Delft University of Technology

MSc. Thesis

The inclusion of model uncertainty

Preliminary examination on how model uncertainties affect frequency lines for water levels

Supervisors:
Dr. Chris Geerse
Ir. Robert Slomp

Author:
Bart Strijker

Supervising professor:
Prof. dr. ir. Matthijs Kok

Other Committee member(s):
Ir. Richard Jorissen
Prof. dr. ir. Hubert Savenije

Department of Hydraulic Engineering - Flood Risk
Delft University of Technology

Tuesday 10th April, 2018

Rijkswaterstaat
Ministerie van Infrastructuur en Milieu

The inclusion of model uncertainty

Preliminary examination on how model uncertainties affect frequency lines for water levels

by

B. Strijker

to obtain the degree of

Master of Science
in Civil Engineering

at the Delft University of Technology,
to be defended publicly on Tuesday April 17, 2018 at 15:30.

Project duration:	August 2017 – April 2018	
Thesis committee:	Prof. dr. M. Kok,	Delft University of Technology
	Prof. dr. ir. H. H. G. Savenije	Delft University of Technology
	Dr. C. P. M. Geerse,	HKV Consultants
	Ir. R. M. Slomp,	Rijkswaterstaat WVL
	Ir. R. Jorissen,	Delft University of Technology / Rijkswaterstaat HWBP

TU Delft

“ *Remember that all models are wrong;*
the practical question is how wrong do they have to be to not be useful ”

George E. P. Box, *Empirical Model-Building and Response Surfaces*, 1987

Abstract

The transition towards a new risk approach for the Dutch national safety assessment of primary flood defences has been taken as an opportunity to improve the dealings with uncertainties. The probabilistic models for the new safety assessment (WBI2017) not only deal with the natural variability, but also with so-called epistemological uncertainties. One class of epistemological uncertainties is the model uncertainty in the hydrodynamic models used. According to WBI2017 the model uncertainty related to water levels is included by considering the water level as an additional stochastic variable. The quantification of the water level uncertainty depends on the dominant hydraulic processes in each water system and is chosen to be independent of the return period. However, water level frequency lines derived including model uncertainty sometimes conflict with the physics. The main objective of this thesis is to analyse how model uncertainties affect the water level frequency lines in different fresh water systems in the Netherlands and to provide insights into different methods to deal with model uncertainties for the Dutch national safety assessments.

A comparative analysis is carried out to assess the performance of Hydra-NL w.r.t. observations and to analyse if physical processes that play an important role in each fresh water system are represented correctly after the inclusion of model uncertainty with the WBI2017 method. This study shows that the estimated exceedance probability of actual water levels in the tidal river area and lake area are often overestimated by the Hydra-model even without model uncertainty. When model uncertainties are included the overestimations become even larger. Furthermore, the inclusion of model uncertainty sometimes gives an incorrect representation of the underlying physical processes. The effect of the model uncertainty gets larger when the water level frequency line becomes flatter. This is e.g. the case at locations where the water levels are influenced by the closure of the Europoortkering and upstream of the flood channel near Veessen-Wapenveld. It can be argued that water level uncertainties are likely to decrease in these situations, because a reservoir is more predictable than a flowing river and the operation of the flood channel results in an enlarged conveyance that is less sensitive than an average river profile. The hypothesis that the inclusion of model uncertainty according to WBI2017 results in an incorrect representation of the underlying physical processes is further analysed for several case studies in the upper river area and the lake area.

According to WBI2017 the peak discharge at Lobith is the only stochastic variable in the upper river area apart from the model uncertainty. The most important sources of model uncertainty in the hydrodynamic model used are the calibration approach, hydraulic roughness and physical processes or morphological changes under extreme conditions. In this study a simplified physical model is set up based on normal flow equations and backwater equations. This model is able to transpose discharge to corresponding water levels for different hydrodynamic systems. Varying river schematisations and two river interventions (flood channel and retention area) are modelled to compare the WBI2017 method with a more physics-based approach. In this physics-based approach the hydraulic roughness of the main channel and/or floodplains is incorporated as additional stochastic variable for the derivation of water level frequency lines. Rivers have a dynamic behaviour where the geometry of the river varies for every branch and even within the branch. It is shown that the water level uncertainty becomes smaller for wide rivers where the water level frequency line becomes flatter. The cases where river interventions are present the water level uncertainty is bounded, because of the environment/geometry that influences the water levels. According to the physics-based method the water level uncertainties are location and discharge dependent, which are both not integrated in the WBI2017 method. The physics-based approach shows potential possibilities to include model uncertainty that represents the physical processes in a better manner.

For the lake area the focus is on locations where high water levels are predominantly determined by the wind that is causing a set-up on the lake. Wind transformation, modelling of the drag coefficient and schematisation of the wind fields are for wind-dominated locations the most important model uncertainties in the hydrodynamic model used in the lake area. In this study, a simplified physical model is used

that simulates an one-dimensional closed basin in which the wind causes shear stresses at the water surface that result in a water level gradient. By considering an empirical parameter of the "capped Wu" formula as additional stochastic variable the uncertainty of the drag coefficient is modelled for the physics-based method. The quantification is performed pragmatically by matching the 95 % confidence bounds of the water level for the physics-based method and the WBI2017 method. The decimate heights along a lake gets larger for locations that are further away from the center of gravity of the water surface, where the wind set-up is the largest. The physics-based method demonstrates that the water level uncertainty is larger for higher decimate heights and vice versa. It can be concluded that it is not valid to choose one uniform standard deviation of the water level for all wind-dominated locations, as it is done for WBI2017. Including the model uncertainty according to the physics-based method shows potential possibilities to address this problem.

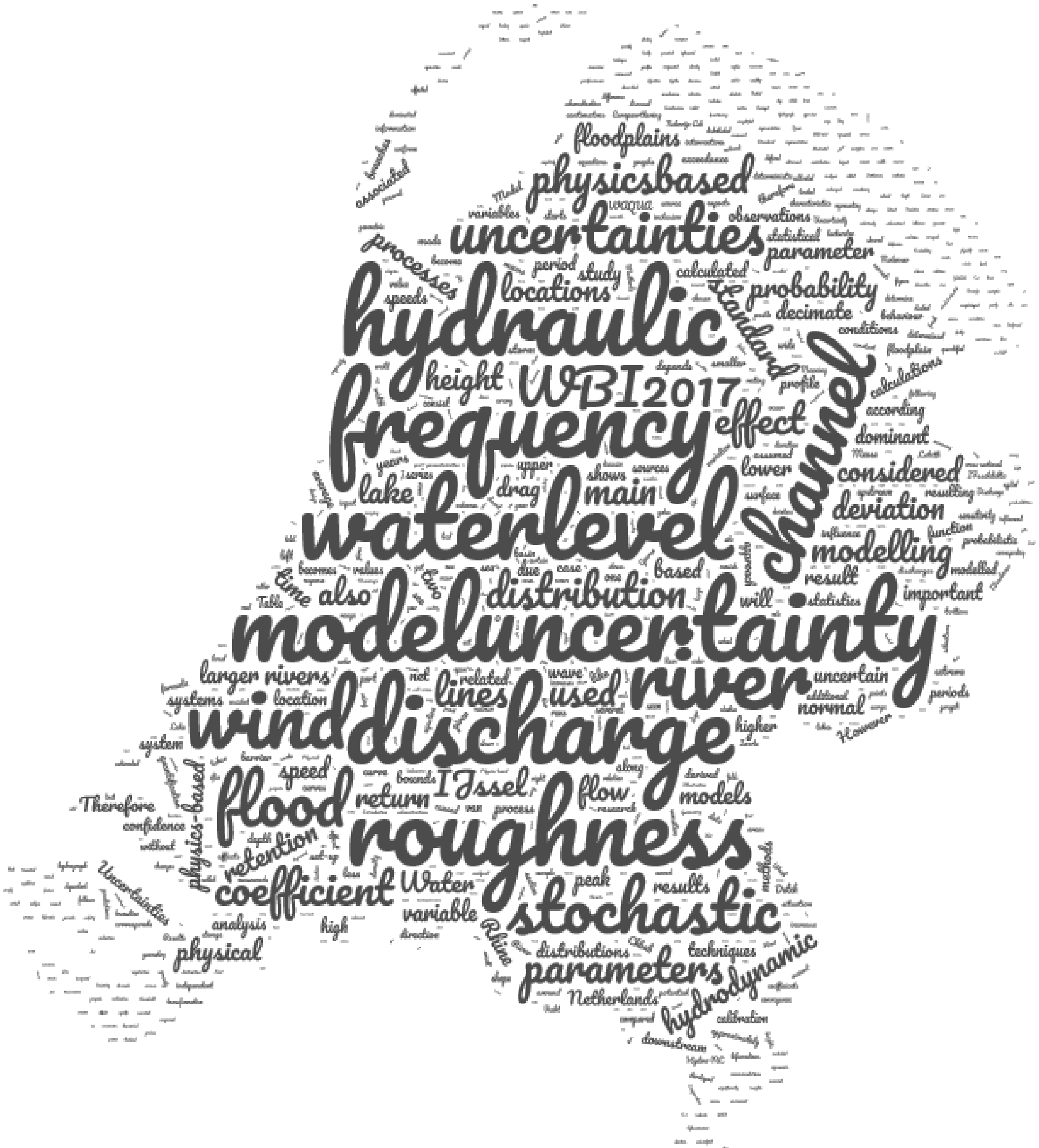
The physical models used provided rapid insights into system behaviour, but the findings should be verified by using more sophisticated hydrodynamic models in which the water dynamics are represented in more detail. It is recommended to include the model uncertainty by considering a model parameter in the hydrodynamic model for the upper river area and lake area as additional stochastic variable in the Hydra-models, to model uncertainty in the water level. For the upper river area the hydraulic roughness seems the best candidate and because of computational restrictions the main channel and floodplain roughness could be considered fully dependent. For wind-dominated locations in the lake area it is recommended to use an empirical parameter of the "capped Wu" formula as additional stochastic variable. In other water systems it becomes more complex, because several equally important sources of uncertainty are present. Still, the most important model uncertainty sources could considered stochastically, but computational effort would become the limiting factor.

Acknowledgements

This thesis examines the interface between physics and statistics, where shortcomings of physical models are covered by statistical techniques. Knowledge of both fields is necessary to assess related problems properly. Understanding the assumptions and limitations of used physical models and the comprehension of the hydraulic processes that are dominant in different physical environments regarding the outcomes were necessary to obtain meaningful conclusions. A large part of this necessary information is obtained during the thesis by reading and meetings with professionals who gave useful advice. I am very grateful to all those people who helped me during this study by making time for meetings or providing me relevant reports. I also would like to thank all the people at HKV for the pleasant work environment and inspiring discussions.

I would explicitly like to thank several persons whom are closely involved during this thesis. First of all I would like to express my sincere gratitude to my daily supervisors - Chris and Robert - for the time they invested in me, their help to discover which direction of this research is relevant and their constructive feedback. Chris has helped me a lot with the understanding of probabilistic models and the interpretation of the results. Not only your expertise and extensive knowledge about statistics, also your reviews of my report has improved my thesis a lot. Robert helped me a lot by placing this study in the right context. You have introduced me to several people and provided plenty of reports to improve this study. Furthermore I want to thank Richard for your adequate comments and helping me to get my conclusions sharper and Huub for your valuable input and fresh perspective. At last I thank professor Matthijs who put me in touch with Chris and Robert to establish this study and your scientific criticism enhanced the progress meetings and made me think twice about assumptions and/or limitations of this study.

Bart Strijker
Rotterdam, April 2018



modeluncertainty
frequency
hydraulic
waterlevel
roughness
discharge
wind
river
channel
stochastic
flood
uncertainties
WBI2017
standard
physicsbased
parameters
coefficient
distribution
modelling
floodplains
probability
locations
processes
height
effect
considered
deviation
main
lake
also
time
lines
used
larger rivers
IJssel
return
retention
physics-based
hydrodynamic
Water
Rhine
physical
speed
flow
models
results
peak
variable
distributions
techniques
Netherlands
calibration
downstream
important
statistics
result
based
will
normal
higher
uncertain
However
decimate
calculated
statistical
parameter
study
period
observations
speeds
values
conditions
calculations
according
dominant
lower
upper
shows
drag
due
case
two
related
wave
location
systems
system
Therefore
confidence
analysis
high
Uncertainties
bounds
process
research
along
profile
sources
function
modelled
periods
associated
waqua
variables

Contents

List of Figures

List of Tables

1. Introduction

1.1 Background

The role of probabilistic models in determining hydraulic loads
In the context of the Wettelijk Beoordelingsinstrumentarium 2017 (WBI2017), a set of tools and guidelines has been developed for the safety assessment of primary flood defences along the major water systems in The Netherlands. Up until 2016 the flood defences were required to withstand a hydraulic load (water levels and wave action) with a certain probability of exceedance. This approach strongly focussed on the failure mechanism overflow and overtopping of the flood defences, although the requirements for geotechnical failure mechanisms are derived among others from the hydraulic loads. The new standard (since 2017) is based on the probability of flooding of an area which is translated to the failure probability of a dike segment. This new standard does not only focus on the hydraulic load, but also on the strength of the flood defences. Therefore, the new standard expresses the probability of flooding more completely.

The hydraulic loads are caused by threats that occur in an area, such as wind, tides, sea level and river discharge. The combinations of these threats determine the hydraulic load levels and differs for every water system. The hydraulic load level, in m+NAP, equals the sum of the local water level and the wave overtopping height, which are also known as the hydraulic load parameters. These parameters operate as starting point in the process of establishing the dimensions of flood defences for the failure mechanisms overflow and overtopping and are also used to analyse additional failure mechanisms.

A probabilistic Hydra model (e.g. Hydra-NL and Hydra-Ring) is one of the tools from WBI2017 and is used to determine the hydraulic load levels for representative locations in the Netherlands. A Hydra model uses statistical properties of the governing variables (discharge, sea level, wind speeds etc.) and a large number of calculated water levels and wave loads (calculated with physical models like WAQUA and SWAN, resp. a hydrodynamic and wave model) in order to determine the probabilities of occurrence of hydraulic load levels in an area. These models are able to determine the magnitude and the likelihood of water levels at a certain location, yielding so-called frequency lines for water levels. Figure 1.1 illustrates a water level frequency line, where the probability that a water level will occur is expressed as a return period. Return periods refer to the average time between years in which a certain water level is exceeded.

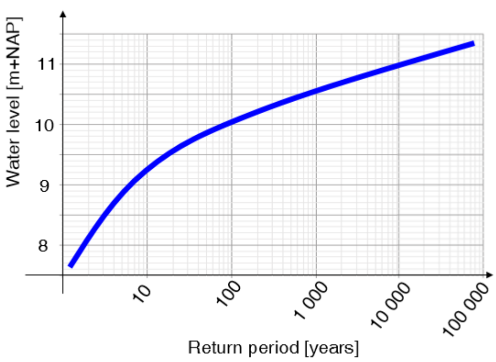


Figure 1.1: Illustration of a frequency line for water levels. The spacing of the scale on the x-axis is proportional to the logarithmic of the return period

Not only extreme water levels play a role in flood defence assessments and design, also lower water levels with larger probabilities of occurrence are important for geotechnical failure mechanisms. Next to flood defence assessments and designs, these water level frequency lines are also used for management and maintenance of rivers and hydraulic structures and design of bridges (vertical bridge clearances). So, the whole range of hydraulic loads are of interest for different purposes. This illustrates the importance of a thorough understanding and accurate derivation of water level frequency lines.

The aforementioned transition from old to new standards has been taken as an opportunity to improve the issue of dealing with uncertainties. The probabilistic models for the new safety assessments (WBI2017) do not only deal with the natural variability, but also with the epistemological uncertainties that consist of model and statistical uncertainty (further explained in Section 1.1). The incorporation of epistemological uncertainties should result in 1) a more realistic estimation of the exceedance probability of hydraulic loads and should result in 2) an improved possibility or incentive to invest more efficiently in further research or in structural solutions for water safety. Therefore, this study focusses on the inclusion of model uncertainties in the process of deriving water level frequency lines.

The science of uncertainty and the art of probability
Models consist of equations that are based on experimentation, observations and intuition. A model is always a simplified description of physical reality expressed in mathematical terms and will therefore always produce incorrect outcomes. Full verification and validation of models of natural systems is impossible, because natural systems are never closed and model results are always non-unique (Oreskes et al., 1994). The confirmation of model results by demonstration of agreement between observations and prediction will always by inherently partial for a system in which mass or energy can be lost to or gained from the environment. The degree of inaccuracy can be reduced by means of calibration processes, where model parameters are tuned such that there is a match (sufficient agreement) between observed and simulated distributions. Complete confirmation is precluded by the fallacy of affirming the consequent and by incomplete access to natural phenomena.

Model calibration in hydrodynamic models in river areas is often done by the adjustment of only one parameter in the model: the roughness coefficient. In doing so, this parameter can be considered as a "dust-bin" that also consists of physical processes that are not captured by the model, due to errors or simplifications. Furthermore the calibration process is done by using a limited range of observations. Both aspects contribute to uncertainties in predicted water levels in physical models and subsequently in water level frequency lines calculated by Hydra-NL. These uncertainties should be taken into account before a hydraulic load is adopted in flood risk analyses to prevent underestimates. Legitimately, the relative performance of a physical model with respect to observational data is the most feasible that can be said about the performance of a model. Equifinality of the results and the lack of observational data for all possible outcomes lead to uncertainties in the performance of the model. In scientific literature many classifications of uncertainties and many methods for factoring into calculations exist. In hydraulic engineering, a distinction is generally drawn between three different classes:

- **Inherent uncertainty** (natural variability): Uncertainty that arises from pure randomness, which cannot be reduced by further analysis and lead to fluctuations in time and/or space of a physical process e.g. discharge and wind.
- **Statistical uncertainty** (epistemological uncertainty): Uncertainty that is related to estimations/determination of probability distributions and the associated distribution parameters of a stochastic variable based on measurements with a limited duration range.
- **Model uncertainty** (epistemological uncertainty): Uncertainty that is associated to models and occurs due to inaccurate parameterised models, model simplifications and wrong mathematical equations.

The inherent uncertainty is the reason why flood risk analysis are carried out, because floods are natural occurring processes and it is unknown when and where they will occur. This class of uncertainty was the first one that flood assessments took into account in the Netherlands, where uncertainties like sea level, discharge, wind and lake level have a presence. Inherent uncertainties in other natural processes, statistical information or physical models were characterised by representative values. Only limited natural invariabilities were considered in previous probabilistic assessments (before WBI2017) to determine the probability of occurrences of hydraulic load levels for the assessment of flood defences. Already in 1939, statistical techniques were used to deal with the frequency of occurrences of water levels and therefore inherent uncertainties (Wemelsfelder, 1939).

The statistical distributions that are estimated for inherent uncertainties are usually based on limited set of observations, which may induce uncertainty in the parameters of these distributions and also uncertainty of the type of distribution should be used. Currently, part of this source of uncertainty is explicitly taken into account and is incorporated into the stochastic variables that are modelled (Chbab, 2017; Geerse, 2016).

The last class of uncertainty is the model uncertainty. Model uncertainty is the result of a simplified representation of reality and uncertainty in model parameters in the models that are used to simulate physical phenomena such as movements of water and wave development. Several input parameters of these models are considered as deterministic values, while in reality these parameters could experience natural variability. An example is the geometric profile and roughness coefficients in these models. Computational power and storage are limiting factors and only the most dominant uncertain parameters that affect the hydraulic loads can be considered as a statistical distribution. Model uncertainties in water levels and wave conditions are considered in computations in WBI-2017. These uncertainties have been quantified separately for each region in the Netherlands, because of different dominant physical processes with different model uncertainties at every location.

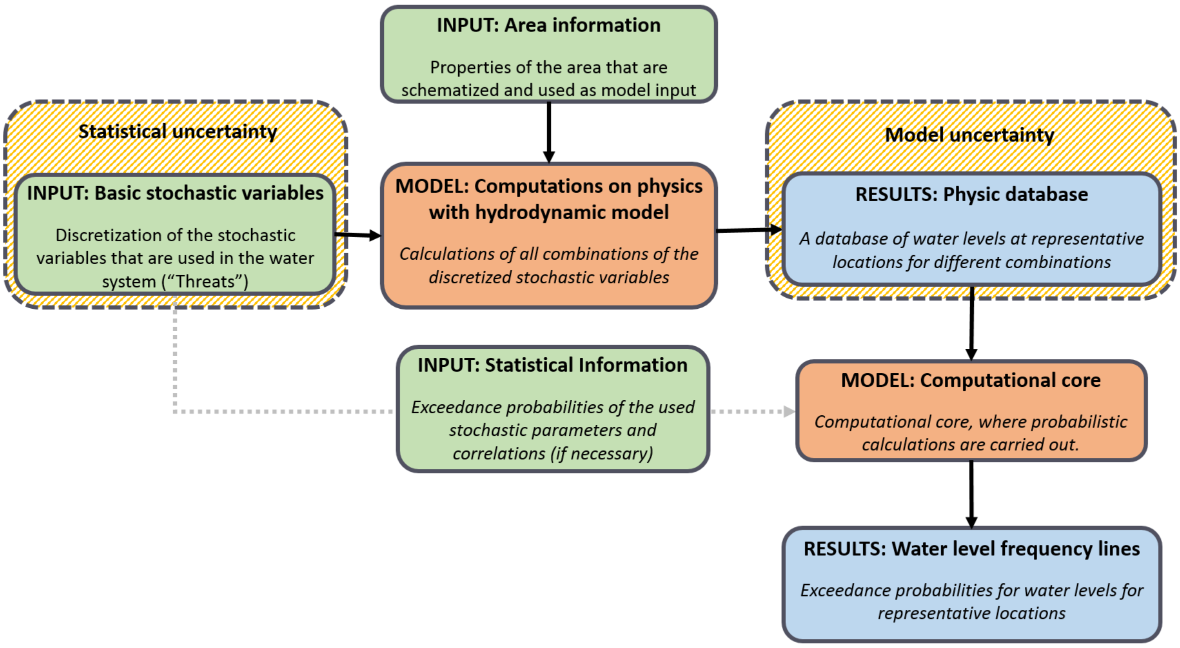


Figure 1.2: Schematic overview showing required information and the relations between different parts of the process for determining water level frequency lines

Figure 1.2 shows schematically the process for deriving water level frequency lines. The position of the epistemological uncertainties are illustrated by the yellow boxes and the inherent uncertainties are called the "basic stochastic variables" (in Dutch "Basisstochasten"). The copper coloured boxes indicate the models that are used. These models make physical and probabilistic calculations for which input data is required that are indicated by the green boxes. The model uncertainty is added to the outcomes of physical calculations after which the computational core translates the physic database (including model uncertainty) in combination with the statistical information of the stochastic variables to water level frequency lines.

1.2 Problem description

Parameters in models can be considered as a stochastic variable or a deterministic variable and the amount of stochastic variables are limited due to computational restrictions. The trade off depends on the influence of the uncertain parameter on the results. Sensitivity analyses can be used to determine which parameters will be incorporated as stochastic variables and which are considered as deterministic values in probabilistic calculations (Geerse, 2013; Nicolai et al., 2011). There is no standard procedure for the establishment of the deterministic values, which results in problems in the characterisation of these parameters. Different parameter characterisation can affect the accuracy of the model results (over- or underestimations) and flood defences are not constructed according to the safety standard.

These possible conservative results are not verifiable for all model outcomes, because only a small range of model outcomes can be compared with observational data. To illustrate this, we look into the discharge levels of the river Rhine over the past 120 years of which the highest observed Rhine discharge near Lobith is 12.850 m^3/s (homogenised discharge level according to Parmet et al. (2001)). Although we have a relative long time series we want to draw conclusions about water levels with an exceedance probability of for example 1/10.000 per year. Statistics derived using the GRADE instrument show that this corresponds to a discharge level of 16.270 m^3/s (without statistical uncertainties). Well-founded substantiation about the reliability of model results can only be achieved for hydraulic load levels with small return periods, because of the limited duration of observations and statistical noise for higher return periods.

In previous safety assessment regulations, before WBI-2017, only the inherent uncertainty of parameters was taken into account for the most dominant physical processes that determine the hydraulic load parameters. The probabilistic calculations of WBI-2017 are done with statistical and model uncertainty in local water levels and wave conditions. Statistical uncertainties are quantified in the distributions of the stochastic variables and the model uncertainties are included by introducing three stochastic parameters: water level, significant wave height and the wave period. These parameters are not the source of model uncertainties, but are used as representation of several parameters that introduce model uncertainties (e.g. roughness, geometric schematisation, lateral inflow and fetch). Figure 1.2 shows at which stage of the process statistical and model uncertainties are added. The statistical uncertainties are processed in the frequency distribution of the stochastic variables and the model uncertainties are introduced at the computational core.

The quantification of the model uncertainties is done for every water system where physical models are used to determine the hydraulic load levels. The shortcomings and uncertainties associated to the model vary for different areas and therefore quantification as well. Comparison analyses between measured and calculated loads and sensitivity studies, complemented with expert judgement are used to substantiate the size of the model uncertainty (Chbab & Groeneweg, 2017).

The motivation for this research is that water level frequency lines that are modelled in accordance with the model uncertainty technique can result in physically unfounded effects, for example in the river IJssel around Olst. Olst is located just upstream of a river reach which is given more space, so-called Room for the River (RfR) projects. This project consists of a flood channel that starts to flow during high water using an adjustable weir, so the conveyance part of the river is enlarged and the result is lowered water levels for discharges that exceeds the threshold value at which the inlets are opened. Another physical aspect that influences hydraulic processes around Olst is the strategy that discharge distributions at the bifurcations are controlled at design conditions. If the discharge at Lobith exceeds 16.000 m^3/s, the additional discharge will be diverted to the Waal and IJssel, which results into non-increasing discharges and water levels at the Lek, but enlarged water level at the IJssel.

These physical interventions result in the two bends in the water level frequency line without model uncertainty (blue line) in Figure 1.3. The frequency line starts to decline around a return period of 100 year (the moment at when the flood channel starts to flow) and increases around a return period of 4000 year (when more discharge is diverted to the IJssel). However, if the model uncertainty is added by making the water level a stochastic parameter (normal distribution with in this case no bias and a standard deviation of 20 cm) the effect of those physical interventions is partly revoked (see green line in Figure 1.3), which is also demonstrated by Geerse et al. (2017) at a nearby location.

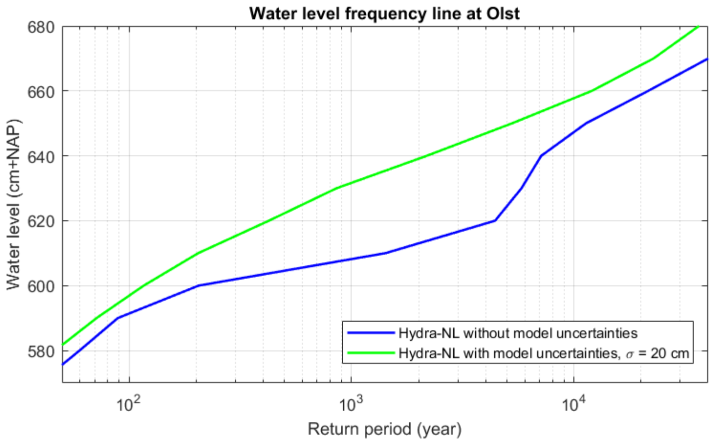


Figure 1.3: Frequency lines for water levels that are calculated by Hydra-NL, with and without model uncertainty, in the IJssel near Olst. The high water channel Veessen-Wapenveld is located a few kilometres downstream of Olst. The calculations of Hydra-NL were carried out with statistical uncertainty.

The extent to which model uncertainties for water levels (normal distribution with zero bias and a standard deviation) effectively modifies the water level frequency lines, depends on the chosen standard deviation and the so-called decimate height (water level difference that reduces the exceedance probability by a factor 10). For frequency lines with lower decimate height, the contribution of the standard deviation to the frequency line with model uncertainties is larger (Geerse & Rongen, 2016). This behaviour can be found in the water level frequency line at Olst, but also applies in other water systems like the Lake area or Vecht- and IJsseldelta. It is unknown if the actual sources of the model uncertainty also result in this statistical behaviour of water level frequency lines.

1.3 Aim and objectives

The overall goal is to improve the performance of Hydra-models, which have the purpose to give the best possible representation of the hydraulic load parameters at which flood defences need to be assessed. The performance depends on various aspects, like model structure, chosen numerical techniques and model input and parameters. This study focusses on the representation of model uncertainties and the objective of the research presented in this thesis is:

> (1) Analyse how model uncertainties affect the water level frequency line in different water systems and (2) provide insight into different methods to quantify model uncertainties in Hydra-models.

In addition of the aim of this research the following research questions are defined:

1. Do water level frequency lines for different fresh water systems calculated by Hydra-NL correspond to observations and are the physical processes that play an important role in each water system represented correctly?
2. What is the effect of dominant uncertain model/input parameters on the water level frequency lines for different hydrodynamic systems for the upper rivers and are these results consistent with the current WBI2017 uncertainty modelling technique?
3. How do dominant uncertain model/input parameters effect the water level frequency lines for the lake area and is this consistent with the current WBI-2017 uncertainty modelling technique?

1.4 Remarks and study limitations

- Only model uncertainties associated to the local water level are investigated.
- This study uses previous inventories of uncertainties and applies them on case studies in different water systems. Different uncertainty modelling techniques are assessed, which can help to set out a strategy to cope with model uncertainties in a proper and physical substantiated way.
- In this study the hydraulic roughness is parameterised by the Manning's coefficient and is assumed to be discharge independent. Dune formations during flood waves could increase the hydraulic roughness during higher discharge levels, but these discharge dependent processes are not taken into account. The uncertainty in the hydraulic roughness is also assumed to be the same for different river schematisations.
- The model that is developed in this study for the upper river area is based on simple normal flow and backwater equations and therefore not all physics are in detail described. In reality, complex flow fields will occur in rivers that are captured by 2D WAQUA-model calculations. The model in this study gives a clear indication how model parameters affect water level frequency lines, but for practical cases a more accurate solution is required.
- The uncertainties in the water level caused by uncertain discharge distribution at the bifurcations are not taken into account in the developed model. River stretches are assumed of which the discharge frequency line does not contain uncertainties. In reality, uncertainties related to roughness and geometry of the river can influence the water levels near the bifurcations and subsequently the discharge distribution.

1.5 Research contribution

The main objective of this research is to gain insights about how sources of model uncertainties effect the hydrodynamics and water level frequency lines of different water systems in the Netherlands. In WBI-2017 is chosen for a pragmatic approach to include model uncertainties by considering the water level as a stochastic variable in the probabilistic calculations. The insights of this master thesis are instrumental to enhance the implementation of model uncertainties in software wherein the probabilistic framework for assessing the safety of flood defences is embedded. This can be used to weigh in which way uncertainties are modelled in the hydraulic load model of WBI2023. In general the following techniques can be applied to implement uncertainties in model parameters, related to the water level (H. de Waal et al., 2013):

1. The parameter can be used as a full stochastic variable in the probabilistic model and thus affects the hydrodynamic calculations.
2. The effect on the water level of the uncertainty in the parameter can be added as a surplus on the calculated water levels within the probabilistic calculations, so no additional calculations are required (current method in WBI-2017).
3. A representative, deterministic value of the parameter is used in the production runs and no surplus will be added afterwards.

The trade-off between these techniques will be enhanced, knowing the performance at different water systems in the Netherlands. These modelling techniques should reflect the reality closely regarding the sources of model uncertainties on the water level frequency lines. A limiting factor is the computational power, so the amount of hydrodynamic calculations should also be taken into account for each water system. If a stochastic variable is added to the hydraulic load models, the amount of calculations will be multiplied by the partitioning of the variable and thus leading to much more process time.

1.6 Structure report

This section gives an overview of the report structure, and with that a short overview of the methods used to achieve the aim and objective of this research. The chapters of this report are structured as follows:

- Chapter 2 describes the comparative analysis that compares water level observations with hydra-NL calculations for several gauge stations in the Netherlands.
- Chapter 3 deals with the model uncertainties associated to the upper-river area. An one-dimensional semi-analytical model is developed that is used to derive hypothetical water level frequency lines for different hydrodynamic river systems.
- Chapter 4 treats the model uncertainties for the lake area. To examine how uncertainty in the input and/or model parameters (e.g. wind drag coefficient) effects the wind set-up and resulting water level frequency lines in the lake area a one-dimensional semi-analytical model is used.
- Summarising conclusions and recommendations are set out in Chapter 5.

2. Comparative analysis

2.1 Introduction

In this chapter a comparative analysis is executed to analyse the model outcome of Hydra models for several locations in different water systems in the Netherlands. The performance of the Hydra model (calculations with and without model uncertainties) will be compared to actual observations. Only water levels are considered in this study. It is tried to have widely distributed locations in the Netherlands of different water systems where also different hydraulic processes shape the water level frequency lines. The availability of water level time series affect the choice of the locations, because not at every location in the Netherlands there is a measuring station and it is tried to have at least 25 years of observations (we not always met this requirement). The water level observations where obtained from Waterbase (`live.waterbase.nl/`) which is an application of Rijkswaterstaat where historical water level time series of locations in the Netherlands are stored. No further checks were performed to verify the reliability of the measurements.

This study focusses on model uncertainties associated to water levels which limits the comparative analysis to the fresh water systems; In the salt water system the water levels are determined by statistical analysis and model uncertainties are not considered. The fresh water system in the Netherlands incorporates the following water systems:

1. **Vecht- and IJsseldelta**; consisting of the deltas of the rivers Vecht and IJssel
2. **Lakes**; consisting of Lake IJssel and Lake Marken,
3. **Upper rivers**; consisting of the upper reaches of the Rhine branches (Nederrijn-Lek, Waal and IJssel) and Meuse.
4. **Tidal rivers**; consisting of the lower reaches of the Rhine and Meuse and their branches.

For every water system, several locations are discussed depending on the dominant processes that determine the water level frequency line. The quantification of the model uncertainty is also adjusted to these dominant processes and varies for different areas in the subsystems. The following research question is answered in this chapter:

> *Do water level frequency lines for different fresh water subsystems calculated by Hydra-NL correspond to observations and what is the effect of the inclusion of model uncertainties (and are the physical processes that play an important role in each water system represented correctly)?*

2.1.1 The need of probabilistic models

Why do we need a probabilistic model like Hydra-NL and why are we not using statistical techniques for observations to determine the hydraulic load parameters? These probabilistic models are, among others developed because it is not feasible to measure continuously at any location in the Netherlands to have sufficient insights about probabilistic hydraulic load parameters. In a probabilistic model the probability of occurrence of hydraulic loads is determined by making use of other dependent stochastic variables, where sufficiently long records are available.

Another important aspect is that long time series of hydraulic load parameters consist of inhomogeneities. These inhomogeneities should filtered out which in general result in limited time series that makes statistical techniques for observations unreliable. An alternative is to homogenise the time series, which is a challenging task. Physical processes may deviate considerably for different ranges of hydraulic load levels, particularly if there is a discontinuity in this process. A clear example is the effect floodplains in a river, which starts to count from a certain water level in the river. As displayed in Figure 2.1, the water level frequency curve (right graph) shows a bend at h_{crit}: the height at which the floodplain starts to contribute. Statistical analyses of observed water levels, for the whole range, does not make sense because the extreme conditions (water levels during high water) are physically different processes. Therefore only statistical analyses of water levels exceeding the bank full discharge should be used, but that gives a limited amount of observations that leads to unreliable statistical analyses. Other causes of non-homogeneous measurement series are changes of the river geometry and different vegetations in flood plains. These changes may be corrected by vertical shifts, but this will makes the statistics more complex and also less reliable.

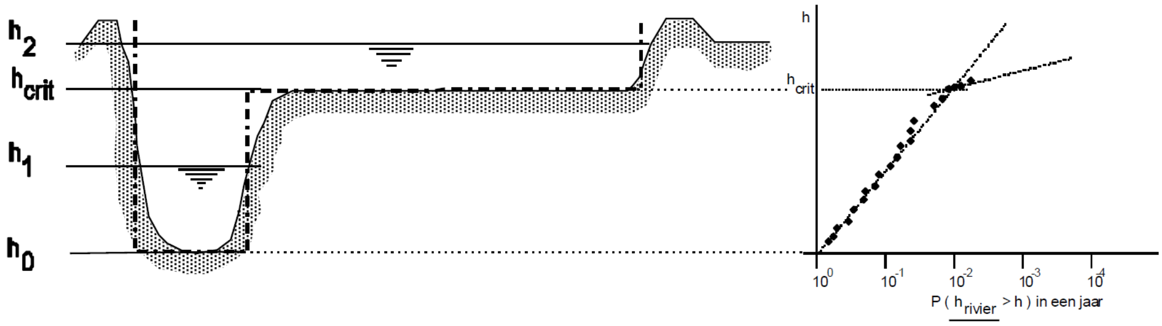


Figure 2.1: Left: visualisation of representative water levels in a cross-sectional profile of a river. Right: Exceedance probabilities per year for water levels that show a kink due to the contribution of the floodplain (Vrijling and Van Gelder (2002)).

Hydrodynamic models are used to calculate water levels based on different combinations of threats, where for the fresh water systems the water levels are computed by WAQUA. For each water system the hydrodynamic model is calibrated and/or validated against measurements. This set of WAQUA results is collected in a database and are used as input of the computational core of a probabilistic Hydra model. Next to this database, the probabilistic hydra-model uses probability and frequency distributions of the stochastic variables that are derived by statistical analyses. Also correlation models are used to describe the correlation between the stochastic variables. Water levels for a range of exceedance frequencies can be derived by translating the stochastic variables to the calculated water levels, resulting in water level frequency lines.

2.2 Methods

2.2.1 Plotting position of observations

The time series of observations consist of hourly measured water levels. A threat (flood wave in a river or storms at sea) is an event that occurs in time of which we are interested in the peak values. These events need to be filtered out of the time series such that the peaks of every event occur independently from each other in time. Therefore, it is assumed that every 720 hourly period (30 days) these variables assume new values, independently from the values in the preceding period. Events are selected in such a way that no higher peaks are present in a window running from 15 days before until 15 days after the selected peak. Since threats are much more probable in the months October - March (winter months) then in April - September (summer months), the analysis is restricted to data of the winter months. Calculations and statistical data in Hydra models are also based on winter months where a base duration of 30 days is used to ensure independence of the peak values of evolving variables. For the remainder of the analysis, only the selected events are considered.

In order to make a comparison between observations and resulting water level frequency lines derived by a Hydra model (in this case Hydra-NL), the events need to be ranked in order of magnitude and plotted on probability paper. Different plotting rules can be used to obtain order-ranked data by which the return periods of specific events are estimated. The first step is to rank the dataset in increasing order of magnitude ($x_i > x_{i+1}$) from the smallest i=1 to the largest i=N. The exceedance probability in one year associated with each event can be calculated by Gringorten plotting position (Gringorten, 1963):

$$P(X > x_i) = \frac{i - 0.44}{N + 0.12} \frac{N}{T} \tag{2.1}$$

i and N are associated to the selected events and T is the duration of the water level time series. The return period of an event corresponding to this exceedance probability is given by:

$$T_{x_i} = 1/P(X > x_i); \tag{2.2}$$

There are a number of plotting formulas that result in different positions of the observations. The estimate of the exceedance probability associated with the observations of the stochastic sample set (water levels in this case) depends on the used plotting formula. For example, the largest return period of the Gringorten method is 1.78 T, while the Gumbel plot gives T+1.

Trend correction

The water levels in the tidal-river area are affected by storms at the North Sea. Depending on the exact location in this area, the sea level has a major influence on deriving water levels. The sea level has been non-stationary the last decades and therefore observations need to be corrected. The increase of water levels due to sea level rise depends on the location. Duits (2006) showed that water levels in the tidal-river area where both sea and river discharge are felt could increase by about 60 % of the sea level rise. At Maasmond (located between the sea and the storm surge barriers) the increase of computed normative water levels are exactly the sea level rise in the climate scenario, while this increase would be less at location where discharge become more important. There is chosen to correct the water levels at all locations in the tidal-river area with the same trend. This trend correction is set to a water level rise of 20 cm per century, so for measured water level of 20 years ago 4 cm is added. This uniform trend correction results in too large corrections at locations in the tidal-river area where the sea level is less dominant.

2.2.2 Flood protection programs

After the storm surge of 1953 the Dutch government established the Delta Commission to draw up plans for preventing any such disaster in the future. The Commission recommended closing off a number of sea inlets, shortening the coastline by about 700 kilometres and a couple of years later the building of the Delta works started. Except for the threats of flooding along the coast by storm surges, parts of the Netherlands may also be flooded due to high river discharges. The events of 1993 and 1995 along the

Rhine River initiated large-scale river works in the Netherlands, because the design discharge increased with approximately 1000 m^3/s. This increase should be compensated by the so-called Room For the River projects. River intervention works were planned and built which had the aim to increase the flood conveyance capacity as well as biodiversity.

These kind of flood protection programmes influence the hydrodynamics in the water systems and result in non-homogeneous time series of the water level. The behaviour of the water systems could differ in time and could not be representative for the current situation. The two programmes that affect the hydrodynamics in the fresh water systems the most are discussed below.

Delta Works
The Haringvliet dam (completed in 1971) bridges 4.5 kilometres of water between Goeree-Overflakkee and Voorne Putte. This Dam has got two functions: 1) flood protection and 2) drainage of water from the Rhine and the Maas into the North Sea. Openings in the Dam regulate the amount of water which flows into the North Sea. The Haringvliet Dam results in less tidal influences in the Haringvliet and surroundings and hydrodynamics of the system are affected.

Other important delta works affecting the hydrodynamics of the fresh water systems are storm surge barriers like Maeslantkering (1997), Hartelkering (1997) and Ramspolkering (2002). Their primary function is to protect densely populated areas near the river mouths of the Rhine and the Meuse against storms and high sea levels. These movable barriers can close when water levels are threatening the dikes in the environment. These recently constructed barriers influence the water level time series, because the barriers are closed during certain conditions. Such a physical process result in non-homogeneous water level time series if closures affect the time series. To ensure these inhomogeneities starting date of the time series is often chosen later than the construction date of a surrounding barrier.

The storm surge barriers are implemented in Hydra-NL with a failure probability. The failure probability of a barrier is the probability that the closure procedure fails, while the conditions for closure are met. In general, the Hartelkering and Maeslantkering are considered to be one barrier (Europoortkering) with a failure probability of 1/100 per event. The failure probability of the barriers that are related to the fresh water system (Europoortkering and Ramspolkering) are used in Hydra-NL to calculate the effect on water level frequency lines. Two model runs are made in areas that are influenced by the operation of these barriers: 1) normal failure probability ($P_{EP} = 1/100$) and 2) always failing ($P_{ep} = 1$).

Room for the rivers and Meuse Works
Other projects of flood protection programs, like Room for the River and the Meuse works, can not be adjusted easily in order to analyse the situation before and after these interventions. The model calculations of Hydra-NL are using the current situation of the river geometry, while the actual observations include different geometrical states of the river. The water level time series at locations that are influenced by these measures are non-homogeneous. In the appendix you can find 1) an overview of all Room for the River projects in the Netherlands and 2) an overview of the Meuse works.

The comparison between water level observations at locations where river interventions (widening and deepening of the rivers) affect the systems are invalid and therefore the results of Hydra-NL are not compared to the observations. In these cases, only water level frequency lines with and without model uncertainty are analysed.

2.2.3 Approach and principles of WBI2017 uncertainty modelling

Model uncertainty in the water level is included by considering the calculated water level as a stochastic variable that is described by a normal distribution with a mean and standard deviation. The model uncertainty parameters are independent of the return period of the other stochastic variables and dependent on the location. The dominant hydraulic processes may deviate for each water system under consideration and the model uncertainties as well. The mean of the model uncertainty is assumed to be zero, because these models are calibrated and no bias is expected. For the different water systems the sources of model uncertainty are quantified and the total model uncertainty is calculated as the sum of the sources, which are assumed to be independent. The assumption is made that the source uncertainties are normally distributed in which the total standard deviation is the square root of the sums of the squares. Background information about the substantiation and the quantification of the standard deviation can be found in Chbab and Groeneweg (2017) and are qualitatively discussed for every water system in the next section.

Effect of the model uncertainty on water levels

In general, the inclusion of model uncertainty results in a water level frequency line that is located higher than the original line. Geerse and Rongen (2016) derived a formula that estimates the net effect (Δh) of the model uncertainty on water level frequency lines (net effect is illustrated in figure 2.2): water level increase given a certain return period. This formula only applies to situations where the water level frequency line approximates a straight line when it is plotted semi-logarithmic as a function of the return period. Furthermore, the validity is also dependent on the decimate height and the standard deviation of the stochastic variable representing the model uncertainty. The formula is based on the net effect around the return period of 1000 years and is essentially only valid for high return periods (T $\geq$ 100).

$$\Delta h = \frac{1}{2}\ln(10)\frac{\sigma_h^2}{h_{dec}}, \quad \text{if } h_{dec} \geq 0.3\text{m and } \sigma_h \geq 0.1\text{m} \tag{2.3}$$

where σ_h is the standard deviation of the stochastic variable representing the model uncertainty and h_{dec} is the decimate height of the water level. The decimate height is commonly used for the assessment and design of flood defences, because it expresses the relation between flood probabilities and changes of the water levels. In fact, the decimate height can only be calculated if the return periods have got an exponential dependence on the water levels. Often, but not always (see Figure 1.3), this is a reasonable approximation, according to results of Hydra-calculations. A practical approach to estimate the decimate height at a specific return period is to average the absolute difference in height between the water level calculated by ten times larger and smaller return period.

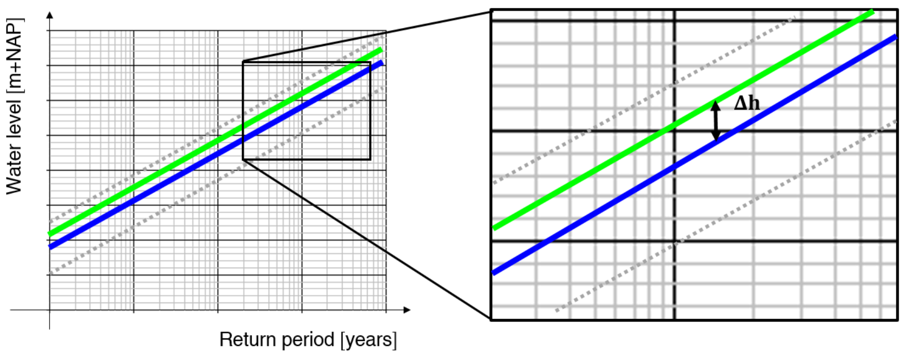


Figure 2.2: Illustration of the net effect on the water level frequency line by introducing model uncertainty. The blue line represents a water level frequency line without model uncertainty and the green line includes model uncertainty.

The decimate height depends on two aspects 1) the frequency curves of the stochastic variables or threats (including correlations between each other) and 2) the environment that translates the threats to water levels. For example, in the upper river area the frequency curve of the water level is mainly determined by the discharge. The frequency curve of the discharge gives an increase of the discharge level for a reduction of the exceedance probability by a factor 10. The environment (e.g. roughness and geometric profile) is the link between discharge changes and water level response in a river. For example, an increase of the discharge in a wide river results in a lower water level response than for a narrow river.

The functioning of the WBI2017 method will be further substantiated in the next section where water level frequency lines with and without uncertainty are showed for different water systems.

2.3 Results and discussion

The results and discussion are elaborated together. For every fresh water system the dominant processes and used stochastic variables are discussed. The major uncertainty sources associated to the water levels are mentioned as well as the quantification of the model uncertainty that is used in calculations in Hydra-models. The substantiation and the quantification of the model uncertainty is carried over from Chbab and Groeneweg (2017). That study is based on different hindcast studies, like Chbab (2014), Chbab (2015) and Tijssen et al. (2014).

The water level frequency lines are, where possible, compared to observations. If the data points are coloured red, the water level time series at that location are not significantly influenced by inhomogeneities and the comparison is made. However, if the data points are coloured grey, the systems inhomogeneities are too large and nothing can be said about the performance of Hydra-NL w.r.t. observations.

Besides water level frequency lines, hydra-models are able to calculate illustration points. These illustration points provides information about the most probable circumstances conditional on the occurrence of the calculated water level (Geerse et al., 2011). Also the contributions of different stochastic variables to the exceedance frequency of the water level are calculated. These illustration points are used to determine the dominant hydraulic processes for different locations.

2.3.1 Lake area

The lake area consists of the lakes IJsselmeer, Markermeer (including IJmeer, IJburg, Gouwzee), the Randmeren (Gooimeer, Eemmeer, Ketelmeer and Vossemeer) and the Eemvallei. The stochastic variables that are considered to derive hydraulic load levels in this area are the lake level, wind speed and wind direction. Water levels in the lake area are computed by the 2-dimensional hydrodynamic model WAQUA and possible combinations of the stochastic variables. For the largest part of the lake area, wind is the main driving force for calculated water levels. The wind shear causes a set up of the water level at the downwind side. The wind speeds are highly dependent on the wind direction, where west and west-northwest result in the most extreme wind speeds. Figure 2.3 shows how the lake area is divided in sub-areas based on dominant hydraulic processes. The west side of the lake area is dominated by the lake level, because the wind speeds are in general much lower from the east direction. The following standard deviations for the model uncertainty of the water levels are used in the sub-areas.

Table 2.1: The model uncertainty in different sub-areas in the lake area

	Sub-areas	Standard deviation [m]
1	Wind-dominated locations (IJsselmeer and Markermeer)[a]	0.30
2	Wind-dominated locations (Ketelmeer, Vossemeer, Eemmer and Nijkerkernauw)	0.25
3	Lake-level dominated locations	0.15
4	Combination of wind and lake level	0.35
5	Wind dominated and discharge from the river Eem	0.10

[a] This is including IJmeer, IJburg and Gooimeer.

The largest and most important source of model uncertainty in the lake area is the wind. The aspects associated to wind modelling that create uncertainty in water levels are windstress, wind fields and transformation of potential wind to open-water wind. The model uncertainty increases around the Randmeren, where the topography and time modelling aspects become important. The effect of storm duration, wind rotation and fetch becomes more uncertain, because these locations are situated after a constriction surrounding lands can result sheltering effects. In lake level dominated area the model uncertainty is relatively small and are mainly caused by parameter setting and numerical aspects of the used model.

The same uncertainty sources as the Eemmeer and Nijkerkernauw apply to the area of the Eemvallei. In addition, the discharge in the river Eem also contributes the total model uncertainty. Therefore, the model uncertainty was estimated at 0.4 metre. However, due to the presence of outer-dike areas and low quays the water level in the Eemvallei rises less quickly than in the adjacent lakes. The difference in decimate heights for the locations of the Eem and the locations in adjacent lakes is about a factor 4. Therefore, the net effect of the model uncertainty becomes four times larger as well. For pragmatic reasons, the standard deviation is also chosen about a factor of 4 smaller than in adjacent lakes.

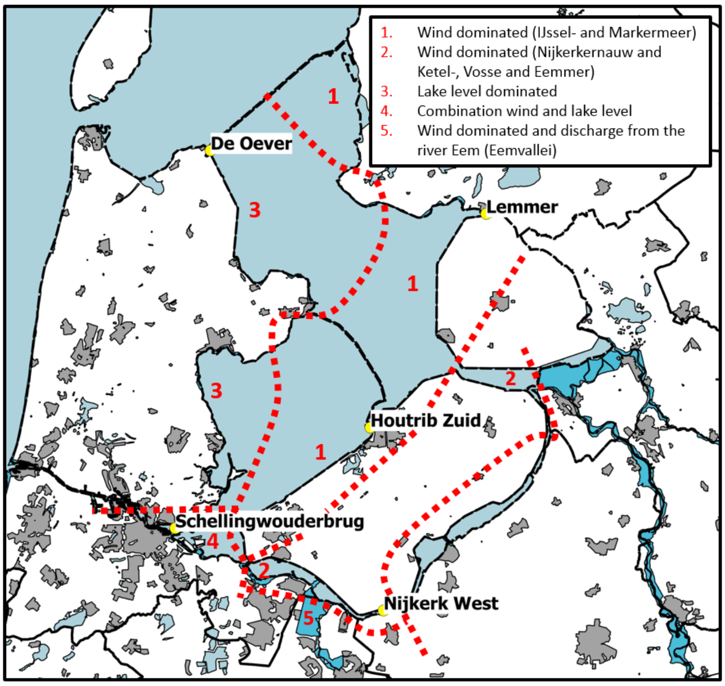


Figure 2.3: The lake area that is divided into sub-areas based on the dominant hydraulic processes [1].

Wind dominated locations

Three wind dominated locations are analysed: Lemmer, Houtrib Zuid and Nijkerk West. Lemmer is located at the IJsselmeer, Houtrib Zuid at the Markermeer and Nijkerk West at the Nijkerkernauw. The water level frequency lines without model uncertainty at Lemmer and Houtrib Zuid are very much in line with the observations. For higher return periods, the model seems to underestimate the water levels at Houtrib Zuid, but this could also be statistical noise. The inclusion of model uncertainty results in a water level frequency line that lies 20-40 centimetres above the line without model uncertainty.

The decimate height at T = 1000 years for Lemmer is smaller than that of Houtrib Zuid. At both locations the stochastic variables of the wind (speed and direction) shape the frequency line. The environment is the other aspect that influences the water levels. The water level tilt induced by the wind occurs around the centre of gravity and therefore the wind set-up would be larger for locations farther away from that point. Houtrib Zuid is in absolute terms further from its centre of gravity than Lemmer. Notice that both locations are located in different lakes and the water level tilt occurs around different points. The effect of the model uncertainty on water levels is 24 centimetres at Lemmer and 34 centimetres at Houtrib Zuid, because of the difference in decimate height.

[1] The division of the lake area is based on Chbab and Groeneweg (2017). The east-side of the Markermeer (along the Flevopolder) consists of locations where the wind and lake level are the dominant parameters according to Chbab and Groeneweg (2017), but these areas are wrongly displayed in Figure 2.3. Furthermore, illustration points show that for wind-dominated locations along the IJsselmeer high lake levels also contribute to high water levels. Dividing a water system into sub-areas is a restrictive approach, because high water levels are often caused by combinations of different threats.

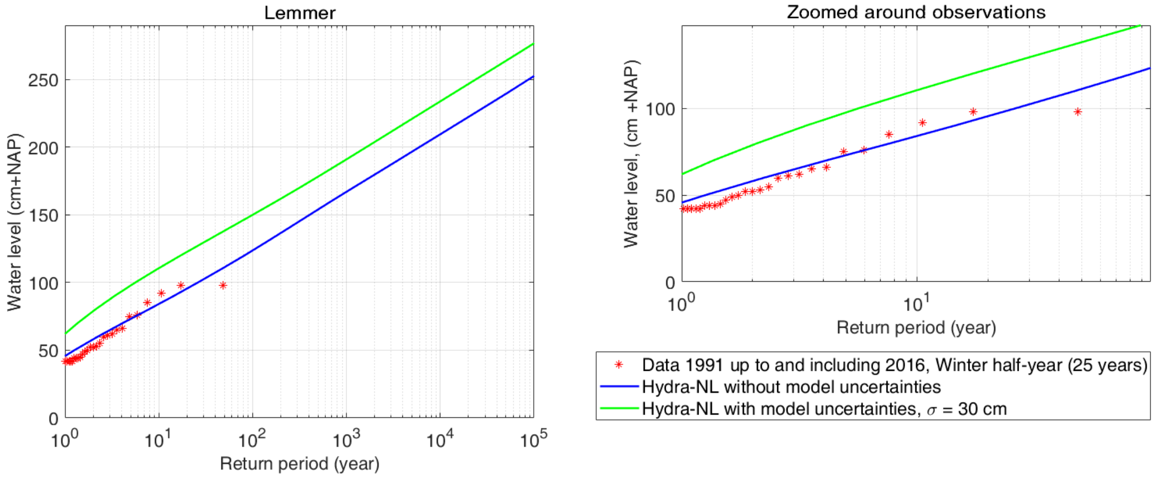


Figure 2.4: Water level frequency line at Lemmer (IJsselmeer).

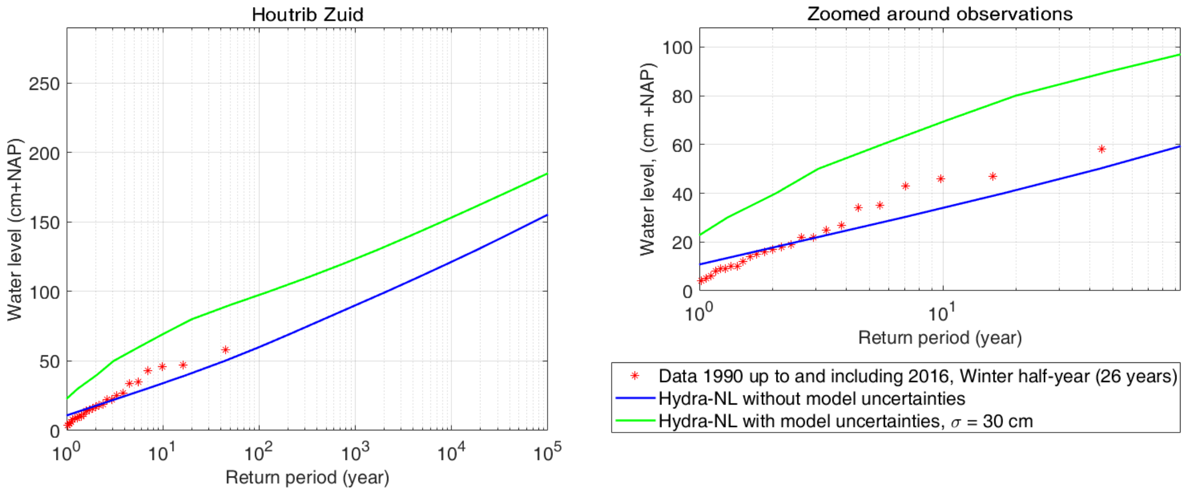


Figure 2.5: Water level frequency line at Houtrib Zuid (Markermeer).

At Nijkerk West the water level frequency line without model uncertainty overestimates the observations for over 20 centimetres. One possible explanation could be that the hydrodynamic model in the lake area is validated by comparing observations to model predictions at several measuring stations along the lake for different storms. In general the model predications correspond to the observations at the measuring stations, but deviations at some locations could occur that are hard to avoid due to model schematisations. The modelled water levels are strongly dependent on a correct schematisation of the wind, which evolves in time and space (van der Mheen, 2013). These schematisations may be too conservative and result in overestimated water levels.

The inclusion of model uncertainty has got a larger effect around lower return periods, because the decimate height is smaller while a constant standard deviation for all return periods is assumed. It is remarkable that the effect of model uncertainties is larger at smaller return periods, while in that region the uncertainties should be smaller. The net effect of model uncertainty at a return period of 10000 years is 27 centimetres, which is lower than at Houtrib Zuid. The model uncertainty at Nijkwerk West is larger, but has got less effect on the water levels that is due to the larger decimate height. A higher standard deviation does not lead to a larger net effect on the water levels, while in general a larger standard deviation of the water level is chosen because the uncertainties are estimated to be larger.

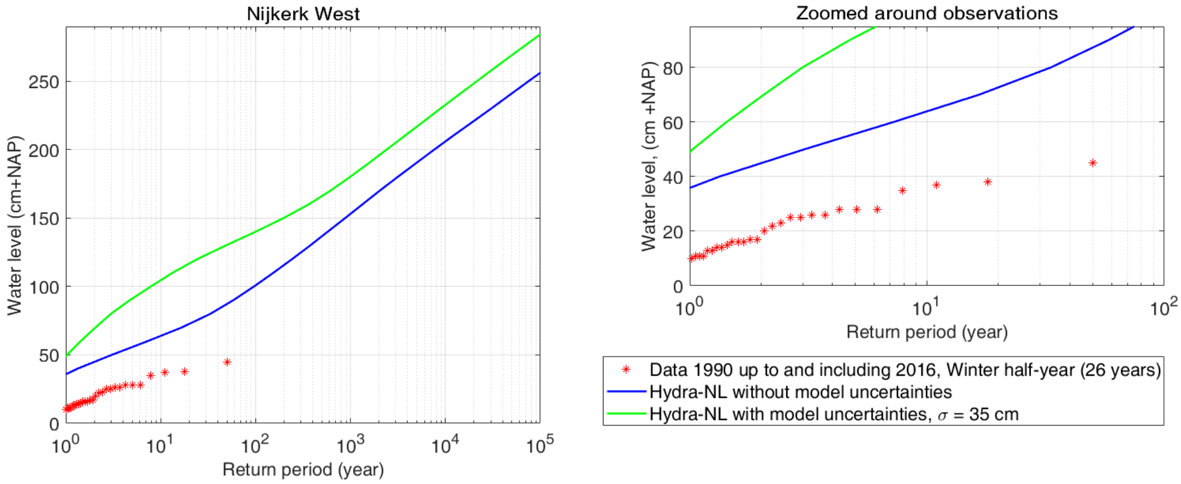


Figure 2.6: Water level frequency line at Nijkerk West (Randmeren).

Lake level dominated

The lake level is considered as a stochastic variable in probabilistic calculations. The frequency distribution of the lake level consists of statistical uncertainties that are incorporated in the statistics of the variable itself. In addition, model uncertainty is added in probabilistic calculations at locations where the lake level shapes the water level frequency line. Figure 2.7 shows the outcomes of Hydra-NL at Den Oever. The water levels seem to be overestimated by the model with approximately 10 centimetres and the net effect of the model uncertainty results in a water level increase of roughly the same magnitude.

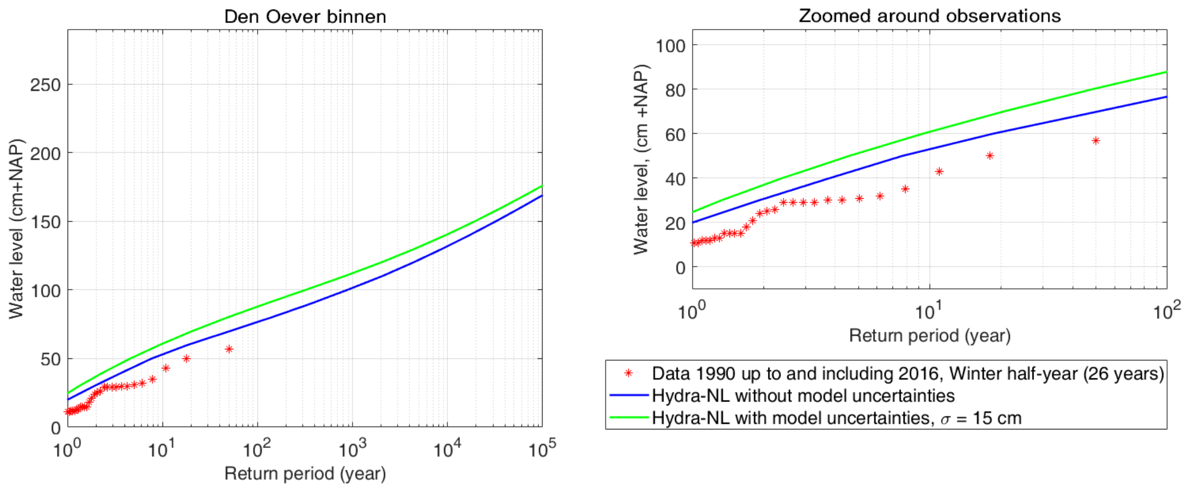


Figure 2.7: Water level frequency line at Den Oever (IJsselmeer).

Combination wind and lake level

In the last area that is analysed the combination of wind and water level are important. The contribution of the wind gets larger for higher return period where lake levels are moderate and the wind speeds are high from the North. Schellingwouderbrug is a place where both processes are important and the resulting water level frequency line can be seen in figure 2.8. Again, the model overestimates the observations slightly and the effect of the model uncertainty ranges between 15 and 20 centimetres.

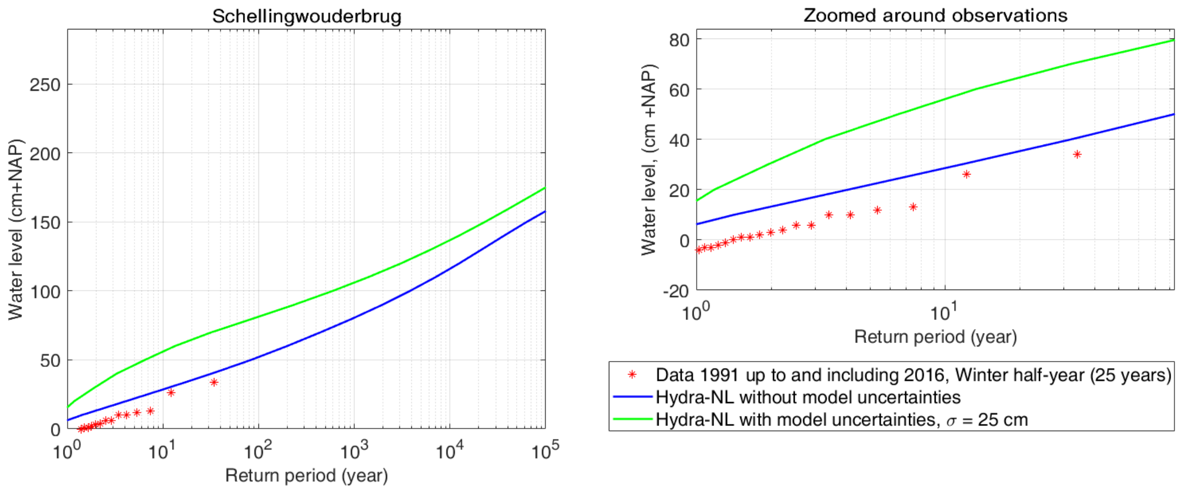


Figure 2.8: Water level frequency line at Schellingwouderbrug (Markermeer).

2.3.2 Vecht- and IJsseldelta

The Vecht delta includes the downstream part of the Overijsselse Vecht, Zwarte Water and Zwarte Meer. The IJssel delta consists of the downstream part of the river IJssel and the Keteldiep. In the deltas the following stochastic parameters are used: Discharge, lake level IJsselmeer, wind speed, wind direction and state of the Ramspol barrier. The discharge refers to the discharge in the river Vecht for locations in the Vecht delta and for locations in the IJsseldelta to the discharge in river IJssel.

In the upstream part of the Vecht delta high water levels are mainly caused by the discharge of the Vecht. Further downstream, at the Zwarte Water and the Zwarte Meer, the influences of the wind and the lake level of the IJsselmeer become more important. The same applies to the upstream part of the IJsseldelta (upstream of Kampen) where discharge is the dominant stochastic variable. Water levels at the downstream part of the IJsseldelta are mainly caused by high winds and small discharge levels in the IJssel. Based on the transition of discharge-dominated to more wind-dominated locations the sub-areas are defined, which can be seen in Figure 2.9.

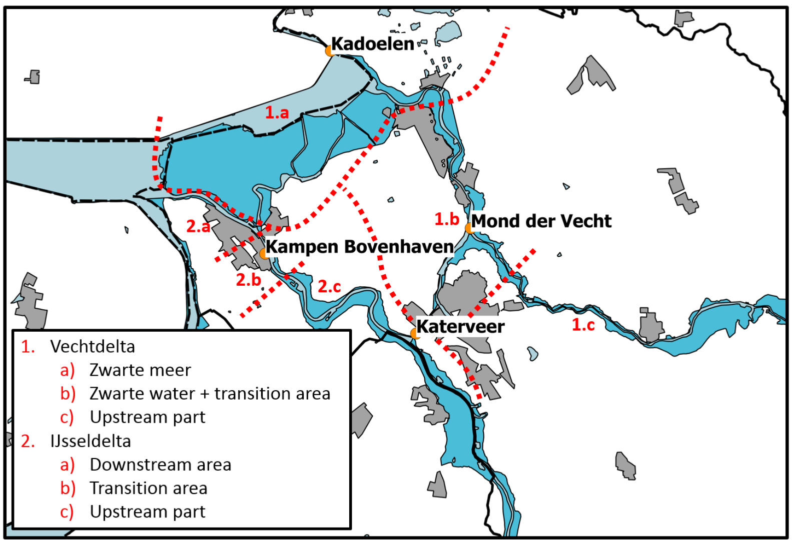


Figure 2.9: Sub-areas of the Vecht- and IJseldelta

The Ramspol barrier is located in the Vechtdelta, which separates the Ketelmeer from the Zwarte Meer and reduces the hydraulic loads at the Zwarte Meer and the Zwarte Water during extreme water levels. The Ramspol barrier closes when the lake level at the Ketelmeer exceeds 0.5 m+NAP, due to a storm from westerly winds, together with an easterly directed current at the location of the barrier. To prevent that water flows from the IJsselmeer into the Zwarte Meer and causes high water levels the barrier was constructed. The functioning of the barrier is taken into account by separate series of WAQUA calculations (barrier open and barrier functioning properly) and the inclusion of a failure probability of the barrier that is set to 1/100 per closure.

Table 2.2: Model uncertainty in the IJsseldelta

	Sub-areas	Standard deviation [m]
1	IJsseldelta downstream	0.30
2	Transition area IJsseldelta	0.30
3	Upstream part IJsseldelta	0.30

Table 2.3: Model uncertainty in the Vechtdelta

	Sub-areas	Standard deviation [m]
1	Zwarte Meer	0.40
2	Zwarte Water and transition area	0.30
3	Upstream part Vechtdelta	0.30

In Table 2.2 and 2.3 the corresponding model uncertainty for each sub-area is quantified. Important uncertainty sources for the upstream part of the Vecht- and IJsseldelta are associated to model calibration, hydraulic roughness and changes of the morphology. For quantifying the model uncertainty in the Vechtdelta there is made distinction between the two riverbanks. This study focusses on water levels along the river axis and the maximum value of the model uncertainty of the two riverbanks is chosen. In the transition and downstream area the wind modelling aspects become important, which result in additional model uncertainties just like the lake area. For further details on how these model uncertainties are quantified, reference is made to Chbab and Groeneweg (2017).

Zwarte Meer and Transition area (Vechtdelta)
The Ramspol barrier reduces the water levels in the Zwarte Meer and the transition zone. During a storm from westerly winds the water is heading up from the IJsselmeer into the Zwarte Meer, but in these situations the barrier blocks the water levels. A closed Ramspol barrier occurs in parallel with relatively low discharge levels in the Vecht. The water level frequency line is shaped by the following combinations of stochastic variables:

- Open barrier: High discharge levels in combination with relative low water level at the lakeside of the barrier, caused by relative high lake levels and low winds speeds.
- Closed barrier: High discharge levels in combination with relative high water levels at the lakeside of the barrier, caused by relative low lake levels and high wind speeds.

The contribution of the barrier state for normative water levels is approximately 50/50 open-closed situations. The resulting water level frequency line at Kadoelen can be seen in figure 2.10. If the Ramspol barrier always fails, the Zwarte Meer is a wind-dominated subsystem and the water levels are higher (dashed lines). The Ramspol barriers limits the influence of the wind and the discharge becomes more important. The model uncertainty is quantified by a standard deviation of 40 centimetres and the net effect is 55 centimetres at T=10000 years. The outcome of Hydra-NL is not compared to the observations, because the water levels are influenced by the construction of the Ramspol barrier and consist of old water level time series that do not represent to current situation.

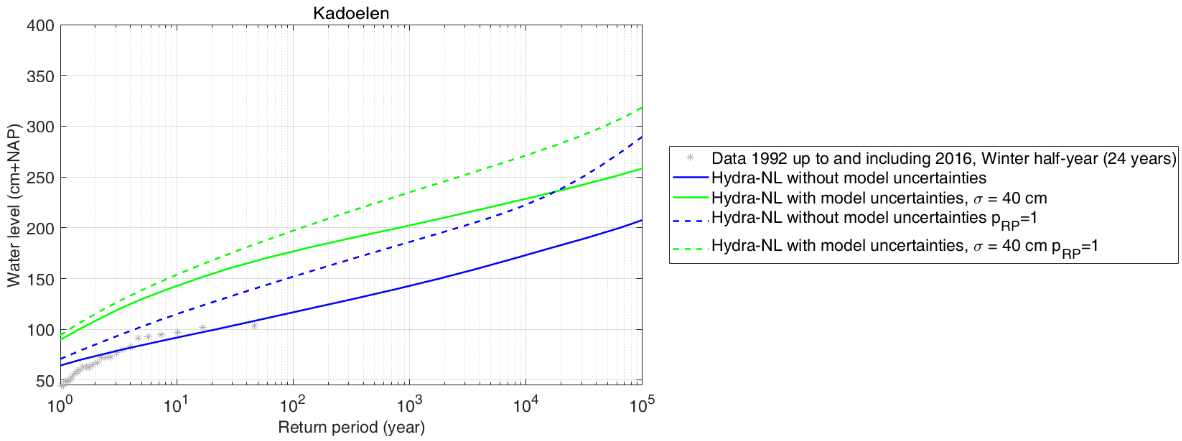


Figure 2.10: Water level frequency line at Kadoelen (Zwarte meer).

The water levels at the Zwarte water and transition area of the IJsseldelta become more discharge dominated. Water level responses are more sensitive to the discharge in the river than for storm surges and therefore the decimate height becomes larger.

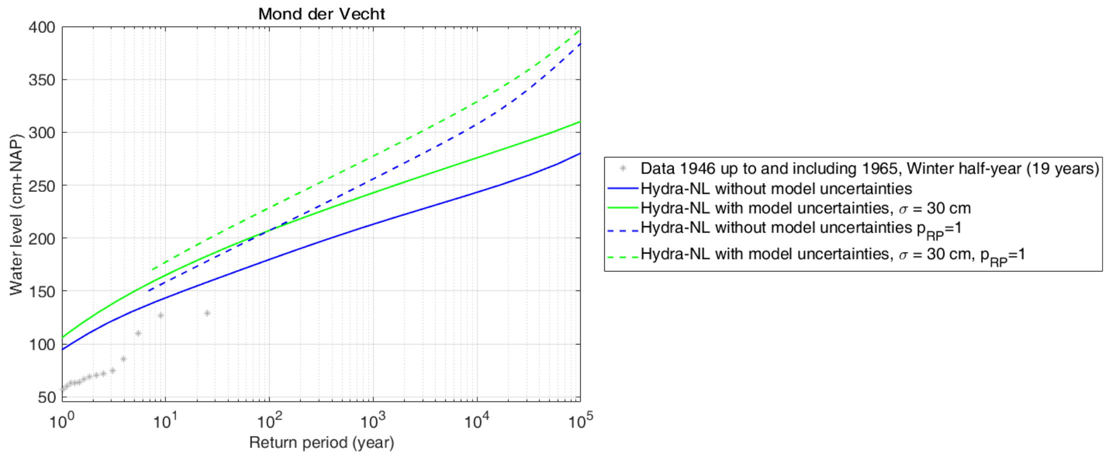


Figure 2.11: Water level frequency line at Mond der Vecht (transition area Vechtdelta).

Transition area and Upstream area (IJsseldelta)

The operation of the Ramspol barrier reduces the water levels in the Vechtdelta, but has got a negative impact on the water levels in the IJsseldelta. During a storm surge, the water levels are blocked by the barrier and diverted to the IJsseldelta. The water levels at Kampen Bovenhaven are for high return periods mainly determined by extreme westerly till north-westerly winds that causes storm surges in the IJsselmeer that is also felt in the transition area. For smaller return period (T<100 years) the discharge is still important and the operation of the Ramspol barrier barely influence the water level. The effect of model uncertainty ($\sigma = 30cm$) result in water level increases by about 10 centimetres. Compared to Mond der Vecht, which is located in the transition zone of the Vechtdelta, the net effect at Kampen Bovenhaven of the model uncertainty is about twice as much, while the model uncertainty is estimated by the same standard deviation. Again the model outcome is not compared to the observations, because Room for the River projects changed the hydrodynamic system.

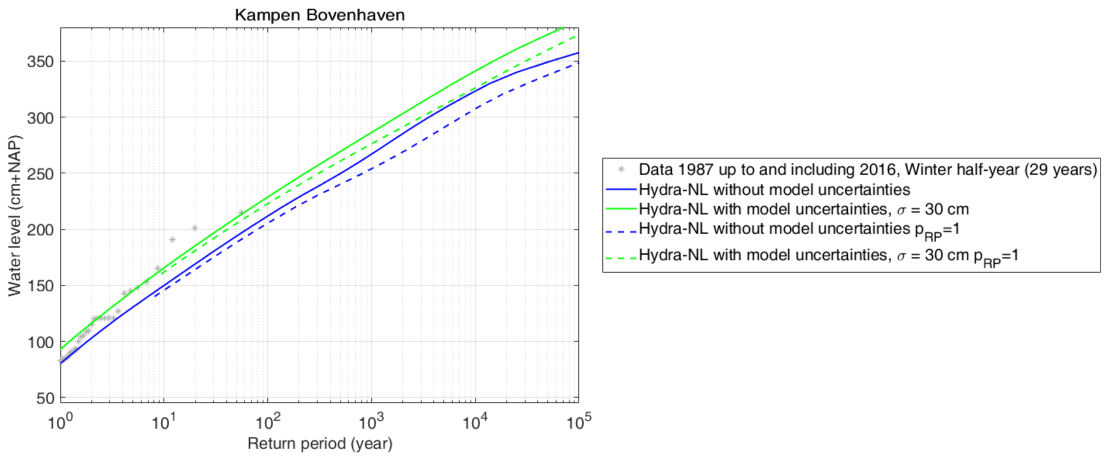


Figure 2.12: Water level frequency line at Kampen Bovenhaven (transition area IJsseldelta)

The water levels at Katerveer are predominantly determined by discharge levels from the river IJssel. The sharply increase of water level at T=4000 years is caused by the flood mitigation measure "Lek ontzien",where more discharge is diverted to the IJssel. The water levels including model uncertainty result in deviant behaviour, because the physical processes that shape the water level frequency line are dampened. The net effect varies from almost no effect a T=7000 years to 20 centimetres at T=4000 years.

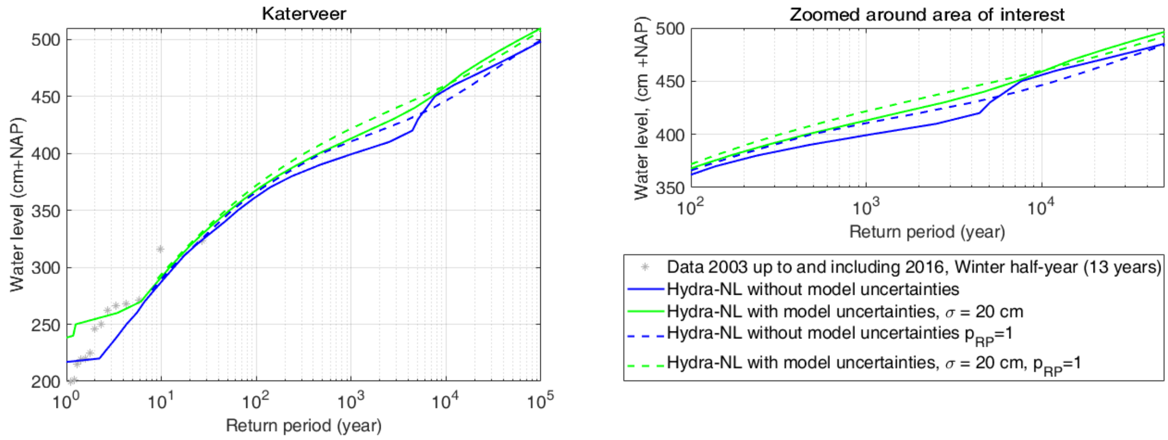


Figure 2.13: Water level frequency line at Katerveer (upstream area IJsseldelta)

2.3.3 Upper rivers

The upper-river area is the part of Meuse and Rhine and their branches, where storms at the North Sea and IJsselmeer do not affect the water level during high discharge levels. In this area high water levels along the river axis are only determined by discharge.

The Rhine branches are bordered by Lobith where the Rhine enters the country and the transition area of the tidal river area and the IJsseldelta, where the water levels are only partly determined by the discharge from the Rhine. The river divides into three tributaries just a few kilometres after crossing the border, starting with the Waal and Pannerdensch Kanaal at the Pannerdensche Kop and a few kilometres downstream, the Pannerdensch Kanaal divides into the Nederrijn-Lek and the IJssel.

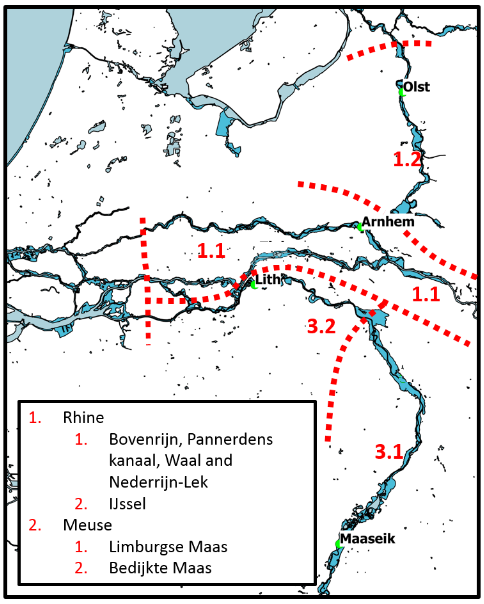


Figure 2.14: Upper river area and the defined sub-areas.

The river Meuse reaches the Netherlands at Eijsden, just south of Maastricht. The stretch where the Meuse enters the Netherlands until the border with Noord-Brabant is known as the Limburgse Maas, which consist of the Bovenmaas, the Grensmaas and the Zandmaas. The river continues in Noord-Brabant, where the river becomes the Bedijkte Maas and ultimately flows into the Hollands Diep and the Haringvliet and via discharge sluices to the sea.

WAQUA calculations are made and the model is forced by a flood wave upstream. This flood wave is characterised by a peak discharge (corresponding to a exceedance frequency) and a standard shape. The Rhine and Meuse receive discharge from a number of tributaries and the downstream boundary consists of water levels derived by rating curves. The model uncertainty includes the calibration approach, hydraulic roughness and morphological changes under extreme conditions. Uncertainties related to the discharge distribution at the bifurcation points in the Rhine are relatively small, but are taken into account at the IJssel.

The model uncertainty of the Meuse is larger than for the Rhine branches that has to do with the correlation between the hydraulic roughness used in hydrodynamic models for the Dutch rivers and GRADE (model that compute discharge statistics). The details are discussed in the next chapter, but the point is that for discharge statistics of the Meuse uncertainties in the roughness coefficient are not considered and therefore need to be fully taken into account. The discharge statistics of the Rhine includes these uncertainties and the model uncertainty in the hydrodynamic models of the Dutch rivers can be partly reduced. Table 2.4 shows the applied model uncertainty.

Table 2.4: Model uncertainty for the Dutch upper-river area

River branch	Standard deviation [m]
Bovenrijn, Pannerdens kanaal, Waal and Nederrijn-Lek	0.15
IJssel	0.20
Meuse	0.30

Rhine branches

Water level frequency lines in the river area are shaped by the transformation of discharge to water levels. Two aspects are important: 1) geometry of the river and 2) discharge statistics. In general, the river widens at higher discharge levels and the rating curve flattens out at these levels and also the slope of the water level frequency line decreases. Upstream floodings, before Lobith, reduce the extreme discharge peaks substantially and the frequency discharge curve becomes flatter. The strategy to divert more discharge to the IJssel and less to the Nederrijn-Lek affects the water level frequency lines in the Rhine branches as well. This positive effect can be observed in the Nederrijn-Lek near Arnhem (see figure 2.15), where water levels decreases around a return period of 300 years.

The inclusion of model uncertainty blurs the physical process of "Lek ontzien", because the net effect of the model uncertainty increases at places where the water level increases slower. This effect is also observed at Olst but the effect of the mitigation measure works the other way around: water levels increase.

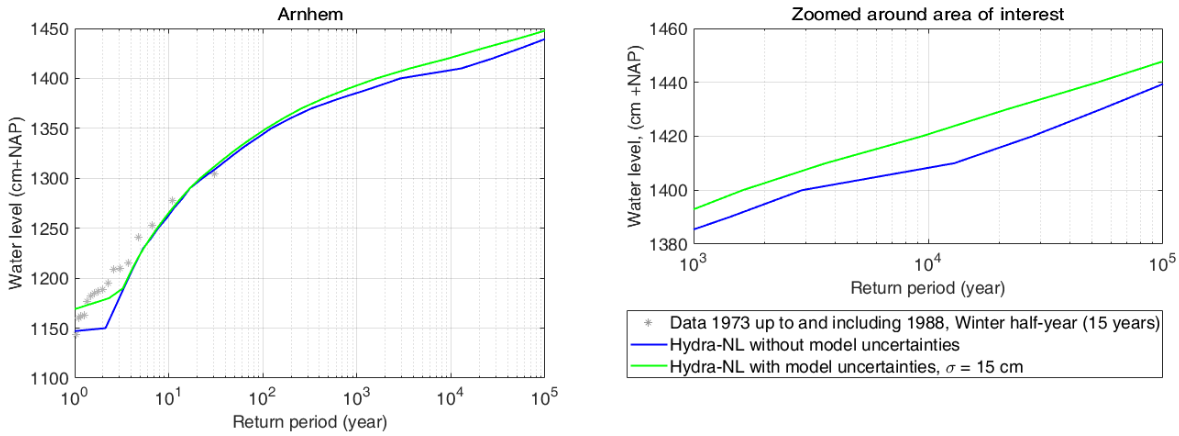


Figure 2.15: Water level frequency line at Arnhem (Nederrijn-Lek)

Contrary to the discharge levels at Arnhem, the discharge levels in the IJssel increase. This result can be observed in the water level frequency line at Olst, see Figure 2.16. Except for this strategy, a flood channel further downstream affect the water levels at Olst. This flood channel starts to function around return periods of 100 years and the water levels hardly increase until the measure "Lek ontzien" affects the water levels. The inclusion of model uncertainty reduces the effect of these measures locally (between return periods of 100 and 4000 years) by several decimetres. Chapter 3 elaborates these effects in more detail.

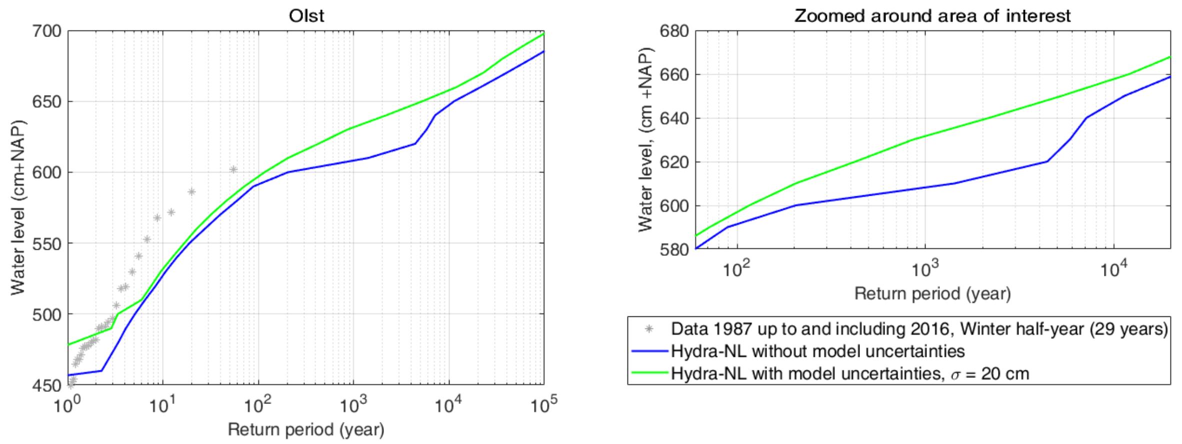


Figure 2.16: Water level frequency line at Olst (IJssel)

Bedijkte Maas and Limburgse Maas

Where the Meuse enters the Netherlands, it is still a fast-flowing river and the stretch known as the Grensmaas can meander freely. This area is also characterised by the influence of the Maasplassen and wide floodplains. Figure 2.17 shows the water level frequency line at Maaseik, where the water levels flatten out for higher return periods.

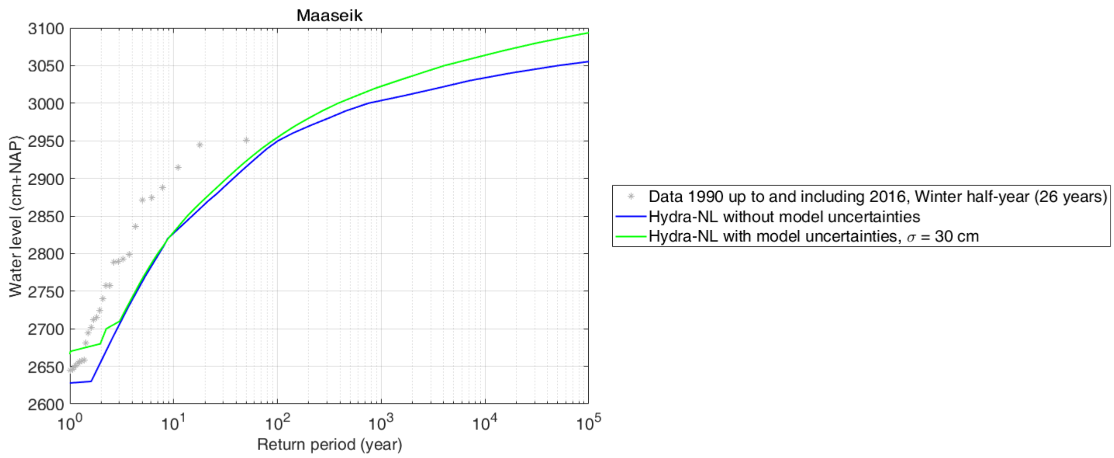


Figure 2.17: Water level frequency line at Maaseik (Limburgse Maas)

The Meuse consists of two retention areas, Lateraalkanaal-West (LKW) and Lob van Gennep (LvG), which extract water from the river. The moment that water withdrawal starts differs for both retention areas. The LKW is designed to have the maximum effect around return periods of 250 years and LvG around 1250 years. These retention areas do not effect the water level frequency lines at Lith to a large extent.

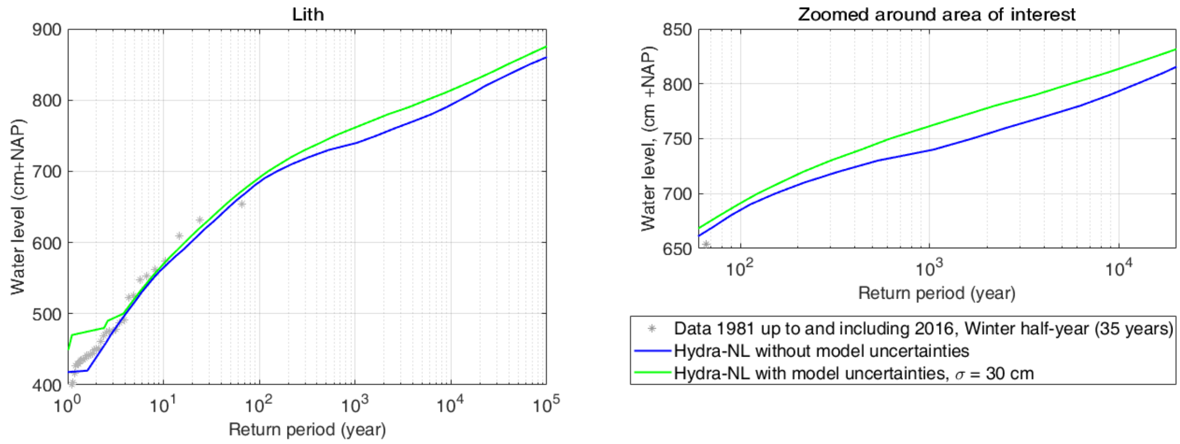


Figure 2.18: Water level frequency line at Lith (Bedijkte Maas)

2.3.4 Tidal rivers

The tidal rivers area is that part of the lower reaches of the Rhine and Meuse, where storms at the North Sea have a significant effect on the water levels. The tide also influences the water levels in this area and play an important role in the determination of hydraulic load levels. The stochastic variables in this area are discharge (Rhine and Meuse), sea level, wind speed, wind direction, the state of the storm surge barriers (open or closed) and prediction of the water level at Maasmond. In general the normative water levels are caused by four hydraulic processes: Discharge, sea level, wind and operation of the storm surge barrier. Based on these different threats the area is subdivided into sub-areas. The model uncertainty is quantified for every sub-area and obviously uncertainties related to the discharge dominant location differ from storm surge dominant locations. In Table 2.5, the quantification of the model uncertainty of each sub-area is given.

The storm surge barriers protect the land from storm surges above the North Sea. The operation of the Maeslant barrier depends on predicted water levels at Maasmond. These predictions contain uncertainties that are included in the model. Besides the predicted water levels, the probability of failure to close the Europoortkering is estimated to be 0.01 per closure. For determining the model uncertainty in each sub-area, there is made distinction between two operating situations of the Europoortkering: open and closed. Further explanation about the dominant hydraulic processes of each sub-area is provided at the paragraphs below.

Illustration points are used to indicate the most probable circumstances. However, other circumstances may also contribute to the exceedance probability of water levels. Illustration points provide limited information and if the amount of stochastic variables increases it should be used with caution.

Table 2.5: The standard deviation (SD) in metres for each sub-area in the tidal river area

	Sub-areas		Open situation	Closed situation
1	Storm surge dominant		0.15	0.25
2	Flood storage area		0.25	0.30
3	Discharge dominant		0.15	0.15
4.a	Transition area	Storm - Storage	0.25	0.30
4.b		Storm - Discharge	0.15	0.25
4.c		Storage - Discharge	0.25	0.30
5	Sea region		0.15	0.15

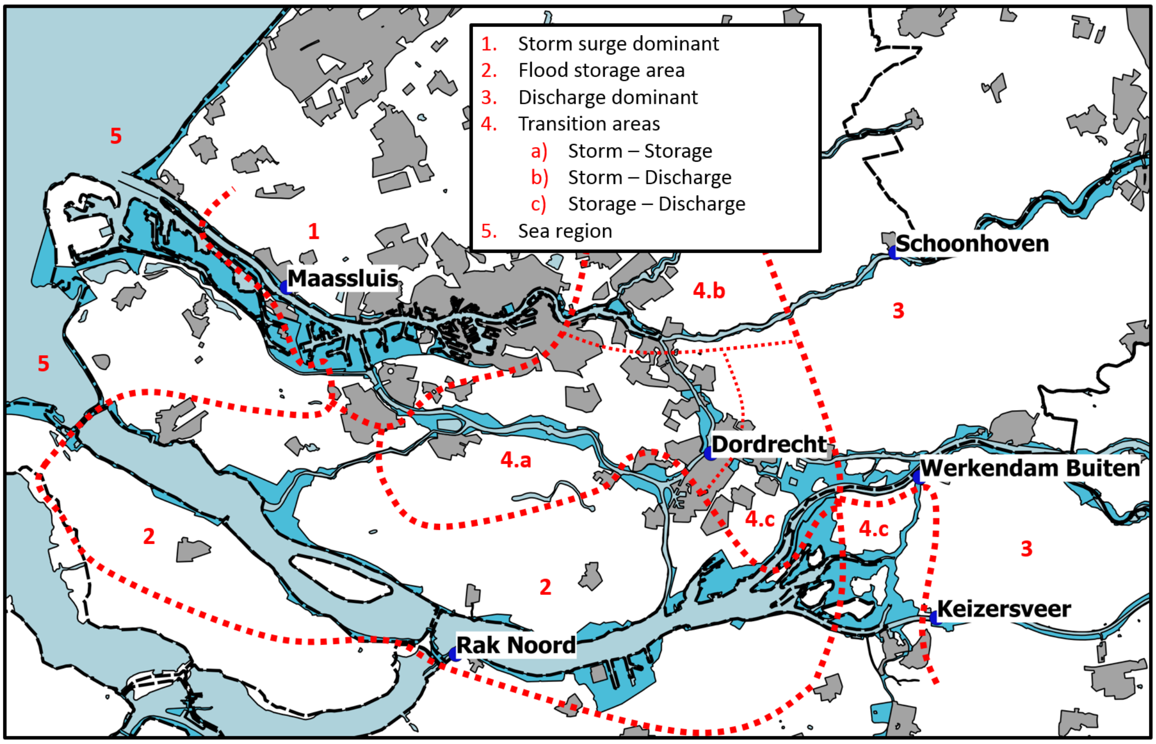


Figure 2.19: Sub-areas of the tidal river area.[2].

The sea region is not considered in this study, which is the area outside the Europoort barrier. In this area normative water levels are mainly determined by storm surges along the Dutch coast. Sudden rise

[2]This division is based on Chbab and Groeneweg (2017). It is a challenging task to define areas in which certain hydraulic processes are the most dominant. The water levels in the tidal rivers area arise from combinations of different threats and the most dominant threat in an area tells a limited story about the hydraulic processes.

in the water, such as a seiche, are also important processes in this area and are modelled in PHAROS. Model uncertainties associated to this phenomena are also take into account, but this area is excluded from the scope of this study.

The water level time series in all sub-areas are corrected for a sea level rise of 20 cm per century. This trend correction is an estimate and at locations where sea level is not the only dominant parameter the water levels are less affected by the sea level rise. Therefore, observations in these areas are corrected too heavily and the water levels are displayed to high.

Storm surge dominant
The water levels in these region are mainly determined by storm surges and the operation of the Europoortkering. The Europoortkering is operated using predicted values for the discharges of the rivers, wind speeds, wind directions and sea water levels at Maasmond. Based on these predictions, water levels at Rotterdam and Dordrecht are calculated and if the predicted water levels exceed the critical levels for at least one of the locations the Europoortkering will close. These critical values are 3.0 m+NAP for location Rotterdam and 2.9 m+NAP for location Dordrecht.

Figure 2.20 shows how the Europoortkering shapes the water level frequency line near Maassluis. If the Europoortkering is functioning, the water levels are clearly lower and the water levels are slowly increasing for higher return periods. In case the Europoortkering will always fail, the water levels are determined by high sea levels and high wind speeds. The outcome of Hydra-NL corresponds well to the observations. The model overestimates the observations with less than 10 centimetres.

The effects of the Europoortkering on the hydraulic system is two-fold (Janssen & Jorissen, 1992): reduction of water levels because the storm surge can not enter the tidal river system and an increase of water levels because of the accumulation of river discharge. The model uncertainty are supposed to be larger in case the Europoortkering is closed, but it can be argued that the physical processes that need to be modelled become less sensitive in case of a closed Europoortkering because it acts more like a reservoir that is filling. If so, the standard deviation for the closed situation should be diminished. The net effect of the WBI2017 model uncertainty is larger for the situations where the Europoortkering flattens the water level frequency line.

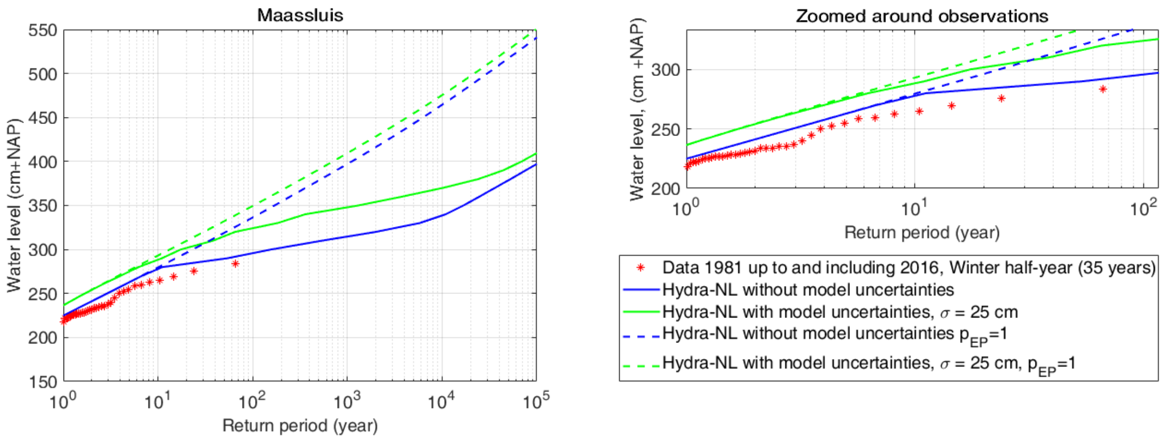


Figure 2.20: Water level frequency line at Maassluis (Storm surge dominant)

Flood storage dominant

The flood storage dominant locations are less sensitive to the functioning of the Europoortkering. This area is also further away from the barriers and discharge becomes more important. Normative water levels are mainly caused by high discharge levels and high water levels at sea that prevent the sluices at Haringvliet to discharge water to the sea. The model overestimates the water levels in this area by about 20 centimetres without model uncertainty. These overestimations are even larger if the model uncertainty is included.

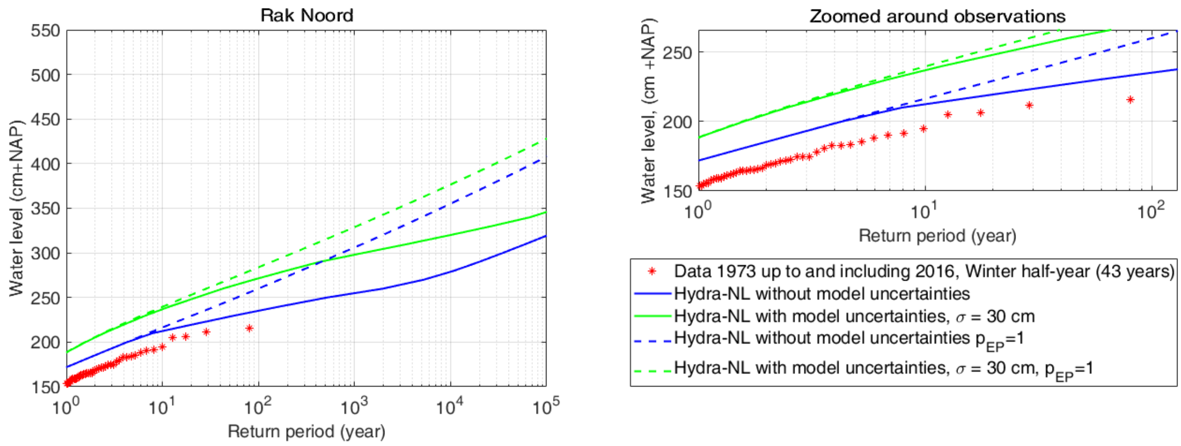


Figure 2.21: Water level frequency line at Rak Noord (Flood storage dominant)

Discharge dominant

The dominant hydraulic process that shapes the water level frequency line is the discharge from the Rhine and/or Meuse in this area. According to Chbab (2015), this area is barely affected by storm surge and tide and the the Europoortkering should not play an important role. However, a failing Europoortkering result in water level increases of 0.5 - 1 metre for higher return periods.

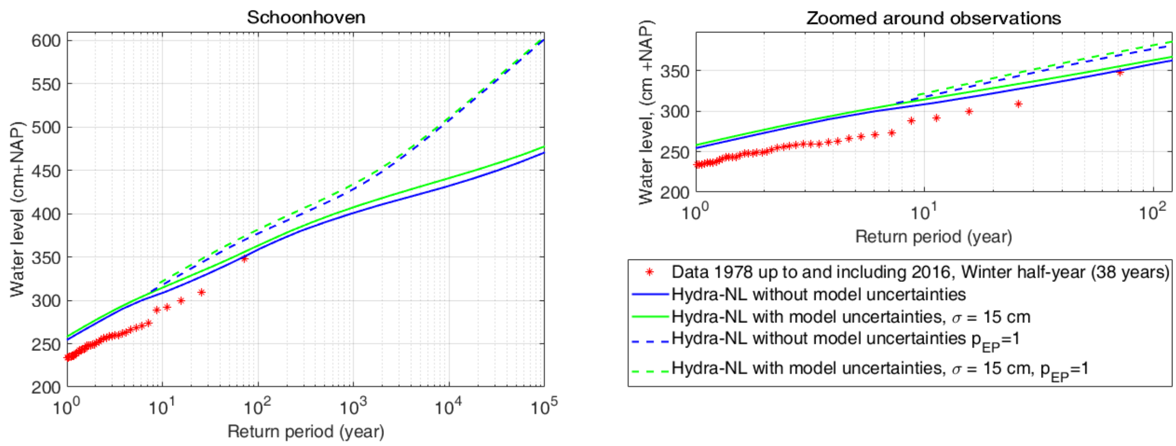


Figure 2.22: Water level frequency line at Schoonhoven (Discharge dominant)

Transition areas

The water levels in the transition area are characterised by combination of the discharge of the rivers and sea level. The varying conditions can roughly be characterised by the combination of the above-mentioned sub-areas. For example, at Dordrecht the storm surge and flood storage are the dominant aspects (see Figure 2.23). The functioning of the Europoortkering still affects the water levels and storm surge and high discharge levels are important threats in this area.

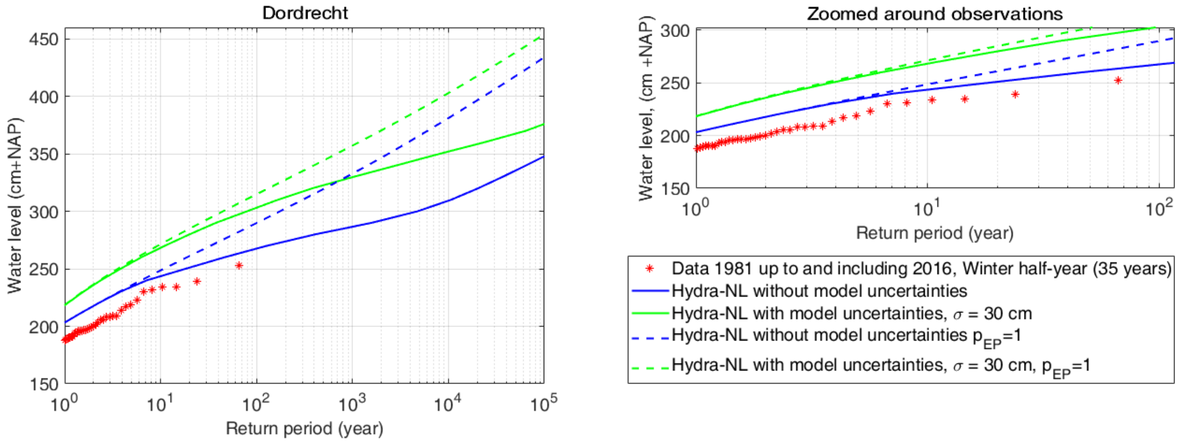


Figure 2.23: Water level frequency line at Dordrecht (Transition area: Storm - Storage)

In the transition area between flood storage and discharge there is still influence of the Europoortkering, however the effect is less pronounced. Figure 2.24 shows the water level frequency line at Keizersveer, where discharge becomes more important. This can also be noticed by comparing the decimate heights of Keizersveer and Dordrecht. The water levels at Keizersveer are influenced by Room for the River projects that result in relative lower water level frequency lines compared with the observations. The overestimations of the water levels become even larger if water levels are corrected for these inhomogenities. In addition, the water level time series are also corrected for sea level rise, while the sea level barely affect the water levels at this location.

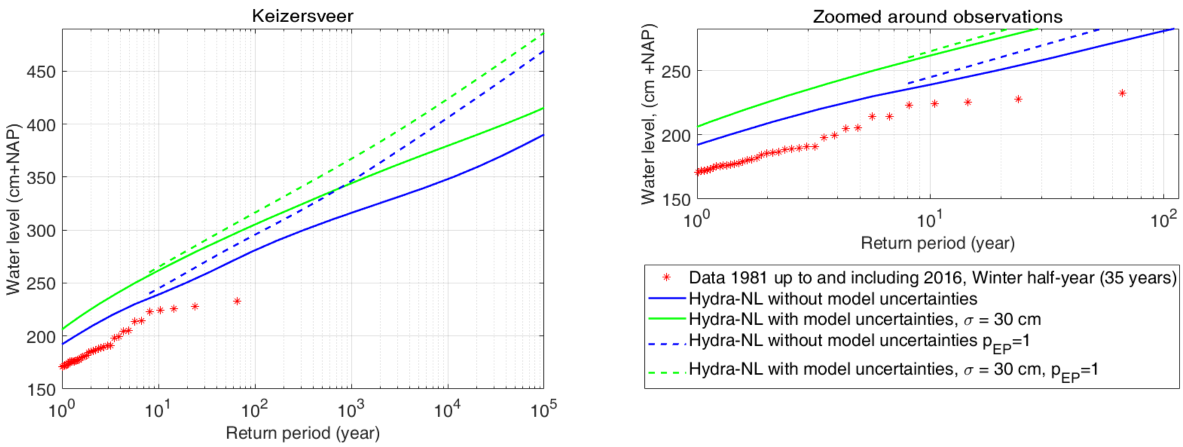


Figure 2.24: Water level frequency line at Keizersveer (Transition area: Storage - Discharge)

2.4 Conclusions

The first research question concerns the performance of Hydra-NL and is recalled here:

> *Do water level frequency lines for different fresh water systems calculated by Hydra-NL correspond to observations and are the hydraulic processes that play an important role in each water system represented correctly?*

Considering the comparison between the statistical characteristics of the measurements and the outcomes of Hydra-NL without considering model uncertainty, it can be said that exceedance probability of actual water levels in the tidal river area and lake area seems to be systematically overestimated by Hydra-NL. Drawing conclusions regarding the performance of Hydra-NL for other water systems (Upper-river area and Vecht- and IJsseldelta) is hard, because of inhomogeneous water level time series. Furthermore, it is difficult to point out the source of the overestimations, because these systems consist of multiple threats and an incorrect model schematisation could occur for different modelled processes. In general, deviations between model outcomes and observations can be caused by systematic deficiencies in physical or statistical models, consistently conservatism by estimating representative values for deterministic variables (double counting) or incorrect statistical properties of stochastic variables.

In each water system the water level frequency line is shaped by different hydraulic processes. Threats are translated by means of hydrodynamic models and water levels are derived. The decimate height is determined by the statistics of the threats as well as the environment and gives an illustration of the hydraulic processes. For the lake area the decimate height of wind-dominated locations appears to differ due to varying distances to the centre of gravity and in the upper river area the topography of the area contributes to the shape the water level frequency lines. The model uncertainty is chosen to be uniform for sub-areas where the same hydraulic processes predominantly determine the water levels. However, the net effect of the model uncertainty increases if the decimate heights become smaller and therefore the net effect of model uncertainty differs within these sub-areas. It is unknown if the model uncertainty is represented correctly in this way.

Also the functioning of the storm surge barriers, flood channels and other flood mitigation processes affect water level frequency lines. All these processes are clearly visible for calculations that are made without model uncertainty. However, the inclusion of model uncertainty distorts the water level frequency lines at several locations and seems to conflict with the physics. The net effect of model uncertainty around water levels that are reduced by flood mitigation projects, like a flood channel and a storm surge barrier, is larger compared to the situation in which these projects would not be carried out. Intuitively, the uncertainty in the water level decreases around return periods for which these measures affect the water levels, because these adjustments reduce the water level response to the threats in a system. For example, the operation of the moveable gates of flood channels controls the water levels and results in an enlarged conveyance seems is less sensitive to uncertainties than an average river profile. Reservoirs are more predictable than a flowing river (like the functioning of a storm surge barrier). The inclusion of water level uncertainty is chosen to be independent of the return period and results in physically unfounded water levels for situations where flood mitigation projects .

To summarise: Assuming that the observations obtained from the Waterbase are reliable, Hydra-NL overestimates the water levels in the measuring range for locations in the lake area and the tidal river area. This is based on Hydra-NL outcomes where model uncertainty is not taken into account, which in turn results in even a larger overestimation. The inclusion of model uncertainty seems to conflict with the physics because of two reasons. Firstly, the net effect of model uncertainty differs in each sub-area, because of the varying decimate heights. The effect becomes larger for smaller decimate heights, while a smaller decimate height means that water levels in a system are less sensitive to threats. Secondly, the net effect around water levels that are reduced by flood mitigations projects is larger, while the operation of the flood mitigations projects controls the water level locally. A uniform standard deviation of the model uncertainty, that is independent of the return period, seems not adequately reflect the physics in water systems. This hypothesis is further investigated for the upper river area and the lake area in respectively Chapter 3 and Chapter 4.

3. Uncertainties in hydrodynamic modelling of rivers

3.1 Introduction

This chapter elaborates the effect of uncertain model parameters on frequency lines of water levels for different hydrodynamic systems in the upper-river area. The peak discharge at Lobith is apart from the model uncertainty the only stochastic variable that is considered for the derivation of water level frequency lines in WBI2017. The water levels are the result of WAQUA runs of which the peak discharge is varied in such a way that the stochastic variable is accurately represented. In this study, no WAQUA runs are carried out, but analytical equations are used that describe the physical processes of a river and are able to translate discharge to corresponding water levels. The applications are simplified hypothetical case studies of which the results can be used as a guidance to set-up WAQUA models that do describe the actual situations in the Netherlands. The developed model is not calibrated on measurements or an existing calibrated model, but the impact of model uncertainties for different hydrodynamic processes can be assessed.

The basis of this model is the Manning formula that is able to describe the relationship between discharges and water levels under normal flow conditions, based on geometric parameters of a river channel: hydraulic roughness, cross-sectional profile and channel slope. Flow velocity and depth immediately after an intervention, like a flood channel, can induce backwater effects and flow velocity become non-uniform. The resulting water levels can be approximated by the Bresse equation, which are also considered in the developed model. A rating curve, which describes the relationship between discharge and water level, has been drawn for rivers with varying hydrodynamic systems: different cross-sectional profiles, retention area along the river and the impact of a flood channel. These different hydrodynamic systems are chosen, because the relative importance of any source of uncertainty depends largely on the hydraulic conditions in the river reach (Pappenberger et al., 2006). Therefore it is relevant to analyse the response of water level frequency lines on model uncertainties for different hydrodynamic systems. In addition, these systems occur in the Netherlands and it is demonstrated that the current uncertainty modelling technique in WBI2017 might result in physically incorrect water level frequency lines (see Chapter 2).

First, the causes of uncertain water levels are elaborated in this chapter. Previous studies already focussed on the sources of uncertainties in hydrodynamic models and the effect on water levels during flood conditions (only one discharge level). However the effect of model uncertainties on the full range of flow conditions or exceedance frequencies has not been considered, at least to our knowledge. Secondly, the methodology is discussed, where the set-up of the developed model will be explained, which consist of a physical and probabilistic part. A selection between important and less important uncertainty sources is obtained with a sensitivity analysis, where model input variables are one by one varied to estimate their impact on the water level. The hydrodynamics of the two river interventions (retention area and flood channel) are described and two different methods to include model uncertainty are discussed. In the third section, the resulting water level frequency lines for different hydrodynamic systems are evaluated. After that, the developed model will be discussed and justified and the research is placed in the context of WBI2017. In the last section the conclusions are elaborated and the second research question is answered:

> *What is the effect of dominant uncertain model/input parameters on the water level frequency lines for different hydrodynamic systems for the upper rivers, where the local water levels primary result from high discharges, and are these results consistent with the current WBI2017 method that includes model uncertainty?*

3.1.1 Causes of water level uncertainties

River systems are of a dynamic and stochastic nature and the underlying processes are not completely understood. An imperfect description of physical processes, along with the inability to accurately quantify the model inputs and parameters, leads to uncertain water level predictions. Identifying the uncertainty sources and assessing their contribution to the overall uncertainty in water level predictions is necessary in order to come to grips with system behaviour and a reliable safety assessment of primary flood defences along rivers. Uncertainties introduced by the model structure, numerical solution technique and the specification of future scenarios are not considered in the analysis of this chapter. The focus is rather on uncertainty associated with quantifying model parameters and input parameters.

Previous research aimed to identify and quantify the sources of uncertainty that contribute most to the uncertainties in the modelled water levels by means of expert judgement (Warmink et al., 2011; Klis, 2003; Kok et al., 2003; Tijssen et al., 2014). These studies were used to produce the list of model inputs and parameters below that will be assessed later on in the sensitivity analysis to determine the most sensitive ones (see Section 3.2.2). At the end of every item, the source of uncertainty is quantified by means of a standard deviation.

1. **River Discharge**
   Uncertainty in river discharge is inherent to nature and future discharge levels can not be predicted exactly, unless large interventions are undertaken (e.g. a dam in the river). River discharge is considered as a fundamental stochastic variable in probabilistic models used for flood risk assessments, which means that the uncertainty is explicitly taken into account using exceedance probabilities of the discharge. In addition, the uncertainty in the computed water levels, associated to the discharge, is a combination of inherent and epistemological uncertainties.

   There are several possibilities to derive frequency discharge curves. The traditional way is based on statistical analysis of observed discharge levels. However, a new method has been developed to derive the design discharges using the instrument GRADE (Generator of Rainfall And Discharge Extremes). This method is meant to provide a more physically based method for the estimation of the design discharge. This instrument consists of stochastic simulation of the weather and hydrological (rainfall-runoff model) and hydrodynamic (hydraulic SOBEK model) modelling. Prinsen et al. (2015) and Hegnauer et al. (2014) constructed uncertainty bands for the discharge statistics of the Rhine based on the uncertainty in the climate, uncertainty in the hydrology and uncertainty in the hydraulic modelling (the three components of GRADE). In table 3.1, the uncertainty of discharge peak levels are quantified by means of a standard deviation for different levels. The same is done for the river Meuse.

Table 3.1: For different discharge levels the uncertainty is quantified for the Rhine at Lobith based on an uncertainty analysis in GRADE (source: Prinsen et al. (2015))

Peak Discharge (Q_p) [m^3/s]	Standard deviation (σ) [m^3/s]
5 940	340
7 970	440
9 130	500
10 910	600
12 770	700
14 000	560
14 840	620
14 970	640
15 520	750
16 270	930
16 960	1120
17 710	1350

These uncertainties are considered as statistical uncertainties and are incorporated in the discharge frequency curve. It can be assumed that uncertainties associated to the peak discharge that is entering the Netherlands near Lobith is taken into consideration. However, the discharge distributions at the bifurcations Pannerdensche Kop and IJsselkop (rhine branches) are not incorporated and result in additional discharge uncertainties downstream.

Discharge distribution

The discharge distribution at the bifurcations in the Netherlands depends on the water level at the bifurcation and in the branches. Ogink (2006) concluded that the most important sources for the uncertainty in the discharge distribution are the morphodynamics and the hydraulic roughness of the main channel and floodplains. The fact that uncertain discharge distribution is an implicit source of uncertainty and dependency of parameters related to different Rhine branches makes it rather difficult to deal with. The uncertainty of the discharge distribution at Pannerdensche Kop is estimated at a standard deviation 130 - 180 m^3/s and 85 - 100 m^3/s for the IJsselkop according to Ogink (2006). This quantification is based on the situation at that time, while nowadays regulating structures are constructed at the Pannerdnesche Kop and IJsselkop that aim to manipulate the discharge distribution at the bifurcations. Therefore the uncertainties related to the discharge distribution could differ but in the absence of scientific evidence, the quantification of Ogink (2006) is used in the sensitivity analysis in Section 3.2.2

Table 3.2: Standard deviations of the discharge, due to uncertainties at the bifurcations in the Netherlands.

	Standard deviation of the discharge [m^3/s]	
	Lower limit	Upper limit
Pannerdensche Kop	130	180
IJsselkop	85	100

2. **Hydraulic roughness**

Physical meaning

Hydraulic roughness has many sources, such as vegetation resistance, resistance due to channel shape and bends, resistance due to velocity differences in the flow, grain resistance and resistance due to subaqueous bedforms (Knighton, 1998). The number of processes that are involved in energy dissipation makes it a challenging task to parameterise the roughness in a river.

In many lowland rivers, like the Netherlands, the resistance in the main channel is dominated by bedforms that develop on the river bed and increase in height with increasing discharge (Julien et al., 2002; van Rijn, 1993). The relationship between roughness and the development of bedforms is not yet fully understood. It is often represented by an empirical relation that has been derived from flume studies and then applied to a 'real' river case. These empirical relations or parameters that are used to represent the hydraulic roughness of the main channel are largely uncertain. Warmink et al. (2013) selected five roughness parameterisations that predict the bed form roughness of the main channel and calibrated them on bed forms and flow measurements during three large discharge waves in the river Rhine (still significantly lower than design discharge levels). The calibrated models were applied to situations beyond the calibration events to a discharge level of 16000 m^3/s (corresponding return period of 1250 years according to the HR2006 statistics). Warmink et al. (2013) showed that different available roughness parameterisations diverge significantly if applied to extreme discharge conditions.

During floods, the floodplains are completely inundated and floodplain roughness influences the total bed roughness as well. The floodplain roughness is in many lowlands river dominated by the presence of vegetation. These roughness values are often parameterised based on a land cover map, which links the type of vegetation to average vegetation structural parameters, such as vegetation height and density, and a drag coefficient. However, the different formulas that describe the roughness of submerged and non-submerged vegetation show significant deviations when these calibrated models are extrapolated to extreme discharge conditions (Augustijn et al., 2008). Another challenge is the naturally variation of the vegetation during the year, where the composition and the density of the vegetation effect reliable roughness predictions. In cases where the vegetation has no leaves (e.g. in winter), the hydraulic roughness will be less compared to a situation where there are leaves.

Calibration parameter

In hydrodynamic models, like WAQUA and SOBEK, the roughness of the main channel is often considered as the calibration coefficient. The hydraulic roughness of other parts of the river (e.g. floodplains, side channel and flood channel) are derived from a RWS report (Velzen van et al. (2003)) that indicates estimations of the hydraulic roughness based on the land use. Wrong estimates of the hydraulic roughness in those parts of the river result in a physically incorrect hydraulic roughness of the main channel, because of the calibration process. Moreover, physically omitted processes and inaccurate model schematisations are also compensated by the hydraulic roughness of the main channel. The number of complex processes that are involved in flow energy dissipation (water flow over weirs, embankments, groynes or slopes in the landscape) and eddy viscosities in real river are all compensated, but also vary for different stages.

To demonstrate these uncertainties several combinations of main channel and floodplain roughness values and resulting water levels are considered. A model needs to be calibrated to reproduce a certain water level for a given discharge. Several combinations of main channel and floodplain roughness produce the same water level (see Figure 3.1). For example, a model is calibrated with a discharge level of 12 600 m^3/s and a water level of 22 metre + NAP. If the floodplain roughness is estimated too high, the water level for higher discharges will be overestimated and the same applies vice versa. So, compensations of the main channel roughness for wrong estimated model parameters only apply to the calibration range and extrapolation of the calibrated roughness parameter to circumstances that have not occurred, introduces uncertainty in the model results. The size of this uncertainty source depends on the geometric parameters of the river, where in general large floodplains result in steeper contour lines.

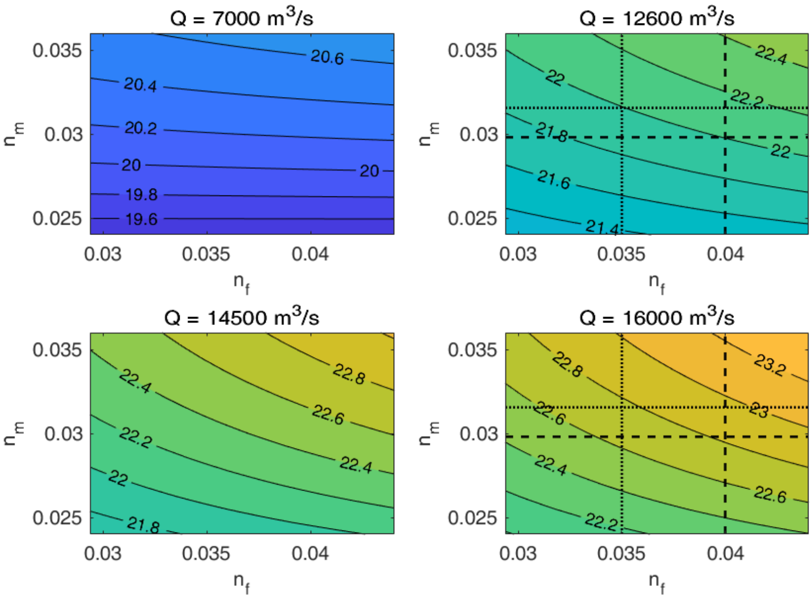


Figure 3.1: Water levels (m+NAP) as a function of Manning coefficients ($s/m^{1/3}$) of the main channel and floodplain for different discharges. The dashed line indicates the calibrated roughness parameter for a water level of 22 m + NAP and the dotted line shows the actual roughness parameters

To conclude, a distinction can be drawn between inherent uncertainties and model uncertainties associated to the hydraulic roughness. First, it is uncertain how the hydraulic roughness will be behave physically under design conditions. Warmink et al. (2013) quantified the uncertainty in the bedform roughness in the main channel due to the choice of the roughness model and the uncertainty due to extrapolation to design conditions. He showed that the 95 % confidence interval of the Nikuradse roughness height for the main channel (the combined effect of both uncertainty sources) of the river Rhine for a discharge level of 16 000 m^3/s ranges from 0.32 m to 1.03 m. This corresponds to a range of +/- 10 % for Manning's roughness coefficient. Secondly, Warmink et al. (2007) quantified the variation in the roughness parameter due to differences in the calibration discharge predictions and he showed that the variability of the roughness is on average (average of three river sections) +/- 10% (based on a case study in the river Rhine). The estimation of the variability of the different aspects associated to the hydraulic roughness coefficient is rather difficult. This is not the focus of this research, but a coefficient of variation of the hydraulic roughness coefficient is estimated by 10%.

3. **Geometric information**
The height of the bed level of the main channel and floodplains, that are model parameters in hydrodynamic models, are based on different data sources. These data sources are measured with different techniques and their accuracy may vary. A distinction can be made between the sources of uncertainty associated to these measurements; 1) systematic error and 2) local variation. A systematic error is related to an error having a non-zero mean, so that its effect is not reduced when observations are averaged. The local variation means that the measurements could deviate from the actual value. An example of a systematic error is that the actual average bed elevation of the IJssel is 10 cm higher than the measured over its entire length. A local variation could be that one bottom grid cell in the model schematisation deviates 15 cm from the actual level. Because our model is based on the normal flow equations, variations are considered to occur in the entire longitudinal direction and will be considered as systematic errors.

The geometric information of the area above the water level is based on the Digitaal Topografisch Bestand (DTB) that consists the height information of all objects and the ground surface. The DTB makes use of aerial- and ground measurements, depending on the application. In general, this means that the height of the floodplains is based on this data source and the topography of the main channel (part underneath the water level) should be measured by means of another technique; single- and multi beam echo sounders. These techniques measure the depth of the riverbed below the water surface and the accuracy depends on several aspects, like temperature, the density of the water and the soil structure at the bottom.

Except that the measuring techniques could contain errors, the riverbed is exposed to physical processes that cause a dynamic bed elevation. The bed elevation in the upstream Dutch Rhine delta decreases by about 1 to 2 cm per year due to a difference between the amount of sediment transport downstream and the amount supplied from upstream, which are caused by river training works (Blom, 2016). Morphological processes are not considered in the hydrodynamic models and only the measured state of the riverbed is used. Depending on the times measurements of the bottom level are done, incorrect geometric information could influence the accuracy of predicted water levels in the river.

Geometrical errors of the river cause wrong model schematisations and could result in inaccurate water level predictions. The morphological processes that influence the bed elevation are not considered in this research. Table 3.3 shows the quantification of geometric errors of the different data sources, which are based on *Productspecificaties Digitaal Topografisch Bestand* (2017) and Wiegmann et al. (2005).

Table 3.3: The standard deviations of the measurement errors. A distinction is made between the floodplains and topography of the main channel, because the elevation of these parts are measured by different techniques.

	xy-plane σ [cm]	z-axis σ [cm]
Floodplains	13	9
Topography main channel	12.5	7.5

The model uncertainty of the geometry also depends on the interpolation technique that is used to schematise the bed elevations in the hydrodynamic model. The measurements are interpolated to grid cells and the accuracy between different measurement techniques could be equal after processing it. All these aspects makes the quantification of this uncertainty source rather difficult. It is assumed that the standard deviation of both river compartments (main channel and floodplains) is 10 cm in the vertical axis and 15 cm in the horizontal plane.

4. **Other parameter setting**

Uncertainty due to other parameter settings of the 2D WAQUA model, such as eddy viscosity and energy loss parameters due to weirs and groynes, fall within this uncertainty source. However, when doing model calibration, the contribution of model parameter settings and model schematisation are largely reduced. Accordingly, the uncertainty due to model schematisation and these parameter settings can be considered to be small compared to the uncertainty due to calibration procedures, morphological changes, and differences between calculated and actual discharge distribution Tijssen et al. (2014).

This study is not focussed on the exact quantification of uncertainty, but on how dominant sources of uncertainty effect the water level frequency curves in different hydrodynamics systems, such as geometric varying situations or river interventions. However, a first indication of the magnitude is relevant to assess the importance or sensitivity of these sources with respect to the water level. Spatial variations and dependencies could also play an important role in these uncertainty source, but in this study the model uncertainty is assumed to be systematic. This means that the model uncertainties are spatially fully correlated.

According to Tijssen et al. (2014) and Chbab (2015) the most important sources of uncertainty are the calibration approach, hydraulic roughness and physical processes or morphological changes under extreme conditions for the Dutch upper rivers. The study Tijssen et al. (2014) estimated the overall model uncertainty for the upper river area of the Netherlands of a bias of 0 metre and a standard deviation of 0.8 metre. This quantification was based on expert judgement from Deltares and are the best estimate that the experts could give at that moment. Later, this standard deviation is adapted because it seems to be rather large according to the experts afterwards. It was decided to lower the standard deviation of the upper-river area to 0.3 metre.

Later on, this standard deviation was lowered again to 0.15 metre. The roughness coefficient plays an important role in hydrodynamic models that are used for 1) the rivers in the Netherlands and 2) the determination of the discharge entering the Netherlands using GRADE (Generator of Rainfall and Discharge Extremes). It is likely that the roughness coefficients that are used in those models are correlated to each other, because the models are both related to the Rhine or Meuse but only used for different countries (Chbab & Groeneweg, 2017). And therefore, there is a correlation between the model uncertainty in the WAQUA models for the Dutch rivers and the statistical uncertainties that are accounted for in GRADE. In short, a rougher main channel in the hydrodynamic models of GRADE results in higher water levels in the river before it enters the Netherlands. Therefore, more flooding will occur that result in lower peak discharges. At the other hand a rougher main channel in a hydrodynamic model of the Dutch rivers result in higher water levels. A positive correlation between the roughness of the main channel in both models result in a negative correlation between uncertainties and associated water levels in the Netherlands. Therefore, the standard deviation was lowered again.

According to Chbab (2015) the uncertainty in discharge distributions at the bifurcations are relatively small compared to above mentioned uncertainty sources (calibration approach, roughness and morphological changes). Uncertainties in discharge distribution only contributes in the IJssel and therefore the uncertainty in this river is larger than the rest. The estimation of the model uncertainty, that is implemented in the hydraulic load models, for the upper rivers can be found in Table 2.4 of the previous chapter.

3.2 Methods

The model considered in this study concerns a highly schematised situation in which the river is assumed to be a symmetric channel with an initially plane sloping bed. The hypothetical model is appropriate to make a first investigation of the water level response induced by uncertainties in model and input parameters. It provides rapid insight into the physical system behaviour and the uncertainties involved. The major drawback of such a model is that because of its simplification, it is of little use to real-life rivers. The reason for still using the hypothetical model, is that the potential of a probabilistic approach can best be investigated by first examining simple cases in which the hydrodynamic processes are fully transparent. More real-life complexity incorporates aspects like hydraulic structures, variations in geometry and flow resistance, multiple branches and bifurcation points. This results in complex propagation of model uncertainties through the system, but these are rather difficult to describe by non-numerical modelling techniques.

By schematising hydrodynamic behaviour, the relationship between discharge and water level may be derived using the Manning formula and equations that describe the backwater effects near interventions. Rating curves can be determined for different sets of parameters and simulate all states of the river system (situations varying from discharges during regular flow conditions to discharges under extreme conditions). The established database of rating curves is combined with the exceedance probability of discharge to obtain frequency lines of the water level for different input parameters. In the simplest case (without model uncertainty), only the discharge is considered as stochastic variable and the discharge frequency line is translated to a water level frequency line by means of the derived rating curve. However, if other input, model or outcome parameters are considered to be stochastic, the calculations become more complex. By utilising probabilistic calculations in the computational core, the following methods to deal with model uncertainty are applied:

- The resulting water levels are considered as stochastic variables (WBI2017 method)
- The input or model parameters are modelled as stochastic variables (Physics-based method)

An overview of the complete model can be seen in Figure 3.2. The stage of the process at which these modelling techniques are implemented differ from each other. In the calculations according to WBI2017 the discharge is the only basic stochastic variable in the upper river area and the model uncertainty is applied on the resulting water levels. The water levels for all discharge levels at every location along the river are considered to have the same amount of uncertainty. The Physics-based method applies the uncertainty one step earlier in the process. Model runs are made in which additional model parameters are considered to be stochastic. The whole process to obtain water level frequency lines is discussed in this section.

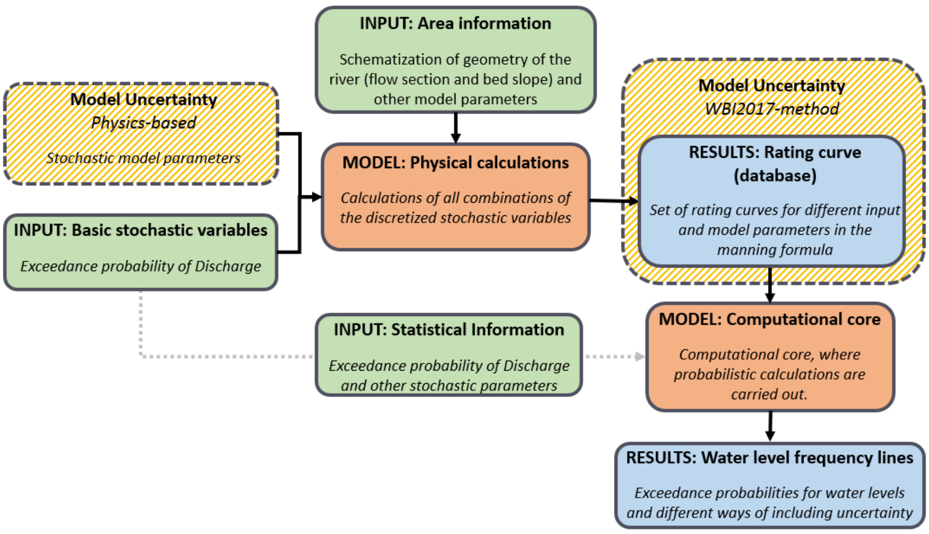


Figure 3.2: Overview of different elements in the process to derive water level frequency lines.

3.2.1 Model set-up

Physical calculations

The water levels in rivers 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. This all together is captured by the Manning formula that describes the discharge of steady uniform flow in a single channel (Manning, 1891). The basis of this formula is the force balance between gravity and bed friction. The Manning formula is defined as follows:

$$Q = \sqrt{S_w}\frac{1}{n}AR^{2/3} \tag{3.1}$$

where n is Manning's coefficient representing the hydraulic roughness [s/m$^{1/3}$], S_w is the water surface slope [-] (or the gradient of the riverbed (S_b) assuming uniform flow), A the cross sectional area [m^2] and R is the hydraulic radius [m]. The hydraulic radius is defined as $R = A/P$, where P is the wetted perimeter [m]. The last part in the Manning formula, $AR^{2/3}/n$, is also called the conveyance part and can be seen as a measure of the discharge capacity of a section, because of their linear relationship. The terminology used for $AR^{2/3}$ is the flow section. A clear distinction is made between conveyance and flow section, because of uncertainty modelling of parameters.

In general, lowland rivers do not have a single channel but consist of a compound channel. During flood events, the main channel does not have enough flow capacity and the floodplains are submerged. In the floodplains the water depth is lower and, in many cases, the hydraulic roughness is higher. Also flood channels could be part of the conveyance of a river and the total discharge Q_T in a river equals the sum of the discharges in the different compartments. The Divided Channel Method (DCM) propose a division through vertical lines, where the total discharge is given by the sum of the sections:

$$Q_T = \sum_{n=1}^{i}\left(\frac{A_i R_i^{2/3}}{n_i}\right)\sqrt{S_b} \tag{3.2}$$

where the index i indicates each subsection. This compartment-averaged approach has the advantage of requiring little input and being straightforward to calculate, while recognizing the different properties of the compartments. However, velocity gradients between the flows in different compartments generate mixing patterns and secondary currents and induce lateral momentum transfer (Fernandes et al., 2012), which is ignored in this method. DCM produces reasonable overall discharge predictions but systematically overestimates the main channel flow and underestimates the flow in the floodplains (Weber & Menéndez, 2004). The flow field becomes even more complex, as the intermediate region influenced by groynes participate during high discharges. These processes are included in 2D WAQUA calculations, but are not covered in the above equations.

Area information

The geometry of the Dutch rivers varies as well as the branches of the rivers. For example the width-ratio between the main channel and the floodplain and the bed level of the floodplains compared to that of the main channel differ for each Rhine branch. The presence (or absence) of summer dikes and the storage capacity in the floodplains also differ for each branch, but are eliminated in the model in this study. The developed model considers characteristic geometric profiles that are representative for the hydrodynamic processes and differences of the Rhine branches in the Netherlands. The river characteristics used in the model runs can be found in Table 3.4.

Table 3.4: River characteristics used in the model runs. The row of the main channel width consist of two values of which the first indicates the bottom width and the second the width at bank full depth.

			Bovenrijn	Waal	Nederrijn-Lek	IJssel
Percentage of the discharge at Lobtih	K	%	100	63.53	21.10	15.37
Bed slope	S_b	m/km	0.13	0.12	0.13	0.1
Width main channel	W_m	m	330/440	260/370	130/200	90/120
Width floodplain	W_f	m	850	550	400	550
Bankfull depth	h_m	m	9.5	8	6	7.5
Manning coefficient main channel	n_m	s/m$^{1/3}$	0.03	0.03	0.03	0.03
Manning coefficient floodplain	n_f	s/m$^{1/3}$	0.0367	0.0367	0.0367	0.0367

A visualisation of the terminology can be seen in figure 3.3. The cross-sections represent a symmetric two-stage channel with two identical floodplains, where the orange dashed lines show the interface between the main channel and the floodplains. A fixed discharge distribution at the bifurcations is assumed, which is in reality uncertain and depends on many factors (Ogink, 2006).

The simplified profile makes it difficult to parameterise the profile in order to describe the real rivers in the Netherlands, because the summer dykes separate the main channel and floodplains and ponds are located in the floodplains. They increase the bankfull discharge and affect the inundation process of the floodplains. The bankfull depth in table 3.4 is based on the AHN database (to estimate the ground level of the floodplains) and a database that contains the bed level of the main channel (from Emiel Kater of Rijkswaterstaat).

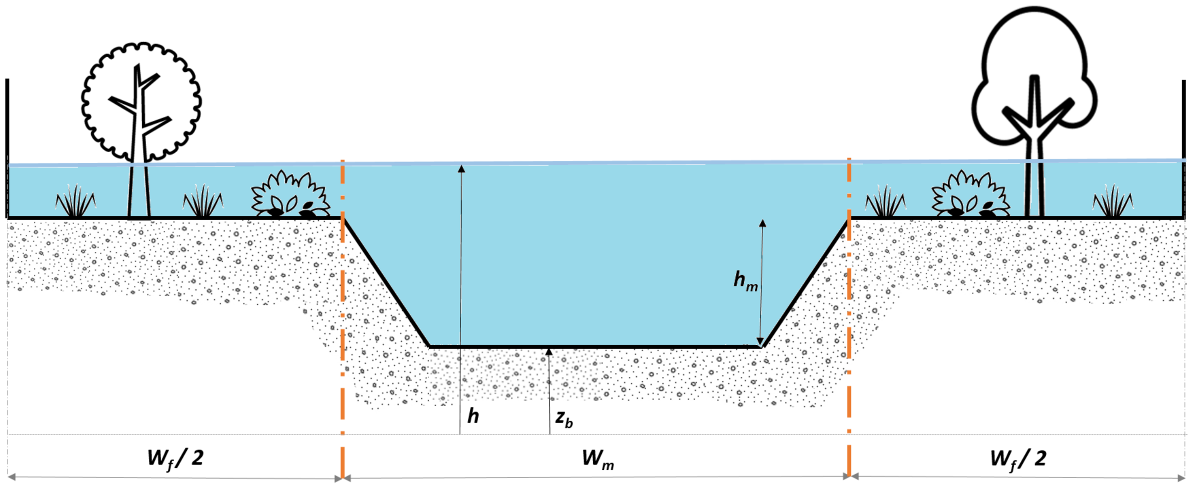


Figure 3.3: Terminology of cross-sectional profile of a river

Modelling of non-uniform flow conditions

The Manning's formula assumes steady and uniform flow. The river flow is said to be steady if at any point in the river the important characteristics such as velocity and pressure do not change in time. It is reasonable to assume that the flow in rivers is quasi-steady, which means that at any instant the flow through any cross-section has fully adapted to the discharge, as if these were not changing in time. The flow is called uniform if the $\delta d/\delta x = 0$ if $d = d_n$, where d is the depth and d_n is the normal depth that corresponds to water level according to the DCM method. Assuming that the river geometry does not change significantly, the DCM method gives a reasonable representation of the water levels in a river.

However, in some cases the river geometry in the longitudinal direction varies significantly, as in the case of side channels or flood channels and thereby non-uniform flow will occur. The conveyance of the river is locally enlarged and the normal flow depths decreases compared to up- and downstream from the flood channel. Water levels respond to these interventions but the normal flow depth will not be obtained. The water level will gradually vary in the longitudinal direction and a so-called backwater curve occurs. The Bélanger equation provides the basis to calculate these backwater curves:

$$\frac{\mathrm{d}d}{\mathrm{d}s} = S_b \frac{d^3 - d_e^3}{d^3 - d_c^3} \tag{3.3}$$

where d_e is the normal flow depth and d_c is the critical flow depth (depth for which the flow at a given discharge is critical). This above equation can be approximated for flows with low Froude numbers ($Fr = u/\sqrt{gh} << 1$), where d is much larger than d_c as follows:

$$\frac{\mathrm{d}d}{\mathrm{d}s} = i_b \Big[1 - \frac{d_e^3}{d^3}\Big] \tag{3.4}$$

Low Froude numbers are most common for lowland rivers, like the Dutch rivers. The parameters water level (h) and bottom level (z_b) are absolute values above a reference level, in this case NAP, while water depth (d) is the absolute difference between the water level and bottom level. The Bélanger equation assumes a straight canal with a shallow, rectangular cross-section. In our situation a compound channel is modelled and the water depth varies over the width. Therefore the discharge-weighted averaged water depth is estimated as follows:

$$d_e = \frac{Q_m(h - z_b) + Q_f(h - z_b - h_m)}{Q_T} \tag{3.5}$$

where Q_m is the discharge in the main channel, Q_f is the discharge in the floodplains. Equation 3.3 makes the calculation process difficult and therefore the Bresse equation is used:

$$h = h_e + (d_0 - d_n)2^{-\dfrac{x - x_0}{L_{1/2}}} \tag{3.6}$$

where d_0 is the water depth at the downstream edge of a river stretch (at x=0) and the so-called 'halflength' $L_{1/2}$ is given by:

$$L_{1/2} = 0.24\frac{d_n}{i_b}(\frac{d_0}{d_e})^{4/3} \tag{3.7}$$

The formulations above calculate the upstream water level of a river stretch that is influenced by backwater curves. The equilibrium water levels will be determined by the Divided Channel Method and used to calculate the actual water levels. This method is only used to simulate the backwater effects for the case in which a river is influenced by a flood channel. This is further discussed in Section 3.2.3.

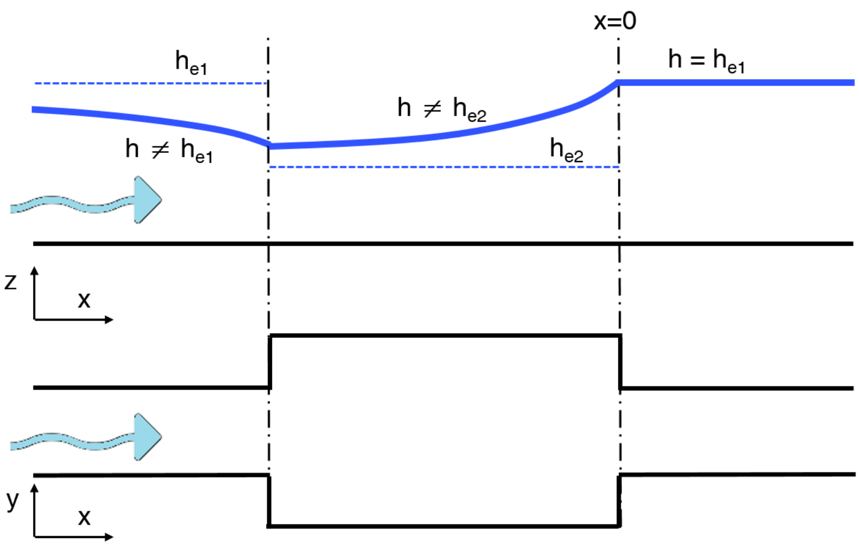


Figure 3.4: Schematic illustration of the effect of backwater curve on water levels at a wide cross section.

Hydrographs
The model should be able to model the time evolution of discharge levels during a flood wave. In cases like retention and a flood channel, the associated rating curves are shaped by a so-called hydrograph (time evolution of the discharge at one specific location). These systems respond differently on natural varying hydrographs and therefore the effect of uncertain model parameters on water levels do as well. The hydrograph depends among others on the rainfall series that vary in time, space and intensity that results in a large set of possible shapes of a hydrograph. The water level response at a specific location could depend on the shape of the hydrograph as in the case of a retention area. A river with a retention area starts to extract water from a certain water level to the retention area until the retention area is full. If a high water wave with a certain peak discharge is wider, the effect of the retention area is smaller, because the retention area could be already full before the peak discharge occurs. The same peak discharge, but a narrower wave could result in lower water levels, because the retention area is able to top off the whole peak. The shape of the hydrograph and the peak discharge determine the volume of water that could be withdrawn from the river (Kok et al., 2003).

However, most of the other hydrodynamic systems are insensitive to the shape of the hydrograph for the failure mechanisms overflow and overtopping (Geerse (2013) and Stijnen et al. (2002)). Furthermore, the shape of hydrographs may be important for the process of deformation (peak height reduction and widening) as it propagates downstream. However, the shape of the flood wave at Lobith does not significantly influence the maximum water levels in the Rhine branches as calculated by WAQUA (Beckers et al., 2009), because of the broad flood wave that results in a small attenuation. Therefore, this natural behaviour is not modelled and a design flood wave with a fixed shape is assumed also because of computational restrictions.

Design flood waves are characterised by peak discharges, peak duration and a rising and falling limb. In this research the hydrograph is schematised by a trapezium that has got an increasing front flank, a top duration of one day, and a decreasing back flank. A base duration B = 30 days (720 hours) is taken for the trapezium and the peak occurs at time t = 15 days (half a day sooner and half a day later, because of the top duration). A time frame is chosen that describes the propagation of a high water wave in the following time domain: $0 \leq t \leq B$. The hydrograph is parametrised in the model as follows (See Figure 3.5):

$$Q(t, q_p) = \begin{cases} q_{min} + \dfrac{(q_p - q_{min})}{t_{rise}} t, & \text{if } t < t_{rise} \\ q_p, & \text{if } t_{rise} \leq t \leq t_{fall} \\ q_p - \dfrac{(q_p - q_{min})}{t_{rise}} (t - t_{fall}), & \text{if } t > t_{fall} \end{cases} \tag{3.8}$$

where t is the time [hours], q_p is the peak discharge[m^3/s], t_{rise} is the time that the hydrograph rises and it reaches the peak discharge [hours], t_{fall} is the time at which the discharge starts to fall [hours] and q_{min} is the minimum discharge that all relevant discharges exceed [m^3/s]. To make simulations in a model, the time frame of the hydrograph should discretised into a number of steps (n) with a step size of Δt: $t_0 = 0$, $t_1 = \Delta t$, $t_2 = 2\Delta t$, ... , $t_n = n\Delta t$. A whole range of peak discharges is modelled in order that the rating curves and resulting water level frequency lines can be constructed with sufficient accuracy. Table 3.5 gives the parameters that are used to discretise the hydrographs and figure 3.5 shows an example of a resulting hydrograph for a peak discharge of 16.000 m^3/s.

Table 3.5: The parameters used in the discretisation of the hydrographs

Description	Symbol	Value	Unit
Minimum discharge	q_{min}	750, 500, 150 and 100 [a]	m^3/s
Hydrograph rise time	t_{rise}	348	hours
Time at which discharge starts to fall	t_{fall}	372	hours
Numerical time step	Δt	0.5	hours
Step size Discharge	ΔQ_p	20	m^3/s

[a] Minimum discharge for resp. Bovenrijn, Waal, Nederrijn-Lek and IJssel (aprox 1/3 of the average discharge of the Rhine branches)

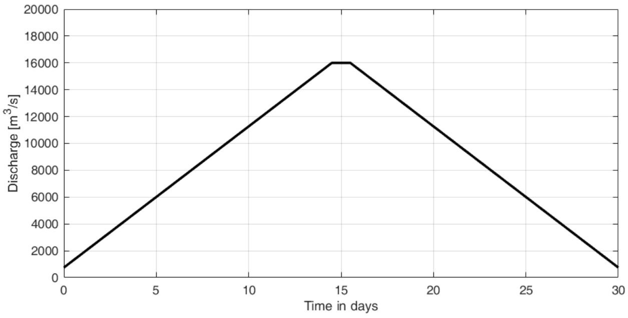


Figure 3.5: Schematisation of a hydrograph by a trapezium with peak discharge of 16.000 m^3/s and a minimum discharge of 750 m^3/s

The modelled peak discharges are linked to an empirical exceedance frequency function $F_Q(q)$: average number of exceedances of the random variable Q of the level q, in times per year. An exceedance frequency of level q is derived from the exceedance probability of the peak discharge q of a high water wave, related to the base duration B = 30 days. This relation is given by (Geerse et al., 2011):

$$F_Q(q) = N_{trap} P(Q > q), \qquad q \geq q_1 \tag{3.9}$$

where N_{trap} is the number of discharge trapezia. In applications there are 6 discharge trapezia, because only the winter half-year (October − March) is considered because extreme discharges at Lobith are often associated with large multi-day precipitation in winter (Hegnauer et al., 2014). Equation 3.9 is valid for discharge levels with return periods of at least one year (denoted by q_1). The hydrographs for lower discharge levels are too irregular and proper waves can no longer be detected in discharge time series. Instead of exceedance frequencies or exceedance probabilities often the return period of discharge level q is noted, which is given by

$$T(q) = \frac{1}{F_Q(q)} \tag{3.10}$$

The discharge statistics for the Rhine at Lobith are used for the model runs, after which the discharge is fixed distributed at the bifurcation points to the other Rhine branches (see table 3.4). The frequency discharge curves can be seen in Figure 3.6. The upper graph shows two different frequency discharge curves: 1) HR2006 that are based on a statistical analysis of observed discharges and 2) WBI2017 that includes a more physically based (and thus more realistic) assessment of extreme discharge statistics based on GRADE (Hegnauer et al., 2014). The WBI2017 frequency curve has got a bend (due to upstream flooding) that gives a distorted water level frequency line. The HR2006 discharge curve is used in all cases, since we want to focus on the geometrical aspects of the river. The use of other discharge statistics and non-fixed discharge distributions will be further elaborated upon in the discussion.

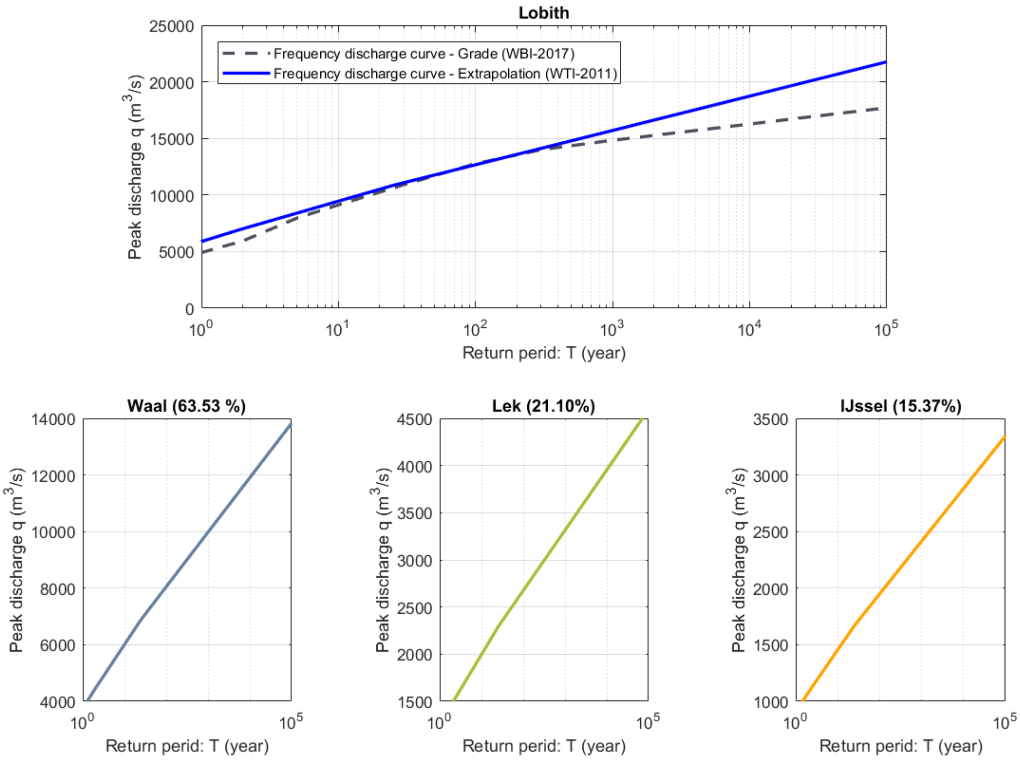


Figure 3.6: Frequency discharge curves for the Rhine branches. The upper graph shows the two different discharge statistics (HR2006 and WBI2017), while the lower graphs only represents the HR2006 statistics

The statistical information can be combined with the rating curves in order to derive water level frequency lines. If the input and model parameters are considered to be deterministic the discharge statistics can be logarithmically interpolated with the calculated rating curves. However, if other model parameters are considered to be uncertain a dataset of rating curves is obtained. Therefore additional calculations need to be made which are explained in detail in Section 3.2.4.

3.2.2 Sensitivity analysis

To fully incorporate uncertainties into a model, every uncertainty should be modelled as a random variable as well as their interdependencies. However, a probabilistic model becomes more computationally intensive with the addition of each random variable. Therefore, it is useful to identify the parameters that are most influential and limit the number of implemented random variables. The identification and quantification of several uncertainty sources is done in section 3.1.1 and this section selects the most sensitive input/model parameters in the developed model with respect to the water levels. The most sensitive parameters are considered as additional stochastic variable in the physics-based method.

The standard deviation reflects the amount of spread or dispersion and one standard deviation above-average and below are modelled to assess the sensitivity of each uncertain parameter. The model inputs and model parameters are perturbed one by one while the remainders are set on the average values (characterised in Table 3.4). A design discharge level of 16 000 m^3/s at Lobith (Bovenrijn) is used in the sensitivity analysis, which corresponds to a return period of 1250 years (according to the HR2006 discharge statistics). The sensitivity of the parameters are analysed for each Rhine branch in the Netherlands, because the sensitivity of each parameter can vary for different river characteristics. Table 3.6 shows the perturbations for which the sensitivity is tested. The following remarks related to the sensitivity analysis:

- The geometrical information consists of an horizontal and vertical uncertainty. Uncertainties in the horizontal plane are neglected, because the influence on the flow section is proportional to the water depth while the influence of vertical changes are proportional to the width of the river. Therefore the water level is much more sensitive to vertical uncertainties rather than horizontal uncertainties.

- The standard deviation of the discharge at the bifurcations are obtained from (Ogink, 2006). The discharge uncertainty associated to the discharge distributions in a river Rhine branch consists of one or two bifurcations. The uncertainty in discharge distribution at the bifurcations is considered to be independent and therefore the Nederrijn-Lek and IJssel discharge uncertainty is calculated as follows: $\sigma = \sqrt{\sigma_{pk} + \sigma_{yk}}$. The discharge in the river Waal only consists of uncertainties related to the Pannerdensche Kop and the Bovenrijn does not have any discharge uncertainty related to the discharge distribution, because the bifurcations are located further downstream.

Table 3.6: The standard deviation of the model inputs and parameters that are analysed in the sensitivity analysis and the associated standard deviation.

Description	symbol	unit	Variation
Discharge distribution	Q_{dis}	m^3/s	$\sigma_{pk} = 155$, $\sigma_{yk} = 92.5$ [a]
Manning coefficient main channel	n_m	$s/m^{1/3}$	$\sigma = 0.003$ (CV = 0.10)
Manning coefficient floodplain	n_f	$s/m^{1/3}$	$\sigma = 0.00367$ (CV = 0.10)
Bed level main channel	z_B	m	$\sigma = 0.1$
Bank height	h_m	m	$\sigma = 0.1$

[a] Standard deviations of the uncertain discharge distribution at the Pannerdensche Kop (pk) and IJsselkop (yk), which are based on the average of the upper and lower limit from Table 3.2.
[b] CV = Coefficient of Variation (μ/σ).

Figure 3.7 shows the sensitivity of the water level response to the above-mentioned input and model parameters. In general, the water level response appears to be most sensitive to the roughness coefficient of the main channel. However, the uncertainties related to the discharge distribution become more important for the IJssel and Nederrijn-Lek, where the uncertainty of both bifurcations affect the discharge. The total width (main channel + floodplains) of the Nederrijn-Lek is larger than the IJssel and therefore the water levels in the IJssel river are more sensitive to the discharge distribution.

The absolute sensitivity of the water levels to the main channel roughness varies strongly in the Rhine branches. One of the reasons that the water levels become less sensitive in the Nederrijn-Lek and IJssel is that the ratio between the width of the river and the discharge does not remain constant. If the cross-sectional profile is relatively wide compared to the discharge, a change in the conveyance (due to roughness uncertainty) results in relatively small water level response. For example, approximately 15% of the discharge at Lobith (Bovenrijn) flows into the river IJssel, while the total width of the IJssel is approx 50 % w.r.t. the Bovenrijn. Therefore, a discharge perturbation at the IJssel has got minor effect on the water level. However, other geometrical aspects also play a role in the sensitivity of the water level such as the .

The relative sensitivity between the roughness of the main channel and floodplains depends on two geometrical parameters (Huthoff, 2004): 1) the ratio between the main channel width and the total width (parameter α) and 2) the ratio between the water depth in the floodplains and the bank height (parameter β). Larger relative floodplain widths (smaller α) result in larger increases in water level rise when floodplain roughening occurs. If the water level during high flow conditions, just exceeds the bank-full height (small β), while the river has got relatively large floodplains, the water level response is more sensitive to the roughness of the main channel. However, a channel with equal geometrical parameters α and β, but different absolute width results in a different water level response.

The sum of the two geometric uncertainties (bed level of the main channel and floodplains) will always be 20 cm, however, the distribution of this variation depends on the ratio between the flow sections of the main channel and floodplains. This can also be seen in the absolute values of the river IJssel, where the flow section of the floodplains are relatively large.

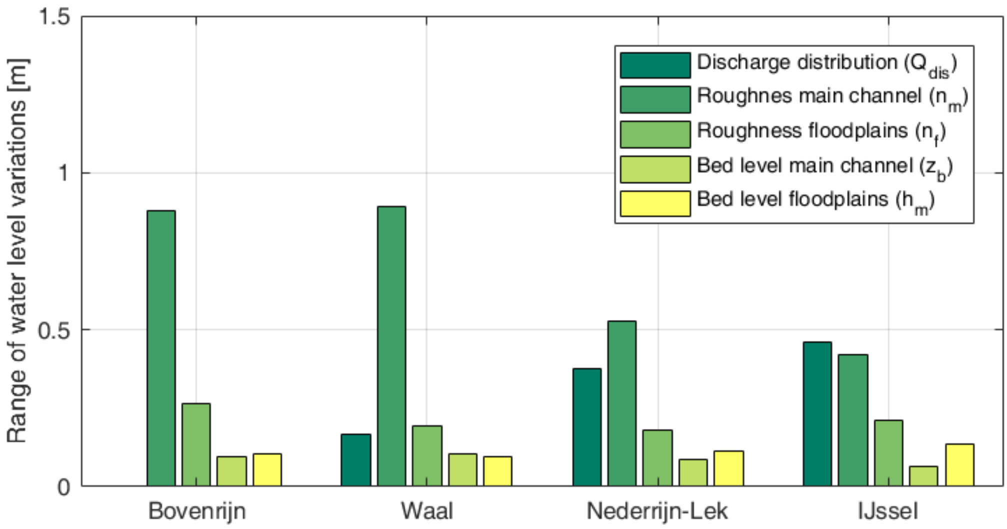


Figure 3.7: Sensitivity of the water level response for the cross-sectional profiles of the Rhine branches.

Table 3.7: The absolute water level response for different model parameters in metres (upper part) and the geometrical parameters (lower part; these are not water level responses, but just ratios without units).

	Bovenrijn	Waal	Nederrijn-Lek	IJssel
River discharge	-	0.16	0.38	0.46
Roughness main channel	0.83	0.79	0.50	0.4
Roughness flood channel	0.25	0.22	0.17	0.2
Bed level main channel	0.10	0.10	0.09	0.06
Bed level floodplains	0.10	0.10	0.11	0.14
α parameter	0.35	0.40	0.25	0.18
β parameter	0.51	0.59	0.54	0.37

Several studies stated that the upstream discharge and the hydraulic roughness of the main channel have the largest influence on the uncertainty in the modelled water levels of lowland rivers (Warmink et al. (2011); Kok et al. (2003)). This corresponds to the outcomes of this sensitivity analysis. Model uncertainties in the upstream discharge entering the Netherlands is already incorporated in the discharge statistics. Therefore, only model uncertainties related to the discharge distribution contribute to the upstream discharge in the Rhine branches. Uncertainties related to the discharge distribution is implicitly dependent on the morphodynamics and hydraulic roughness of the Rhine branches (Ogink, 2006). Therefore, this uncertainty is not further considered in this research, but further research is recommended.

To conclude, the uncertain model parameter used in the physics-based method is the hydraulic roughness and especially the roughness of the main channel. It is shown that the relative importance of the floodplain roughness become larger in cases like the IJssel where the ratio between floodplains and main channel becomes larger. In these situations both roughness coefficients could be considered as stochastic variables.

3.2.3 Physical processes of river interventions

In this subsection two river interventions are described that needs further explanation, because they differ substantially from the normal physical calculations that are described in Section 3.2.1. So far, the average river profiles of the Rhine branches in the Netherlands are considered. However, projects that are meant to mitigate flood risks result in abnormal (bends or changes in inclination) water level frequency lines and differ from average river profiles. As was indicated in the introduction, the effect of river interventions, like a flood channel near Olst, is partly revoked if model uncertainty (according to the WBI2017 method) is added. Next to this system, the modelling of a retention area is further explained in this paragraph.

Duits and Noortwijk (1999) discussed the uncertainty involved in flood level predictions in the Rhine for several river interventions. Monte Carlo Simulations were used to quantify the impact of various uncertain model parameters on simulated flood levels based on 1D and 2D hydrodynamic models. It is concluded that not all uncertainty sources are of equal importance (as showed in the sensitivity analysis in subsection 3.2.2) to the flood level predictions and that the 90 % confidence bounds of flood level predictions are almost independent of measures that increase the conveyance of rivers (Room for the River projects). However, this study only focussed on the design discharge level, while we are interested in the influence on the complete water level frequency line. Uncertainties associated to a retention area, flood channel or side channel were also not elaborated in Duits and Noortwijk (1999).

A. Flood channel

After the 1993 and 1995 (near) flood events, new programmes were introduced which have to lower design water levels by increasing the discharge capacity. In addition to raising the dikes, it seeks solutions by increasing river conveyance by opening up more room for the water to flow through in order to decrease the local water levels. This is done by, amongst others, moving the dikes further inland, lowering floodplains, lowering groins in the river or the construction of flood channels. An overview of the set of measures to be implemented according to the national flood protection programme Room for the River at approximately 30 river locations in the Netherlands can be found in Appendix A.

A flood channel creates extra room to manoeuvre for the river and act as a 'bypass' in extreme high water conditions that causes an increase of the conveyance of the river. The bypass is restricted by dikes or higher grounds and up- and downstream are connected to floodplains of the main river. When the flood channel is fully in use, it may provide several decimetres water level reduction and the effect will also be observed further upstream, due to backwater effects. The presence of a flood channel operates as a water withdrawal followed downstream by a water input into the river. The water withdrawal starts from a certain threshold value that is defined by a water level. From a hydrodynamic point of view, the difference between a side channel and flood channel is actually only the level of use. A side channel functions more frequently than a flood channel and does not contain a gate that separates the flood channel from the main river. The scheme of a flood channel is depicted in figure 3.8.

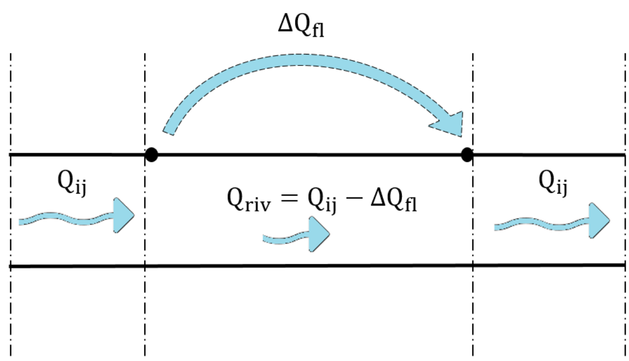


Figure 3.8: Top view of a river with a flood channel

The discharge capacity of the flood channel depends on several aspects, where channel characteristics (geometry, hydraulic roughness and water surface slope) and in- and outlets of the flood channel are the most important parameters (*Voorspellen afvoer nevengeulen*, 2010). The effective water level decrease due to the flood channel strongly depends on the discharge capacity and the length of the channel along which the backwater curves develops. Figure 3.9 shows how backwater curves develop along the river and illustrate the water level reduction of flood channels with different lengths. In these simulations a stationary discharge level is assumed and the flood channel is fully functioning.

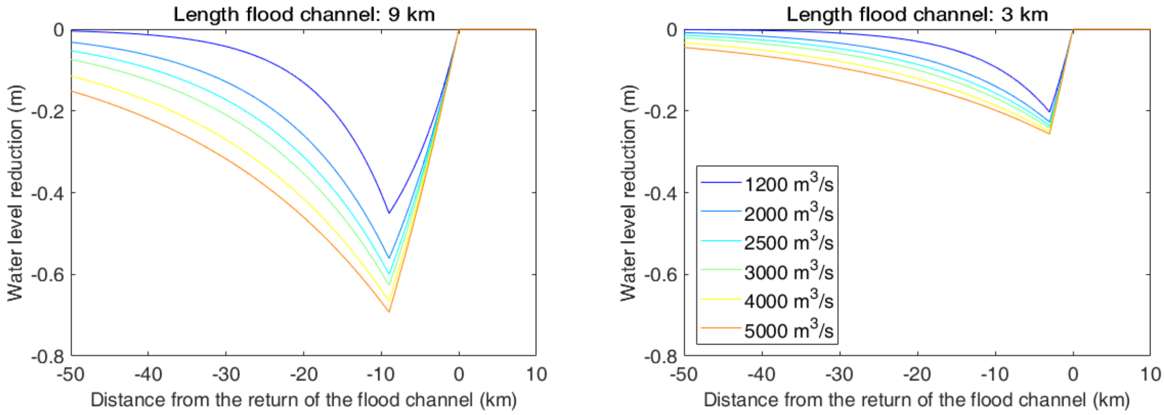


Figure 3.9: Illustration of water level reduction upstream of the flood channel outlet and the effect of the flood channel length. The water level reduction is illustrated for several discharge levels upstream of the flood channel.

To further explain the functioning of a flood channel, the outcomes of WAQUA runs (consistent with WBI2017 calculations) are analysed for the flood channel Veessen-Wapenveld. This flood channel is separated from the main river by moveable gates. The height of these gates, when closed, is 5.65 m+NAP and if the water level exceeds that height, the gates are opened and the flood channel starts to function. One of the advantages of these moveable gates is that the discharge capacity is not limited by the crest height.

First, the discharge distribution of the system is analysed. The model is forced by a flood wave with a peak discharge of 18 000 m^3/s at Lobith. Figure 3.10 shows the hydrographs at different locations in the river IJssel. The grey line shows the hydrograph just after the IJsselkop, which is at the beginning of the IJssel. The peak discharge at the flood channel is higher, due to lateral inflows from tributaries at the IJssel. The hydrograph is getting steeper around 2300 m^3/s (corresponds to approximately 16000 m^3/s at Lobith) which is caused by the strategy "Lek ontzien" that diverts more discharge to the IJssel. After approximately 13 days the water level exceeds the height at which the gates are opened. Discharge is diverted to the flood channel (dashed blue line) and the discharge in the main river between the inlet and outlet is lower (solid blue line). The hydrograph downstream, just after the outlet, drops for about one day and than follows the original hydrograph (case without flood channel). It seems like the flood channel operates as a small retention area.

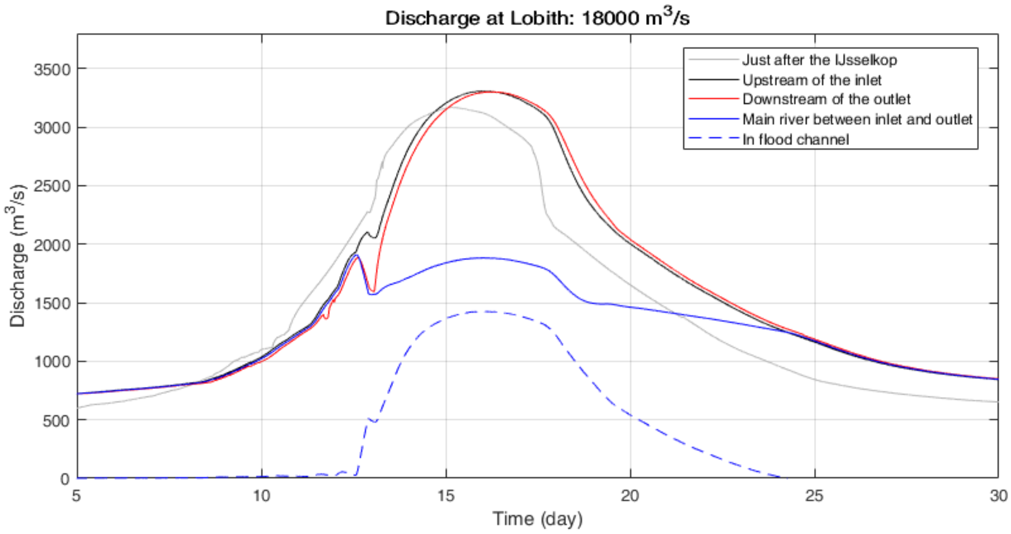


Figure 3.10: Hydrographs in the IJssel river around the flood channel

Secondly, the water level time series is considered. The local water levels are affected by backwater curves that are caused by the downstream boundary condition. After the opening of the gates, the water levels at all locations near the flood channel are temporally lower. There are two reasons: 1) the flood channel behaves like a retention area and 2) the water levels respond to the new equilibrium situation at which the geometry is enlarged. The water levels downstream of the flood channel are temporally lowered, due to discharge reduction, and then follow its "original" line (in case there was no flood channel). At the locations upstream of the outlet, the water level also drops, but then adapts to a new situation where the conveyance of the river is enlarged. In case there were no moveable gates, the adaptation would be different and the in- or outlet could be an obstruction of the flow.

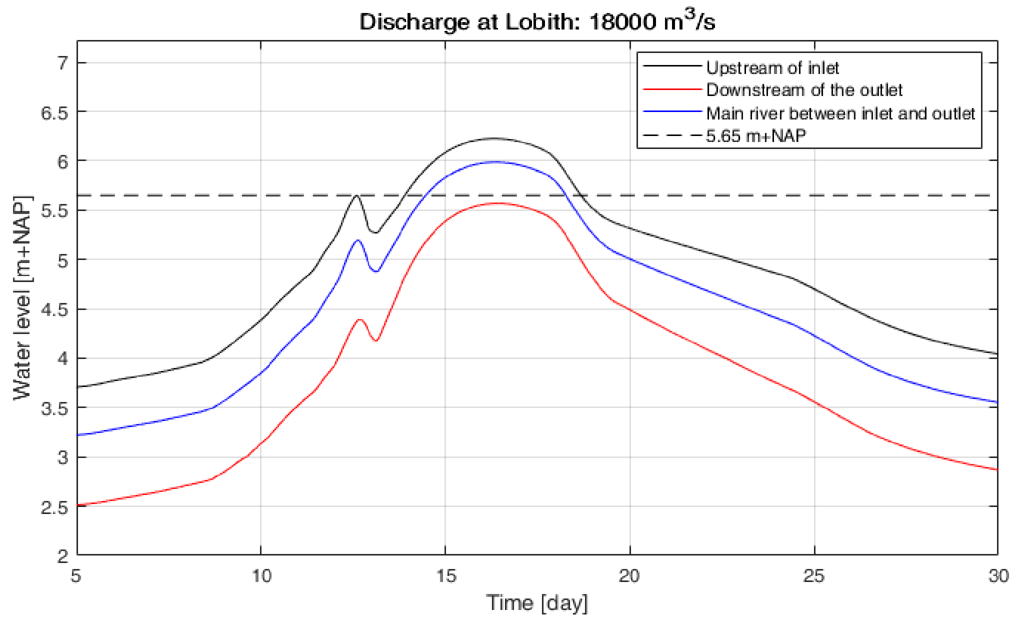


Figure 3.11: Water level time series at locations up- and downstream of the inlet and downstream of the outlet.

A1. Model schematisation
The operation of the flood channel depends on how the water levels evolve in time. If the water level exceeds the height of the gates the flood channel starts to function. Over the length of the flood channel, the water discharge in the main river will be reduced. The amount of discharge that is withdrawn from the main river is not explicitly calculated, but is the result of the conveyance of the flood channel. The flood channel is schematised as an additional compartment over the reach of the flood channel and flow restrictions due to the in- and outlet of the flood channel are not considered. The water depth downstream of the outlet remain unchanged for all discharge levels (the small retention effect is neglected) and follow the equilibrium depth. The flow depths in the main river, between the in- and outlets, are larger than the equilibrium flow depth and a backwater curve will be established upstream of the outlet with depths that can be calculated by the Bresse function (see Equation 3.6 and 3.7). The flood channel is schematised as follows[3]

Schematisation of the Flood channel	
Length	: 10 000 m
Width	: 850 m
Bottom level	: 17.5 m+NAP
The height of the gates	: 19.5 m+NAP
Hydraulic roughness (n_{fl})	: 0.0367

To compute the water levels the following steps are taken:

1. Establish the rating curves (downstream of the outlet and between the in- and outlet) of the equilibrium water depths, where the moveable gates are also taken into account (see figure 3.12). The discontinuity in two right rating curve is caused by the assumption that water levels respond instantaneously to the new geometric profile if the gates are opened. If the water level exceeds 19.5m+NAP the grates are opened, but this also means that the water levels of the rising limb of the hydrograph differ from the and falling limb. To illustrate this effect the water level will follow the grey dash-dot line in figure 3.12. This effect is not very interesting, because it will never affect the maximum water level during a flood wave.

2. Use the Bresse function to calculate the water levels that are affected by backwater curves.

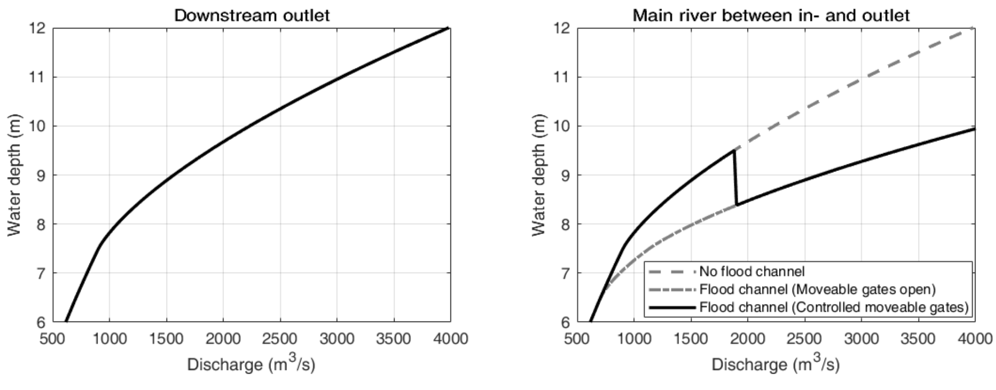


Figure 3.12: Rating curves of the equilibrium water depths (w.r.t. bottom main channel) downstream of the outlet and between the in-and outlet of the flood channel

The resulting hydrograph of a flood wave with a peak discharge of 3500 m^3/s in the IJssel can be seen in figure 3.13. The time evolution of the discharge is roughly equivalent to the situation at Veessen-Wapenveld.

[3]The flood channel near Veessen-Wapenveld is used as reference case for the schematisation of the flood channel in our developed model. However, it differs from the WAQUA calculations that are used to analyse the physical behaviour. On the basis of this study, no firm conclusions can be drawn regarding the flood channel near Veessen-Wapenveld. Therefore additional WAQUA calculations are required.

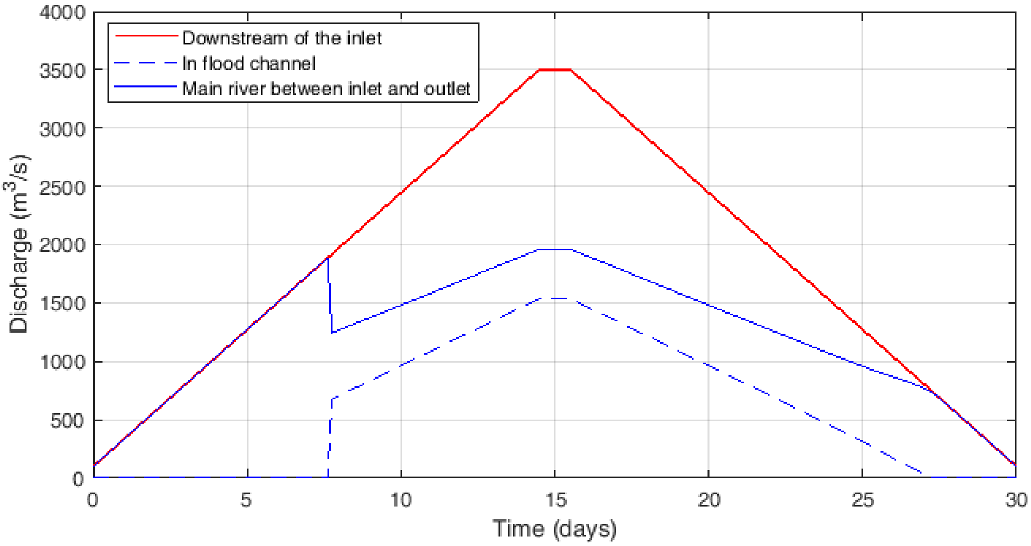


Figure 3.13: Modelled hydrographs where sharp discontinuities resulting from the assumption that the river has got an instantaneous response to an enlarged conveyance.

B. Retention area

The purpose of a retention area is to top off the peak of a flood wave by a discharge-extraction from the river to a retention area. In this case, a passing flood wave is corrected by the participating retention area with a certain storage capacity (S_{cap}), which starts to extract all the discharge at a critical water level until the retention area is completely filled. This critical water level corresponds to a certain water level exceedance probability, which need to be chosen such that the water level frequency curve is lowered at the appropriate flood probability of dike sections downstream (and partly upstream due to backwater effects) of the retention area. In this case, the retention area is designed according to one specific threshold water level, while the corresponding discharge and exceedance probability that the discharge extraction starts are uncertain. This is because the relation between discharge and water level depends on several parameters that are uncertain. Except for uncertainties in the discharge-water level relation, the operation of a retention area is also sensitive to uncertainties associated to the hydrograph shape, exact volume and inlet (type and height of the inlet).

The retention area starts to fill if the water level exceeds a pre-designed threshold value and all discharge above that level is extracted. This value is calculated with a design peak discharge and estimated (average) model parameters. The process of discharge extraction is defined by the fill function (Z_f), where the retention are starts to extract if Z_f is smaller than zero:

$$Z_f(t) = h_{tv} - h_e(Q(t, q_{peak})) \tag{3.11}$$

where h_{tv} is the threshold value at which the retention area starts to fill and h_e is the equilibrium water level in the channel without discharge extraction. Both water levels can be calculated by the DCM method (see equation 3.2). The water levels downstream are decreased until the maximum storage capacity of the retention area is reached. The available storage volume as function of time is given by:

$$S(t + \Delta t) = S(t) - max\Big(0, \Delta t\Big(\frac{Q(t) + Q(t + \Delta t)}{2} - Q(h_{tv}, n)\Big)\Big) \tag{3.12}$$

given the initial value of $S_0 = S_{cap}$. The term $Q(h_{tv}, n)$ represents the discharge below the threshold value which cannot being topped off by the retention area. So the retention area will extract discharge if 1) the water level exceeds a certain threshold value and 2) the available storage volume is larger than zero. The resulting water level downstream of a retention area can be described in mathematical terms as follows:

$$h(t) = \begin{cases} h_{tv}, & \text{if } Z_f(t) < 0 \text{ and } S(t) > 0. \\ h_e(Q(t, q_{peak}), n), & \text{otherwise.} \end{cases} \tag{3.13}$$

Figure 3.14 shows how the wave corrections take place for a location downstream of a retention area with a storage volume of 50 Mm^3. The retention area instantaneously extracts all discharge above the critical value of 20.35 m + NAP, which in this case corresponds to a discharge level of 2500 m^3/s. A flood wave with a peak discharge of 2400 m^3/s is not affected by the retention area, while the wave with a peak discharge of 2700 m^3/s is completely topped off. The storage capacity of the retention area is large enough to extract all discharge that exceeds the critical discharge level. If the peak discharge of the waves get larger, the retention area is completely filled and from that moment the water levels are not corrected by the retention area any more. The retention area is completely filled before the entire flood wave has passed, which is for example the case for waves with a peak discharge of 2780 m^3/s and 3300 m^3/s. For the first mentioned wave the peak of the wave is already passed and the retention area still reduces the maximum water level during a flood wave. For the latter wave, the retention area is completely filled before the peak of the wave occurs and the retention area does not reduce the maximum water level during a flood wave.

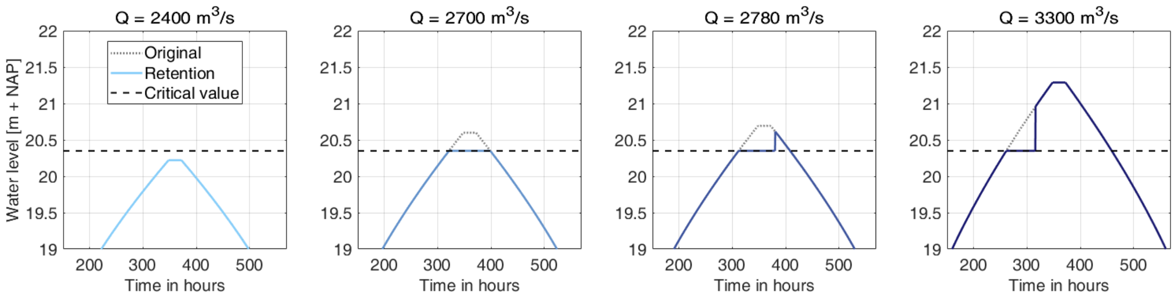


Figure 3.14: Corrections on water level time series to simulate the effect of the retention area. The subplots show resulting water levels of flood waves with increasing peak discharges resp. from upper left to lower right. The grey dashed line indicates the threshold value at which the retention area starts to fill.

A rating curve can be drawn, where the peak discharge levels of the flood wave and the associated maximum water level are plotted against each other. The water levels remain stable between 2500 m^3/s and 2700 m^3/s, after which the frequency line rapidly increases till it attains the water level for which the retention area is completely filled. So, for flood waves with peak values higher than 2850 m^3/s the peaks remain unaffected by the retention area.

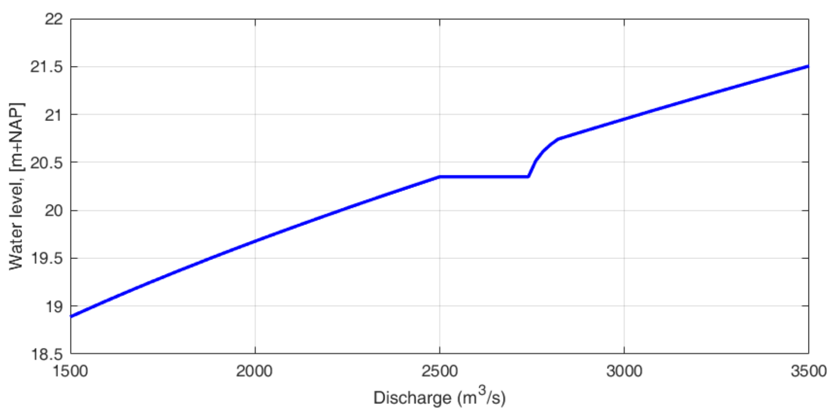


Figure 3.15: The rating curve of a river at a location that is influenced by a retention area of which the uncertain model parameters are considered to be deterministic.

3.2.4 Uncertainty modelling techniques

A deterministic approach treats the input and model parameters as constants during simulations, which does not capture the variations or uncertainty. A probabilistic approach makes it possible to vary the input and model parameter according to an assigned probability distribution function. The objective of a probabilistic approach is to quantify uncertainties in the model output, which is in this case the water level frequency line.

In WBI2017, the model uncertainties of water levels in the fresh water system are taken into account by means of a posteriori integration method, which means that the uncertainty is taken into account after the water levels are calculated (Diermanse (2017); *Hydra-Ring 17.1 - Technical reference manual* (2016)). This study uses a method where an uncertain quantity of the model is treated as a stochastic variable after which the distribution of the resulting water levels are calculated. This method can be seen as a priori method, because the uncertainty is included beforehand.

Current method consistent with WBI2017
In the current WBI2017 method, the model uncertainty is not considered as an additional stochastic variable in a hydrodynamic model, but it is included in the resulted water level. These water levels are considered to be uncertain, which is done by adding a normal distribution to the water levels that describes the model uncertainty:

$$V = X + Y \tag{3.14}$$

where X is stochastic variable representing the water level, Y is a normal distributed stochastic variable representing the model uncertainty and V is the stochastic variable representing the water level including the model uncertainty. The normal distribution of Y is parameterised by a mean, that represents a bias, and a standard deviation: $\mathcal{N}(\mu_h,\, \sigma_h^2)$. The bias is zero, because it is assumed that the physical models calculations are carried out by calibrated and validated models, with the goal to minimise the bias. Furthermore only one standard deviation is used for the whole range of water levels. The basis for the establishment of the standard deviation are the uncertainties during extreme flow conditions and are already discussed in Section 3.1.1.

The probability density function of V can be drawn from the joint probability distribution function of X and Y, which is formulated as the product of the conditional probability distribution (representing uncertainty) and the base variable (representing the water level):

$$f(x, y) = f(y|x)f(x) \tag{3.15}$$

If we translate this to exceedance probabilities:

$$\begin{gathered} P(V > v) = P(X + Y > v) \\ \int dx f(x)\ P(x + Y > v|X = x) \\ \int dx f(x)\ P(Y > v - x|X = x) \\ \int dx f(x)\ [1 - F_{Y|X=x}(v - x)] \end{gathered} \tag{3.16}$$

where $F_{Y|X=x}(v-x)$ is the cumulative density function of y, given realisation $X = x$. Discretisation of this formula gives the final result that is implemented in the semi-analytical model:

$$P(V > v) = \sum_{i=1}^{n} f(x_i)\Delta x[1 - F_{Y|X=x}(v - x)] \tag{3.17}$$

Adding uncertainty result in a probability distribution V of which the percentiles for water levels generally exceed a larger range, which means that the water level is more dispersed or spread out if model uncertainty is included. How this modelling technique effects the water level frequency line is illustrated in figure 3.16. The left graph shows the stochastic variable X representing the water level (black line) and the added normal distributed stochastic variable Y representing the model uncertainty of which the grey dashed lines illustrate the 90 % confidence bounds. The stochastic variable Y has got a bias of zero and a standard deviation of 20 cm being constant for all return periods. Applying equation 3.17 gives the stochastic variable V that represents the water level including the model uncertainty (red line).

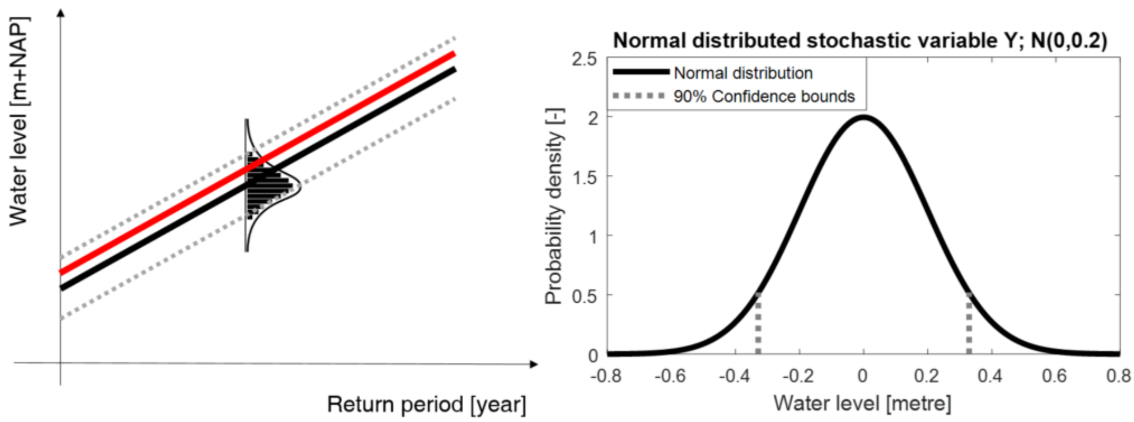


Figure 3.16: Illustration of the inclusion of model uncertainty according to WBI2017

A variant of the normal distribution is the truncated normal distribution. A truncated distribution is a conditional distribution that results from restricting the domain of some other probability distribution. These kind of distributions arise in practical statistics in cases where the ability to record, or even to know about, occurrences is limited to values which lie within a specified range. The stochastic variable Y that is distributed according to the normal probability density function with a mean μ_h and standard deviation σ_h could be applied to cases where the model uncertainty at a specific location is bounded. Let $Y \sim \mathcal{N}(\mu_h, \sigma_h^2)$ with $f(y)$ the probability density of Y. Denote by $\int_{-\infty}^{x} dy f(y)$ the corresponding cumulative distribution. Then the truncated probability density function is defined by

$$g(y) = \begin{cases} \dfrac{f(y)}{F(b) - F(a)} & \text{if } a \leq y \leq b \\ 0 & \text{if } y < a \text{ or } y > b \end{cases}$$

Figure 3.17 shows an example of truncated normal distribution. In this case the truncated normal distribution has got an upper and lower limit of twice the standard deviation away from the mean. The mean value and standard deviation of the original distribution are used.

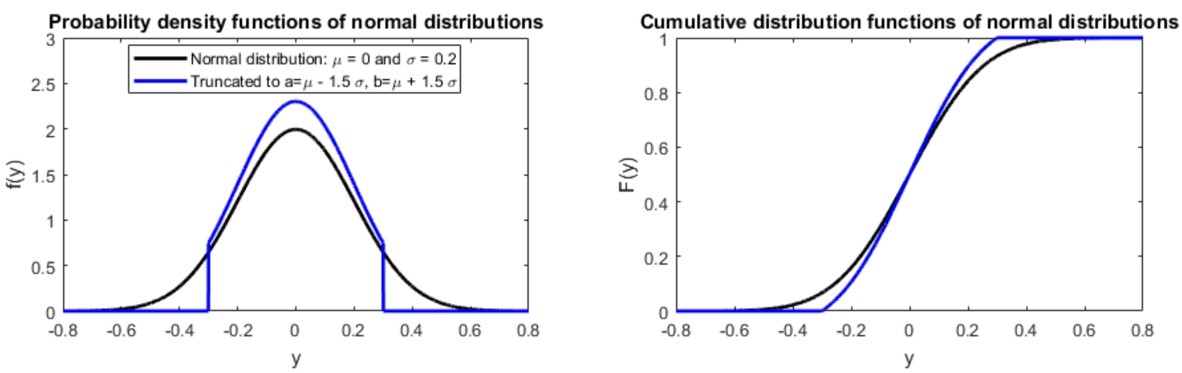


Figure 3.17: The pdf and cdf of a normal distribution and a truncated normal distribution at 1.5σ

Physics-based method

Another way to include model uncertainty is by considering an input or model parameter (that represents the source of the model uncertainty) in the model as a stochastic variable with a probability distribution. The sensitivity analysis showed that the hydraulic roughness of the main channel is the most sensitive parameter in the Rhine branches and therefore introduces the major uncertainty of water level predictions. In general two types of distribution to describe the uncertainty are used: the normal distribution (unbounded) and the lognormal distribution (semi-bounded). These two distribution functions are widely used, because they are relatively easy to work with. A detailed explanation about the properties of those distributions and the trade-off between the methods to be used can be found in appendix C. A normal distribution is applied to the hydraulic roughness coefficient of the main channel and the formulas applied to model the effect on the water level frequency line are given below.

The Manning's roughness coefficient n is considered as a stochastic variable ($\mathcal{N}(\mu, \sigma^2)$) with a probability density function $f(n)$. The stochastic variable may take particular values (positive in the case of log-normal) with positive probability and for these particular values the associated rating curve is derived using the Manning formula. Figure 3.18 shows a discritesed roughness coefficient of the main channel (right) and the associated rating curves (left).

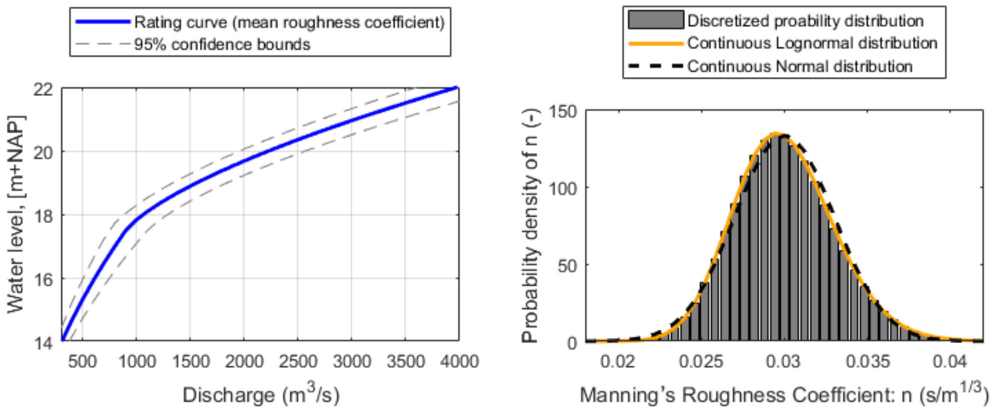


Figure 3.18: Left graph: Rating curve of a schematised IJssel cross-sectional profile and the 95 % confidence bounds resulting from the stochastic roughness coefficient of the main channel. Right graph: Probability density functions of the Manning's roughness coefficient

The probability that water level x exceeds h given a roughness coefficient n_i can be calculated using the discharge frequency curve and the derived rating curve. The so-called "Law of Total Probability" can be applied, which expresses the total probability of an outcome that can be realised via several distinct events, to determine the water level frequency curve:

$$\begin{aligned} P(H > h) &= \int dnf(n)P(X > h|N = n) \\ &\int dnf(n)[1 - F_{X|N=n}(h)] \end{aligned} \tag{3.18}$$

where $F_{X|N=n}(x)$ is the cumulative density function of x, given realisation $N = n$. The following formula is found for the case of discrete outcomes, which is implemented in the semi-analytical model:

$$P(H > h) = \sum_{i=1}^{j} f(n_i)\Delta n[1 - F_{X|N=n_i}(h)] \tag{3.19}$$

where $f(n_i)\Delta n$ is the probability that the roughness coefficient n_i occurs. The sum of all possible Manning's coefficients and corresponding exceedance probabilities gives the exceedance probabilities of a water level (h) where the uncertainty is incorporated into the water level.

The sensitivity analysis showed that the water level response is in general most sensitive to the roughness of the main channel. However, in some cases the roughness of the floodplains become important (in cases where the floodplains are large and the bank height is small) and should also be considered as a stochastic variable as well. In these cases, the model will consist of two stochastic variables, which are assumed to be independent:

$$P(H > h) = \int\int dn_m dn_f f(n_m) f(n_f) \;\; P(X > h | N_m = n_m, N_f = n_f) \tag{3.20}$$

where n_m is the roughness of the main channel and n_f is the roughness of the floodplains. The discrete function can be expressed as:

$$P(H > h) = \sum\sum f(n_{mi}) \Delta n_m \;\; f(n_{fj}) \Delta n_f \; [1 - F_{X|N_m = n_{mi}, N_f = n_{fj}}(h)] \tag{3.21}$$

Note: The hydraulic roughness of the main channel and floodplains are not considered to be dependent. The parameterisation of the floodplain roughness is based on ecotope classification of the land that is translated to roughness values, while the roughness of the main channel is calibrated. Generally, every model parameter can be made stochastic as described above. This makes it very easy to investigate the effect of other uncertainty sources on water level frequency lines.

Water level distributions

For both methods (WBI2017 and physics-based) a range of water levels can occur (with probability of occurrence p) given a certain discharge. The resulting water level frequency lines (described by equations 3.17 and 3.19) will be compared in the following section, but it is also interesting to see how the uncertainty evolves along the water level frequency line. In fact, a water level frequency curve of the upper rivers is nothing more than a translation of the rating curve to frequency of occurrence. Therefore, the probability distributions of the water level given a discharge level are also compared for several discharge levels, together with the associated standard deviations. The WBI-2017 method assumes an uncertainty that is independent of the return period, which can be assessed in this way. The following formulas are used to determine the statistical parameters of the physics-based method:

$$\begin{aligned} \mu_{H|Q=q} &= \int dn f_N(n) h(n, q) \\ \sigma_{H|Q=q} &= \int (h(n, q) - \mu_{H|Q=q})^2 f_N(n) dn \end{aligned} \tag{3.22}$$

The mean and standard deviation of the uncertain water levels given a certain discharge for the WBI2017-method can be easily determined, because the uncertainty is imposed by means of a symmetric normal distribution on the resulting water level.

$$\begin{aligned} \mu_{V|Q=q} &= x_{Q=q} \\ \sigma_{V|Q=q} &= \sigma_h \end{aligned} \tag{3.23}$$

Note: In case two stochastic variables are modelled, the comparison between the two methods is done by means of the 95% confidence bounds of the water level frequency lines.

3.3 Results

The methodology to derive water level frequency lines is applied to different river schematisations. The uncertainty modelling techniques (WBI2017 and physics-based) are assessed by looking at varying geometric properties and two river interventions (flood channel and retention area). The standard deviation of the stochastic variable representing the model uncertainty (σ_h) in the WBI2017-method is set to 20 centimetres. The stochastic roughness coefficients are described by a normal distribution with a mean value corresponding to the river characteristics in Table 3.4 and a variation coefficient of 10 %. The sensitivity analyses showed that for an average IJssel profile a 10% variation coefficient of the main channel corresponds approximately to a standard deviation of 20 centimetres for the water level.

3.3.1 Varying geometric properties of rivers

First the geometric properties of a river are varied where it is assumed that these geometric profiles apply to a long river stretch such that the uniform flow assumption is valid. Different model runs are made with varying flow sections and water surface slopes. During these runs the hydraulic roughness is considered to be stochastic and depending on the situation the main channel and/or floodplain roughness are modelled stochastically. In a situation where a river consist of wide floodplains, the roughness of the floodplains become equally important as the main channel roughness or even more and both roughness coefficients are considered stochastically. At the beginning of every paragraph a short summary of the used model settings is given.

IJssel river characterised by average dimensions
An average IJssel river profile is modelled that is schematised as follows:

Model schematisation average IJssel

River characteristics	: Average IJssel profile
Physical calculations	: Divided Channel Method
Stochastic model parameters (Physics-based)	: Roughness (main channel)

Figure 3.19 shows the resulting water level frequency lines and the 95% confidence bounds of the water levels. The 95% confidence bounds illustrate how water level uncertainties propagate as function of the return period. Both methods seem to give similar results.

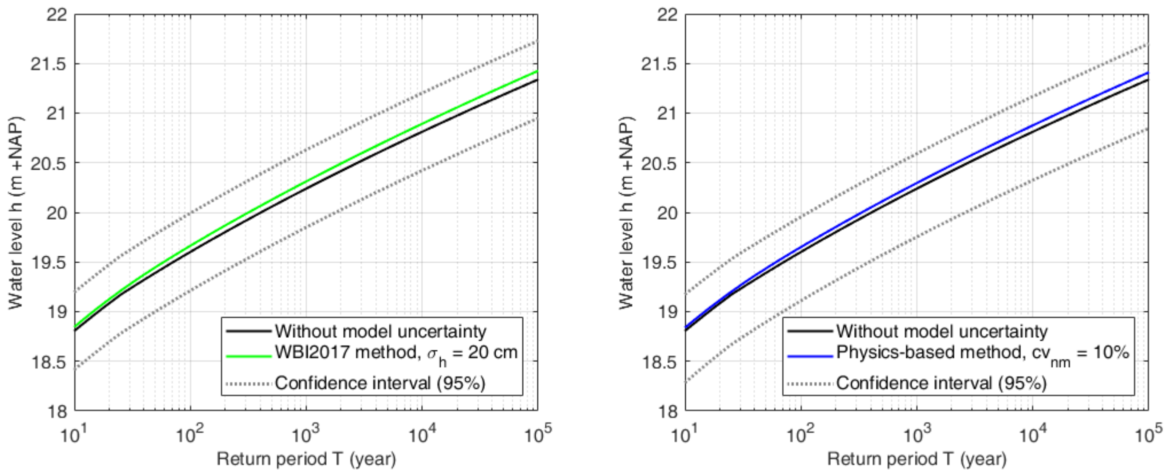


Figure 3.19: Water level frequency line derived with different uncertainty modelling techniques: without model uncertainty (black), WBI2017 method (green) and physics-based method (blue). The 95 % confidence bounds of the water level resulting from the two uncertainty modelling techniques are shown in both graphs.

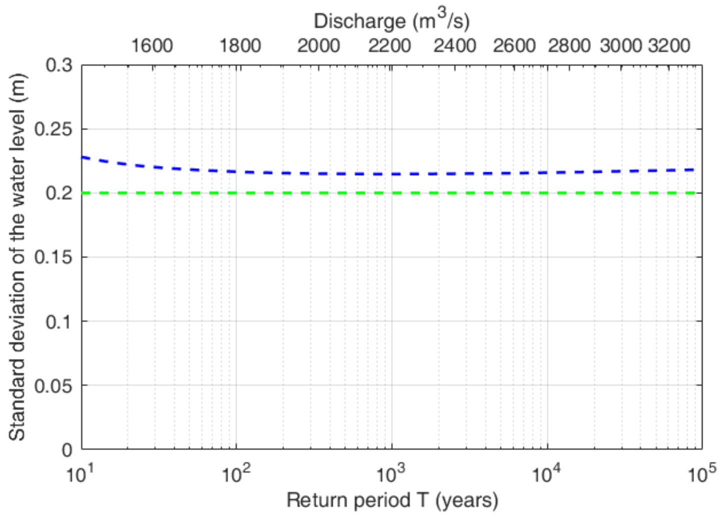


Figure 3.20: Uncertainty of the water level plotted against return period and discharge. Blue line indicates the physics-based method and the green line the WBI2017 method.

The uncertainty in the water level can be indicated by the standard deviation which can be determined for both uncertainty modelling techniques as described in 3.2.4. Figure 3.20 shows that the WBI2017 method gives a constant water level uncertainty that is consistent with the principles of the method: normal distribution with a standard deviation which is independent of the return period. The physics-based method results in a slightly curved line that is caused by 1) the inundation process of the floodplains 2) the increase of the flow section. The width of the main channel is relatively small (compared to the total width of the river) and therefore an uncertain main channel roughness results in a larger water level response if the floodplains are not inundated. When the floodplains are completely inundated the water levels become less uncertain because the geometry becomes more important. For higher discharge levels an uncertain main channel roughness increases the water level uncertainty slightly, because the flow section becomes larger.

The water level uncertainty is higher for the physics-based method, while the water level frequency line of the physics-based method lies approximately 1-2 centimetres below the one derived by the WBI2017 method. This can be explained by the non-linear translation of a normal distributed roughness coefficient to water levels. Therefore, the water level distribution has got a heavier tail for lower water levels. In Appendix D this effect is visualised by showing the resulting water level distribution at four discharge levels. Furthermore, a sensitivity analysis has been carried out to identify the difference between a log-normal and a normal distribution (see Appendix C). A normal distributed manning roughness coefficient resulted in the least skew of the water level distribution. To conclude, the two different uncertainty modelling methods give approximately the same result for an average IJssel profile.

IJssel river characterised by wide floodplains

Previous results showed that the quantification of the uncertain main channel roughness by a variation coefficient of 10 % corresponds to a water level uncertainty (expressed by the standard deviation) of 20 centimetres for an average IJssel profile. Now the width of the floodplains are varied to see how uncertainties affect the water level frequency line. Rivers in the Netherlands may become several kilometres wide. The system behaviour of a wide river deviates from the average situation as well does the rating curve. In this case, a symmetrical compound channel is modelled with varying floodplain widths. The characteristics of an IJssel river are assumed, but the width of the floodplains are varied by 550, 1100 and 2200 metres. These dimensions are representative for the river IJssel, which may become locally 3 kilometres wide e.g. around rkm 890 - 910. For these situations, the flow section of the floodplains becomes more important and therefore both roughness coefficients (main channel and floodplains) are considered as independent stochastic variables. The following model schematisation is used to derive water level frequency lines:

Model schematisation wide IJssel	
River characteristics	: IJssel (varying floodplain width)
Physical calculations	: Divided Channel Method
Stochastic model parameters (Physics-based)	: Roughness (main channel + floodplains)

The resulting water level frequency lines calculated by the two different uncertainty modelling techniques deviate significantly. For the physics-based method both roughness coefficients are made stochastic, but the water levels that includes the WBI2017 model uncertainty are still much larger. The decimate height of a wide profile is smaller than an average cross-sectional profile and the net effects of the WBI2017 depends on the decimate heights and increase significantly.

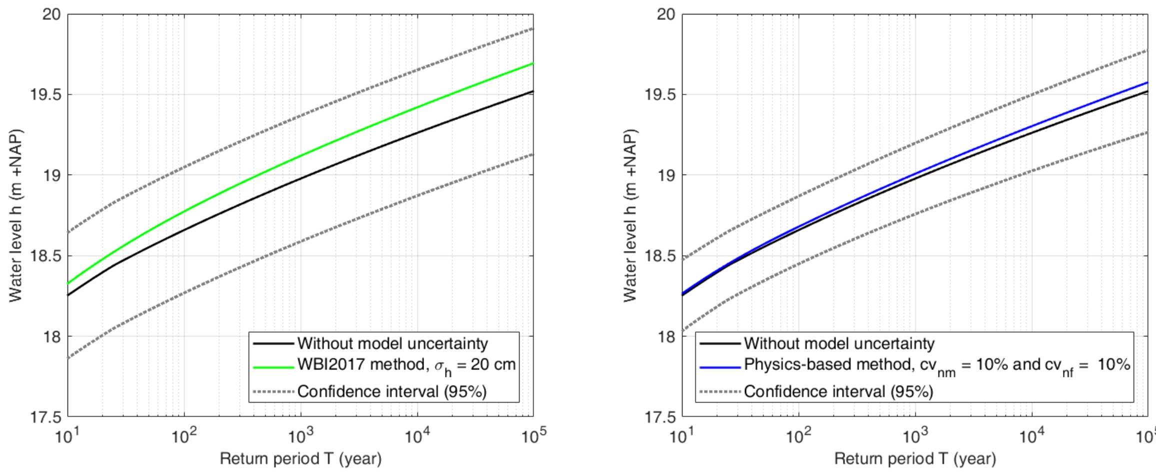


Figure 3.21: Water level frequency lines for a wide river (width floodplains = 2200 metres) derived with different uncertainty modelling techniques: WBI2017 method (left) and physics-based method (right).

The uncertainty in the water level due to both uncertainty modelling techniques are shown in Figure 3.22 as well as an average IJssel cross-sectional profile. The WBI2017 assumes an uniform standard deviation for the model uncertainty for all river profiles. The physics-based approach shows that the standard deviation of the water level for a wide IJssel profile is almost two times smaller.

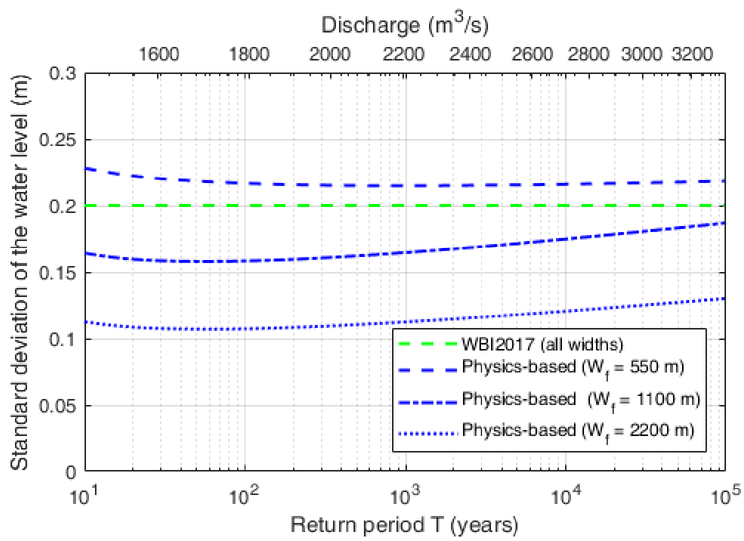


Figure 3.22: Standard deviation of the water level for different widths of the floodplains where the remainder characteristics are based on an average IJssel river. The green line shows the WBI2017 method and the blue line the physics-based method.

Lets compare the net effect of the two different uncertainty modelling methods for an average profile and wide profile for which the decimate heights differs. The net effect of the WBI2017 becomes larger for a smaller decimate height, just as seen in the Comparative Analysis, while the physics-based method results in smaller net effects (see Table 3.8). The latter is also in line with the expectations that a wide river is less sensitive to uncertainties.

Table 3.8: Net effect of the uncertainty modelling methods on the water levels at a return period of 10 000 years.

		WBI2017	Physics-based	Difference
IJssel average (W_f = 550 m)	hdec = 0.55 m	8 cm	9 cm	-1 cm
IJssel wide (W_f = 1100 m)	hdec = 0.40 m	11 cm	6 cm	5 cm
IJssel wide (W_f = 2200 m)	hdec = 0.25 m	16 cm	4 cm	12 cm

The influence of the water surface slope

The characteristic water surface slope for each Rhine branch is the fall of the river (difference between water level upstream and downstream) divided by the length of the river. However, locally a river could become steeper than on average. For these model runs, the water surface slope is varied and the effect of different modelling uncertainty methods on the water levels is investigated. For the physics-based method the hydraulic roughness of the main channel and floodplains are considered as independent stochastic variables.

Model schematisation steep rivers	
River characteristics	: Average IJssel profile
Physical calculations	: Divided Channel Method
Stochastic model parameters (Physics-based)	: Roughness (main channel + floodplains)

First, the water surface slope is considered twice as steep as normal and the resulting water level frequency lines can be seen in Figure 3.23. For lower return periods the confidence bounds of the physics-based method are wider than the WBI2017 method. The reason for this is that the inundation of the floodplains occurs during higher discharge levels and the uncertainty of the water levels is larger if the water levels do not exceed the bankfull height. For higher return periods both uncertainty modelling methods result approximately the same water level uncertainty.

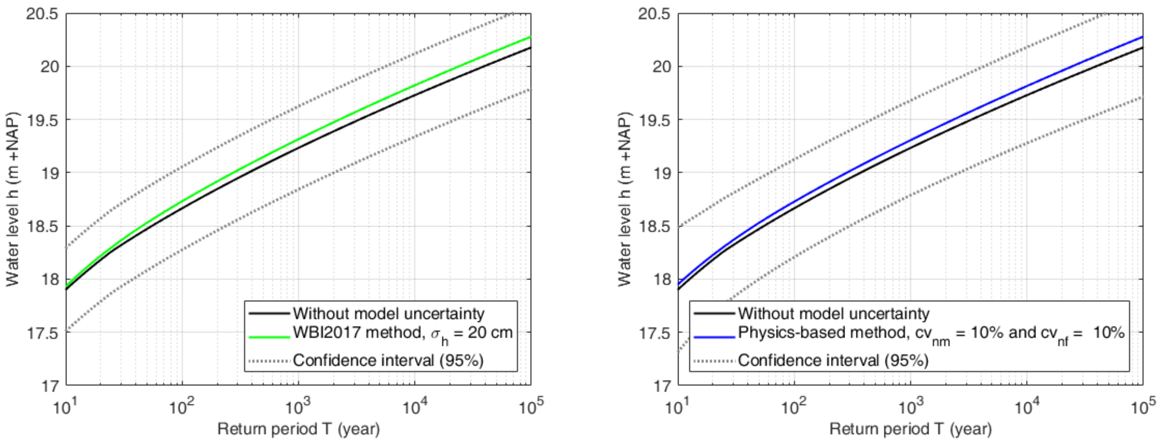


Figure 3.23: Water level frequency lines for a steep river (S_w = 2 10^{-4})derived with different uncertainty modelling techniques: WBI2017 method (left) and physics-based method (right).

Next, the water surface slope is made twice and five times as steep as the normal situations. The resulting uncertainties in the water levels can be seen in Figure 3.24. The peak in water level uncertainty corresponds to the discharge level for which the water levels calculated with different roughness values do not exceed the bankfull height. If the water level exceeds the bankfull height the uncertainty decreases, because the water level uncertainty reduces if the river becomes wider. For steeper rivers the discharge capacity of the river increases and the calculated water levels are lower given a certain discharge level. Therefore, the frequency of the floodplain inundation becomes smaller and the peak of the water level uncertainty moves to higher discharge levels. The water level uncertainty is shifted, but the uncertainty for water levels when the flood plain are inundated does not differ significantly.

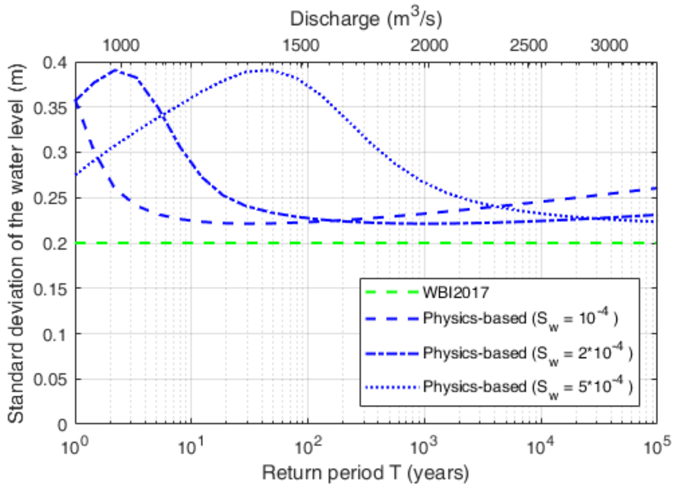


Figure 3.24: The standard deviation of the water level for different water surface slopes.

The decimate height becomes smaller for steeper rivers, but this does not result in smaller water level uncertainties as seen in Figure 3.24. For both uncertainty modelling methods the net effect increases for smaller decimate heights (see Table 3.9). This is in line with the expectations, since the water level uncertainty does change significantly at a return period of 10 000 years, the net effect becomes larger for smaller decimate heights. It can be concluded that a smaller decimate height in the river system does not necessarily mean that the water level uncertainty decreases. It does, if the decimate height gets smaller due to wider cross-sectional profiles. However, a decrease in the decimate height due to a steeper river does not result in a reduction of water level uncertainty.

Table 3.9: Net effect of the uncertainty modelling methods on the water levels at a return period of 10 000 years.

		WBI2017	Physics-based	Difference
IJssel average ($S_w = 10^{-4}$))	hdec = 0.55 m	8 cm	9 cm	-1 cm
IJssel steep ($S_w = 2 \cdot 10^{-4}$))	hdec = 0.47 m	9.5 cm	9 cm	0.5 cm
IJssel steep ($S_w = 5 \cdot 10^{-4}$))	hdec = 0.42 m	10 cm	10.5 cm	-0.5 cm

3.3.2 Flood channel

Model schematisation Flood channel	
River characteristics	: IJssel profile
Physical calculations	: Divided Channel Method + Backwater effects
Stochastic model parameters (Physics-based)	: Roughness (main channel)

A flood channel is defined as an additional channel outside the existing floodplains of a river and is only used at high discharge levels. In this case, the flood channel is designed for a use of approximately once every hundred years, which means that the flood channel starts flowing at an IJssel discharge of approximately 2000 m^3/s (HR2006 statistics). In section 3.2.3 the system behaviour of a river with a flood channel is described based on a deterministic approach. Water levels downstream of the flood channel affect the water levels upstream and therefore uncertainties in the water level also propagates in the upstream direction. It is assumed that uncertainties in the river stretch, of which the flood channel affects the water levels, are fully correlated.

Figure 3.25 shows the water level frequency lines that are determined by different methods: deterministic approach (without model uncertainty), the WBI2017-method and the physics-based method. For deterministic calculations the water levels remain constant between return periods of approximately 100 and 1000 years due to the enlarged conveyance of the river. From a return period of 1000 years onwards the water level frequency line becomes more gently because the width becomes larger and the water level becomes less sensitive to the discharge. The confidence bounds of both uncertainty modelling methods clarify how model uncertainties propagate along the water frequency line. The WBI2017 method assumes the 95 % confidence bounds to be constant ($\sim$ 80 cm) for all return periods, while the uncertainty in the water level decreases according to the physics-based method (the confidence bounds become narrower). The narrower confidence bounds start when the water level calculated with the highest roughness coefficient exceeds the height of the moveable gates. This physical behaviour is illustrated in figure 3.26.

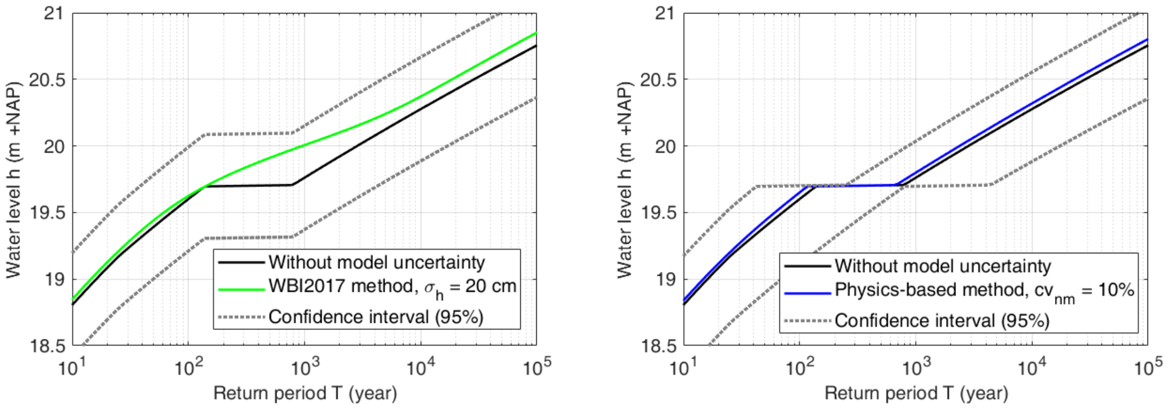


Figure 3.25: Upper graph: The water level frequency lines for different uncertainty modelling techniques. Lower graphs: The 95% confidence bounds of both methods (Left; WBI2017 method, right; physics-based method).

The moment at which the water level exceeds the height of the moveable gates the conveyance of the river is suddenly enlarged. The water level drops down instantaneously and the water levels are lower for the same discharge level. The process of lowering the gates is based on a pre-fixed water level, which means that if the hydraulic roughness is underestimated w.r.t. the actual water level the flood channel starts to function during lower discharge levels. However, the maximum water level during a flood wave remains the same, because the water level at the enlarged conveyance is lower. On the other side, overestimated roughness coefficients result in water levels that do not exceed the height of the moveable gates. In reality the process in which the water levels adopt to the new river profile will be smoother, but the system behaves in the same manner.

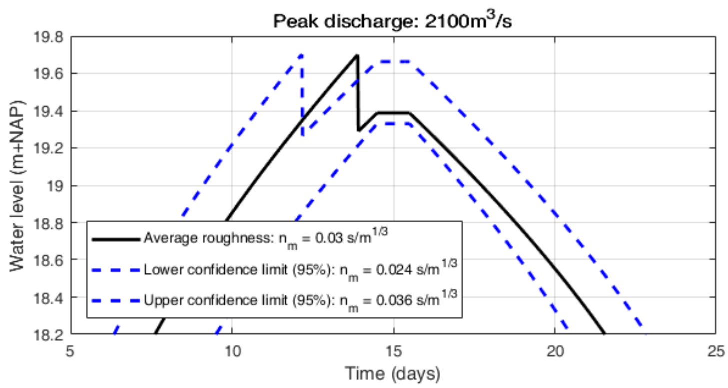


Figure 3.26: Water level time series during a flood wave with a peak discharge of 2100 m^3/s. The water level time series of the 2.5th and 97.5th percentile (upper and lower limit of 95% confidence bounds) of the stochastic roughness are illustrated by the blue dashed line.

The WBI2017 method does not consider the physical presence of a flood channel and the confidence bounds of the water level remain constant for all discharge levels (and return periods). The assumption of a normal distributed water level with a constant standard deviation seems not valid for a situation where a flood channel influences the hydrodynamic system.

Uncertain hydraulic roughness of the flood channel

The area of the flood channel is only inundated in case of high-water with return periods of more than 100 years. Just like the roughness in floodplains, the roughness in the flood channel is based on ecotope classification of the land. Errors in floodplain roughness parameterisation are reduced by calibration of the main channel roughness for discharge levels at which the floodplains are inundated. The roughness of the flood channel is not included in this calibration process, because discharge goes rarely through the flood channel. However, any changes in the discharge carrying capacity of the flood channel due to vegetation roughness affect the water levels. During conditions when the flood channel is fully functioning a large part of the water flows through the flood channel and roughness uncertainties related to the floodplains become more important. The influence is assessed by considering the roughness of the floodplains (including flood channel) as additional stochastic variable in the physics-based approach.

Model schematisation Flood channel

River characteristics	: IJssel profile
Physical calculations	: Divided Channel Method + Backwater effects
Stochastic model parameters (Physics-based)	: Roughness (main channel + floodplains)

Figure 3.27 illustrates how the water level frequency line is affected. The flattened part of the line is shortened because there is a probability that the discharge capacity of the flood channel is lower then expected. Uncertainty in the floodplain roughness also affects higher water levels for which the flood channel is fully functioning.

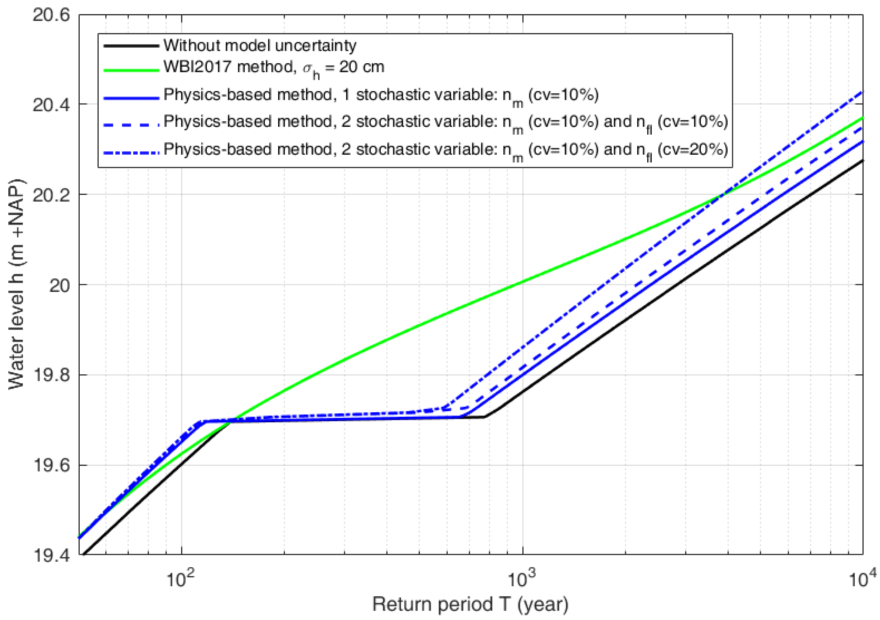


Figure 3.27: The effect of uncertain hydraulic roughness in a flood channel.

In Figure 3.28 the water level uncertainty as function of the return period is displayed for different uncertainty modelling methods. In rivers where a flood channel influence the hydrodynamic the water level uncertainty decreases, because the operation of the moveable gates decreases the maximum water levels during a flood wave. In addition, the flood channel results in a wider cross-sectional profile and the water level uncertainty becomes smaller compared with an average IJssel profile. This effect is clearly visible for high return periods ($T > 10^4$). Considering both hydraulic roughness (main channel and floodplains) as stochastic variables result in slightly larger water level uncertainties for high return periods. To conclude, the WBI2017 does not count for the reduction in water level uncertainty around the discharge levels for which the operation of the moveable gates control the water levels. The effect of wrongly modelled water level uncertainty with the WBI2017 method is also noticeable at higher return period, because the upper tail of the water level uncertainty also contributes to higher return periods.

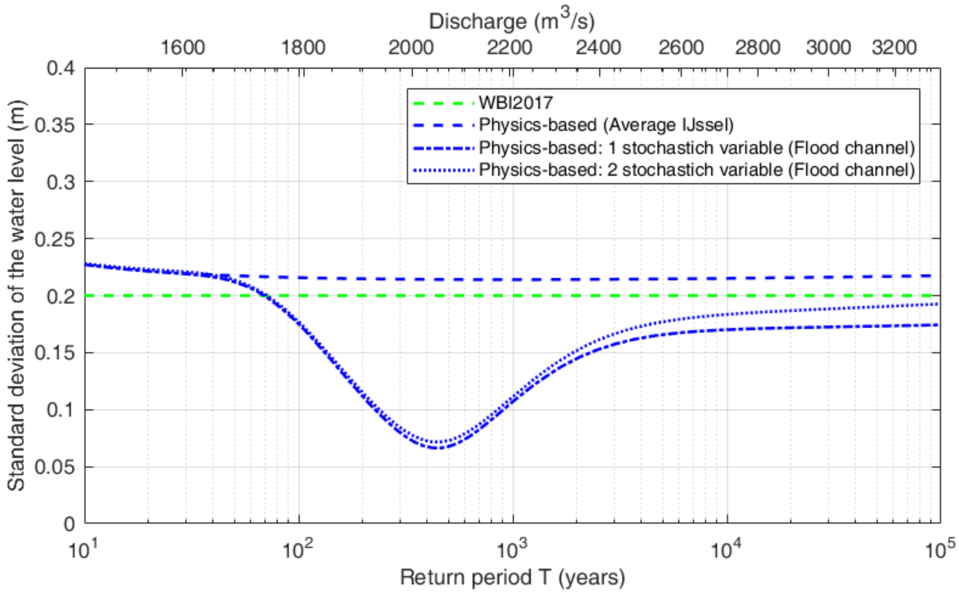


Figure 3.28: Water level uncertainty for different uncertainty modelling methods.

Truncated normal distribution WBI2017 method

The previous results showed that the inclusion of model uncertainty by the WBI2017 method does not represents the physical behaviour of a flood channel. A possibility to capture this effect in a pragmatic way is to truncate the normal distribution of the model uncertainty. This is modelled with the following model set-up:

Model schematisation Flood channel

River characteristics	: IJssel profile
Physical calculations	: Divided Channel Method + Backwater effects
Stochastic model parameters (Physics-based)	: Roughness (main channel)
Stochastic model parameters (WBI2017)	: Truncated water level

By truncating the model uncertainty, the water levels for higher return periods are less influenced by the tails from lower return periods. The standard deviation of a truncated normal distribution is also smaller and therefore, the model uncertainty for higher return periods could be underestimated. This pragmatic approach will not result in a better representation of physical processes.

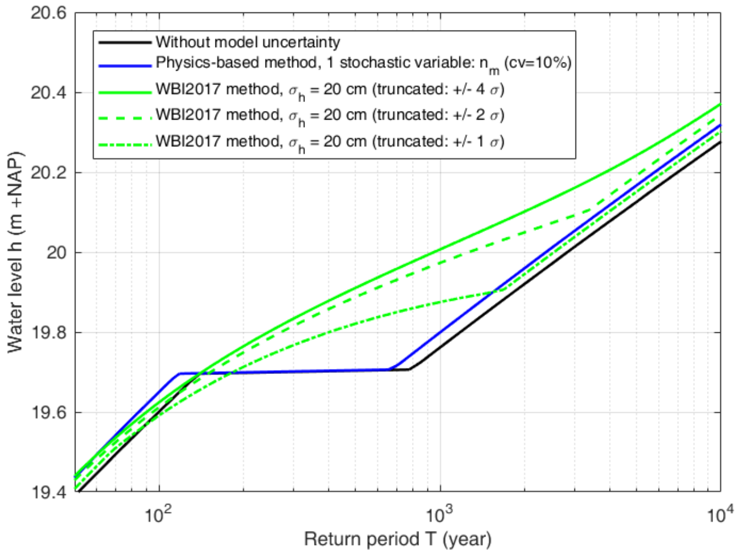


Figure 3.29: Water level frequency lines where the normal distribution of the water level (WBI2017) is truncated at different intervals.

3.3.3 Retention area

A retention area could be used to lower water levels in a river downstream of the retention area by extracting discharge above a critical level. In this way the passing flood wave is corrected by the available storage volume in the retention area. At two locations along the river Meuse retention areas are used to reduce the water levels downstream: Lateraalkanaal-West and Lob van Gennep. This study does not aim to mimic the reality in detail but to gain insights into the concept. An IJssel profile is considered while in reality there is no retention area along the IJssel river. The model is set-up as follows:

Model schematisation Retention area

River characteristics	: IJssel profile
Physical calculations	: Divided Channel Method + Retention process
Stochastic model parameters	: Roughness (Main channel)

Retention with a fixed inlet height is only effective over a limited range of water or discharge levels and no longer works when the maximum volume of the retention area is reached. In this case, a retention area extracts instantaneously all discharge above the designed threshold value that corresponds to a discharge level of 2500 m^3/s (return period is approximately 1500 years). Assuming the roughness coefficient to be uncertain the retention are could start to function at different discharge levels, but still at the same water level. The storage capacity (S_{cap}) of the retention area is first assumed to be $25 \cdot 10^6$ m^3/s, but later on is varied.

The black line in figure 3.30 shows the deterministic approach, where model uncertainties are not considered. It can be seen that the water level stops increasing from the moment the threshold value is exceeded. The behaviour of the confidence bounds of the two uncertainty modelling methods are similar to a flood channel. The confidence bounds of the water level for the WBI2017 method remains constant for all return periods, while the physics-based shows a decline in the confidence bound if the water level calculated with the highest roughness exceeds the threshold value at which the retention area starts to function. However, the uncertainty of the water levels between return periods of 10^4 and $2{\cdot}10^4$ years increases. We observe that the operations of the moveable gates reduces the water level uncertainty around the flood channel according to the physics-based method, while a retention area redistributes the water level uncertainty.

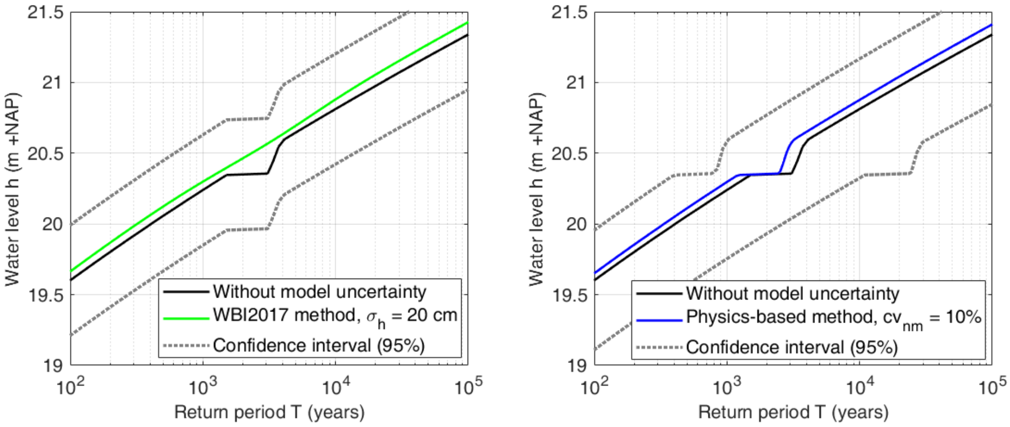


Figure 3.30: The frequency lines of the water level calculated by different methods: deterministic method (black), WBI2017 method (green) and physics-based method (blue). The 95% confidence bounds of the water level both methods due to the uncertainty modelling techniques are also shown.

The variation of the standard deviation of the water level as function of the discharge/return period is analysed. The introduction of a retention area results in the redistribution of water level uncertainty according to the physics-based method: locally accumulation and reduction of water level uncertainties. Accumulation takes place to discharge levels where the water level modelled with the mean roughness values are not yet topped off. For these situations, water levels corresponding to higher roughness values are topped of. Reduction of the water level uncertainty occurs for discharge levels where the water levels modelled with the mean roughness values are not influenced by the retention area, while lower roughness values are still influenced. Therefore, the lower tail of the water level distribution becomes heavier (see Figure D.3 in the Appendix).

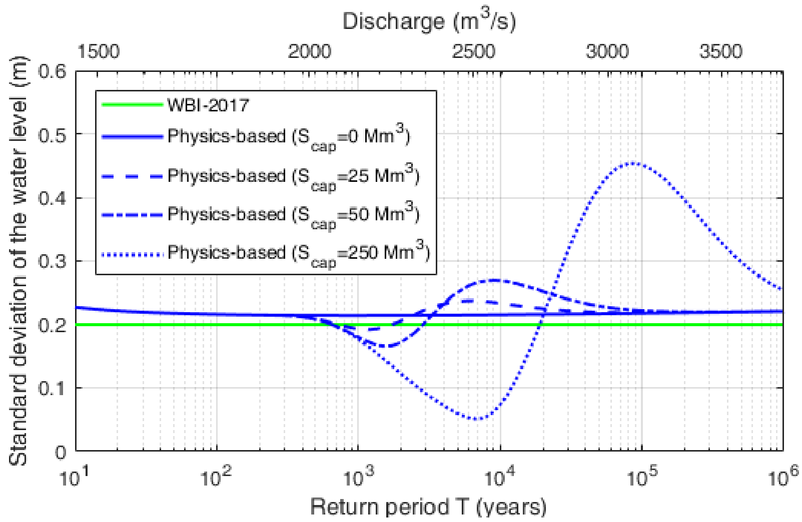


Figure 3.31: Water level uncertainty for a river with a retention area. Different storage volumes are modelled ranging from 0 to 250 Mm^3.

Intermezzo: Additional uncertainties associated to the operation of a retention area

The hydrodynamic behaviour of retention is much more sensitive to uncertainties than other river intervention measures like flood or side channels. Uncertainties that influence the effectiveness are e.g. hydraulic roughness, inlet height and shape of the flood wave. The rating curves of a location downstream of a retention area is often based on deterministic calculations, in which expected values are assumed for uncertain parameters. This research focused on the most dominant uncertain model parameter that is determined based on river schematisations that are not affected by river interventions.

For rivers where retention areas affect the water levels, the important model uncertainties could deviate. The water levels that occur during the discharge extraction strongly depend on the exact height of the inlet and the discharge capacity towards the retention area. Therefore, water levels become locally more uncertain. To illustrate this effect model runs have been made which consider the water levels around the inlet uncertain. The resulting water level frequency lines can be seen in figure 3.32. The physics-based approach behaves more like the WBI2017 method. This study does not focus on uncertainties related to the operation of a retention area, but the developed model can also be used to address the effectiveness of retention area if model uncertainties are taken into account.

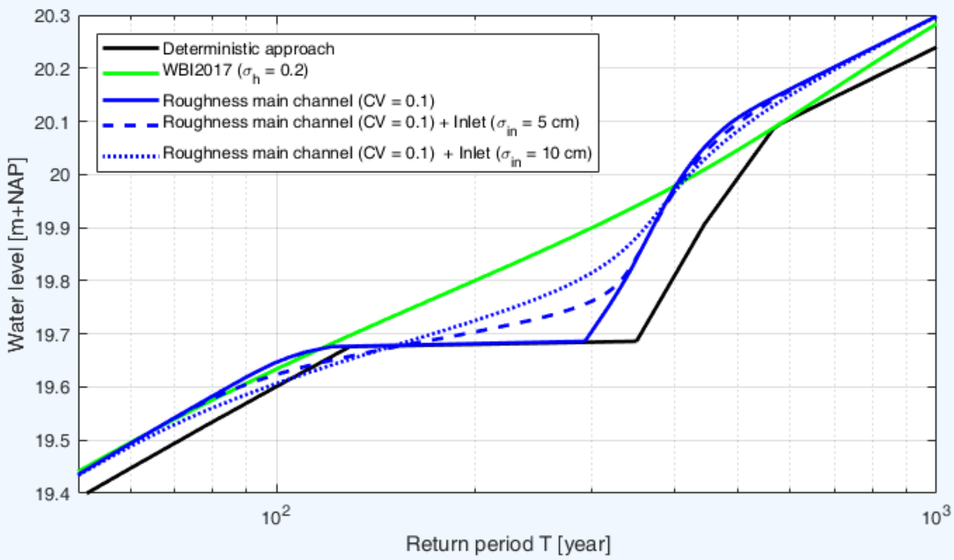


Figure 3.32: The influence of additional water level uncertainty at the inlet height of the retention area

3.3.4 Practical applicability in the Netherlands

It has been demonstrated that the geometry shapes the water level frequency line, which in turn affect the net effect of model uncertainty in the WBI2017 method. A lower decimate height, caused by a wide river, results in a larger net effect of the model uncertainty in the WBI2017 method. Considering the hydraulic roughness as stochastic parameter, it has been shown that the uncertainty in the water level becomes smaller. A pragmatic way to avoid these artefacts is to adjust the model uncertainty locally for the situations where geometry flattens the water level frequency line. Therefore, the decimate heights along the Dutch rivers are analysed.

Bouw (2008) computed the decimate heights along the Dutch rivers, based on the discharge statistics and geometric information consistent with HR2006. Therefore, the Room for the river projects are not included. In general these projects result in lower decimate heights, because the river has become wider. The decimate heights are calculated at the norm frequency according to the old safety standard. The Rhine branches and Bedijkte Maas have got a norm frequency of the water level of 1/1250 per year and the Limburgse Maas 1/250 per year. Figure 3.33 shows the variation of the decimate height of locations in the upper rivers area in the Netherlands. The dynamic behaviour of a river causes the spatial variability of the decimate height. In natural conditions the river slope, roughness and cross section are irregular and a river will always be non-uniform. Even within a branch differences in geometry exist. The Limburgse Maas shows strong local variations of the decimate height. Therefore, it is not practical to adjust the model uncertainty locally for every situation.

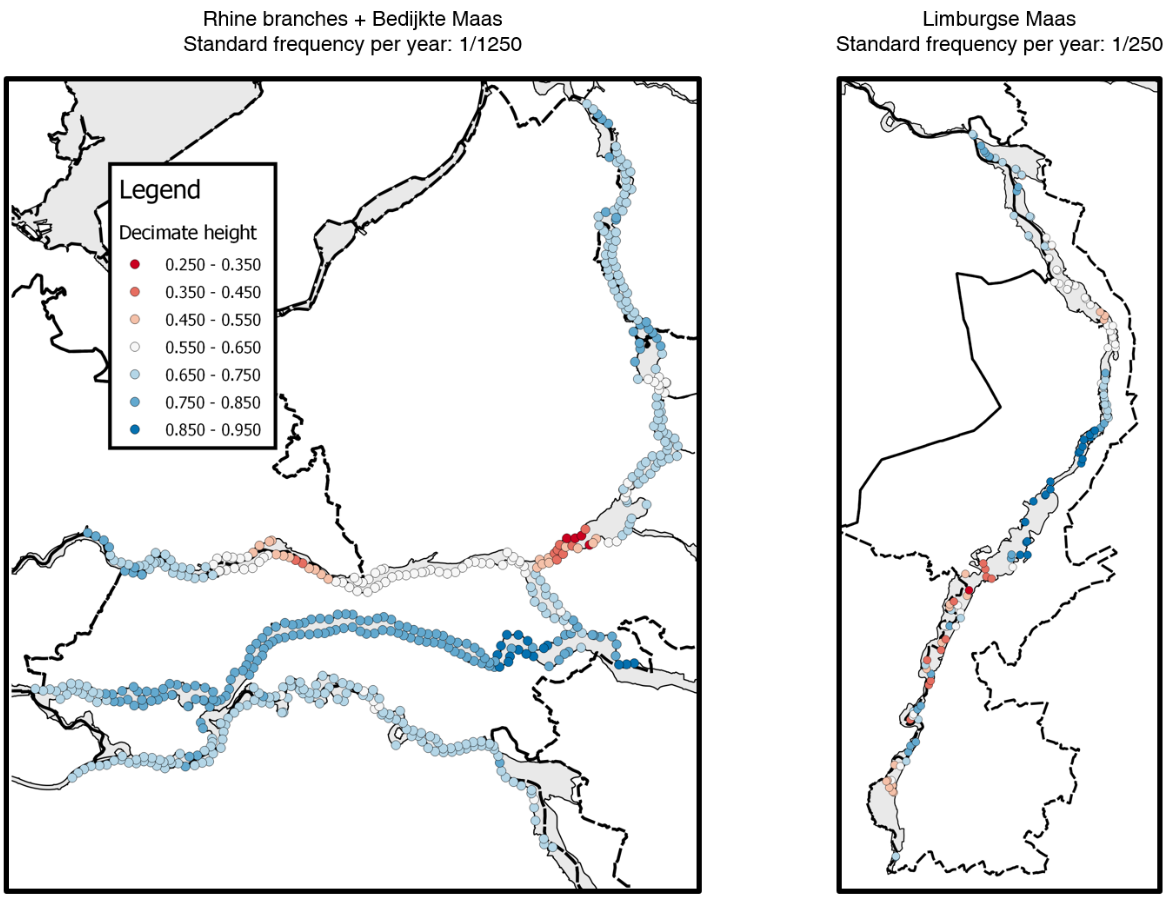


Figure 3.33: Spatial variations of the decimate height in the upper river area. (Source: Bouw (2008))

3.4 Model discussion

A question that needs to be answered, is to what extent the relevant phenomena can be modelled and predicted with the developed model. The used physical equations consist of several assumptions and limitations and assumptions are made to isolate geometric behaviour. These limitations and assumptions are discussed in this section in order to place the results in the right context. It is also tried to establish the link between results in this study and the hydrodynamic models that are used in hydra-models, so a follow up study can build upon the results obtained.

3.4.1 Hydrodynamic modelling

The calculation of water levels in the river is based on the normal flow and backwater curve equations which consist of many approximations and simplifications. These calculations are appropriate to draw conclusions for this study, because they focus on physical system behaviour. However, the calculated water levels cannot be used for practical applications. The following remarks may be made regarding the hydrodynamic modelling:

- The flow is assumed to be steady and uniform (normal flow). The river flow is said to be steady if at any point in the river the important characteristics such as the velocity and pressure do not change in time. In natural flowing rivers this will never be the case, but at locations where the water level fluctuates rather slowly and gradually steady flow conditions can be assumed. During flood waves the flow may become unsteady in situations where the water surface slopes deviates from the bed slope and the rating curve could transform into a loop. The phenomena that at a given water level, the discharge is larger when the surface elevation rises than when it falls is called Hysteresis. Therefore, the highest water level during a high flood wave follows after the peak discharge.

- Idealised cross-sectional profiles are assumed; a straight compound channel with floodplains, which has a constant width in the flow direction. The velocity, slope and discharge are assumed to be constant in the longitudinal direction of the river. In natural conditions the river slope, roughness and cross section are irregular, asymmetrical and continuously changing. Perpendicular to the flow, groynes and a variety of embankments are located in the Dutch rivers that divide the floodplains and the main channel. Lateral flow from the main channel towards the floodplains would occur and the floodplains could function more like a storage area than a conveyance area. The flow of a river can therefore not be assumed to be uniform. The DCM makes distinction between the main channel floodplain flows, but lateral momentum transfer has been ignored when estimating flow velocities in the compound channel.

- The Bresse function is used to calculate the backwater effects, for which equilibrium water depths are used as input. This function is actually only valid for rectangular cross-section of which the bottom level is constant. The model schematisations in this research assumes a compound channel and therefore a discharge-averaged water depth is chosen. So, depending on the amount of water that flows through the floodplains/main channel the water depth is calculated.

3.4.2 Uncertainty modelling

The roughness parameter(s) of the cross-section is assumed to be stochastic during the model runs for the physics-based method. One of the main assumptions is the fully spatial dependency of the roughness stochastic variable in the longitudinal direction of the river. In reality, it is possible that these uncertainties occur in a limited stretch of a river. The uncertainties partly influence the water levels in the river, because the downstream boundary conditions (which may not consist of deviations from the estimated model parameter) is also involved in the water level determination.

The influences of spatial correlations on normative water levels are investigated by Duits and Noortwijk (1999). A Sobek-model was forced by a stationary discharge of 16000 m^3/s at Lobith and the roughness of the main channel was varied to analyse the confidence bounds at five locations along the river IJssel. The 90% confidence bounds of the water level are calculated by the 90% confidence bounds of the main channel roughness which is set to 14 $m^{0.5}/s$ for the IJssel. The model consist of representative cross-sectional profiles for approximately every 500 metre and different correlation coefficients are imposed to describe the spatial dependency of model uncertainties. The resulting confidence bounds of the normative water levels can be seen in Figure 3.34. The correlation of the roughness of the main channel in the longitudinal direction has a major influence on the confidence bounds of the normative water levels. However, these spatial correlations can also be taken into account by adjusting the standard deviation of the roughness coefficient.

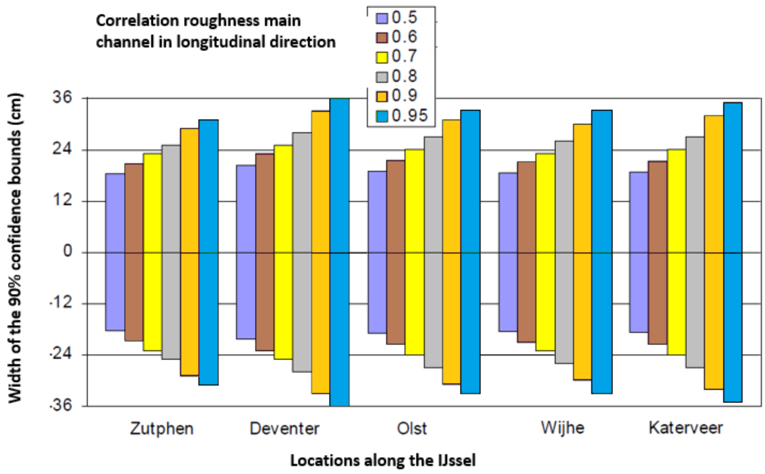


Figure 3.34: Comparison of the 90% confidence bands of the water levels at a discharge level of 16000 m^3/s at Lobith for five locations along the IJssel for six different correlation coefficients of main channel roughness in longitudinal direction (Duits and Noortwijk (1999)).

This study only considered the uncertainties related to the roughness, but the uncertain discharge distributions at the bifurcations also play an important role for the Nederrijn-Lek and IJssel. There are more uncertainties that play an important role near river interventions, like a retention area or a flood channel. The shape of the high water wave becomes important near retention areas, where in this study a representative shape of a high water wave has been adapted. The control of the inlet and the flow capacity of the inlet could limit the effectiveness of these interventions. If the opening of a moveable gate fails the discharge capacity of a flood channel is lower and therefore the water levels in the main river higher. These kind of uncertainties are not included in this study, as well as in WBI2017.

3.4.3 The influence of discharge distribution at bifurcation points and discharge statistics

The discharge frequency curve plays an important role in flood risk analysis. The quantification of the water level uncertainty (expressed in a standard deviation) does not depend on the discharge statistics, but the net effect and the resulting water level frequency including model uncertainty is affected by the discharge statistics. In this study, the old discharge statistics (HR2006) are considered. Changes in the decimate height are only caused by the geometrics of the river, because a nearly exponential discharge frequency line is assumed (see Figure 3.6). The discharge statistics of WBI2017 include upstream flooding that leads to a reduction of the discharge compared to the HR2006. Because of this the water level frequency lines becomes flatter for higher return periods. The net effect of model uncertainties becomes larger since the decimate height is lower.

In addition, the river intervention "Lek ontzien" results in more discharge towards the IJssel and less in the Nederrijn-Lek. This discharge distributions at the bifurcations are manipulated by regulating structures at Pannerden and Hondsbroeksche Pleij. The policy of this river intervention is to keep the discharge towards the Lek as long as possible at 3.380 m^3/s by controlling these regulating structures properly. The resulting discharge distribution can be seen in left graph of Figure 3.35 and is based on WAQUA runs where peak discharges of flood waves after the IJsselkop (in the IJssel river) are compared to peak discharge levels at Lobith. The resulting discharge frequency line for the IJssel can be seen in the right graph of Figure 3.35.

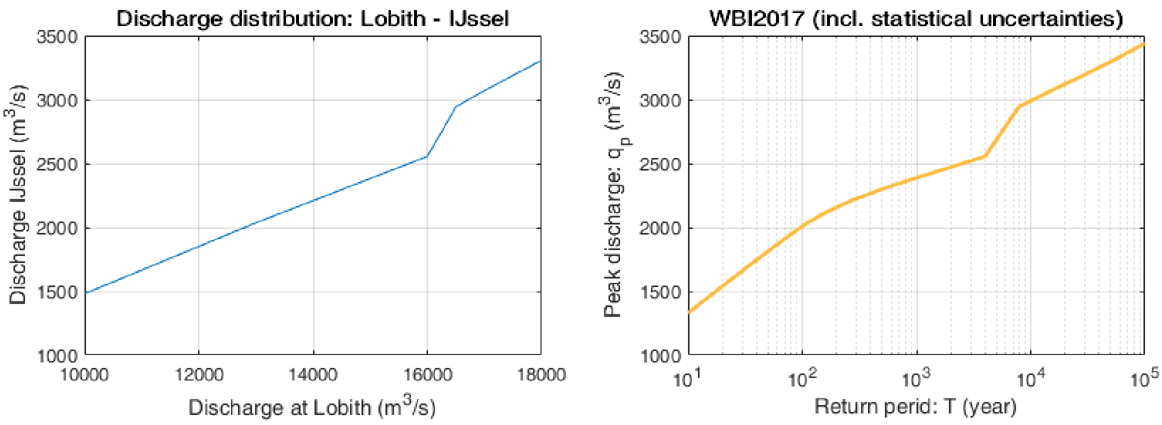


Figure 3.35: Left: relation between discharge at Lobith and the IJssel discharge (based on peak discharge levels of WAQUA calculations). Right: frequency curve based on discharge statistics at Lobith consistent with WBI2017 (including statistical uncertainties) and the discharge distribution of the left graph.

During a model run of WAQUA, every discharge level corresponds to one specific water level and the regulating structures are operated based on a water level. The principles of operating the regulating structures is same as a flood channel and also result in physical unfounded results. This can probably be improved by applying the physics-based approach.

3.5 Conclusions

The second research question is recalled, after which it is evaluated:

> *What is the effect of dominant uncertain model/input parameters on the water level frequency lines for different hydrodynamic systems for the upper rivers and are these results consistent with the current WBI2017 method that includes model uncertainty?*

According to Tijssen et al. (2014) and Chbab (2015) the most important sources of water level uncertainty in the Dutch upper river area are: the calibration approach, hydraulic roughness and physical processes or morphological changes under extreme conditions. The sensitivity analysis performed in this study confirmed that the hydraulic roughness is in general the most uncertain model parameter, and especially the hydraulic roughness of the main channel in which the most water is flowing through.

In this study, the effect of dominant uncertain model parameters on water level frequency lines is assessed by considering the hydraulic roughness (main channel and/or floodplain roughness) as stochastic variable. The WBI2017 method includes the model uncertainty by considering the calculated water levels as stochastic variables with a uniform standard deviation independent of the return period for all river schematisations. In order to compare the uncertainty modelling methods the following two aspects are analysed: 1) relation between the effect of water level uncertainty and the decimate height and 2) dependency between water level uncertainty and the return period.

1. **Relation between decimate height and the effect of water level uncertainty**

   Geerse and Rongen (2016) derived mathematically that the net effect of model uncertainty is larger for lower decimate heights according to the WBI2017 method. The decimate height in the upper river area depends on discharge statistics and the environment (e.g. cross-sectional profile and water surface slope) that establishes the relation between discharge and water level. The water level uncertainty related to the most important sources is equally quantified for the Rhine branches in WBI2017. According to the physics-based method, the water level uncertainty depends on the geometry of a river: the water level uncertainty reduces if the river becomes wider. Therefore a uniform standard deviation of the model uncertainty in the upper-river area is unjustified. The water level uncertainty could be adjusted to the decimate height, where lower decimate heights result in smaller uncertainties, but this makes the inclusion of model uncertainty rather pragmatic.

2. **Dependency of water level uncertainty on the return period**

   In WBI2017 the model uncertainty is chosen independently of the return period. In the upper river area the discharge is the only stochastic variable in computing the water level frequency line, so the model uncertainty is independent of the discharge. In situations where the river is not affected by river interventions, this is a reasonable assumption. However, the water level uncertainties that are calculated by the physics-based method show local deviations in case river interventions are present. These deviations are caused by the environment/geometry that influence the water levels at specific discharge levels during a flood wave. Peak water levels cannot become higher during a flood wave, because of the physical processes that occur: discharge extraction above a certain water level for a retention area and the operation of the moveable gates that enlarges the conveyance if the water level exceeds a critical height (flood channel).

It can be concluded that the uncertainties in water levels are location (locally/micro-level and not meso-level) and discharge dependent, which are both not integrated in the WBI2017 method. An average river profile is modelled properly by the WBI2017 method. However, behaviour and uncertainty in water levels in rivers is a dynamic process. The operation of river interventions results in discharge related reduction or redistribution of water level uncertainty, according to the physics-based method. The two uncertainty modelling methods differ from each other and the physics-based approach seems to represent the underlying physical processes more correctly.

4. Uncertainties in the Lake area

4.1 Introduction

The largest Dutch inland lakes are Lake Marken and Lake IJssel and the size of those lakes are 696 km^2 and 1140 km^2 respectively. Their average depths are about 3.5 and 4.2 metre. The size of these lakes allows significant wave generation and wind-induced set-up of the water levels. Together with the water volume of the lakes this may cause significant flooding damage of the land that is protected by the surrounding dikes. Lakes in the Netherlands are filled by rivers and pumping stations that are discharging into the lakes. Under normal circumstances, the water from the lakes flows into the sea during low tide. High lake levels arise when no, or insufficient, discharge into the sea is possible over prolonged periods with north-westerly winds, causing elevated sea levels. The dominant process controlling the water levels are (combinations of) high lake levels and elevated high water levels caused by wind set-up.

The probabilities of elevated water levels due to wind set-up are obtained from the statistical information from the wind and the transformation to water levels. This transformation is done by a physical model, like WAQUA, which introduces model uncertainties in the local water levels of the lake. These uncertainties should be taken into account and therefore the resulting water level is considered to be a stochastic variable with a zero mean and a standard deviation. This standard deviation depends on the dominant hydraulic processes that determine high water levels. The normative hydraulic conditions for locations along the lake area can be categorised in three different classes:

- Wind dominant conditions
- Lake level dominant conditions
- Combination of lake level and wind

In this study we focus on the water levels of the wind-driven locations. The water levels of the lake-level dominated location depend mainly on the statistically determined lake levels, where model uncertainties are less relevant. These statistical derived lake levels are used as variable in the hydrodynamic WAQUA model. Uncertainties in the water level are mainly determined by parameter settings and numerical aspects, which are not assessable in a simplified physical model. At the end of this chapter the following research question will be answered:

> *How do dominant uncertain model/input parameters effect the water level frequency lines for the lake area and is this consistent with the current WBI-2017 method uncertainty modelling technique?*

4.2 Background

Storms can produce large rises in water level in lakes and near coasts, which are known as wind set-up or storm surges. Accurate water level predictions in these systems require the application of mathematical models that solve the Navier-stokes equation. These equations are derived from mass and momentum conservation, and consist of the continuity equation and the momentum equations. The general assumption is made that the flow is essentially depth averaged, observing that coastal seas, rivers and lakes are much longer than they are deep. Under this shallowness condition, conservation of mass implies that the vertical velocity of the fluid is small. The depth-averaged Navier-stokes equations include wind-induced forces and atmospheric pressure variations and are solved in the hydrodynamic WAQUA model.

To produce accurate high water levels predictions, time and space evolving wind fields need to be modelled. Spatial variations of the water depth and time varying wind speeds result in constantly change water levels during a storm. However, the natural invariability of the wind needs to be schematised such that computed water levels correspond to observations. Therefore, the stochastic variables representing the wind (speed and direction) are translated to a wind event: a wind field in space and time 10 metres above the open water level. In model runs, a uniform wind field is imposed of which the open water wind speed and direction are varied in time. The storm duration is schematised as a trapezoidal profile of which the wind speeds increases linearly from zero to the maximum value in 23 hours, after which the maximum wind speed last for two hours and the wind speed decreases linearly to zero again for 23 hours (Geerse, 2006). Wind rotation during a storm is modelled, where distinction is made between different wind sectors.

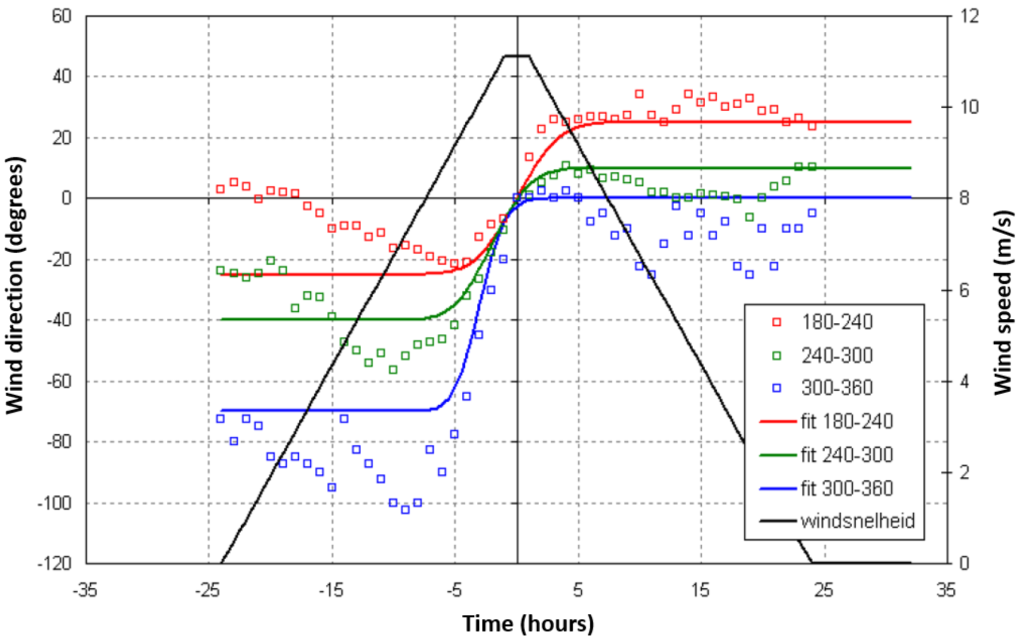


Figure 4.1: Rotation of the wind for different sectors, in degrees with respect to the relevant wind direction (Chbab and de Waal (2017)). The black line shows the time variation of the wind speed (Geerse (2006)).

The wind speed of uniform wind fields is represented by the potential wind speed. The potential wind speed is defined as a standardised wind speed corrected for local roughness effects, representing the one hour averaged wind speed at 10 metres height at a location with a local roughness of 3 cm, corresponding to short grass. The potential wind speed need to be transformed to open-water wind speed (10-m above the open water level) in order to model wind shear stresses in hydrodynamic models. At the Gooimeer and Eemmer the wind speeds are influenced by sheltering effects by nearby land. From the Hollandsche Brug (in between the IJmeer and Gooimeer) the wind speed decreases linearly to 90 % at Nijkerk (at the end of Nijkerkernauw) (Chbab & de Waal, 2017). Figure 4.2 gives an overview of the lake area.

Figure 4.2: Overview op the IJsselmeer and Markermeer and adjacent lakes (Kramer et al. (2017)).

4.2.1 Wind stress modelling

The wind shear stress is the sum of three components; the viscous, turbulent and form stresses. The water surface will be relatively smooth at low wind speeds that result in low turbulence levels. At these stages, the wind stress is dominated by molecular forces that create the viscous stress (Sterl, 2017). At higher wind speeds ($U_{10} > 30$ m/s) the turbulent stress and form stress become dominant. The stress τ_s exerted by the wind is usually parameterized in hydrodynamic models as follows:

$$\tau_s = \rho_a u_* = \rho_a C_D U_{10}^2 \tag{4.1}$$

where ρ_a is the density of air ($\cong 1.25$ kg/m^3), u_* is the so-called friction velocity, C_D is the drag coefficient (-) and U_{10} is the wind speed at a height of 10 metres above the water surface (m^3/s). The drag coefficient is vital for modelling storm surges above a lake and expresses the surface roughness of the area that can be defined as: $C_D = (u_*/U_{10})^2$. The drag coefficient depends on the surface roughness. In the lake area, the water surface become rougher with increasing wind speeds because of wave generation. This process occurs for low to moderate wind speeds, but the behaviour of the drag coefficient for higher wind speeds (U_{10}>30 m/s) is uncertain. Several studies try to describe the relation between wind speeds and drag coefficient. The majority of these studies arrive at comparable results regarding the behaviour of the drag coefficient at high wind speeds: the drag coefficient reaches a maximum of approximately (2.5 +/- 0.2)$\cdot 10^{-3}$ between 30 m/s and 40 m/s. These studies uses laboratory measurements or measurements in real hurricane conditions on sea, but measurements in a lake are not considered.

The fit of Wu (1982) is often used to describe the drag coefficient in hydrodynamic models. The relation of Wu (1982) is a linear relation between the drag coefficient and the wind speed:

$$C_d = (a + b\, U_{10}) \cdot 10^{-3} \tag{4.2}$$

where a and b are empirical parameters. As can be seen, this relation does not contain an upper bound. One simple modification to remedy relations consisting of a linear drag coefficient is to embed a cap whenever it exceeds a specific value (wind speed or drag coefficient). An often used method is to cap the drag coefficient if the wind speed reaches 30 m/s, see e.g. van Vledder (2017). Furthermore a lower bounds can be applied and the relation between wind speed and drag according to Wu (1982) becomes:

$$C_d = (a + b \max{(U_{10,Cd_{min}}; \min{(U_{10,Cd_{max}}; U_{10})})}) \cdot 10^{-3} \tag{4.3}$$

where $U_{10,Cd_{max}}$ is the wind speed that corresponds to the upper limit of the drag coefficient and $U_{10,Cd_{min}}$ the wind speed corresponding to the lower limit. Figure 4.3 shows the relationship between wind speed and drag coefficient with and without an upper- and lower bound

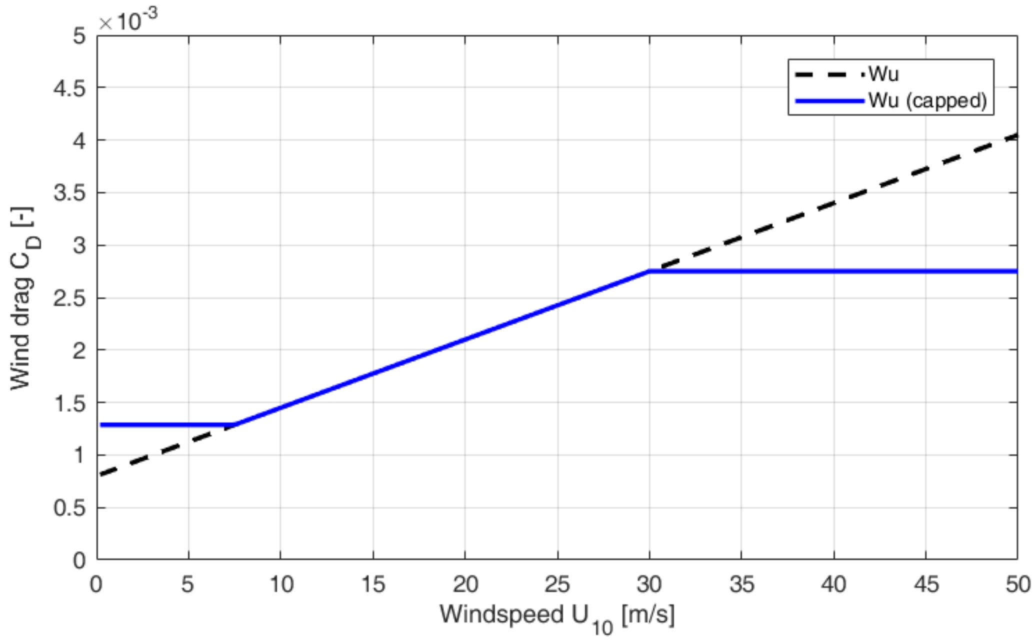


Figure 4.3: Relationships of the drag coefficient and wind speeds according to Wu (a=0.8 and b=0.065). An upper and lower bounds are set to resp. 30 m/s and 7.5 m/s.

4.2.2 Causes of water level uncertainties

Uncertainties of the water levels in the lake area are location dependent. The dominant hydraulic processes vary in this area, where there can be made distinction between wind-dominant areas and lake-level dominated area. The sources of uncertainty related to the hydrodynamic modelling of water levels in the lake area are discussed and are based on Chbab (2014) and the associated hindcast studies. At the end, the quantification of every uncertainty source is listed.

1. **Transformation of potential wind to open-water wind**

   Measurement series for stations at or near open water are too short for an accurate evaluation of extreme wind speeds (Bottema, 2007). This is why long term statistics of measured wind speeds at land-based meteorological stations are used. These measured wind speeds are transformed to potential wind speed. In Hydra-models the extreme potential wind statistics (speed and direction) is used for the inference of the water levels and wave conditions.

The potential wind speed (U_{pot}) needs to be transformed to open water wind speed at 10 metre height ($U_{10,ow}$), which is the input of physical models like WAQUA (see figure 4.4. The transformation from potential wind speed to open water wind speed can be written as:

$$U_{10,ow} = R \cdot U_{pot} \tag{4.4}$$

where R is the transformation factor that also depends on the wind speed $U_{10,ow}$. For this factor different procedures are available for lake IJssel and lake Marken that also result in different transformation results (e.g. generalised logarithmic relations and power law). These transformations invoke uncertainties that need be take into account.

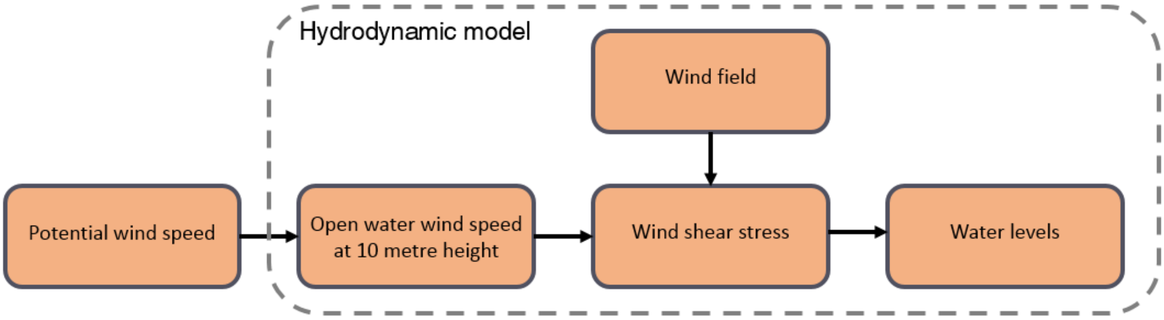


Figure 4.4: This overview only schematises the steps in the process regarding the wind speed. In the hydrodynamic model itself other additional processes and schematisations are modelled to finally compute the water levels.

2. **Wind shear stress (modelling of the drag coefficient)**

   The wind stress depends on the drag coefficient, density of air and the wind speed at a height of 10 m. The drag coefficient is related to the wind speed and it is well-established that for low to moderate wind speeds the drag coefficient increases linearly. However, at wind speeds U_{10}>30 m/s the behaviour of the drag coefficient is less established. These wind speeds are also missing in the measurement range in the lake area. Several studies argue that the drag coefficient is levelling off or is even dropping at these high wind speeds. It is unknown whether this is valid for shallow lakes and the level at which the drag coefficient needs to be capped could occur during lower or even higher wind speeds. The input for the hydrodynamic models consists of 10-m wind velocities that are internally converted to surface stress by applying a particular drag relation. This transformation faces uncertainties, due to the lack of knowledge and natural variability.

3. **Wind fields**

   In the hydrodynamic model for the lakes the wind fields are assumed to be spatially uniform where the wind speed and direction varies in time. Spatially uniform wind fields overestimate the water levels locally (Nicolai et al., 2011). The effect depends on the location and possible sheltering effects by nearby lands. Therefore the effect of this uncertainty source on the water levels is larger at the Eemmeer, Nijkerkernauw, Ketelmeer and Vossemeer.

4. **Storm duration and wind rotation**
   The hydrodynamic models are forced by means of a 10-m time- and space-evolving uniform wind fields that are associated with extreme wind speeds. These wind fields should be chosen such that it includes proper time and space variations. The water levels in Lake IJssel adapt relatively fast to wind fields above the lake and a stationary situation is often established during a storm (Chbab, 2014). However, the duration and shape of these wind fields are inherently uncertain. In reality the storms could differ from the schematisations made in the model.

5. **Other uncertainty sources:**

    - Morphology, bed level, spillways and swaying effects influence the performance of the model. Previous uncertainty sources are related to the wind that is forcing the model, these aspects are related to the model parameters and area schematisation. The bottom level is not calibrated (as well as the roughness parameter) and it is expected that the bottom uncertainty does not significantly influence the water levels. However, in the shallower areas in the lake area (Ketelmeer, Vossemeer, Eemmeer and Nijkerkernauw) the bottom could play a more important role, because the wind set-up becomes larger for shallower areas.
    - The river discharge of the Eem is considered as a deterministic value and set to 75 m^3/s. This corresponds to the daily averaged discharge level that exceeds ones every year. If a two- or three-day averaged discharge level is chosen, the water levels in the Eem area decrease with about 0.15-0.20 metre (Lodder, 2006).
    - According to experts the model consist of uncertainties related to the parameter settings and numerical aspects (Chbab, 2014).

Table 4.1: Uncertainty sources and associated quantification

	Sources of uncertainty	Standard devation [m]
E1	Transformation of potential wind to open-water wind	0.15
E2	Wind shear stress (modelling of the drag coefficient)	0.25
E3	Windfields	0.05 - 0.15
E4	Storm duration and wind rotation	0.05 - 0.10
E5-1	Parameter setting and numerical aspects	0.10
E5-2	Morphology, spillways and swaying effects	0.05 - 0.10
E5-3	Discharge Eem (Lake Marken)	0.10
E5-4	Precepatation, Evaporation and Calibration (bottom) e.d.	0.10

The model uncertainty for locations along the lake area depends on the dominant hydraulic processes that determine the normative water levels. For lake level dominated area, the uncertainties related to the wind are not considered. The used standard deviations of the model uncertainty for different areas can be seen in figure 4.5.

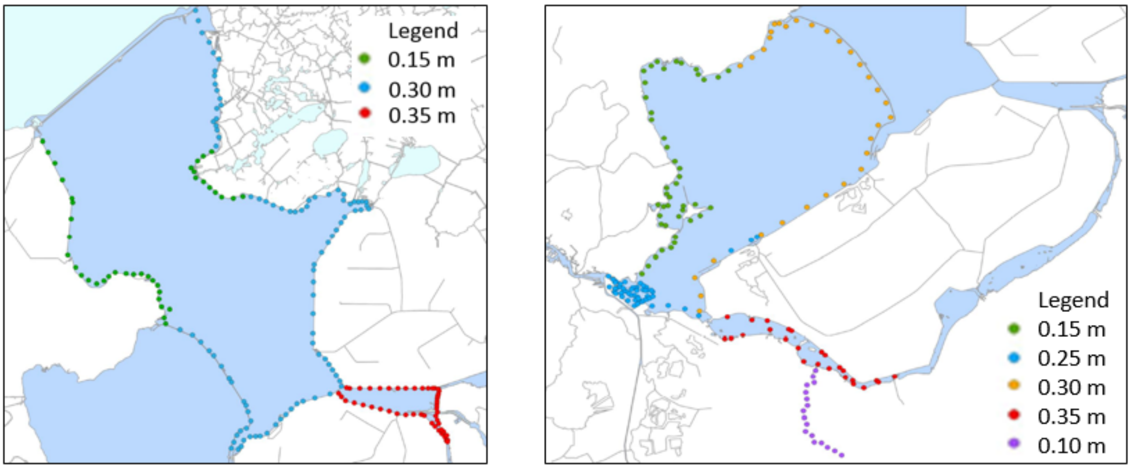


Figure 4.5: Quantification of the model uncertainty in the lake area (Chbab and Groeneweg (2017)).

4.3 Methods

First, the simplified physical model is discussed that is used to simulate the water levels in a 1D lake that are influenced by the wind. The wind will blow the water to one side and wind set-up will occur on the downwind side. These calculations provide the relationship between the open-water wind (U_{10}) and wind set-up for a closed basin given the information about the area. Secondly, the wind statistics are explained that are used to link the calculated water levels to an exceedance probability. Water level frequency lines can be drawn for situations where the other parameters are considered to be deterministic. Thirdly, the uncertainty modelling techniques are elaborated. The corresponding formulas of the WBI2017 method and physics-based method are already emphasised in the previous chapter. This subsection will only discuss the parameterisation of the stochastic variable for the physics-based method. An overview of these elements that are used in the process of deriving water level frequency curves can be seen in Figure 4.7.

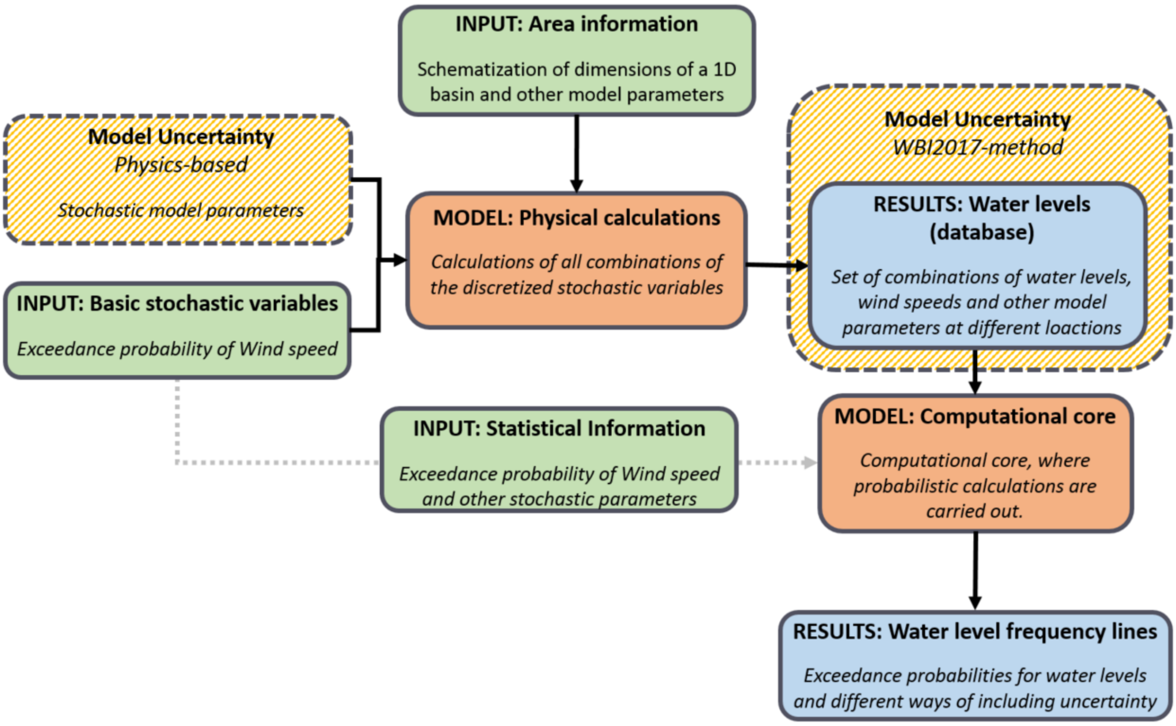


Figure 4.6: Overview of different elements in the process to derive water level frequency lines at locations in a 1D basin that are caused by wind set-up.

4.3.1 Physical calculations

Model schematization

In this study the physical processes are highly schematized, but appropriate to assess the influence of uncertainties related to the drag coefficient on water levels for different systems. A one-dimensional situation is considered in which the wind causes shear stresses at the water surface that result in a water level gradient. A water flow near the surface layer will occur in the direction of the wind, while at the same time continuity requires a return flow in the deeper water layers (the discharge is zero everywhere; no net flow). So the wind forcing and continuity will give a wind driven circulation and with constant wind forcing a stationary circulation will be established eventually (see figure 4.7)

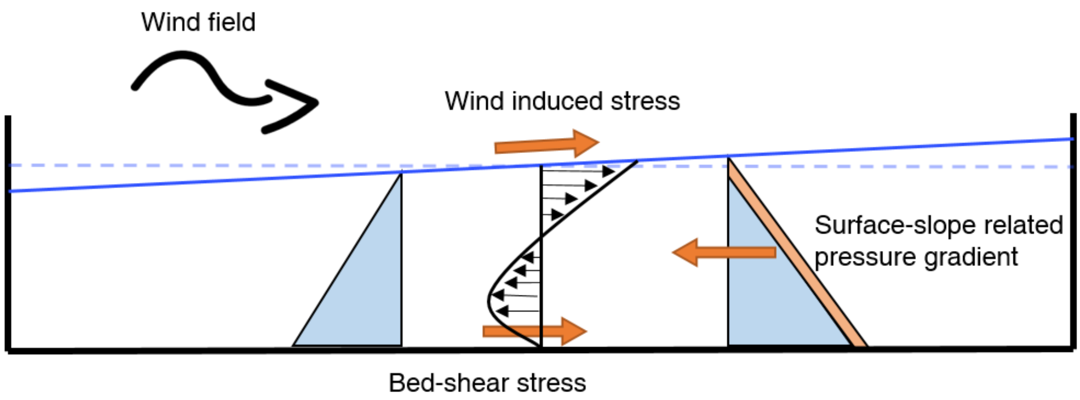


Figure 4.7: Wind-induced circulation and stresses involved

The simplified equation of motion that corresponds to the above 1D situation is:

$$\rho_w g h \frac{dh}{dx} = \tau_{s,x} - \tau_{b,x} \tag{4.5}$$

where ρ_w is the density of water (kg/m^3), g is the gravitational force, h is the water level, $\tau_{s,x}$ is the wind shear stress at the surface and $\tau_{b,x}$ is the bottom shear stress. The bottom shear stress is in most cases unknown, because the velocities near the bottom are unknown. Therefore, the bottom shear stress is related to the wind shear stress at the surface. This is a highly simplified situation, but in most cases the bottom shear stress is less important (J. P. de Waal, 2003). The relation is formulated as follows: $\tau_b = (1-\delta)\tau_s$, where δ is in the order of 1.2. The equation of motion becomes:

$$h\frac{dh}{dx} = \frac{\delta \tau_s}{\rho_w g} \tag{4.6}$$

Substitution of equation 4.1 gives

$$h\frac{dh}{dx} = \frac{\rho_a \delta}{\rho_w g} C_D U_{10}^2 \tag{4.7}$$

This differential equation needs to be solved to determine the water level, but first we simplify the differential equation by combining all the constant in one new parameter $c = \dfrac{\delta \tau_s}{\rho_w g}$. The equation is then,

$$h\frac{dh}{dx} = c \tag{4.8}$$

The solve this differential equation both sides are integrated, which results in a new constant (K) that needs to be solved based on the boundary conditions.

$$h(x) = \sqrt{2cx + K} \tag{4.9}$$

An one-dimensional closed water system is assumed, where a basin is schematised with a length L. Conservation of mass should apply to a closed system: $\int_0^L h\,dx = D \cdot L = A$, where A is cross-sectional water surface (representing the water volume). To derive the boundary conditions, a distinction is made between a wet bottom and partly dried bottom:

- **Flooding**: The water level at the upwind side of the basin (x=0) functions as boundary condition. Substitution in equation 4.9 result in $h(0) = \sqrt{K} = h_0$ and the differential equation that needs to be solved is:

$$h(x) = \sqrt{2cx + h_0^2} \tag{4.10}$$

where h_0 is the only unknown parameter. Using the conservation of mass, this differential equation can be solved iteratively.

- **Drying**: In this case, the wind set-up is so large that the tilted water level leaves parts of the basin dry. In this case the point where the transition between a water depth and dry bottom takes place, denoted by x_0, is considered as boundary condition: $h(x_0) = \sqrt{2cx_0 + K} = 0$. Rewriting and substitution in the differential equation gives:

$$h(x) = \sqrt{2c(x - x_0)} \tag{4.11}$$

where x_0 is the unknown parameter and can also be solved by using the conservation of mass. For this case no iteration process is required and the solution is explicit.

This study investigates the physical system behaviour of the lake area, by means of this one-dimensional. To schematise the IJssel- or Markermeer is rather complex with our model because of their irregular shape. Therefore, the parameters are chosen in such a way that the range of the parameters correspond to the Dutch lakes. The following parameters have been used:

Table 4.2: Used model parameters

Model parameters	Symbol	Unit	Value
Length Basin	L	m	50 000
Depth Basin	D	m	4.5
Relation bodemstress and windstress	δ	-	1.2
Density water	ρ_w	kg/m^3	1000
Density air	ρ_a	kg/m^3	1.2265

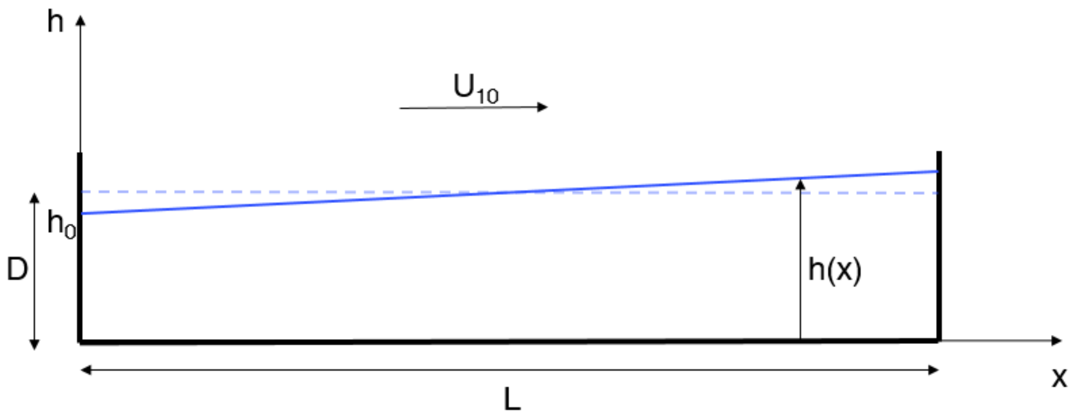


Figure 4.8: Illustrating terminology and parameters used in the model.

As already mentioned, the drag coefficient depends on the surface roughness. As the water surface gets rougher with increasing wind speeds, the drag coefficient increases with wind speeds as well. The drag coefficient can be described by different formulas, but in this model the capped Wu (1982) formula is used (see Equation 4.3). For a deterministic model run, the parameter settings of the capped Wu formula are as follows:

Wu (1982) formula - capped	
a	= 0.8 [-]
b	= 0.65 [-]
$U_{10,Cd_{min}}$	= 7.5 [m/s]
$U_{10,Cd_{max}}$	= 30 [m/s]

As shown in the Section 4.2.2 the drag coefficient is an import source of uncertainty. Therefore, this parameter is not only considered to be deterministic during model runs. How the drag coefficient is modelled stochastically is discussed later.

4.3.2 Statistical information

The physical model requires open-water wind speeds to calculate wind set-up. These open-water wind speeds are derived from the potential wind statistics. The wind statistics consist of wind speed and direction of the potential wind and are based on the measuring station at Schiphol. The wind statistics consist of 16 wind directions and 12-hour maximum conditional on the considered wind direction. For the developed model only one wind direction is considered, which is the west-north west direction. This direction has got the most extreme wind speeds and contributes significantly to the normative water levels at locations along the Dutch lakes. The probability that the potential wind speed U exceeds the value u, given a wind direction r, in one year is given by

$$F(U > u|r) = 360 \cdot P(r)\ P(U > u|r) \tag{4.12}$$

$P(U > u|r)$ is the exceedance probability of the wind speed given a wind direction in the base duration B. This base duration B corresponds with the duration of the time fluctuations of the variable and since the wind speed is a fast variable a base duration of 12 hours is chosen(Geerse et al., 2011). For deriving statistics, only the winter half-year is considered that is assumed to consist of 180 days. This means that there are N = 360 'wind blocks' of duration b = 12 hours.

These statistics correspond to the potential wind speed (wind speed at a height of 10 m and a standard roughness 0.03 m of a flat and open surrounding landscape), while the input of the physical model is open water wind speed u_{10}. The open water wind speed will be larger than the potential wind speed, because the roughness of water is smaller than the roughness of land. The conversion of the potential wind speed u to u_{10} is done with a so-called open water transformation (J. P. de Waal, 2003). This transformation consist of some degree of pragmatism, which is already discussed in Section 4.2.2. Figure 4.9 shows the relation between the potential wind speed and open water wind speed by means of the open water transformation.

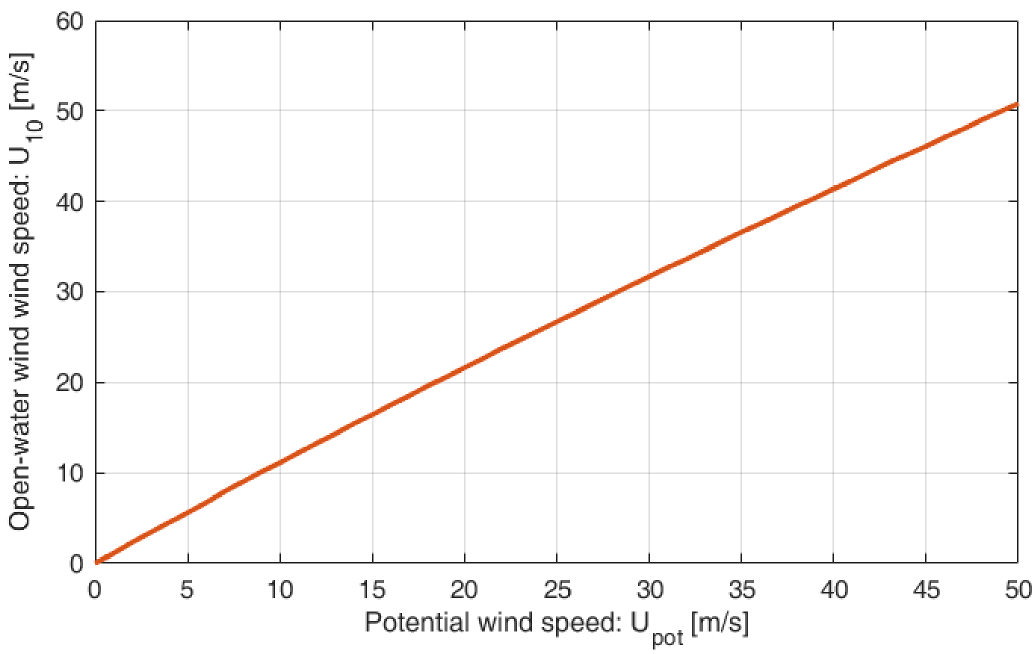


Figure 4.9: Transformation of the potential wind speed u to the open water wind speed u_{10} (source of data: (Agtersloot & Paarlberg, 2016)).

4.3.3 Uncertainty modelling techniques

In the previous chapter the WBI2017 and physics-based approaches are elaborated in detail, where respectively the computed water level is considered as a stochastic variable or parameter(s) in the physical model itself. The same principles and equations are used for the model that is used for the lake area. The only difference is the parameter that is considered to be stochastic in the physics-based approach.

Physics-based

In subsection 4.2.2 the sources of model uncertainty are listed and the most important source is the modelling of the drag coefficient. Figure 4.10 shows the C_d values from nine authoritative studies (Zijlema et al., 2012), which consist of a wide range of wind speeds. These observations show convincingly that the drag coefficient increases approximately linearly with the wind speed up to approximately 20 m/s. The behaviour of the drag coefficient after 20 m/s is rather uncertain, because measurements could become less reliable and the data points vary widely. Zijlema et al. (2012) fitted a 2nd and a 4th-order polynomial to the data using the number of independent observations in each data set as a weight. This parameterisation of the wind drag coefficient propose considerably lower drag coefficients at high wind speeds than the parameterisation of Wu (1982) (see Equation 4.3). However, the minor observations for high wind speeds strongly determine how the polynomial is curved in this extreme region. A study conducted by van Vledder (2017) concluded that a reasonable safe choice for the parameterisation of the drag coefficient is the Wu (1982) formula capped at wind speeds above 30 m/s.

Figure 4.10 provide insights in the variation of the drag coefficient. The uncertainty gets larger for higher wind speeds, because 1) the observations are widely distributed and 2) there are less data points. It can be concluded that the uncertainty related to the drag coefficient increases with increasing wind speeds.

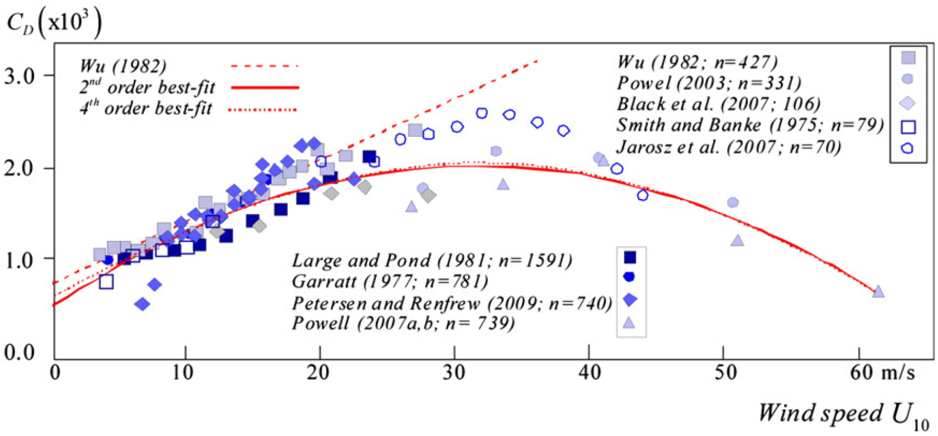


Figure 4.10: Observed values of the wind drag coefficient from various studies and the weighted best-fit 2nd- and 4th-order polynomial (n is the number of independent data points per study) (Zijlema et al. (2012)).

To parameterise the drag coefficient uncertainty the empirical b parameter in the Wu (1982) formula is considered as a stochastic variable. The drag coefficient is linearly related to the open water wind speed and the b parameter. By considering the b parameter stochastically the uncertainty in the drag coefficient increases for higher wind speeds (see Figure 4.11). The quantification of this the stochastic b parameter is rather pragmatic. The confidence bounds of the water level uncertainty according to the WBI2017 approach that is related to the drag coefficient ($\sigma = 0.25\ m$) are analysed. The standard deviation of the b parameter is chosen such that at the edge of the downwind side of the basin the confidence bounds of the water level at a return period of 10 000 years correspond to each other. A coefficient of variation of 15 % is obtained.

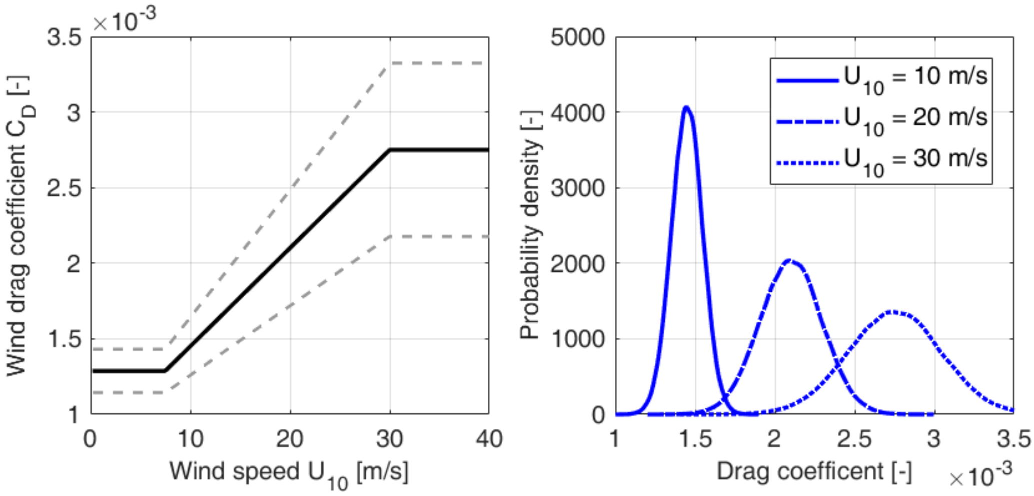


Figure 4.11: Left: 95 % confidence bounds of the drag coefficient as function of the wind speed according to the capped Wu (1982) formula. Right: Probability density function of the drag coefficient for different wind speeds.

4.4 Results

To assess the impact of important sources of uncertainty the different modelling techniques are compared at different locations along the closed basin. The absolute wind set-up for downwind locations closer to the middle point of the basin (or centre of gravity) will be lower than further away. It can be expected that also the decimate height will be different, because it works like a lever: the output force will be react stronger further away from the rotating point if the input force gets greater. This is also illustrated in Figure 4.12.

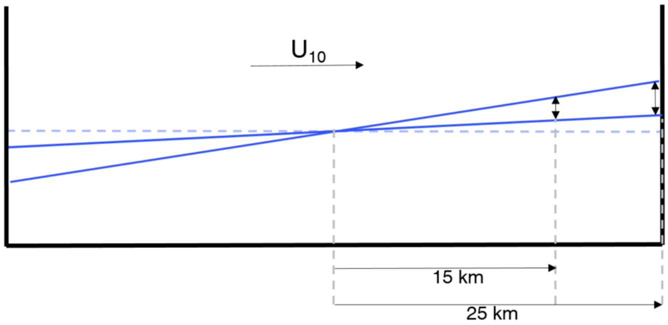


Figure 4.12: Illustration of larger wind set-up at locations further away from the centre of gravity.

During the model runs, the lake level is always considered at 0 m+NAP. Therefore, the water levels also correspond to the wind set-up at every location along the basin.

Sensitivity analyses

First, a small sensitivity analysis is performed to analyse how the wind wet-up is affected by the water depth of the basin and the location along the basin. The depth of the basin is varied between 4.5 and 7.5 metre and the locations are varied between 5 and 25 km from the middle point of the basin. Figure 4.13 shows contour plots of the wind set-up for four different wind speeds (25, 30, 35 and 40 m/s). The wind set-up becomes larger as the wind speed increases, and as the water depth decreases. Distances further away from the middle point of the basin also have got a larger wind set-up. This trend is in line with the expectations. For wind speeds of 40 m/s and water depths smaller than 5.5 metre drying occurs.

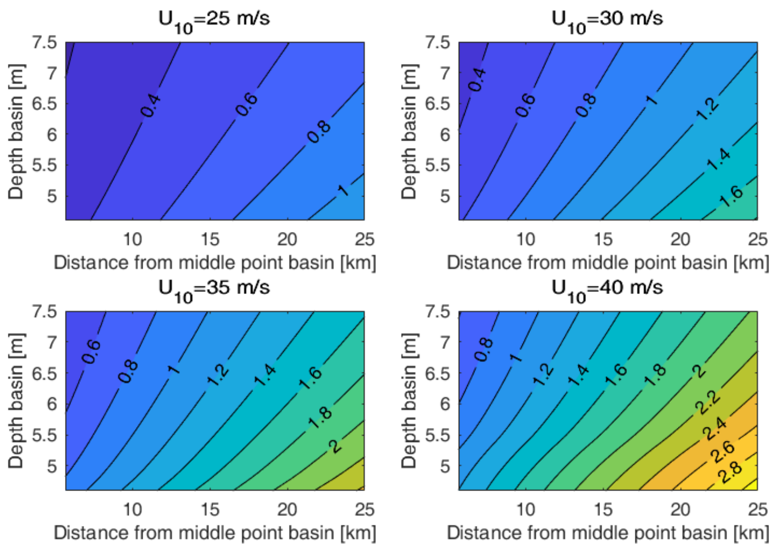


Figure 4.13: Contour plots of water levels for varying basin depths and distances from the middle point of the basin at the downwind side.

For higher wind speeds, the water surface tilt should be larger. The difference in wind set-up between location 5 km and 25 km away from the middle point of the basin is also larger. The model seems to reflect the physical processes correctly.

Water level frequency lines

Next, water level frequency lines for two different locations along the basin at the downwind side are computed: Location A at the edge of the basin (x = 50 km) and location B at three quarters of the basin length (x=37.5 km).

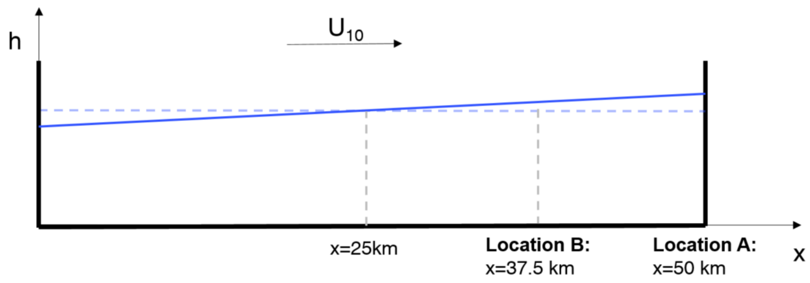


Figure 4.14: Locations for which water level frequency lines are computed.

The decimate height at location A is approximately 60 centimetres, while at location B the decimate height is smaller and is approximately 35 centimetres. According to the WBI2017 method the water level uncertainty for all wind-dominated locations is quantified by a standard deviation independent of the location and return period. For these model runs a standard deviation of 25 centimetres is chosen, which corresponds to the estimated water level uncertainty caused by the uncertain wind shear stress modelling. As known, the net effect of the WBI2017 method is larger for lower decimate height and therefore the location at three quarters of the basin length is probably affected more by model uncertainty.

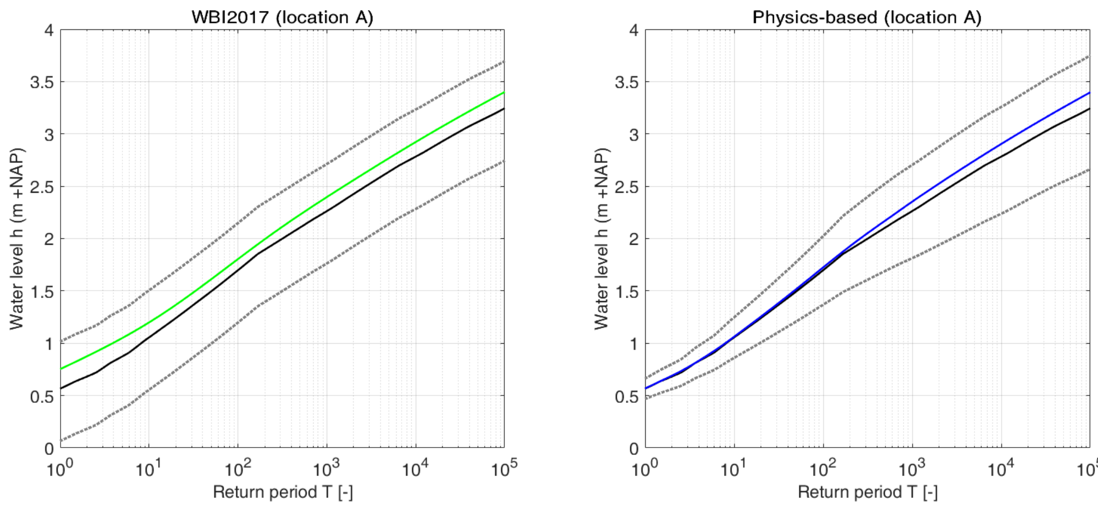


Figure 4.15: Water level frequency lines with different uncertainty modelling methods at location A. Black line is calculated without model uncertainty, the green line according to the WBI2017 method (left graph) and the blue line according to the physics-based method (right graph)

First, the water level frequency lines for location A are presented (see Figure 4.15). The width of the confidence interval of the physics-based method increases for higher return period, which is more in line with the physics compared to the WBI2017 method. Table 4.3 shows the net effect of both uncertainty modelling methods for different return periods as well as the difference between those two methods. At a return period of 10 000 years the net effect of both methods is approximately the same. This is because the stochastic empirical parameter is quantified such that the water level uncertainty is approximately the same. The WBI2017 method has the largest effect for low return periods where the decimate height becomes lower. For these return periods the net effect of the physics-based method is smaller, which corresponds better to physics.

Table 4.3: The net effect in centimetres for two different uncertainty modelling techniques at different return periods at location A

	T = 10 years	T = 100 years	T = 1 000 years	T = 10 000 years
WBI2017	14.5	10.5	13.5	14.0
Physics-based	0.5	2.5	9.0	12.0
Difference	13.0	8.0	4.5	2.0

Now the location at three quarters of the basin length is considered. For this location the decimate height is smaller compared to location A and therefore the net effect of the WBI2017 is larger. If we know consider the physics-based method it can be seen that the net effect is 10 to 30 centimetres smaller than the WBI2017 method. The physics-based method shows smaller water level confidence bounds and the water levels are less uncertain compared to the WBI2017 method. So, the water level uncertainty depends on the location along the basin and decreases for locations closer to the middle point of the basin.

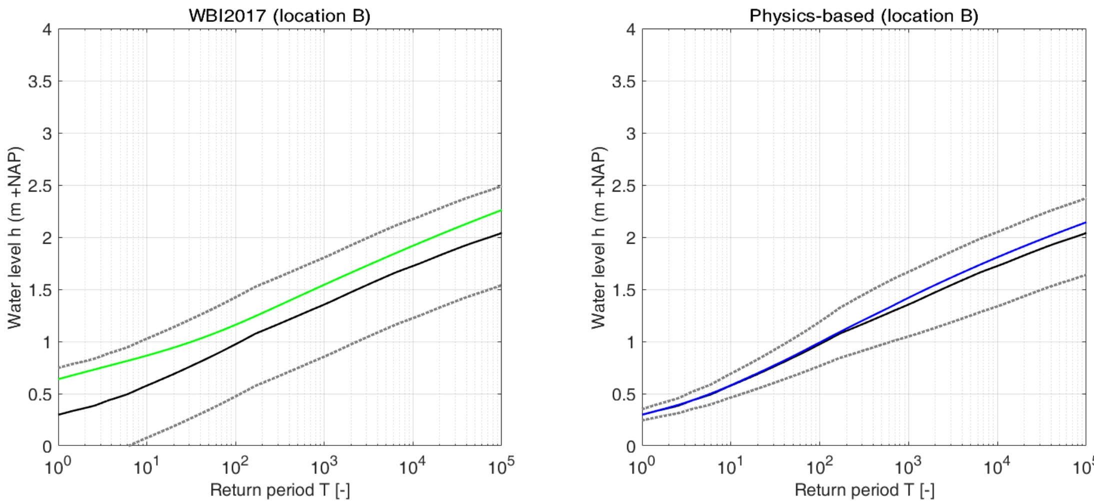


Figure 4.16: Water level frequency lines with different uncertainty modelling methods at location B. Black line is calculated without model uncertainty, the green line according to the WBI2017 method (left graph) and the blue line according to the physics-based method (right graph)

Table 4.4: The net effect in centimetres for two different uncertainty modelling techniques at different return periods at location B. The last row

	T = 10 years	T = 100 years	T = 1 000 years	T = 10 000 years
WBI2017	29.0	18.5	19.0	19.5
Physics-based	0.5	1.5	6.5	8.5
Difference	28.5	17.0	12.5	11.0

The 95 % confidence bounds of the WBI2017 method at location B even consist of water levels that are below the lake level. This caused by the principle that the standard deviation is based on water level uncertainties during extreme conditions. In reality the water level uncertainty is smaller for lower return periods, because model outcomes can be validated by measurements. These kind of artefacts do not occur with the physics-based approach.

4.4.1 Practical applicability in the Netherlands

The decimate height at locations along the lake area varies from approximately 0.2 metre till 0.8 metre according to the HR2006 (Bouw, 2008). It depends on the physical processes that shape the water level frequency line: lake levels and/or wind. Wind dominated locations have got a larger decimate height than lake level dominated locations. Also within the wind-dominated locations the decimate height varies significantly.

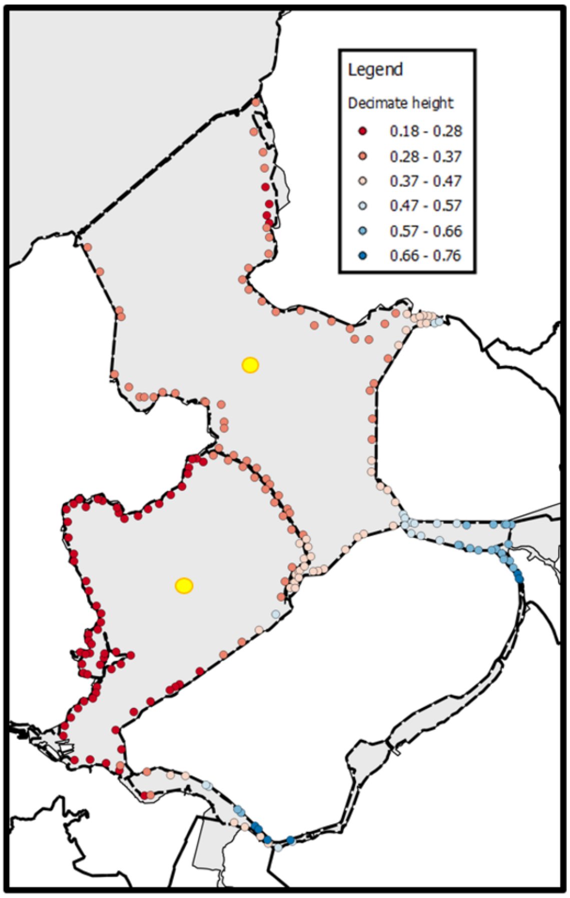


Figure 4.17: Spatial variation of the decimate height in the lake area. The two yellow dots in the middle of the lakes indicate the locations of the centre of gravity of the IJsselmeer and Markermeer (source of data: Bouw (2008)).

The two yellow circles indicate the centre of gravity of both lakes. If the wind blows over the lake, the water level tilt occurs around the centre of gravity of the water surface area. Locations further away from the centre of gravity, given a wind direction, result in larger wind set-ups and are also more sensitive to the wind speed. The highest tilt of the water level in the lake occurs in combination with west and west-north-westerly winds. The blue points in the Marker- and IJsselmeer are in the west-east direction furthest away from the centre of gravity. Winds from the west and west-northwest direction will contribute the most to the normative water levels and the decimate height at these locations is therefore larger compared to other locations. The modelled water level frequency lines in this study are in line with these findings.

4.5 Discussion

The physical model that is used consists of many simplifications, however, the most relevant aspects concerning uncertain parameterisation are captured well, like the modelling of the drag coefficient. In reality, time variation processes and 2D flow patterns will occur, but the parameterisation of the drag coefficient in a 2D hydrodynamic model is the same. Therefore, the results provide relevant information about the processes that are related to model uncertainties.

It has been shown that the current WBI2017 method does not agree with the physics-based approach. It is not valid to choose one constant standard deviation of the water level for wind dominant locations, considering the uncertainty of the drag coefficient. The 1D model is not able to assess the effect of all uncertainty sources by sufficient substantiation, due to the assumptions and simplifications. However, this model provides some insights on system behaviour and in addition qualitative information can be given to explain the issue regarding wind transformation. The uncertainty related to the wind transformation of potential wind to open-water wind (standard deviation of the water level of 15 cm) is the second important uncertainty source. The wind set-up is smaller at wind dominant locations closer to the centre of gravity, but also the sensitivity of the water level w.r.t. the wind speed is smaller. Figure 4.18 shows the wind set-up as function of the wind speed and the 95% confidence bounds, due to the uncertain drag coefficient. The fact is, the wind set-up is less sensitive to the wind speed for locations closer to the centre of gravity (this is also shown in Figure 4.12). Therefore, water levels at locations with a low decimate height are also less sensitive to the errors in the wind transformation. This is contrary to the inclusion of model uncertainty according to WBI2017.

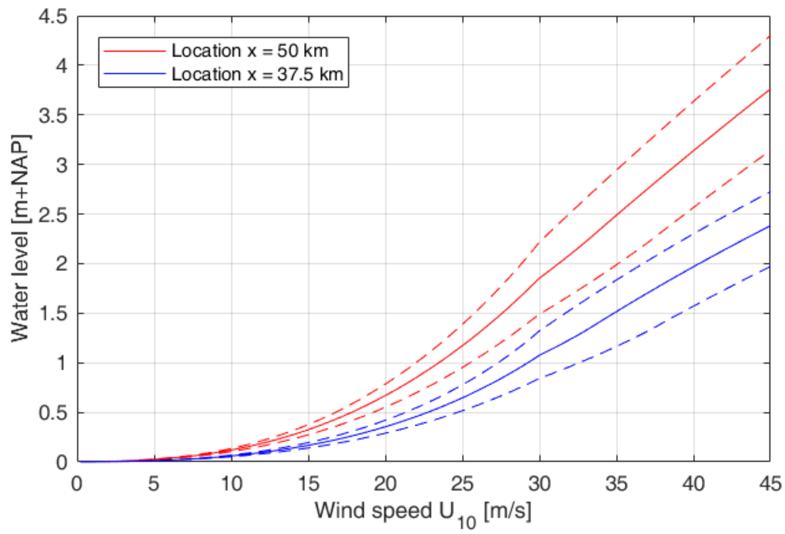


Figure 4.18: Sensitivity of the water levels w.r.t the wind speed for different locations in a closed basin.

4.6 Conclusions

First the research question related to the lake area is recalled, after which it is evaluated:

> *How do dominant uncertain model/input parameters effect the water level frequency lines for the lake area and is this consistent with the current WBI-2017 uncertainty modelling technique?*

A one-dimensional semi-analytical model is used to determine wind set-up in a 1D closed basin where a wind blows over the basin for which a stationary situation is established. The wind speed is considered as basic stochastic variable and water level frequency lines can be drawn by translating the wind speed to water levels at different locations along the closed basin. The physics-based method, where an additional parameter in the model is considered to be stochastic, is compared to the WBI2017 model uncertainty method. The parameter that is considered stochastically in the physics-based method should reflect the dominant source of model uncertainty in the lake area.

The quantification of uncertainty sources in the lake area are retained from Chbab (2014) and wind related uncertainties result in the largest contribution to the model uncertainty. Normative lake levels are obtained from statistical analysis and therefore the model uncertainties related to lake level dominated locations are relatively small. Modelling of the drag coefficient appears the most important uncertainty source for wind-dominated locations in the lake area.

The wind drag coefficient in hydrodynamic models is often computed by the capped Wu (1982) formula. The empirical b parameter in the Wu formula is considered as a stochastic variable and therefore the uncertainty in the drag coefficient becomes larger for higher wind speeds. The quantification is performed pragmatically by matching the 95 % confidence bounds of the water level for the physics-based method and WBI2017 method.

The WBI2017 method assumes a uniform standard deviation of the water level for all wind-dominated locations, which is also independent of the return period. The decimate heights along a lake become larger for locations that are further away from the centre of gravity of the water surface, where the wind set-up is the largest. Geerse and Rongen (2016) derived the relation between the net effect of the WBI2017 model uncertainty and decimate height: lower decimate heights (closer to the centre of gravity) result in a larger net effect. The physics-based approach showed that the water level uncertainty is larger for higher decimate heights and vice versa. It is not valid to choose one uniform standard deviation of the water level for all wind-dominated locations, as it is done for WBI2017.

5. Conclusions and recommendations

This chapter summarizes the conclusions from the research questions and reflects on this thesis' objective. Next, the recommendations are listed, where distinction is made between policy recommendations and recommendations for further research. The policy recommendations are aimed to advise Rijkswaterstaat in addressing the model uncertainty in WBI2023 based on the conclusions of this study. Possible next steps are listed to better comprehend the inclusion of model uncertainty. Recommendations for further research aims to address subjects where lack of knowledge can hinder this process.

5.1 Conclusions

The objective of this thesis was to analyse how model uncertainties effect the water level frequency line in different water systems. The comparative analysis provided insights on the model performance of Hydra-NL and explored the effect of model uncertainty on water level frequency lines for all fresh water systems. In order to possibly overcome limitation of the WBI2017 method, the river and lake systems were analysed in detail by applying a semi-analytical model that describes the system behaviour based on a simplified physical equations

5.1.1 Summarizing conclusions

Performance of Hydra-models

Estimated exceedance probabilities of measured water levels in the tidal river area and lake area are often overestimated by the Hydra-model even without considering model uncertainty. When model uncertainties are included, the overestimations become even larger. The inclusion of model uncertainty sometimes gives an incorrect representation of the underlying physical processes. The model uncertainty in WBI2017 is quantified for each sub-area where the same hydraulic processes predominantly determine the water levels. The selection of one uniform quantification of model uncertainty in these sub-areas results in different net effects, because of the dependency on the decimate height. In addition, the net effect of model uncertainty is larger for water levels that are reduced by flood mitigation projects, like flood channels and storm surge barriers. This conflicts with the physics and is further elaborated below.

Modelling uncertainty in the upper river area

An average river profile is modelled properly by the WBI2017 method. However, behaviour and uncertainty in water levels in rivers is a dynamic process. Based on the findings of this study, it can be concluded that one uniform standard deviation of the model uncertainty, that is independent of the return period, will not adequately reflect the situation in the upper river area because of two reasons. Firstly, when the cross-sectional profile is wider than the average profile according to the physics-based method, net effect of the model uncertainty is smaller. Therefore, the WBI2017 method overestimates the model uncertainty in these cases. Secondly, according to the physics-based method, the water level uncertainty near flood channels and retention areas is discharge dependent and varies along the water level frequency line. These deviations are caused by the environment/geometry that influence the water levels at specific discharge levels during a flood wave. Peak water levels cannot become higher during a flood wave, because of the physical processes that occur. The location (locally/micro-level and not meso-level) and discharge dependent water level uncertainty are not integrated in the WBI2017 method. These restrictions are justified by incorporating the hydraulic roughness as additional stochastic variable in deriving water level frequency lines.

Modelling uncertainty in the lake area

For wind-dominated locations in the lake area, the decimate height varies depending on the distance to the centre of gravity of the water surface of the corresponding lake. The physics-based method shows that the net effect of model uncertainty decreased when the decimate height becomes smaller. Again, it is not valid to choose one uniform standard deviation of the water level for al wind-dominated locations, as was done in the WBI2017 method. The physics-based method overcomes this limitation by adjusting the water level uncertainty for the wind-dominated locations in the lake area.

5.1.2 Reflecting objective

The objective of this research was formulated in Chapter 1 as follows:

> *(1) Analyse how model uncertainties affect the water level frequency line in different water systems and (2) provide insight into different methods to quantify model uncertainties in Hydra-models.*

The comparative analysis showed that the inclusion of model uncertainty consistent with the WBI2017 method does not correctly represents the physical processes that shapes the water level frequency line. This is supported by the results obtained by the developed semi-analytical models for the upper river and lake area. This study showed that incorporating the most important source of model uncertainty as an additional stochastic variable results in physically better substantiated water levels. In the upper river area and wind-dominated lake area, it is possible to define a single parameter which should be taken into account in the physics-based approach. In other water systems it becomes more complex, because multiple sources of uncertainty are present. Still, the most important uncertainty sources could considered stochastically, but computational effort would become the limiting factor at this moment.

5.2 Recommendations

5.2.1 Policy recommendations

The developed model in this study provides estimates as results are based on simplified physical models. Therefore, the findings need to be verified using more sophisticated tools to model water levels: 2D numerical model, like WAQUA or D-Flow Flexible Mesh. However, this study shows valuable insights in system behaviour and examined different methods to account for model uncertainty. Recommendations related to the uncertainties in hydrodynamic modelling of water levels in the upper river area are:

- **Introduce the hydraulic roughness as an additional stochastic variable to include model uncertainty**: The major uncertainties related to water levels predictions in the upper-river area are caused by the calibration approach, hydraulic roughness (main channel and floodplains) and morphological changes under extreme conditions. These factors are related to the hydraulic roughness of the main channel and floodplains. This calibration process introduces an implicit correlation between all parameters and processes in the model that cannot be ignored (Tijssen et al., 2014). Wrong model schematizations are compensated by the main channel roughness during the calibration process after which the roughness parameters are extrapolated to higher discharge levels. It is recommended to introduce the main channel roughness and floodplain roughness as an additional stochastic variable to include model uncertainty. These stochastic variables can be fully dependent for practical reasons and to reduce the computational effort.
- **Use of a gradually increasing (w.r.t. the discharge level) hydraulic roughness uncertainty**: This study showed the possible water level uncertainty that results from uncertainty in main channel roughness. The quantification of this uncertainty is based on extreme conditions, but the uncertainty in hydraulic roughness is lower in the case of discharge levels closer to the calibration and validation data range. In the current study, the standard deviation of the stochastic hydraulic roughness was chosen independent of the discharge, which resulted in an incorrect representation of uncertainties for lower return periods. Therefore, it is advisable to carry out model runs using D-Flow Flexible Mesh, where a discharge dependent multiplier of the roughness can be introduced. In this way, a lower multiplier can be applied in the case of discharge levels that lie around the calibration range.

Recommendations related to the uncertainties in hydrodynamic modelling of water levels in the lake area where wind is the dominant threat that determines the water levels:

- **Introduce an empirical parameter in the drag modelling formula as an additional stochastic variable**: The system behaviour of wind-dominated locations was assessed in this study. It was shown that the water level uncertainty decreases when decimate heights are lower. The decimate height varies strongly in the lake area and therefore the quantification of the WBI2017 model uncertainty becomes rather pragmatic if it needs to be adjusted to the decimate height. Therefore, it is recommended to introduce an additional stochastic variable in the Hydra-models. The empirical b parameter in the capped Wu (1982) seems to be a suitable parameter.
- **Capping of the wind drag coefficient**: van Vledder (2017) stated that a reasonable safe choice for parameterization of the drag coefficient is the Wu (1982) formula capped at wind speeds above 30 m/s. Considering the literature and the inventory of measured drag coefficients in Zijlema et al. (2012) this upper limit seems to be rather conservative. These kinds of conservatisms result in model overestimations when conditions are more extreme, especially if a model uncertainty is added as well. Parameterization is difficult if there is no consensus on physical behaviour in the academic community, but both being conservative and including model uncertainty does not contribute to reliable model outcomes. A choice should be made between either (1) being conservative in the drag modelling without including additional model uncertainty related to the drag coefficient or (2) using the best estimation of the drag coefficient and additionally including model uncertainty.

Some recommendations related to the potential usage of the WBI2017-method:

- **The use of biased model uncertainty for long term predictions**: The bias of the model uncertainty could be used for long term predictions of hydraulic load levels. The river system changes in time; the occurrence of bed degradation of 1 - 2 cm per year in the main channel is one of those variations (Blom, 2016). On the other side, flood waves deposit sand over the floodplains, which can result in accretion (Peters & Kurstjens, 2012). Hydrodynamic models do not calculate morphological processes and therefore a bias in the model uncertainty could be used to account for these changes in the river system. However, for the contemporary assessment of flood defences these trends should be handled carefully, because maybe next year the one in ten thousand discharge will occur in the river Rhine
- **Account for systematic overestimations of Hydra-models**: At several locations the Hydra-model seems to overestimate the actual water levels for model runs, even without including model uncertainty. After the inclusion of model uncertainty, this difference becomes even larger. It is reasonable to assume a bias for the model uncertainty at areas where the model overestimates the actual water levels, for example at Nijkernauw. The hydrodynamic model for the lake area is validated using measured water levels at locations around the Markermeer, but model outcomes can deviate significantly depending on the location. The overestimations are probably caused by the assumptions associated with the uniform spatial wind fields and the evolution of the wind fields above the Markermeer during the time. The largest standard deviation associated with the model uncertainty is chosen in this area, while the model outcomes are already conservative. This conservatism can be compensated by introducing a bias.
- **Reconsider the boundaries between the sub-areas in the tidal river area:** It is advisable to reconsider the boundaries between the sub-areas in the tidal river area. The water level frequency lines of locations in the tidal river area that are considered to be discharge dominant are still affected by storm surges at sea. Model uncertainties are underestimated by WBI2017, because uncertainties related to discharge dominated locations are smaller than locations that are also influenced by storm surge.

5.2.2 Further research

In this study basic hydrodynamic processes were modelled with semi-analytical models and the impact of different uncertainty modelling techniques was assessed. Despite these efforts, some aspects remain unclear and will require more research. Therefore recommendations and opportunities for further research are indicated based on the findings in this report:

- **Study uncertainties related to the discharge distribution**: This study excluded uncertainties associated to the discharge distribution at the bifurcations. In order to perform first exploratory analyses regarding dealing with uncertainties related to the discharge distribution, the same approach with some extensions can be used. The discharge distribution is the product of the discharge capacity and resulting water levels in the Rhine branches nearby the bifurcations. To estimate the uncertainties related to the discharge distribution, more information about the roughness uncertainties in the Rhine branches and its possible correlations should be obtained.

  Uncertainties in the discharge distribution depend on other model uncertainties, but can also counteract water level uncertainties in the Rhine branches. If the hydraulic roughness of one river branch downstream of a bifurcation is higher than estimated, the water levels in this branch will be higher. At the same time the uncertainty of the water level is compensated by the discharge distribution, because more discharge will flow into the other branch. Therefore, the uncertainty in one Rhine branch could be overestimated, but be underestimated in another one. The spatial correlation makes it difficult to deal with uncertainties related to the discharge distribution.

- **Quantify model uncertainties during calibration process**: Investigate the uncertainties related to the calibration parameter and try to estimate the model error for different flow conditions. Wrong model parameterisations are all compensated by the hydraulic roughness of the main channel (calibration parameter). Extrapolation of the hydraulic roughness is unreliable, because it consists of both a physical component and a model error component. The extrapolation of these elements are now considered together, but these elements are not, by definition, correlated and their behaviour may differ under extreme conditions. By decoupling the function of the calibration parameter, the extrapolation could become more transparent.

  Uncertainties in the roughness coefficient can also be location dependent. Tijssen et al. (2014) showed that different calibration techniques resulted in different predicted water levels in the extrapolation zone. If the floodplain roughness of a wide river is estimated incorrectly, the error in the extrapolated water levels become larger compared to small floodplains. This is also illustrated in Figure 3.1. Therefore, the uncertainty in the hydraulic roughness can also vary spatially. These uncertainties can be assessed by analysing the calibration procedure.

  - **Alternative calibration procedure: characterize model parameters based on physics and introduce a "model error parameter"**:
    Hydrodynamic river models are calibrated and validated using measured data, to assure an acceptable level of performance. Possible errors in the schematization are all compensated by the roughness of the main channel and can sometimes result in physically unrealistic roughness values. For example, calibration of a WAQUA Rhine-model for the normal discharge class (discharge for which the floodplains are inundated) was impracticable (Becker, 2012) and resulted in roughness values in the Pannerdensch Kanaal that correspond to a riverbed made of concrete and gravel (based on tables for Manning values Chow (1959)). These values are physically incorrect, but because of the calibration process, we still accept these unreal values in our models. The calibrated roughness values for the high discharge class correspond better with physics and these values are used for extrapolation to extreme conditions.

    The calibration process introduces an implicit correlation between all parameters and processes in the model, which makes it more complex to interpret. It might also result in physically unreal values for the calibration parameter. Therefore, compensation of wrong model schematizations by one of the most sensitive parameter in the model (main channel roughness) has no meaning under extreme conditions (outside the calibration and validation data range). It is unknown which part of the model is wrongly schematized and therefore introducing a "model error parameter" as calibration parameter can be an alternative to existing calibration procedures. The roughness of the main channel can be parametrized by using roughness models that predict bed form roughness (Dutch rivers are bed form-dominated rivers) based on measured bed form and flow characteristics (Warmink et al., 2013). The behaviour of the model bias for different discharge levels can be analysed and extrapolated. By doing so, the developed model represents the underlying physical processes more adequately.

- **Quantify the drag coefficient uncertainty with a truncated distribution**: The capping of the drag coefficients sets a physical limit, but this limit is exceeded by introducing model uncertainty in the physics-based method. Therefore, it could be reasonable to introduce a truncation of the drag coefficient for wind speeds where the drag coefficient exceeds the upper limit. It is interesting to see how the water levels will react on such a measure.

- **Investigate the selection of probability distributions**: Hydrodynamic processes in a river consist of non-linear correlations, of which the relation between flow velocity and hydraulic roughness is one. The selection of the probability density function that is imposed on the hydraulic roughness influences the uncertainty in the water level. The effect of a normal and lognormal distribution of the hydraulic roughness on the water level frequency lines is assessed (see Appenix C). Different probability distributions on different roughness parameters (Chézy, Strickler and Manning) result in varying water level skewnesses.It is recommended to use 2D models to investigate this effect in more detail.

- **Use flood channels more frequent to reduce uncertainties**: The hydrodynamics near flood channels are rather complex and their behaviour is mainly based on models. As the flood channel near Veessen-Wapenveld only functions once in every hundred years, no measurements of flow characteristics are performed. This, while during an extreme flood wave, a large part of the discharge will flow through the flood channel. Errors related to the schematization of the flood channel cannot be reduced by calibration or validation. By using a flood channel more frequently, uncertainties related to the roughness of the flood channel can be reduced by measuring the flow characteristics.

- **Reconsider the independent summation of uncertainty sources**: Sensitivity studies described the water level uncertainty for different uncertainty sources by a normal distribution with a standard deviation. Depending on the hydraulic processes in an area, these sources are added by assuming they are independent of each other. This assumption might overestimate the model uncertainties. For example, the model uncertainty for locations where wind and lake level are dominant is larger than a wind-dominated location. For wind and lake level dominated locations, the decimate height is smaller and the water levels are less sensitive to threats compared to wind-dominated locations. Still, the model uncertainty is chosen to be larger which also might conflict with the physics. The uncertainty sources might be overestimated for these locations, because the sensitivity analysis addresses the sources individually. This effect might be reduced if the sources of uncertainties are combined in one estimation of uncertainty. By introducing additional stochastic variables that are incorporated in the hydrodynamic model, the relative importance of these sources can be addressed better.

References

Agtersloot, R. & Paarlberg, A. J. (2016). Waqua productieberekeningen ijssel-vechtdelta (wettelijk beoordelingsinstrumentarium 2017).

Augustijn, D., Huthoff, F. & van Velzen, E. (2008, 9). Comparison of vegetation roughness descriptions. In M. Altinakar, M. Kokpinar, I. Aydin, S. Cokgor & S. Kirkgoz (Eds.), *Proceedings of river flow 2008 - fourth international conference on fluvial hydraulics, 3-5 september 2008 Çe¿me, turkey.* (pp. 343–350). Kubaba Congress Department and Travel Services.

Becker, A. (2012). Rijn-modellen 5de generatie (modelopzet, kalibratie en verificatie waqua). *Deltares.*

Beckers, J., Geerse, C. P. M., Kramer, N., Nicolai, R. & Wojciechowska, K. (2009). Uncertainties of the hydra models sbw belastingen. *Deltares.*

Blom, A. (2016). Bed degradation in the rhine river. *Waterviewer.*

Bottema, M. (2007). Measured wind-wave climatology lake ijssel (nl). *Report RWS RIZA 2007.020.*

Bouw, R. (2008). Decimeringshoogten tmr2006. In R. P. N. Pater (Ed.), . Witteveen en Bos.

Chbab, E. H. (2014). Modelonzekerheid in lokale waterstanden l.jsselrneer,markermeer, ijsseldelta en vechtdelta (ijmvd).

Chbab, E. H. (2015). Modelonzekerheid in lokale waterstand voor het rmm gebied. deltares memo.

Chbab, E. H. (2017). Basisstochasten wbi-2017 - statistiek en statistische onzekerheid.

Chbab, E. H., den Bieman, J. & Groeneweg, J. (2017). Hydraulische belastingen rijntakken en maas - wettelijk beoordelingsinstrumentarium wbi-2017.

Chbab, E. H. & de Waal, H. (2017). Achtergrondrapport hydraulische belastingen wettelijk beoordelingsinstrumentarium 2017. *Deltares 1230087-008.*

Chbab, E. H. & Groeneweg, J. (2017). Modelonzekerheid belastingen - wettelijk beoordelingsinstrumentarium wbi-2017.

Chow, V. T. (1959). Open-channel hydraulics. *McGraw-Hill, New York City, New York.*

de Waal, H., Spruyt, A. & Smale, A. (2013). Uitgangspunten productieberekeningen wti2017 - aansturing, schematisaties en uitvoerlocaties. *Deltares 1207807-009.*

de Waal, J. P. (2003). Windmodellering voor bepaling waterstanden en golven: Een analyse van de bouwstenen. *RIZA werkdocument 2003.118x.*

Diermanse, F. (2017). Wbi - onzekerhede; overzicht van belasting-en sterkteonzekerheden in het wettelijk beoordelingsinstrumentarium. *Deltares.*

Duits, M. T. (2006). Hydra-bt voor rbso: Testrapport versie 1.2,. *HKV Consultants.*

Duits, M. T. & Noortwijk, J. M. v. (1999). Nauwkeurigheidsanalyse ruimte voor de rijntakken. *Land + water : magazine voor civiele- en milieutechniek.*

Fernandes, J. N., Leal, J. B. & Cardoso, A. H. (2012). Analysis of flow characteristics in a compound channel: comparison between experimental data and 1d numerical simulations. In S. Rauch & G. M. Morrison (Eds.), *Urban environment: Proceedings of the 10th urban environment symposium* (pp. 249–262). Dordrecht: Springer Netherlands. Retrieved from `https://doi.org/10.1007/978-94-007-2540-9_23` doi: 10.1007/978-94-007-2540-9_23

Geerse, C. P. M. (2006). Hydraulische randvoorwaarden 2006 vecht- en ijsseldelta, statistiek ijsselmeerpeil, afvoeren en stormverlopen voor hydra-vij,. *RWS RIZA werkdocument.*

Geerse, C. P. M. (2013). Belastingmodellen wti2017 - gevoeligheidsanalyses en adviezen voor eventuele aanpassingen aan hydra-ring. *HKV PR2647.40.*

Geerse, C. P. M. (2016). Werkwijze uitintegreren onzekerheden basisstochasten voor hydra-nl - afvoeren, meerpeilen, zeewaterstanden en windsnelheden (update februari 2016). *HKV PR3216.10.*

Geerse, C. P. M. & Rongen, G. (2016). Invloed onzekerheid op bepalen van waterstandsverlopen - verandering percentielen van afvoer en meerpeil bij meenemen statistische en modelonzekerheid waterstand. *HKV PR3280.20.*

Geerse, C. P. M., Rongen, G. & Ouwerker, S. (2017). Modelonzekerheid veessen-wapenveld - invloed hoogwatergeul op de grootte van de modelonzekerheid. *HKV PR 3479.10.*

Geerse, C. P. M., Slomp, R. M. & De Waal, J. P. (2011). Hydra-zoet for the fresh water systems in the netherlands - probabilistic model for the assessment of dike heights. *PR2168 HKV rapport voor Rijkswaterstaat, Waterdienst;.*

Gringorten, I. I. (1963). A plotting rule for extreme probability paper. *Journal of Geophysical Research*, *68*(3), 813–814. Retrieved from http://dx.doi.org/10.1029/JZ068i003p00813 doi: 10.1029/JZ068i003p00813

Hegnauer, M., Beersma, J. J., van den Boogaard, H. F. P., Buishand, T. A. & Passchier, R. H. (2014). Generator of rainfall and discharge extremes (grade) for the rhine and meuse basins - final report of grade 2.0.

Huthoff, F. (2004, 06). Sensitivity analysis of floodplain roughness in 1d flow.

Hydra-ring 17.1 - technical reference manual (Tech. Rep.). (2016). Deltares.

Janssen, J. P. F. M. & Jorissen, R. E. (1992). Integrating forecast effects and operational behaviour in designing the rotterdam storm surge barrier. In A. J. Saul (Ed.), *Floods and flood management* (pp. 327–339). Dordrecht: Springer Netherlands. Retrieved from https://doi.org/10.1007/978-94-011-1630-5_22 doi: 10.1007/978-94-011-1630-5_22

Jonkman, S. N., Steenbergen, R. D. J. M., Morales-Nápoles, O., Vrouwenvelder, A. C. W. M. & Vrijling, J. K. (2016). *Probabilistic design: Risk and reliability analysis in civil engineering* (M. H. G. Baas & M. A. v. d. Lugt, Eds.). Delft University of Technology.

Julien, P. Y., Klaassen, G. J., ten Brinke, W. B. M. & Wilbers, A. W. E. (2002, 12). Case study: Bed resistance of rhine river during 1998 flood. , *128*.

Klis, H. v. d. (2003). *Uncertainty analysis applied to numerical models of river bed morphology.* DUP Science.

Knighton, D. (1998, 01). Fluvial forms and processes : A new perspective / d. knighton.

Kok, M., Stijnen, J. W. & Silva, W. (2003). Uncertainty analysis of river flood management in the netherlands. *European safety and reliability conference*.

Kramer, N., Smale, A., den Bieman, J. & Groeneweg, J. (2017). Hydraulische belastingen meren: Wettelijk beoordelingsinstrumentarium 2017. *Deltares 1230087-001*.

Lodder, C. (2006). Achtergrondrapport hr2006 voor de meren. hydraulische randvoorwaarden 2006 voor het markermeer. *RWS RIZA rapport 2006*.

Manning, R. (1891). On the flow of water in open channels and pipes. *Trans. Inst. Civ. Eng. Ire.*, *20*, 161-207.

Nicolai, R., Wojciechowska, K., Vrouwenvelder, A. & Steenbergen, H. (2011). Effect onzekerheden op de hydraulische randvoorwaarden - kwantitatieve analyse van onzekerheden. *HKV PR1831.10*.

Ogink, H. J. M. (2006). Onzekerheid afvoerverdeling splitsingspunten rijn. *Rijkswaterstaat RIZA*.

Oreskes, N., Shrader-Frechette, K. & Belitz, K. (1994). Verification, validation, and confirmation of numerical models in the earth sciences. *Science*, *263*(5147), 641–646. Retrieved from http://science.sciencemag.org/content/263/5147/641 doi: 10.1126/science.263.5147.641

Pappenberger, F., Matgen, P., Beven, K. J., Henry, J.-B., Pfister, L. & de Fraipont, P. (2006). Influence of uncertain boundary conditions and model structure on flood inundation predictions. *Advances in Water Resources*, *29*(10), 1430 - 1449. Retrieved from http://www.sciencedirect.com/science/article/pii/S0309170805002666 doi: https://doi.org/10.1016/j.advwatres.2005.11.012

Parmet, B. W. A. H., van de Langemheen, W., Chbab, E. H., Kwadijk, J. C. J., Diermanse, F. L. M. & Klopstra, D. (2001). Analyse van de maatgevende afvoer van de rijn te lobith: Onderzoek in het kader van het randvoorwaardenboek 2001. *RIZA rapport 2002.012*.

Peters, B. & Kurstjens, G. (2012). Actief zand (deelrapport project 'rijn in beeld') - het herstel van oeverwallen en stroomdalflora langs de rijntakken.

Prinsen, G., van den Boogaard, H. & Hegnauer, M. (2015). Onzekerheidsanalyse hydraulica in grade.

Productspecificaties digitaal topografisch bestand (Tech. Rep.). (2017). Rijkswaterstaat.

Sterl, A. (2017). *Drag at high wind velocities - a review* (Tech. Rep.). Ministry of Infrastructure and the Environment, Royal Netherlands Meteorological Institute (KNMI).

Stijnen, J. W., Kok, M. & Duits, M. T. (2002). Onzekerheidsanalyse hoogwaterbescherming rijntakken, onzekerheidsbron nen en gevolgen van maatregelen. *HKV Consultants PR464*.

Tijssen, A., Becker, A., Stuparu, D. & Yossef, M. (2014). Quantification of model uncertainty for waqua for the upper river area. *Deltares report 1207807-002*.

van der Mheen, M. (2013). Markermeer waqua model 5de generatie - modelopzet en validatie. *Deltares report, 1207880-009*.

van Rijn, L. C. (1993). *Principles of sediment transport in rivers, estuaries and coastal seas* [Book; Book/Illustrated]. Amsterdam : Aqua Publications. (Includes bibliographical references) van Vledder, G. P. (2017). Stappenplan aanpassing hydraulische randvoorwaarden op basis van nieuwe inzichten winddrag.

Velzen van, E. H., Jesse, P., Cornelissen, P. & Coops, H. (2003). Stromingsweerstand vegetatie in uiterwaarden / deel 1 handboek. *RIZA*.

Voorspellen afvoer nevengeulen (Tech. Rep.). (2010). DHV.

Vrijling, J. & Van Gelder, P. (2002). *Probabilistic design in hydraulic engineering.* Civil Engineering and Geosciences - Hydraulic Engineering.

Warmink, J. J., Van der Klis, H., Booij, M. J. & Hulscher, S. J. M. H. (2007, 11). Uncertainty in water level predictions due to various calibrations. In C. Pahl-Wostl (Ed.), *Proceedings caiwa 2007, 12-15 november 2007, basel, switserland* (pp. 1–18). NeWater.

Warmink, J. J., Van der Klis, H., Booij, M. J. & Hulscher, S. J. M. H. (2011, 1st Jan). Identification and quantification of uncertainties in a hydrodynamic river model using expert opinions. *Water Resources Management*, *25*(2), 601–622. Retrieved from https://doi.org/10.1007/s11269-010-9716-7 doi: 10.1007/s11269-010-9716-7

Warmink, J. J., Van der Klis, H., Booij, M. J. & Hulscher, S. J. M. H. (2013). Quantification of uncertainty in design water levels due to uncertain bed form roughness in the dutch river waal. *Hydrol Process*. doi: 10.1002/hyp.9319.

Weber, J. & Menéndez, A. (2004). Performance of lateral velocity distribution models for compound channel.

Wemelsfelder, P. J. (1939). Wetmatigheden in het optreden van stormvloeden. *De Ingenieur*, *54*((9)), B.31-35.

Wiegmann, N., Perluka, R., Oude Elberink, S. & Volgelzang, J. (2005). Vaklodingen: de inwintechnieken en hun combinaties. (vergelijking tussen verschillende inwintechnieken en de combinaties ervan). AGI Rijkswaterstaat.

Wu, J. (1982). Wind-stress coefficients over sea surface from breeze to hurricane. *Journal of Geophysical Research: Oceans*, *87*(C12), 9704–9706. Retrieved from http://dx.doi.org/10.1029/JC087iC12p09704 doi: 10.1029/JC087iC12p09704

Zijlema, M., van Vledder, G. P. & Holthuijsen, L. H. (2012). Bottom friction and wind drag for wave models. *Coastal Engineering*, *65*, 19 - 26. Retrieved from http://www.sciencedirect.com/science/article/pii/S0378383912000440 doi: https://doi.org/10.1016/j.coastaleng.2012.03.002

A. Room for the river and Meuse works

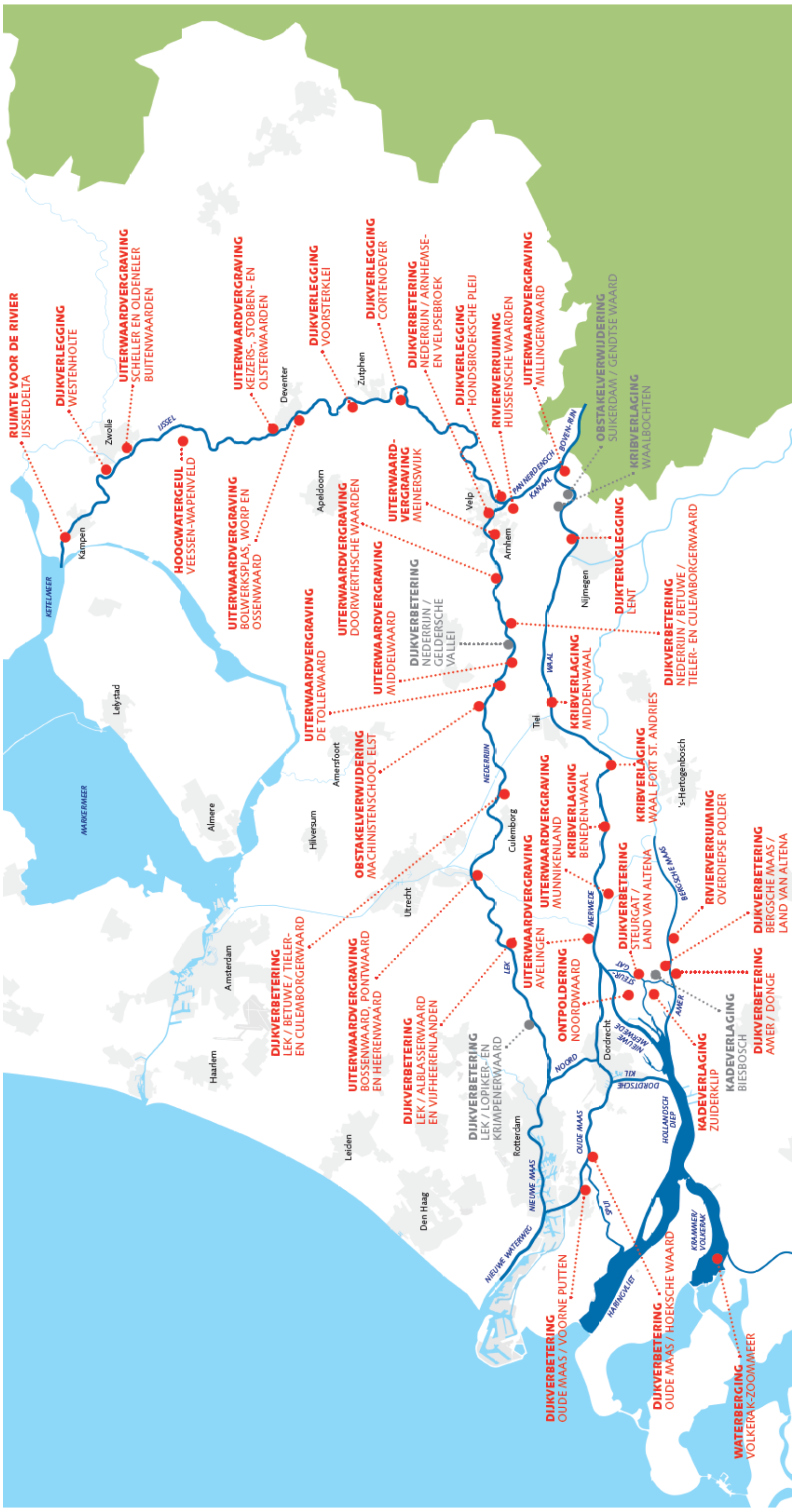


Figure A.1: Map of Room for the River sites in the Netherlands

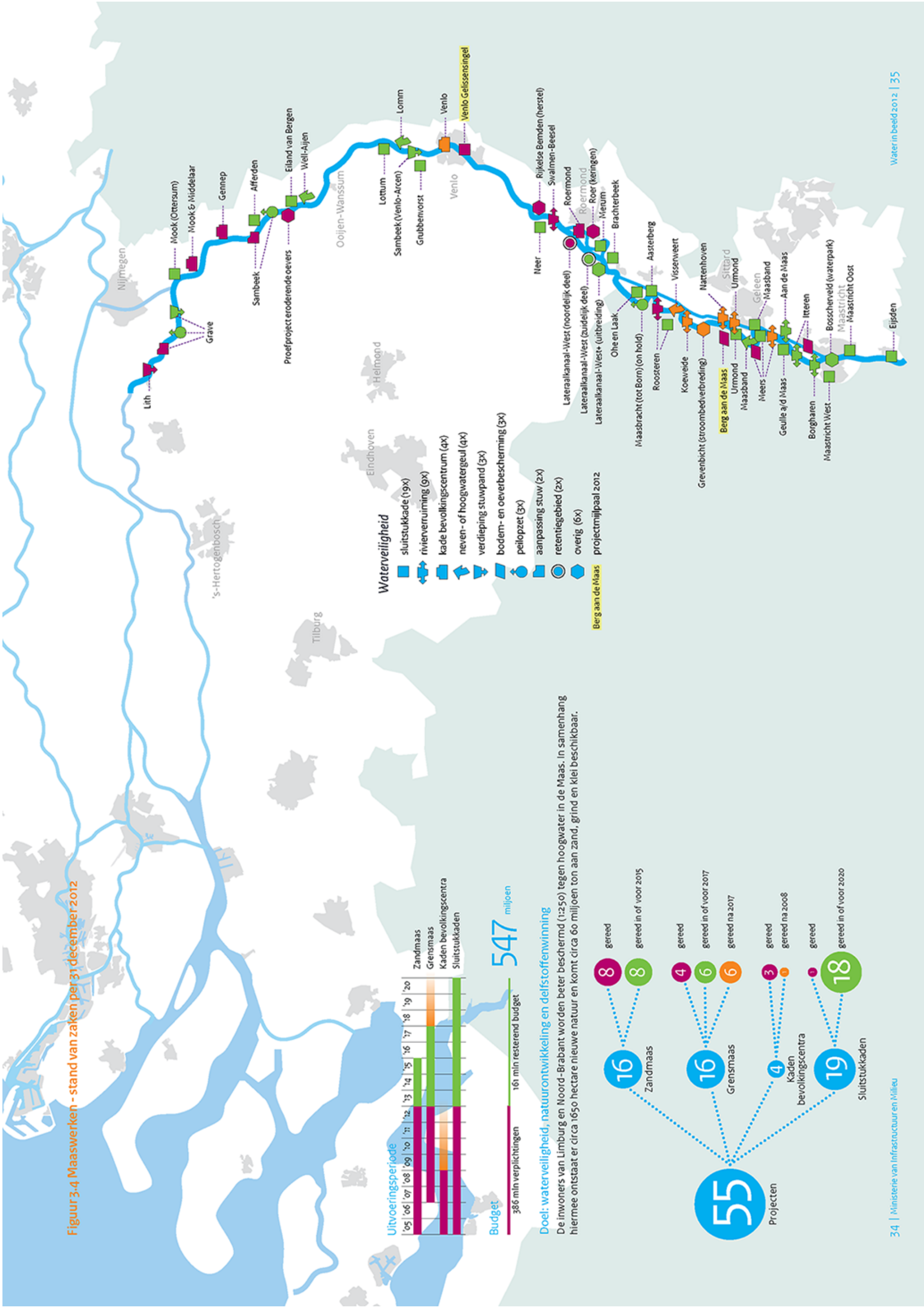


Figure A.2: Map of the Meuse works in the Netherlands

B. The effect of model uncertainty in the Dutch Rhine branches

According to WBI2017 the water level uncertainty in the Bovenijn, Waal and Nederrijn-Lek is quantified by a standard deviation of 15 centimetres. For the IJssel a standard deviation of 20 centimetres is chosen, because of the additional water level uncertainty related to the discharge distribution. The net effect of the model uncertainty consistent with WBI2017 at a return period of 1250 years for the Rhine branches can be seen in Figure B.1 (data obtain from Chbab et al. (2017)). The net effect tells a limited story, because it depends on the estimated standard deviation and the shape of the water level frequency line (see also Section 2.2.3). The inclusion of model uncertainty for the Bovenrijn, Waal and Nederrijn-Lek results in approximately the same water level increase in the branches. The larger water level standard deviation in the IJssel river contribute to the larger net effect of the model uncertainty. But the net effects in the IJssel vary spatially along the river, which are caused by changes in the slope of the water level frequency line.

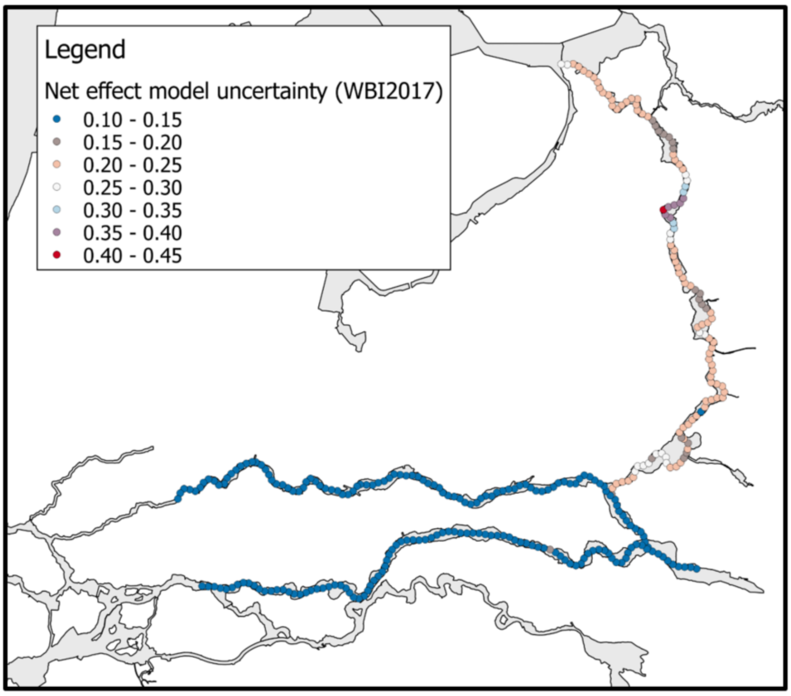


Figure B.1: The net effect of model uncertainty on water levels for the Rhine branches. The net effect is the difference in water levels derived with and without model uncertainty for a return period of 1250 years.

C. Various probability distributions for different hydraulic roughness parameters

C.1 Normal flow equations

For open channels, two well-known equations are commonly used to describe the relation between the mean flow field and channel resistance in the steady uniform case. Originally, the equations are an empirical result that gave validity in several case studies.

$$U = C \sqrt{RS_w} \quad \text{Chézy equation (1769)} \tag{C.1}$$

$$U = (1/n)R^{1/6} \sqrt{RS_w} \quad \text{Manning equation (1889)} \tag{C.2}$$

where U is the mean flow velocity, R is the hydraulic radius, S_w the water surface slope, C the Chézy coefficient and n the Manning's coefficient. C and n represents the channel resistance or hydraulic roughness and both can be expressed by the Nikuradse roughness height: $C = 18\log_{10}(12R/k_n)$ (White-Colebrook (1939)) and $n = k_n^{1/6}/25$ (Strickler (1923)). In literature $1/n$ is also noted by k_s which is called the Strickler coefficient. The Nikuradse roughness height can be used to transform the different roughness parameters into each other that are required in the normal flow equations.

The difference between the two normal flow approaches lies in the determination of the roughness coefficient. The Chézy coefficient is not constant during the time evolution of water levels in a river and depends on the hydraulic radius, while Manning's Roughness Coefficient is independent of the hydraulic radius. Manning's n can be determined by characteristics of the river and several researches investigated the Manning's coefficient and provided tables where the roughness coefficient can be read depending on e.g. the bottom material, shape of the river and vegetation.

C.2 Probability distributions

Probability distributions are imposed on different roughness parameters to see how they influence the resulting water levels. The Normal and Lognormal probability density functions are used to introduce model uncertainty, because it is easy to work with these distributions. This section gives the definitions of these probability density functions.

Normal distribution
The normal distribution is the most common distribution. The distribution function of a normal distribution has no closed form, so positive and negative outcomes can occur. The normal distribution emerges when a large number of independent random variables, from which none dominates any other, are created disregarding the output distributions of these variables (Jonkman et al., 2016). Therefore, physical quantities that are expected to be the sum of many independent processes (such as measurement errors) often have distributions that are nearly normal. This phenomena is known as the central limit theorem. The probability density function of a stochastic variable that is normally distributed is as follows:

$$f_X(x) = \frac{1}{\sigma_Y\sqrt{2\pi}} e^{\left\{-\frac{(x-\mu_Y)^2}{2\sigma_Y^2}\right\}} \tag{C.3}$$

where μ and σ denote the mean and standard deviation of the stochastic variable X and x stand for a realisation. The function is symmetric around the average value.

Lognormal distribution
A stochastic variable X has by definition a lognormal distribution if $Y = \ln(X)$ has a normal distribution. Often, a lognormal distribution is chosen for the roughness coefficient, because of the

semi-bounded property which ensure roughness coefficients cannot take on negative values. This corresponds to the physical property of the roughness coefficient that it is greater than zero. The probability density function of the lognormal distribution is defined as:

$$f_X(x) = \frac{1}{\sigma_Y x\sqrt{2\pi}} e^{\left\{-\frac{(\ln(x)-\mu_Y)^2}{2\sigma_Y^2}\right\}} \tag{C.4}$$

where σ_Y and μ_Y are the mean and standard deviation of the associated normal distribution. If the mean and standard deviation of X (the lognormal distribution) are known, the corresponding values for Y can be determined:

$$\begin{aligned} \mu_Y &= \ln\left(\mu_X\right) - \frac{1}{2}\ln\left(1+\frac{\sigma_X^2}{\mu_X^2}\right) \\ \sigma_Y &= \sqrt{\ln\left(1+\frac{\sigma_X^2}{\mu_X^2}\right)} \end{aligned} \tag{C.5}$$

C.3 Effect of different roughness distributions on water level skewness

The effect of different probability distributions and different roughness parameters are assessed in a simple case study where a rectangular cross-sectional profile without floodplains (width of 1000 metres) and a bed slope of 1/10000 is assumed. In these situations the hydraulic radius approximates the water depth and simplified normal flow equations are used where $R = h$. An average roughness coefficient of n=0.03 $s/m^{1/3}$ is assumed that is transformed to the other roughness parameters by means of the equation in section C.1. A variation coefficient of 10% is used to investigate how different probability distributions of the roughness parameters affect the water level distribution[4]. Three roughness parameters are considered to be stochastic: Manning coefficient (n), Srickler coefficients (k_s) and Chézy coefficient (C). The parameterisation of these stochastic variables can be found in Table C.1.

Table C.1: Characterisation of the stochastic roughness parameters

	Unit	Mean	Standard deviation (cv = 10%)
Manning coefficient	$s/m^{1/3}$	0.03	0.003
Strickler's coefficient	$m^{1/3}/s$	33.3	3.33
Chezy coefficient	$m^{1/2}/s$	51.0	5.1

The effect of the different probability distributions and roughness parameters on the water level distribution can be indicated by the skewness of the distribution. The skewness is a measure of the asymmetry of the probability distribution and can be calculated by using the estimates of the moments of a distribution (Vrijling & Van Gelder, 2002). In this case the stochastic variables are modelled by sample random numbers from a distribution (normal or lognormal). The central moments of the water level distribution can be determined from the samples of the stochastic roughness parameter that are transformed to water levels. The k^{th} central moment of the distribution of the water level (for k>1) is defined as:

$$m_k = \frac{1}{m}\sum_{i=1}^{m}(x_i-\mu)^k \tag{C.6}$$

where m is the sample size of the stochastic variable, x_i the realisations of the stochastic variable and $\mu = \sum_{i=1}^{n} x_i/n$ is the average of stochastic variable. The third central moment represents the skewness of the probability distribution of a stochastic variable. A distribution with a single peak with a skewness greater than zero slants to the right and smaller than zero to the left (see Figure

[4]The standard deviations of different roughness parameters are calculated using a variation coefficient (standard deviation divided by mean value). The same variation coefficient of different roughness parameters does not necessarily result in the same water level uncertainty. This can also be seen in Section C.4 where a variation coefficient of 10 % for Strickler's coefficient results in larger uncertainties in the water level.

C.1). The skewness of a symmetrical distribution such as an uniform or normal distribution equals zero.

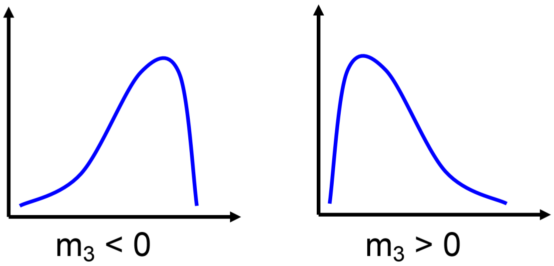


Figure C.1: Illustration of the skewness values and how they correlate with the shape of the probability distribution. (Source: Slides Extreme value modelling in hydraulic engineering by Prof. dr. ir. Pieter van Gelder)

The skewness can be normalised by using the variance (second central moment). The resulting normalised skewness is defined as:

$$b_1 = \frac{m_3^2}{m_2^3} \tag{C.7}$$

The skewnesses of the water level distributions are given in Table C.2. If the probability distribution of roughness parameter has no skew it does not mean that the resulting water level distributions are not skewed. In fact, the normal distributed roughness parameter Chézy or Strickler result in a more skewed water level distribution than a lognormal distribution roughnenss parameter. This is caused by the non-linear terms in the normal flow equations. The skewness of the lognormal distribution are all positive which means the water level distributions have a heavier right tail, as well does the normal distributions of Strickler and Chézy. In this research the distribution of water levels is tried to be approximate normal distributed water levels, because we want to compare these results with the WBI2017 methods. Therefore the normal distributed Manning coefficient seems to be the best candidate, but further assessed in the next section.

Table C.2: Skewness of the resulting water level distribution that are derived from probability distributions of different roughness parameters. A variation coefficient of 10% is used.

	Manning	Strickler	Chézy
Normal distribution	-0.12	0.51	0.53
Lognormal distribution	0.18	0.18	0.20

C.4 The effect of different roughness distributions on water level frequency lines

The normal and lognormal distribution of the Manning's coefficient and Strickler's coefficient are used in the physics-based method to investigate the net effect in the derivation of water level frequency lines. The net effect is the difference in water levels derived by the deterministic approach and the physics-based approach. Water level frequency lines are derived for an average IJssel profile and HR2006 discharge statistics. Figure C.2 shows the result and it can be seen that a variation coefficient of 10% does not result in the same amount of uncertainty in the water level, but this lies outside the scope of this research. The difference between net effects of the lognormal and the normal distribution of the Manning's coefficient is approximately 8.5% at a return period of 10 000 years. This is approximately the same for the distributions of Strickler's coefficient. The selection of the probability distribution of the roughness coefficient (normal or lognormal) does influence the water levels, but only moderately.

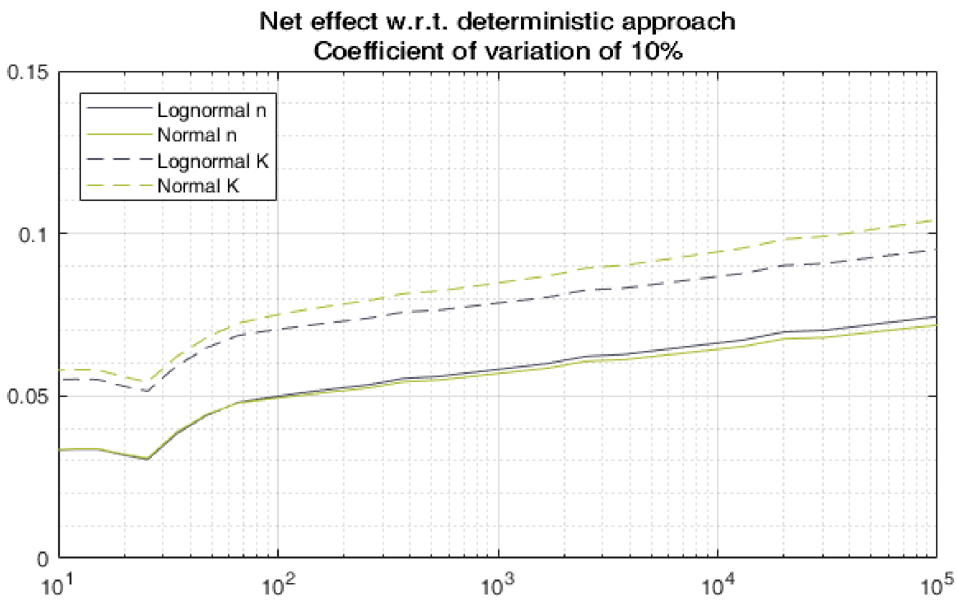


Figure C.2: The net effect of model uncertainty modelled by the physics-based method for different probability density functions of roughness parameters.

D. Probability distributions of the water level at different discharge levels

This appendix shows the empirical probability distributions of the water level for several discharge levels and different model schematisations: average IJssel profile, flood channel and retention area. These situations are in a summary manner explained. The discretisation of the empirical probability distributions for the physics-based approach is rather gross, because of the rediscretised domain.

D.1 Average IJssel profile

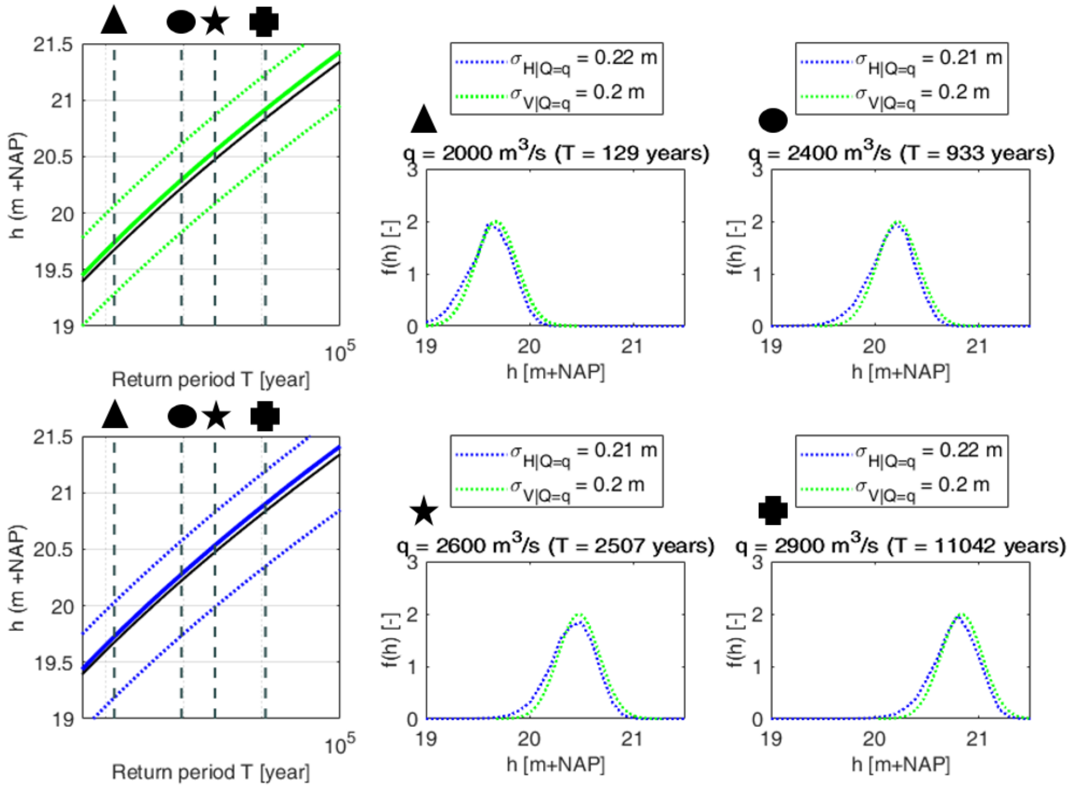


Figure D.1: The two left graphs are water level frequency lines with their 95% confidence bounds for different uncertainty modelling techniques: WBI2017 (Green) and physics-based (Blue). The dashed lines in the left graphs and the symbols next to it indicate four discharge levels for which the water level distributions are shown in the other graphs. The associated standard deviation at each discharge level for the different methods can be read from the legend of the graphs.

In this case the probability density functions of the water levels are approximately similar for the four discharge levels. The left tail of the physics-based method is heavier compared to the WBI2017 method. Therefore the water levels derived by the WBI2017 are slightly larger than the physics-based method in general.

D.2 Flood channel

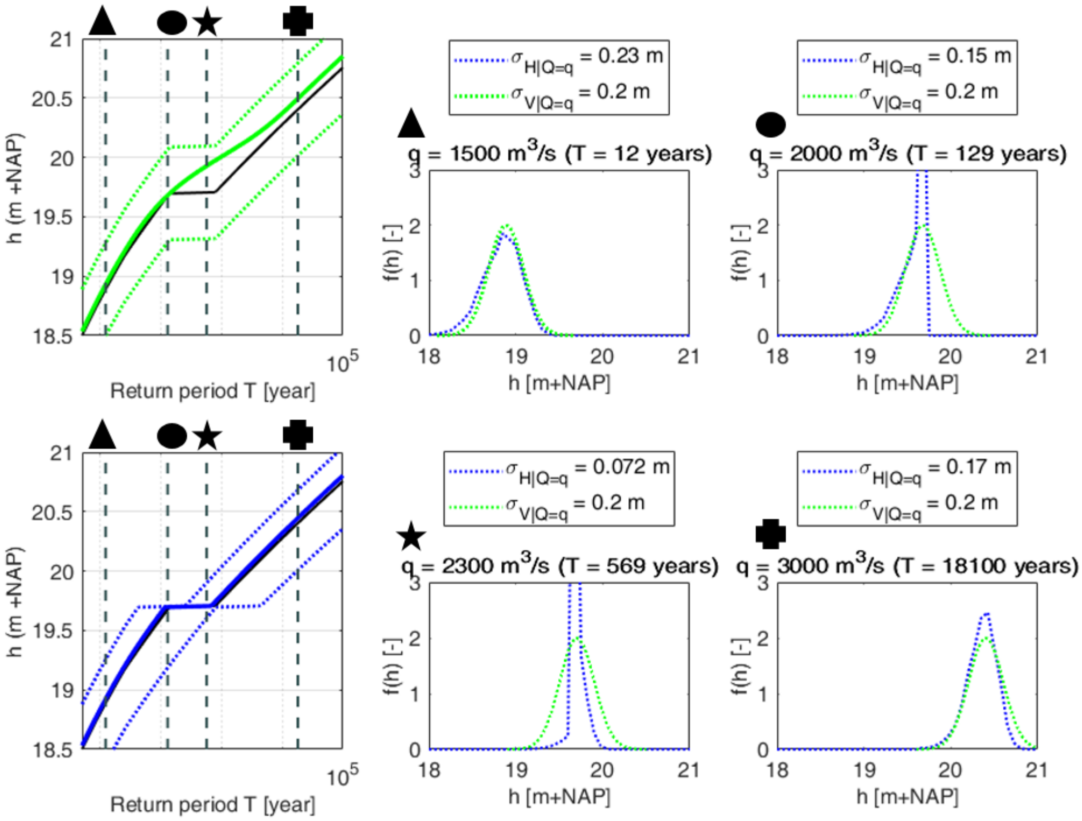


Figure D.2: The two left graphs are water level frequency lines with their 95% confidence bounds for different uncertainty modelling techniques: WBI2017 (Green) and physics-based (Blue). The dashed lines in the left graphs and the symbols next to it indicate four discharge levels for which the water level distributions are shown in the other graphs. The associated standard deviation at each discharge level for the different methods can be read from the legend of the graphs.

At a discharge of 1500 m^3/s none of the water levels in the 95 % confidence interval exceed the height of the moveable gates and the probability distributions of the water level are more or less equal for both methods. If we consider higher discharge levels, like 2000 m^3/s and 2300 m^3/s, it can be seen that the water level distributions that corresponds to the physics-based method are deformed. The water levels are attracted to the height at which the flood channel starts to count. Water levels during flood waves calculated with one peak discharge but multiple roughness coefficients exceed the height of the moveable gates and the flood channel starts to function. Therefore, the maximum water level for runs of different roughness values equals to the height of the moveable gates and the water level at that discharge level become more certain.

D.3 Retention area

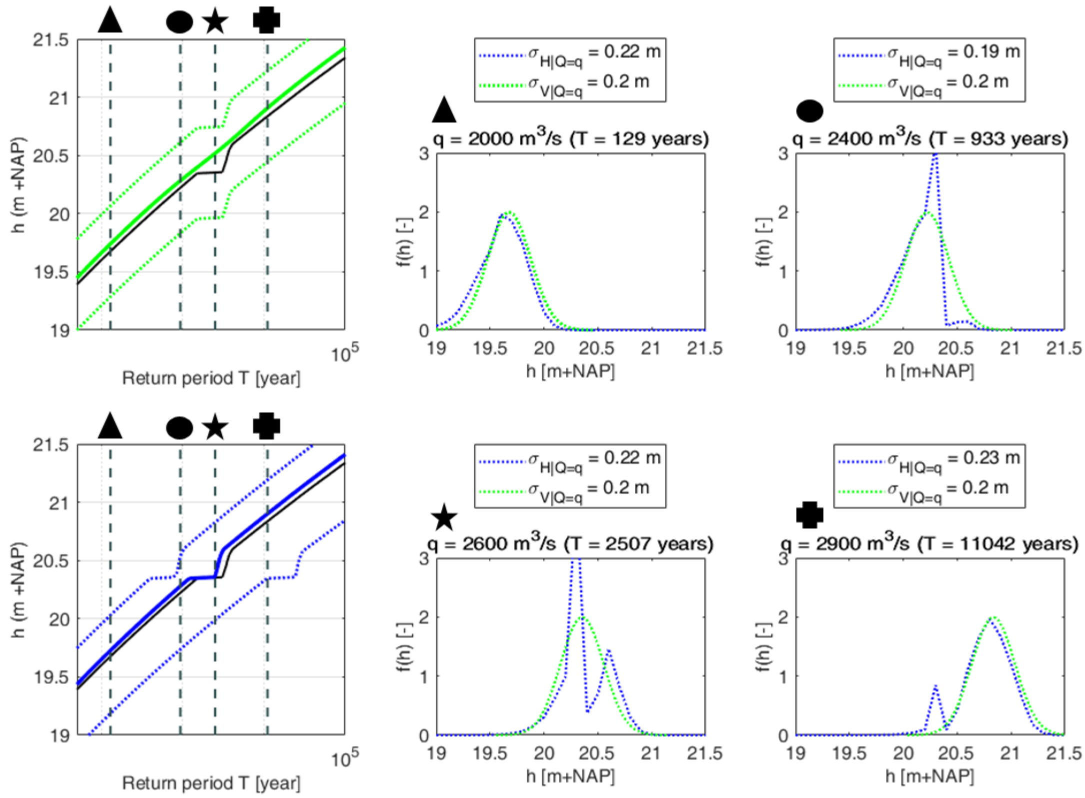


Figure D.3: The two left graphs are water level frequency lines with their 95% confidence bounds for different uncertainty modelling techniques: WBI2017 (Green) and physics-based (Blue). The dashed lines in the left graphs and the symbols next to it indicate four discharge levels for which the water level distributions are shown in the other graphs. The associated standard deviation at each discharge level for the different methods can be read from the legend of the graphs.

The physics-based method results in higher probabilities around the critical water level where discharge extraction starts compared to the WBI2017 method. The water level distribution for a flood wave with a peak discharge of 2400 m^3/s has an flat tail, because the retention area ensures the water levels to be physical bounded (indicated by the solid circle). For a peak discharge of 2400 m^3/s the water levels are more certain (decrease in standard deviation), while for higher peak discharge levels the water level uncertainty increases slightly.