As the discharges observed in 2018 range from 750 to 7500 m^3/s at Lobith, the flood discharges above 7500 m^3/s have to be calibrated by means of results of simulations of the numerical model WAQUA. From the database Hydra-NL, the WAQUA water level simulations (without model uncertainty) are acquired for different return periods. By means of adjusting the hydraulic roughness, the desired accuracy has been achieved

(Table E.1). However, it should be noted that the WAQUA simulations are executed with the bed levels of 2013 instead of 2018.

Return period	Discharge at Lobith [m^3/s]	Difference [cm]
1/100	8090	8
1/1000	9467	2
1/10 000	10522	0

Table E.1: Absolute difference between model results and WAQUA simulations at Nijmegen.

Return period	Discharge at Lobith [m^3/s]	Difference [cm]
1/100	8090	10
1/1000	9467	9
1/10 000	10522	14

Table E.2: Absolute difference between model results and WAQUA simulations at Erlecom.

F
Discharge distribution

As is observed in Figure 4.6, the discharge distribution - the relationship between discharges at Lobith and the Waal river - is highly uncertain. Following Ogink (2006) the most important source of uncertainty is the morphodynamics and the roughness of the main channel and the flood plain. Other uncertainty is the way the regulations works are managed. Despite the uncertainties within this research the discharge distribution is assumed to be fixed following Table F.1 (an interpolation is applied to calculated the discharges in between). However, the discussed uncertainties explain the difference in the simulated Waal discharges and the recorded discharges. In Figure F.1 the hydrograph for the recorded discharges are compared with the simulated discharges for respectively the year 1961 and 2017. As can be seen the trends are followed by the simulated discharge. However, there is a slightly mismatch at the extremes.

Discharge Upper-Rhine [m^3/s]	**Discharge Waal [m^3/s]**
823	673
1013	813
1220	970
1430	1130
1500	1185
1780	1285
2120	1460
2320	1575
2440	1655
2770	1870
3160	2140
3590	2425
4060	2735
4540	3045
5080	3410
5675	3820
6345	4265
7095	4755
8950	5890
10085	6560
11415	7345
13005	8310
14640	9310
16000	10150

Table F.1: Fixed discharge distribution obtained from RWS ON

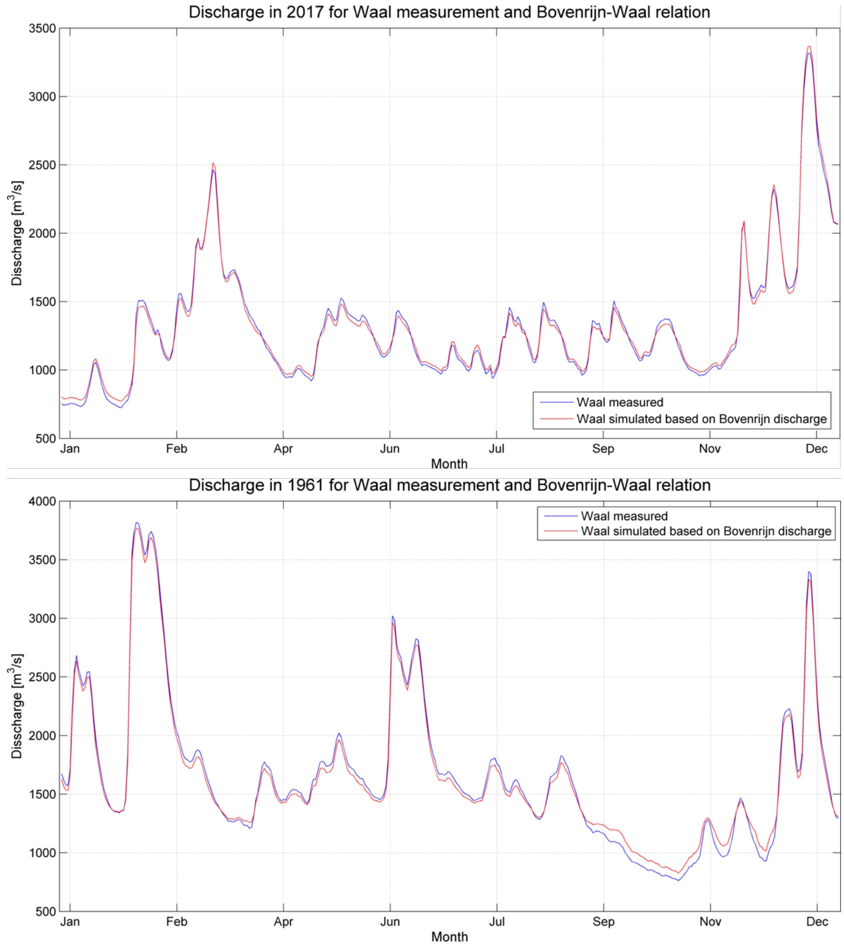


Figure F.1: The hydrograph of the year 1961 (top view) and 2017 (bottom view) for the simulated discharges (red) and the recorded Waal discharges (blue)

G

Data-analysis to determine ALD

The required fairway dimensions are defined for a certain discharge condition ALD, which corresponds with a certain water level (ALW). This discharge condition has been defined, as the discharge at Lobith, that is not exceeded 20 days a year (5 %) without accounting for days with ice. Currently, the ALD has been defined as 1020 m^3/s at Lobith based on the current discharge statistics. However, climate change is likely to affect the discharge distributions, which will eventually also change the ALD.

When applying a data-analysis for the historical discharge records between 1901 and 2017, a value of 1015 m^3/s is obtained for the 95 % exceedance value. This is slightly smaller than the actual ALD, which could be the result of (1) the fact that the data-analysis does not account for days with ice and (2) the current ALD has been defined in another way or based on another dataset. The discharge time series for the climate scenarios $W_{H,dry}$ and G_L has been obtained from the memo of Mens and Kramer (2016), for which a data-analysis has been conducted resulting in a ALD of respectively 825 and 1102 m^3/s. Figure G.1 shows a plot of the exceedance probability of discharges for the historical discharge records, the adapted discharges of $W_{H,dry}$ and G_L.

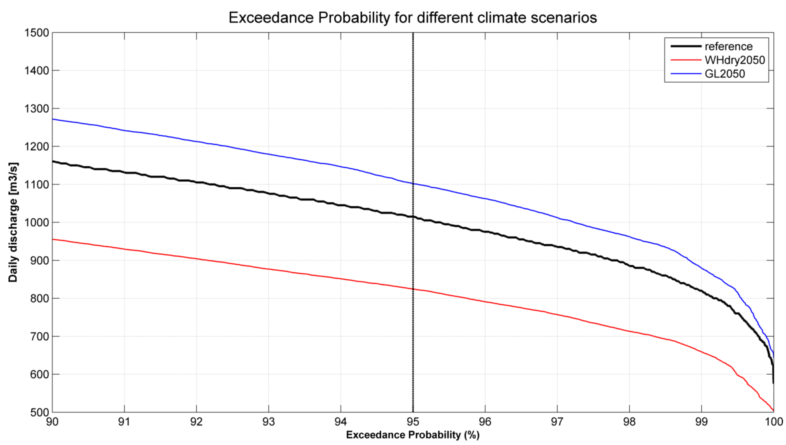


Figure G.1: The exceedance probability of discharges in the dataset of the historical discharge records (black) and the adapted discharge dataset for climate scenario $W_{H,dry}$ (red) and G_L (blue)

H

2D bed corrections

H.0.1. Transverse slope

As discussed in Section 4.4.1 a correction is applied to account for the transverse slope in river bends. As Struiksma et al. (1985) proposed the lateral bed slopes can be approximates by:

$$\tan i_t = -A f_s(\sigma) \frac{d}{R_b} \tag{H.1}$$

in which $f_s(\sigma)$ is a function of the Shields parameters σ, d is the water depth [m] and R_b is the radius of curvature [m] and A is the secondary flow direction coefficient. The water depth and radius of curvature depend on the hydrodynamic conditions, which are constantly changing throughout the year. Hence, the characteristic morphological discharge ($Q_{Lobith} = 2700$ m^3/s) will be used to compute the characteristic hydrodynamic conditions. The secondary flow direction coefficient is defined as ((De Vriend, 1977):

$$A = \frac{2\epsilon}{\kappa^2}\Big(1 - \frac{\sqrt{g}}{kC}\Big) \tag{H.2}$$

in which ϵ represents the tuning coefficient [-], κ the Von Karmann coefficient (in general equal to 0.4) [m/s], Ch is the Chézy coefficient [m$^{1/3}$/s] and g is the gravitational acceleration [m/s^2]. Next, Talmon et al. (1995) defined an approximation for the function $f_s(\sigma)$ as:

$$f_s(\sigma) = 9\Big(\frac{D_{50}}{h}\Big)^{0.3}\sqrt{\sigma} \tag{H.3}$$

with D_{50} [m] defined as the median grain size of the bed material. Now the width-averaged water depth can be corrected as follows:

$$d_{min}(x,t) = \overline{d}(x,t) - d_c or(x) = \overline{d}(x,t) + \Big(\frac{B(x)}{2} - B_S(x) - B_N(x)\Big)(i_t(x) \tag{H.4}$$

in which d_{min} is the critical depth in the cross-section of the fairway [m], $d_c or$ is the transverse slope correction, B is the fairway width [m], B_S is the safety distance from the river bank (typically 25 m) [m], B_N the fairway width [m] and i_b the transverse slope [-].

H.0.2. Bedforms

This section elaborates on the bedforms correction of Section 4.4.1 in which already some details have been discussed to predict bedform dimensions. The equilibrium dune height is predicted by the relationship defined by Bradley and Venditti (2017). However, as Allen (1976) stated the dune height within the river depend on flow conditions in the past and the time scale of adaptation. This is described relaxation behaviour is described by Equation 4.11 in which T_H is the time scale of bedforms adaptation. This parameter is equal to

$$T_H = L_H / c_H \tag{H.5}$$

in which L_H is the characteristic length scale [m] and c_H the velocity of propagation of dune height variations [m/s]. The characteristic length scale is assumed to be 90 m approximately two times the dune length. The velocity of propagation of dune height variations depends on flow conditions and sediment characteristics. It is assumed that this velocity is proportional to the velocity of propagation of disturbances in the longitudinal bed profile. De Vriend (1965) derived the following analytical equation for this velocity

$$c = \frac{s * b}{h * (1 - Fr^2} \tag{H.6}$$

in which c is velocity of propagation of disturbances in the longitudinal bed profile [m/s], s the sediment transport [m^3/s], b the coefficient of non-linearity in the sediment transport formula [-] and Fr the Froude number [-]. The sediment transport is described by the sediment transport fomula of Meyer-Peter and Müller (1948) is defined as

$$s = a * u^b \tag{H.7}$$

in which a is a coefficient that is assumed to be $1.9e-5$. The coefficient of non-linearity (b) is a function of the Shields parameter

$$b = \frac{3}{1 - \frac{\theta_{kr}}{\mu * \theta}} \tag{H.8}$$

Now the velocity of propagation of dune height variations can be derived by multiplying this velocity of propagation by coefficient γ

$$c_H = \gamma * c \tag{H.9}$$

The coefficient γ is assumed to be equal to 1 in the Waal. Figure H.1 shows the results of bedform computations and the affect of this the bedform lagging effect.

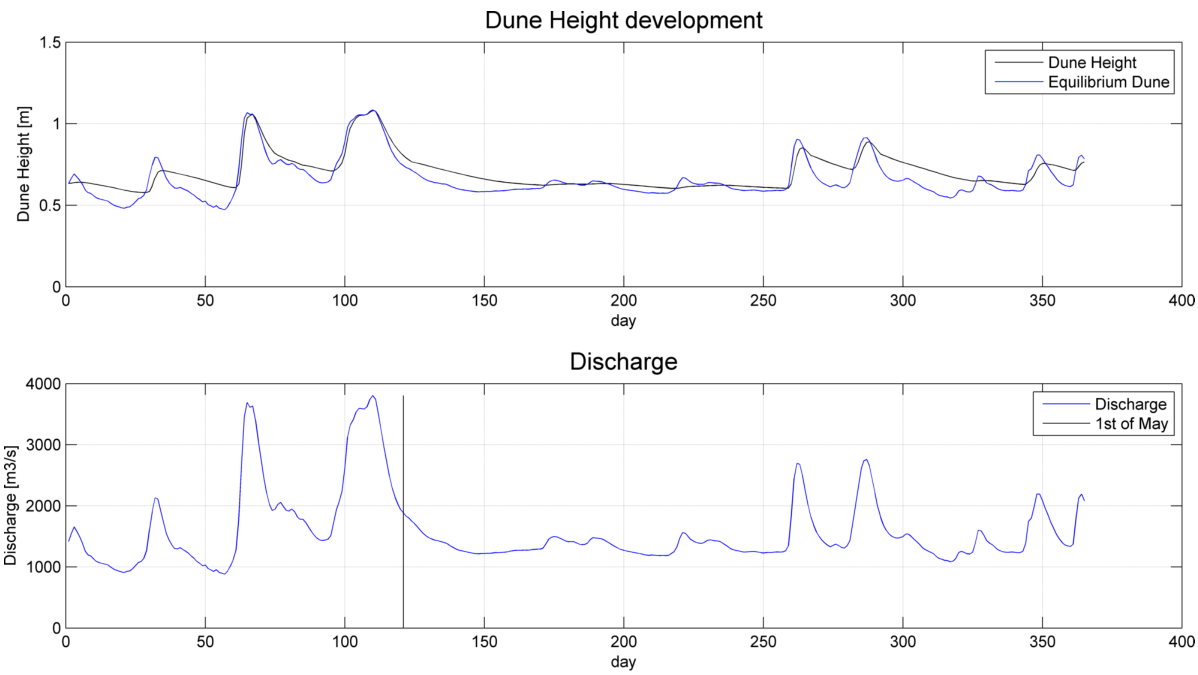


Figure H.1: Simulations of bedform development at the Pannerdense Kop with the discharge time series of the year 1901. With in the bottom view the discharge throuhout the year and in the top view in in blue the equilibrium dune height predicted the approach of Bradley and Venditti (2017) and the lagged dune height based on Equation 4.11.

H.0.3. Non-erodible layers

Figure H.2 shows the design of the bed protection in the outer bend near Nijmegen. This layer consist of a different types of rock up to 10-60 kg. The height of the bed protection at the downstream edge (rkm 885), is elevated 2.15 + m + NAP.

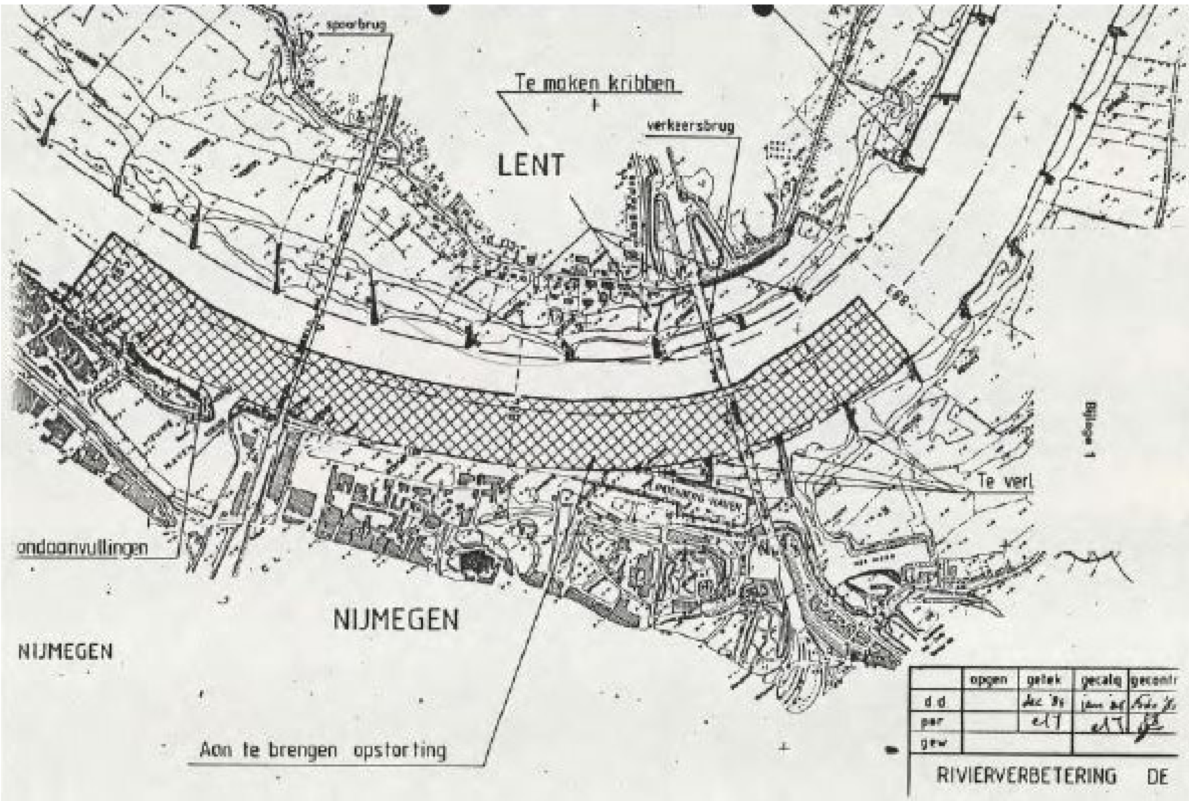


Figure H.2: Design of the bed protection. Reproduced from Franssen (1995)

I Results

I.1. Future river system without stabilization

I.1.1. Navigability typical Rhine vessel

This appendix explains how the statistics of the load factor of a typical Rhine vessel can be obtained. As the water depth for the entire discharge time series (ref: 1901-2017) is simulated, the load factor can be determined for every time step. The entire dataset can be translated in the cumulative distribution of Figure 5.6, a percentage of navigable time with a certain load factor or the annual averaged statistics can be determined. Figure I.1 show the statistics throughout a year (2003 and 2008), indicating the variability. From these figures an annual averaged value can be determined. 2003 is a relatively dry year, which might contribute to the 5^{th} percentile value, while 2008 is a more regular year. From the annual averaged values the Figure 5.9 can be constructed.

On the other side the percentage of navigable time of the various load factor can be calculated from the time series. Figure 5.7 show the statistics of river state 2018 without incorporating climate change. Hence, Figure I.2 will illustrate the impact on various load factors for the river state 2050 with the selected climate scenarios.

I.1.2. Flood protection

In the main text only the DWL at Nijmegen is assessed. As the flood conveyance capacity of stretches in the river section vary, the impact of both bed degradation and a nourishment can be different along the river section. This appendix will assess the DWL at the Dodewaard (rkm 901) which is the downstream edge of the numerical model and Pannerdensche Kop (rkm 867) the upstream edge of the model. Figure I.3a and I.3b show the development of the design water levels in time, while Figure I.4 facilitates comparison between the locations in 2050 without accounting for climate change. It appears that the DWL at Pannerdensche Kop is more affected by the bed degradation and climate change than the other locations. Changes in either the conveyance capacity formula or discharge forcing due to respectively bed degradation and climate change have a greater impact on the DWL at the Pannerdensche Kop than at other locations. This could be dedicated to relatively more conveyance capacity of the main channel than the floodplains in the backwatercurve area

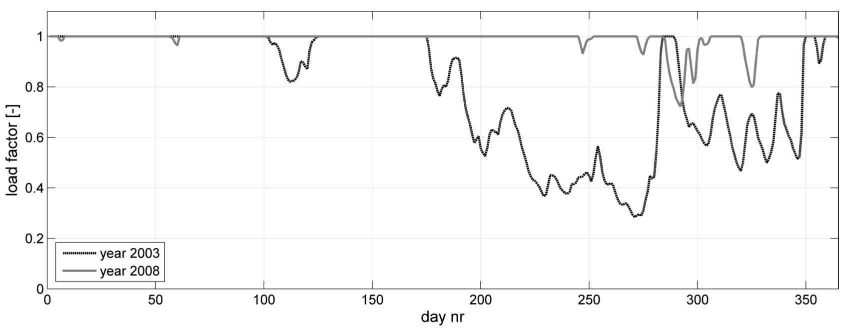
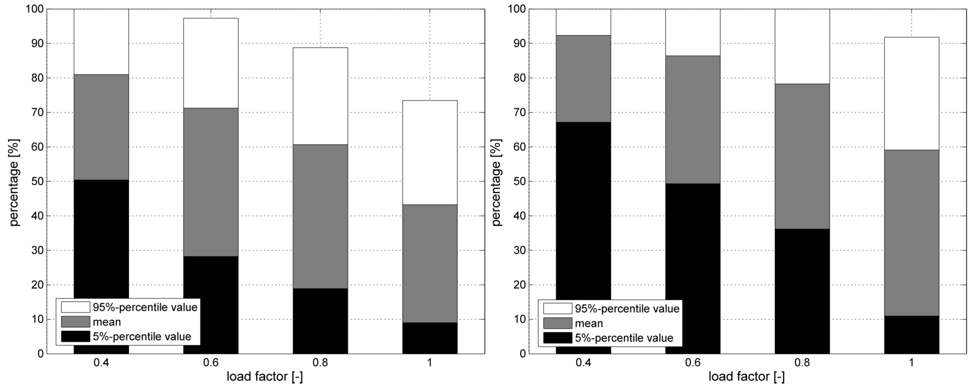


Figure I.1: Load factor of a typical Rhine vessel throughout the year 2003 and 200 for river state 2018.

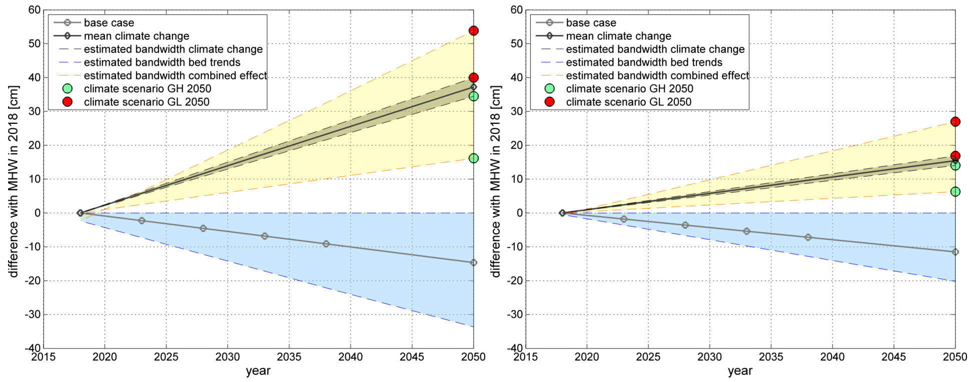


(a) Statistical characteristics of the percentage of navigable time for KNMI'14 scenario $W_{H,dry}$ in 2050.

(b) Statistical characteristics of the percentage of navigable time for KNMI'14 scenario G_L in 2050.

Figure I.2: Percentage of navigable time as a function of the load factor of a typical Rhine vessel.

of the Pannerdensche Kop. Furthermore, it can be noted that this has a considerable impact on the discharge distribution, as the water level at the bifurcation point will directly affect the discharge distribution.



(a) Pannerdensche Kop (rkm 867)

(b) Dodewaard (rkm 901)

Figure I.3: Development of design water levels including the estimated bandwidths.

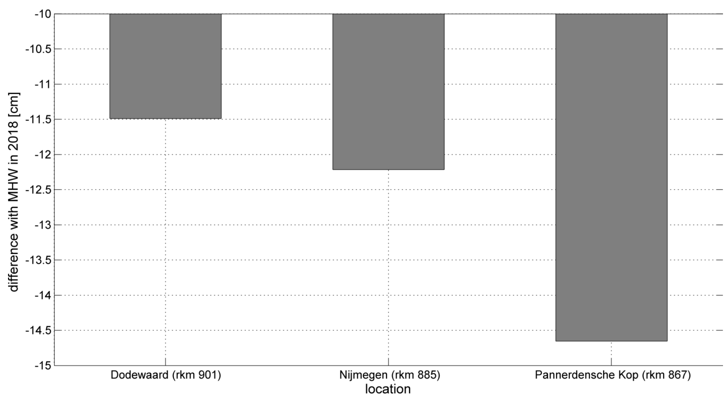


Figure I.4: Difference in DWL in 2050 compared to DWL in 2018 for various locations.

I.2. Sediment management strategies

I.2.1. Dredging activities

In the main text the dredging volumes without climate change were discussed. To also allow an assessment of the impact of a nourishment in combination with the climate change scenarios, this appendix will elaborate on that. Figure I.5 shows the impact of scenario $W_{H,dry}$ and G_L. As discussed in the main text, it is expected that the mean dredging effort will increase. However, Figure I.5 also shows a considerable impact of a dry climate scenario. For 'nourishment 1997' the dredging volume almost reaches the 500 000 m^3 per year.

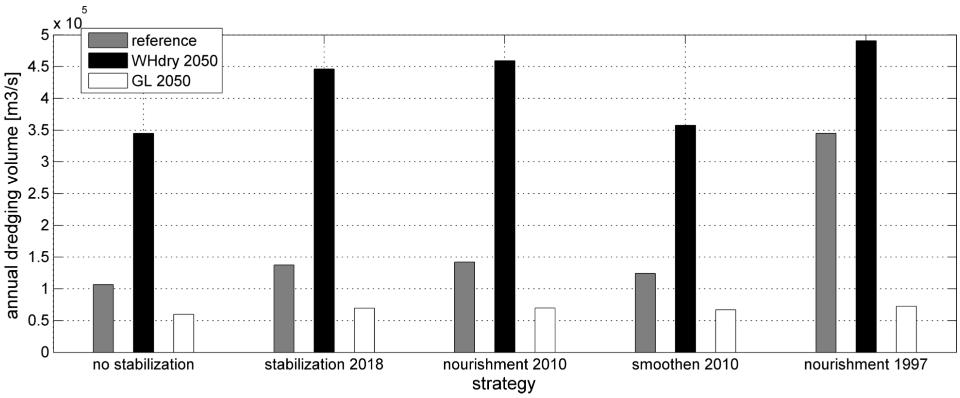


Figure I.5: Predicted mean annual dredging volume for different sediment management strategies in 2050. The range indicates the impact of the climate change scenarios.

Furthermore, the various stabilization frequencies of 'nourishment 2010' are assessed in this appendix. Figure I.6 indicates the minimum and maximum during the period of 2018 to 2050, as after stabilization the river bed will degrade following the expected bed degradation rate. As the river bed of 'nourishment 2010' has to be guaranteed, less frequent stabilization requires a larger nourishment which subsequently increases the dredging effort. This is also observed in Figure I.6.

I.2.2. Navigation efficiency

This appendix section will assess impact of sediment management strategies in combination with climate change. It appears that 'nourishment 1997' limits the damage of a dry climate scenario, as it slightly decreases below a load factor of 0.9. When evaluating the wetter climate scenarios, the navigational efficiency is further increased. Figure I.8 indicates the impact of the stabilization frequency on the mean annual load factor. Less frequent stabilization results in more water depth as shown in Figure 5.18, resulting in an increased load factor. This will eventually result in reduced wellfare loss.

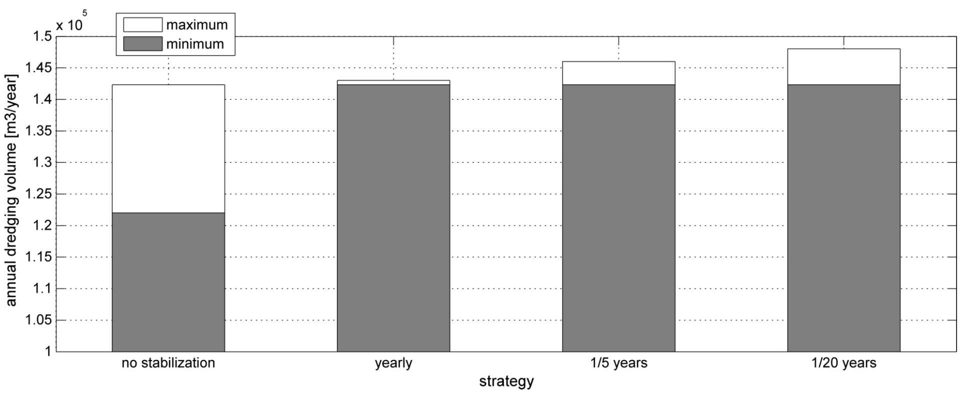


Figure I.6: The minimum and maximum mean annual dredging volume throughout the period 2018-2050 for different stabilization frequencies of 'nourishment 2010'.

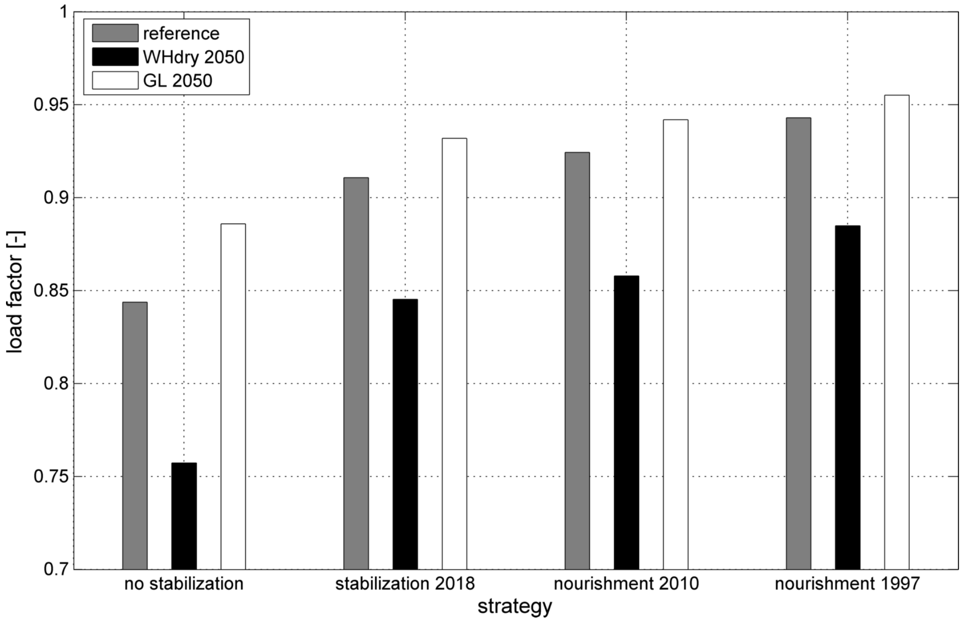


Figure I.7: Predicted mean load factor for different sediment management strategies in 2050. The range indicates the impact of the climate change scenarios.

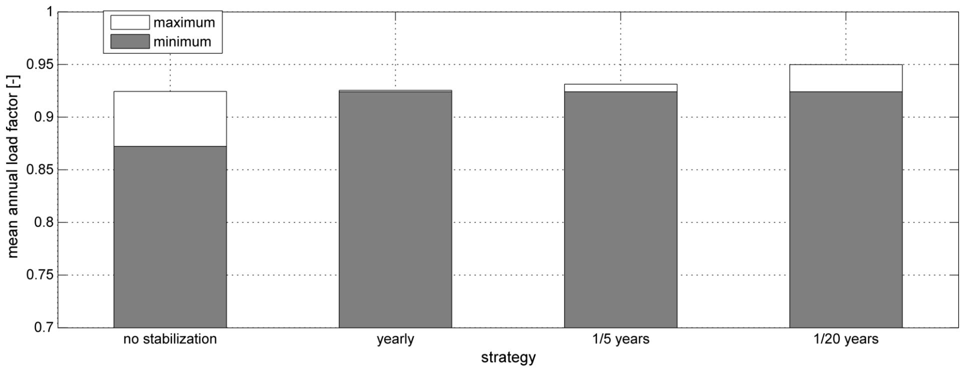
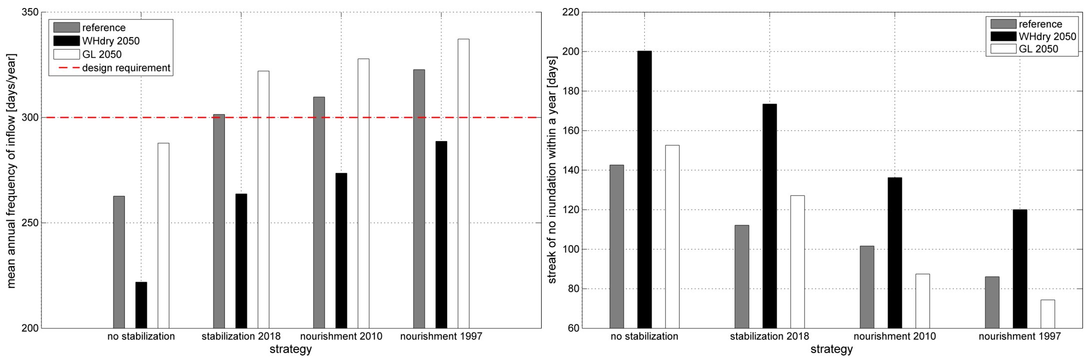


Figure I.8: The minimum and maximum mean annual load factor throughout the period 2018-2050 for different stabilization frequencies of 'nourishment 2010'.

I.2.3. Nature

This section will elaborate on the combined impact of a nourishment and climate change on the functional performance of the both the side channel Klompenwaard and the floodplains of the Millingerwaard. Figure I.9a illustrates that also with the strategy 'nourishment 1997', the design criteria will not be met when the climate will become dryer following scenario $W_{H,dry}$. However, with the current discharge statistics, the performance is improved by the nourishment. A similar analysis can be conducted for the Millingerwaard. It appears that the consecutive days of no-inundation will decrease, as the water level is elevated due to a nourishment. Also with a dry climate scenario, the days on no-inundation for the river with 'nourishment 1997' remains below 120 days, while without stabilization at all this value will increase to 172 days in 2050.



(a) Mean annual frequency of inflow of side channel of Klompenwaard.

(b) Mean consecutive days of no-inundation of Millingerwaard.

Figure I.9: Predicted performance of nature objects for different sediment management strategies in 2050. The range indicates the impact of the climate change scenarios.

J
Costs

J.1. Economic consequences reduced loading

J.1.1. Other methods

The method proposed by Jonkeren et al. (2007) and Flierman (2017) are just an estimation of the costs, while various other methods to quantify economic consequences of navigation exist as well. This section in the Appendix will elaborate on other examined methods to validate the outcomes of the economic consequences of reduced loading.

Taekema (2017) developed a method to assess the cost effectiveness of canalization of the Waal in order to anticipate on the impact of climate change. In his thesis he developed a model to predict the future extra shipping costs due to navigation restrictions, by means an effect model as presented in Figure J.1. The input is again generated by a climate, fleet and economic model. The effect model links the cost to a certain water depth, which is only accounts for extra costs per ton when the water depth drops below 2.80 m. However, in reality at 4 m, some larger vessels experience problems requiring reduced loading. The model has been applied with the simulated water levels and assumed transported load (128 million per year) of this thesis, as shown in Figure J.2. As can be seen, Taekema (2017) accounts for less costs at the moment, while a stronger increase is observed for the driest climate scenario. The fact the model Taekema (2017) predicts less costs in 2018, is dedicated to the the model assumptions that only extra costs will develop when the water depth is smaller than 2.8 m, while Jonkeren, O. (2009) expects extra costs already at water depth smaller than 4 m. «

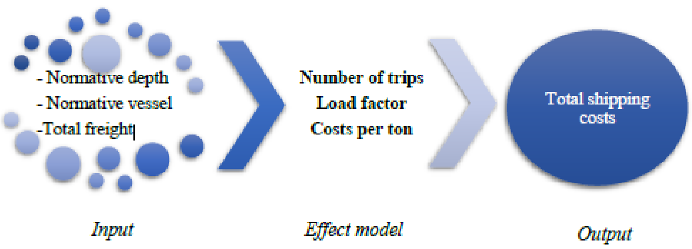


Figure J.1: The effect model reproduced from Taekema (2017).

As a rule of thumb it is assumed that 10 cm less draught in the Waal costs on average €15 000/day. This can be compared by using the Equation 4.17 and 4.19 to examine the wellfare loss of different water depths with method of Jonkeren et al. (2007). A initial price of €8.76/ton is assumed and a daily transported load of 350 000 t/day. Table J.1 shows the calculated daily costs of 10 cm less depth for different depths, showing that the order of €15 000 is realistic for an average depth of 3/4 m.

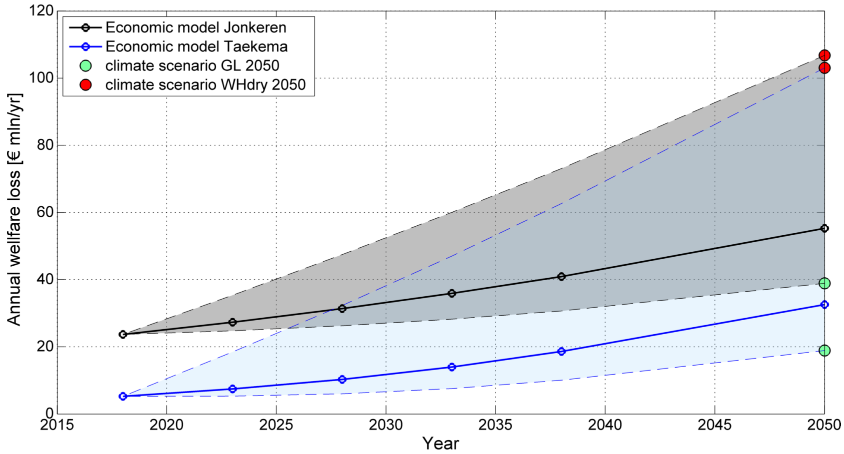


Figure J.2: The difference between the economic models of Taekema (2017) and Jonkeren, O. (2009) for the expected bed degradation and climate scenarios.

Water depth [m]	Costs [€/day]
2.5	46 000
3.0	21 000
3.5	9 000
4.0	2 300

Table J.1: Daily costs of a depth reduction of 10 cm.

J.1.2. Results

This section of the appendix elaborates on the results of the annual wellfare loss based on the approach proposed by Jonkeren et al. (2007) and Flierman (2017). Figure 5.11 shows the result of a future river system without interventions, while Figure 5.21 illustrates the impact of a nourishment with the current discharge statistics. However, as Figure 5.11 already illustrates climate change will affect the future annual wellfare loss due to water depth limitations. Table J.2 lists the annual wellfare loss in 2050 for different strategies and the climate scenarios $W_{H,dry}$ and G_L and the current (reference) discharge statistics. The results of this figure are not incorporated in the cost-benefit analysis. However, they show the increased costs due to a dry climate scenario, while this is mitigated (in a certain extend) by the nourishments.

Table J.3 illustrate the wellfare loss for different stabilization frequencies with the reference discharge statistics. The wellfare loss is fluctuating due to stabilization and the fact the bed degrades again. The range indicates the minimum and maximum value.

Strategy	Annual wellfare loss reference [mln €/year]	Annual wellfare loss $W_{H,dry}$ [mln €/year]	Annual wellfare loss G_L [mln €/year]
No stabilization	55.2	106.7	38.9
Stabilization 2018	23.7	56.3	18.1
'Nourishment 2010'	18.4	44.9	11.1
'Nourishment 1997'	11.8	30.7	7.1

Table J.2: Annual wellfare loss in 2050 for different sediment management strategies. Also different climate scenarios are assessed.

Stabilization frequency	Range of wellfare loss [mln €/year]
Yearly	18.3
1/5 years	15.7 - 18.3
1/20 years	9.5 - 18.3

Table J.3: Annual wellfare loss for different stabilization frequencies. Due to the fluctuating character the range is indicated.

J.2. Dredging costs

Table J.4 shows the effect of different climate scenarios on the maintenance dredging costs. As is observed the dredging costs are increased considerably due to the driest climate scenario (lowered ALW).

Furthermore, this section elaborates on the dredging costs of different stabilization frequencies. Table J.5 shows the dredging costs for different stabilization frequencies. As the strategies allow degradation of the river bed in a certain degree, the dredging volumes are decreased when the river bed degrades. Therefore, Table J.5 shows the range the annual dredging costs vary.

Strategy	Dredging costs - ref [mln €/year]	Dredging costs - $W_{H,dry}$ [mln €/year]	Dredging costs - G_L [mln €/yea
No stabilization	0.37	1.18	0.20
Stabilization 2018	0.47	1.52	0.23
'Nourishment 2010'	0.49	1.56	0.24
'Nourishment 1997'	0.51	1.67	0.25

Table J.4: Annual maintenance dredging costs in 2050 due to climate change for different strategies

Stabilization frequency	Range of dredging costs [mln €/year]
Yearly	0.49
1/5 years	0.49 - 0.50
1/20 years	0.49 - 0.50

Table J.5: Annual maintenance dredging costs for different stabilization frequencies.

J.3. Financial analysis of a future river without stabilization

As Section 5.2.1 elaborates on the separate costs related to maintenance dredging and wellfare loss (reduced loading), this section will show the combined effect representing a net market value of a future river system without stabilization. Table J.6 presents the net market value for the period 2018-2050 of various scenarios including the costs and benefits related to wellfare losses due to restrictions in navigation depth and maintenance dredging. The net market value is the value compared to the situation in 2018. This reveals the net market value range of future river system without interventions between -1126 and +50 million euros.

J.4. Nourishment costs

The total costs of the nourishment depends on the nourishment size and the price per cubic meter. As Section 4.5.5 explains, the price per cubic meter nourishment will decrease, when the total nourishment volume is increasing due to the scale effect. This section in the Appendix will elaborate on the required volumes for the strategies.

A volume is required for both the initial nourishment and the for stabilization. Firstly, will be elaborated on the initial nourishment. The volume within the river section is rather straightforward

$$V = L_{section} * B_n * h_n \tag{J.1}$$

in which $L_{section}$ is the length of the considered river section (25 km), B_n the averaged normal width (256 m) and h_n the height of the nourishment, which depends on the nourishment strategy. However, it is expected that the nourishment is extended till Tiel (rkm 915) requiring another 24 km of nourishing. As a the nourishment has to gradually connect with the river bed, it is assumed that before Tiel the nourishment gradually

Scenarios	Efficiency navigation [mln €]	Dredging [mln €]	Net market value [mln €]
Base case	-248.9	1.3	-248
No stabilization	0	0	0
Doubled erosion	-722.2	1.5	-721
$W_{H,dry}$ and base case erosion trend	- 741.6	-6.3	-748
G_L and base case erosion trend	- 135.8	2.4	-133
$W_{H,dry}$ and no erosion	-291	-9	301
G_L and no erosion	+ 50	2.4	52
G_L and doubled erosion trend	-447	2.6	446
$W_{H,dry}$ and doubled erosion trend bed	- 1126.2	-4.8	1126

Table J.6: Net market value for all scenarios of a future river system without stabilization interventions. All costs and benefits in the period 2018-2050 are considered compared to the situation of 2018.

decrease. To account for this reduction in nourishment volume a 610 000 m^3 is assumed instead of 740 000 m^3 downstream of the river section.

The determination of the required volume for stabilization is more complex. It is expected that the erosion rate will continue at the same rate as before the nourishment. Table D.1 present the volume trends within the considered river section, while also downstream of the river section erosion will take place. As Tiel is considered as the hinge point, the erosion rate slowly decreases to zero at Tiel, which results in an average erosion rate of 0.5 cm/year of rkm 901 to 915. Furthermore, it is known that bed disturbances travel downstream. It is assumed that the nourishment will travel downstream at rate of 1 km/year. Table J.8 shows all elements of the required annual stabilization volume.

Section	Volume trend [m^3 / year]
Study area	110 486
Dodewaard - Tiel	30 600
Top of the nourishment	28 160
Total	169 246

Table J.7: Annual volume trend for stabilization.

Stabilization frequencies

Different stabilization frequencies are analysed that aim to guarantee (not allow exceedance of) 'nourishment 2010' river bed. Table J.8 shows the initial volume required for different stabilization frequencies. Based on the assumptions of Section 4.5.5 the costs for these volume are estimated. The volumes are the sum of the volume required for 'Nourishment 2010' (1.4 million m^3) and the period of degradation times the volume trends of Table J.8. The volumes after the initial nourishment are already considered in Table 4.7.

Stabilization frequency	Volume [million m^3]	Nourishment costs [€/ m^3]
Yearly	1.5	17.6
1/5 years	2.1	17.6
1/20	4.1	16

Table J.8: The initial volumes required for different stabilization frequencies of 'nourishment 2010' and the corresponding nourishment costs per cubic meter.

J.5. Sensitivity analysis cost-benefit analysis

Within this section various elements of the cost-benefit analysis have been adjusted to analyse the sensitivity of those parameters.

J.5.1. Climate change

Table 5.6 presents the elements of the net market value without incorporating climate change, while this section will analyse the cost effectiveness of a nourishment incorporating the climate scenarios $W_{H,dry}$ and G_L. The costs related to an extra increase of the design water levels are neglected, as this is part of the HWBP. The nourishment costs are also assumed to be equal, as the bed degradation rates remain unchanged. However, wellfare loss and dredging costs are expected to be affected by climate change. Chapter 5 elaborates on the future impact of climate change without stabilization intervention, revealing $W_{H,dry}$ will increase the costs related to navigational efficiency and dredging, while G_L counteracts the impact of bed degradation. A similar analysis has been conducted with the various sediment management strategies, which is presented in Table J.9 and J.10 for respectively scenario $W_{H,dry}$ and G_L. It appears that for both climate scenarios a nourishment is cost effective. However, for a dryer climate scenario the net discounted costs of no stabilization increase in such an extend that a nourishment becomes more attractive. Considering a wetter climate scenario, the net market value considering no stabilization is relatively low.

Strategy	**Flood protection**	**Nourishment**	**Dredging**	**Navigation**	**Net market value**
No stabilization	0	0	- 6.3	- 742	- 748
Stabilization 2018	0	- 77	- 9.4	- 291	- 378
'Nourishment 2010'	- 5	- 102	- 9.8	- 135	- 251
'Nourishment 1997'	- 12	- 138	- 10.9	69	- 91

Table J.9: The net market value in the period of 2018-2050 in million euros for different strategies for the expected bed degradation and the climate scenario $W_{H,dry}$. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

Strategy	**Flood protection**	**Nourishment**	**Dredging**	**Navigation**	**Net market value**
No stabilization	0	0	2.4	- 136	- 133
Stabilization 2018	0	- 77	2.4	50	- 25
'Nourishment 2010'	- 5	- 102	2.1	167	62
'Nourishment 1997'	- 12	- 138	2.0	281	134

Table J.10: The net market value in the period of 2018-2050 in million euros for different strategies for the expected bed degradation and climate scenario G_L. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

J.5.2. Different bed degradation scenarios

In a similar way as the previous section, the cost-benefit analysis for a reduced or increased bed degradation compared to the observed trends, can be assessed. As all the cost-benefit analysis are conducted based on the base case erosion trend, this appendix section will apply a sensitively analysis on the impact of an increased trend (doubled) and a case with no erosion. Table J.11 and Table J.12 show the bandwidth due to bed degradation of the net market value with respectively a zero erosion trend and doubled erosion trend.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No stabilization	0	0	0	0	0
Stabilization 2018	0	0	0	0	0
'Nourishment 2010'	- 5	- 25	- 0.3	108	78
'Nourishment 1997'	- 12	- 61	- 0.7	242	168

Table J.11: The net market value in million euros for a zero erosion trend without incorporating climate change. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements in the period 2018-2050.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No stabilization	0	0	1.5	-722	-721
Stabilization 2018	0	-154	0	0	-154
'Nourishment 2010'	- 5	-179	- 0.3	108	-76
'Nourishment 1997'	- 12	-214	- 0.7	242	14.5

Table J.12: The net market value in million euros for a doubled erosion trend without incorporating climate change. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements in the period 2018-2050.

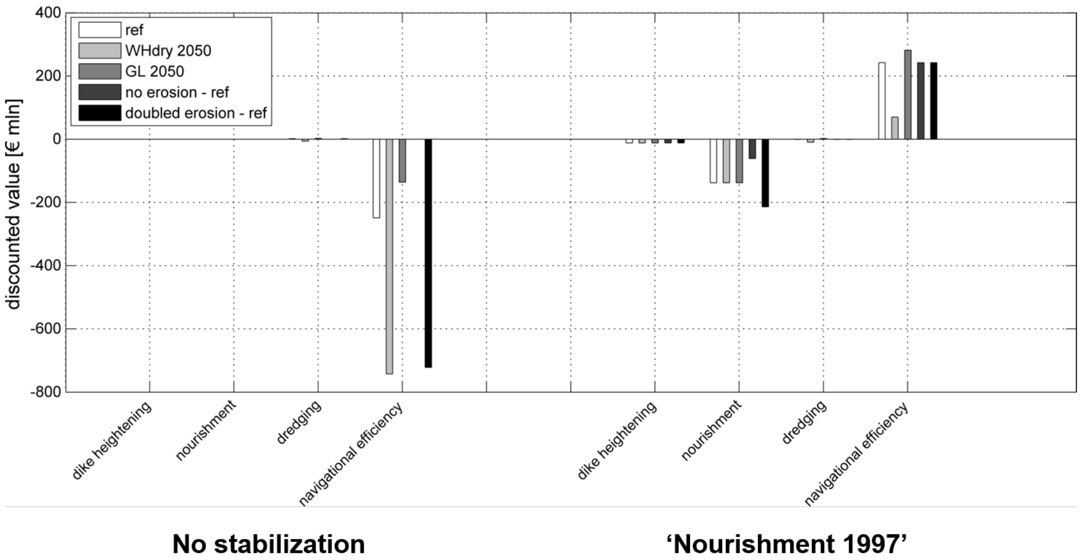


Figure J.3: All cost and benefit elements between 2018-2050 the cost-benefit analysis of the strategies 'no stabilization' and 'nourishment 1997'. The costs of different erosion and climate scenarios are plotted separately. The climate scenario scenario are evaluated with the base case erosion rates.

J.5.3. Cost-benefit analysis stabilization frequencies

In Figure 5.27 already shows the net market value of different stabilization frequencies. The numbers used in the cost-benefit analysis are listed in Table J.13. Less frequent stabilization requires a larger nourishment volume. This subsequently results in more costs for dike heightening, lower nourishment costs due to lower price per cubic meter (economy of scales) and improved navigation. In the end, less frequent stabilization is predicted to be more cost effective.

Stabilization frequency	Flood protection	Nourishment	Dredging	Navigation	Net market value
Yearly	-5	- 104	- 0.3	110	1
1/5 years	- 7	- 96	- 0.4	137	34
1/20 years	- 13	- 92	- 0.6	220	114

Table J.13: The net market value in the period of 2018-2050 in million euros for different stabilization frequencies of the 'nourishment 2010'. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

J.5.4. Source sediment material

Table J.14 shows the net market value of project duration till 2050 for different ways to acquire sediment for the nourishments. With external strategies are considered as sediment from outside study area, floodplain is required sediment from floodplain for nourishment and extra from floodplain is strategy where all sufficient sediment is extracted from the floodplain to compensate for flood protection.

Table J.15 shows the net market value (in 2050) with lower excavation costs (€20/m^3) and higher costs for dike heightening (not connected with HWBP), as for this case sediment extraction from the floodplain is most attractive. As expected, it appears that in this case the cost effectiveness of sediment from the floodplain

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
2010 - external	- 5	- 102	- 0.3	108	1
2010 - floodplain	- 3	- 150	- 0.3	108	- 45
2010 - extra from floodplain	0	- 217	- 0.3	108	- 110
1997 - external	- 12	- 138	- 0.8	242	91
1997 - floodplain	- 7.5	- 269	- 0.8	242	- 36
1997 - extra from floodplain	0	- 446	- 0.8	252	- 204

Table J.14: Net market value in the period of 2018-2050 in million euros for different sediment material.

improves, as it becomes close to the cost effectiveness of external sediment. Furthermore, it appears that when dike heightening becomes more expensive, a smaller nourishment is more cost effective.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
2010 - external	- 63	- 102	- 0.3	108	- 58
2010 - floodplain	- 40	- 138	- 0.3	108	- 70
2010 - extra from floodplain	0	- 179	- 0.3	108	- 71
1997 - external	- 164	- 138	- 0.8	242	- 61
1997 - floodplain	- 103	- 237	- 0.8	242	- 98
1997 - extra from floodplain	0	- 345	- 0.8	252	- 104

Table J.15: Net market value in the period of 2018-2050 in million euros for different sediment material with higher dike heightening costs and cheaper floodplain sediment (€20/m^3).

J.5.5. Different project durations and discount rates

This subsection will elaborate on the effect of different project duration and discount rates. The base case is considered to have a project duration of 32 year (2018 till 2050) with a discount rate of 3 %. Table J.16 shows a project duration of 20 years with the base case discount rate, while Table J.17 shows a project duration of 50 years with the base case discount rate. It appears that a sediment nourishment becomes less attractive when the project duration is shorter, as high investment costs have to be done due to the dike heightening and the initial nourishment. In Table J.19 and Table J.18 show the effect of a changing discount rate with a project duration of 32 years of respectively a discount rate of 1 % and 5 %. A higher discount rate means the benefits of an improved navigability become less worth, as the value of money will decrease in the future. However, also with a discount rate of 5 % a nourishment is considered to be cost effective.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No-stabilization ref	0	0	0.8	- 113.4	- 113
Stabilization ref	0	- 56	0	0	- 56
'Nourishment 2010'	- 5	- 82	- 0.2	79	- 8
'Nourishment 1997'	- 12	- 117	- 0.5	177	47

Table J.16: The net market value in the period of 2018-2038 in million euros for different strategies. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

J.5.6. Different sediment material

Table J.20 shows the net market value of different strategies, when Tout-Venant is used as sediment material, which is more expensive than sand. Also with more expensive nourishment material, such as Tout-Venant, a nourishment appears to be cost effective.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No stabilization	0	0	2	- 472	- 470
Stabilization 2018	0	- 97	0	0	- 97
'Nourishment 2010'	- 5	- 123	- 0.4	136	9
'Nourishment 1997'	- 12	- 158	- 0.9	305	134

Table J.17: The net market value in the period of 2018-2068 in million euros for different strategies. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No stabilization	0	0	1	- 170	- 169
Stabilization 2018	0	- 60	0	0	- 60
'Nourishment 2010'	- 5	- 85	- 0.2	84	- 6
'Nourishment 1997'	- 12	- 85	- 0.6	188	54

Table J.18: The net market value in the period of 2018-2050 in million euros for different strategies with a discount rate of 5 %. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No stabilization	0	0	2	- 377	- 375
Stabilization 2018	0	- 103	0	0	- 103
'Nourishment 2010'	- 5	- 128	- 0.4	114.4	11.1
'Nourishment 1997'	- 12	- 164	- 1	324	147

Table J.19: The net market value in the period of 2018-2050 in million euros for different strategies with a discount rate of 1 %. All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

Strategy	Flood protection	Nourishment	Dredging	Navigation	Net market value
No stabilization	0	0	1.3	- 248.9	- 247.5
Stabilization 2018	0	- 113	0	0	- 113
'Nourishment 2010'	- 5	- 150.3	- 0.3	108	- 47.3
'Nourishment 1997'	- 12	- 203	- 0.7	242	27

Table J.20: The net market value in the period of 2018-2050 in million euros for different strategies with a nourishment costs of Tout-Venant (€47/m^3). All costs are relative to the year 2018. The net market value is the sum of the costs and benefits elements.

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List of Abbreviations

Acronyms

ALD Agreed Low Discharge.

ALW Agreed Low Waterlevel.

CCR Central Commission of navigation on the Rhine.

CEMT Conférence européenne des Ministres des Transports.

DCM Divided Channel Method.

DWL Design Water Level (Dutch: Maatgevend Hoogwater).

GRADE Generator of Rainfall and Discharges Extremes.

GWL Ground Water Level.

HBV hydrological rainfall and run-off model.

HWBP Hoogwaterbeschermingsprogramma.

IPCC Intergovernmental Panel on Climate Change.

IRM Programme Integrated River Management.

IWT Inland Water Transport.

KNMI Koninklijk Nederlands Meteorologisch Instituut.

LAD Least Available Draught.

NAP Normaal Amsterdams Peil.

OI Ontwerpinstrumentarium.

RfR Room for the River.

rkm River kilometer (Dutch: kilometerraai).

WBI Wettelijk Beoordelings Instrumentarium.

WFD EU Water Framework Directive.

WWF World Wide Fund for nature.