Uncertainty in water level predictions due to various calibrations

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Abstract

The modelling of river processes involves numerous uncertainties, resulting in uncertain model results. Knowledge of the type and magnitude of uncertainties is crucial for a meaningful interpretation of the model results and the usefulness of results in decision making processes. This case study comprises an uncertainty analysis of the effect of the discharge data used for calibration on the water levels.

The roughness value of the main channel is calibrated on eight different water levels to assess the influence of the data used for calibration on the uncertainty in the model predictions. Comparison of the data used for calibration with verification model results gives insight in the effects of extrapolation of the calibration parameter.

It is concluded that the uncertainties in the roughness coefficient of the main channel results in a total uncertainty up to 20 cm in predicted water levels similar to water level used for calibration. This indicates that extrapolation of the roughness coefficient introduces significant uncertainty in design water level calculations.

Introduction

The Dutch safety policy against flooding requires every 5 years an evaluation of all primary dikes in the Netherlands. In the evaluation for the River Rhine, the heights of the dikes are compared to a design water level. By law, the design water levels for the Dutch part of the River Rhine have a probability of exceedance of 1/1250 per year. These water levels are computed based on a design discharge which is derived by a statistical analysis of historical discharge peaks. The hydrodynamic river model WAQUA is used to calculate the design water levels based on the design discharge (Rijkswaterstaat, 2005).

Numerical hydrodynamic river models like LISFLOODFP (Bates and De Roo, 2000), SOBEK (WL | Delft Hydraulics, 2007), MIKE-FLOOD (DHI, 2007) and WAQUA (Rijkswaterstaat, 2005) require calibration. Calibration is needed because modelling of the relevant physical processes has multiple shortcomings. These shortcomings are mainly caused by incompleteness of our knowledge about the relevant physical processes, and limitations of computer power which does not enable us to 'perfectly' model all relevant processes (Noordam, 2006). The purpose of calibration is to find a parameter set that minimizes the difference between modelled and observed outcomes as expressed in terms of the objective function (Werner, 2004). In hydrodynamic models, the resistance of the main channel is generally used as calibration parameter (Van den Brink et al., 2006, Werner, 2004). Flow resistance is implemented in these models by means of a roughness coefficient.

During calibration, model errors are compensated by the calibration coefficient. Thereby, uncertainties in the model results are reduced. Uncertainties are present in all parts of numerical models, because these models are inherently a simplification of reality. Extrapolation of the calibration parameter to other circumstances than the circumstances used during calibration introduces uncertainty in the model results. It is important to assess the uncertainty associated with this extrapolation, because extrapolation is done, amongst others, in the Dutch dike safety computations. Therefore, the objective is to assess the uncertainty in water level predictions, due to calibration on different water levels. The questions answered in this paper are:

1. What is the variation in the roughness parameter due to differences in the calibration discharge?
2. What is the quantitative effect of the uncertain roughness on the resulting water levels?
3. What is the influence of extrapolation of the calibrated roughness on the uncertainty of predicted water levels?

The outline of the paper is as follows. Firstly the uncertainty classification method by Walker et al. (2003) is described to give an overview of uncertainties in hydraulic river models. Subsequently, the WAQUA model as used by the Dutch water board and the case study approach are presented. The results from the model analysis are then shown and discussed. In the conclusion, the answers on the research questions are given and plans for future study are presented.

Uncertainty in river models

Uncertainty is defined by Walker et al. (2003) as being any departure from the unachievable ideal of complete determinism. They presented a classification based on the classifications by Janssen et al. (1990) and Van Asselt and Rotmans (2002). The classification method by Walker et al. (2003) distinguishes between the *source* (or nature) of uncertainty, the *location* of the uncertainty in the model and the *degree* of uncertainty.

Two sources of uncertainty can be distinguished: variability (inherent uncertainty) and limited knowledge (epistemic uncertainty). Variability represents the randomness of variations in nature and limited knowledge is a property of the state of knowledge in general or of the modeller. The second feature is the location where the uncertainties manifest themselves within the model, its context or the input and parameters of the model, which are actually parts of the model itself. The third feature is the degree of uncertainty, which deals with the different levels of knowledge, ranging from complete deterministic understanding up to total indeterminacy (in case we do not know what we do not know). These features are combined in the uncertainty matrix (Table 1). The matrix is used to give an overview of important uncertainties (RIVM/MNP, 2003) and show how the uncertainties influence the model.

One of the main sources of uncertainty in modelled water levels is the uncertainty of the resistance coefficient of the river bed (Chang et al., 1993, Van der Klis, 2003). Two main reasons for the uncertain roughness in hydrodynamic models are (a) the roughness coefficient is an approximation of physical resistance processes by empirical relations, and (b) the roughness coefficient is usually applied as a calibration parameter (Noordam, 2006).

According to Hoerner (1965) hydraulic resistance is caused by all retardations in or due to fluid motion. Hydraulic resistance in natural rivers consists, amongst others, of grain resistance, resistance due to bed form (i.e. ripples and subaqueous dunes), vegetation resistance, plan form resistance (i.e. resistance due to channel alignment) and resistance caused by invariability of the flow (i.e. velocity differences in space and time) (Chow, 1959, Hoerner, 1965). These resistances are included in the WAQUA model in empirical relations which cause model uncertainty. Grain resistance, for example, is combined with bed form resistance in an empirical roughness predictor. The results from the roughness predictor are independent of the physical grain size and bed forms, because the predictor relates roughness directly to water depth. The roughness predictor and the empirical parameters in this predictor are included in the uncertainty matrix as two separate entries. Thereby, showing the uncertainties due to grains and bed forms are shown in the uncertainty matrix (Table 1).

Table 1: Uncertainty matrix, modified by Noordam (2006) after Walker et al. (2003). The matrix is filled with examples of uncertainty in roughness, which occur in the WAQUA model.

Feature →	1. Source of uncertainty		2. Location of uncertainty					3. Degree of uncertainty		
				Model						
↓ Uncertainty	Limited knowledge	Variability	Context	Structure	Technical	Input	Parameters	Quantifiable uncertainty	Scenario analysis	Recognised ignorance
Grain resistance		X				X		X		
Bed form resistance	X			X						X
Calibration		X					X		X	

In decision making, numerical models results are interpreted using a deterministic approach. The WAQUA model results are presented in terms of *the* water level and velocities at a certain time and location. Numerous errors are present in the model prior to calibration. During calibration these errors are compensated in the value of the calibration coefficient. At other circumstances than during calibration, the errors in the model have a different effect on the model results. These effects are not compensated by the calibrated coefficient and result in errors in the predicted model results. These uncertainties introduced during calibration are not taken into account in model application. Because the uncertainty caused by extrapolation of the calibrated roughness is caused by the model uncertainties prior to calibration, quantification of the uncertainties in numerical models is of main importance.

Methods

WAQUA model

The design water levels in the Dutch Rhine branches are calculated using amongst others the numerical, two dimensional, depth averaged river model WAQUA. This model consists of: 1) the program environment SIMONA (Rijkswaterstaat, 2005) which holds the shallow water equations to simulate the water flow, and 2) the schematization of a certain period with corresponding input parameters (e.g. measured stage-discharge relations, riverbed roughness, upstream discharge, etc.). In this paper, the term ‘model’ means the combination of model code, schematization, input and parameters.

The schematization consists of a calculation grid, the bathymetry of the river bed, and mapped characteristics of the flow channel (e.g. grain size, vegetation distribution, and other objects like houses, bridges, barriers, spillways, etc.). The most recent version of the WAQUA model of 2006 has grid sizes of approximately 40 m. The time required to simulate one full day for the Dutch distributaries is approximately one hour.

Calibration of the WAQUA model for design water level calculation is carried out using the 1995 model schematization, with corresponding discharge peak and measured water levels at several locations along the River Rhine (Van den Brink et al., 2006). The 1995 peak is used, as it is the highest measured discharge peak in the River Rhine in recent history and is therefore assumed to represent the design discharge of 16000 m^3/s best. The 1995 peak had a maximum discharge of 12000 m^3/s at Lobith, which is 75 % of the design discharge.

Main channel roughness

Hydraulic roughness of the main channel consists mainly of both grain roughness and roughness due to bed forms (ripples and subaqueous dunes) (Van Rijn, 1984). In the WAQUA model, the roughness of the main channel is expressed as an equivalent sand roughness according to Nikuradse, k_n, which is a measure for the irregularity of the bed, expressed in m. For an alluvial river bed with bed forms, this roughness is calculated according to Van den Brink et al. (2006) after Van Rijn (1984):

$$k_n = \alpha \cdot h^{0.7} \left[1 - \exp\left(-\beta h^{-0.3}\right)\right] \qquad (1)$$

where α and β are empirically determined parameters and h is the water depth (m). Herein, α is adapted during calibration, because k_n proved most sensitive to α (Duits and Wijbenga, 1998); β is fixed at 2.5 (Wijbenga and

Duits, 1998). This equation gives the direct relation between water level and roughness due to bed forms. Equation 1 is only valid during high water levels, because only roughness due to bed forms is included and subaqueous dunes do not form during low water levels. Furthermore, equation 1 assumes that the height of subaqueous dunes is in equilibrium with water level. Because the water level is never constant in a natural river, dune height always has a time-lag.

Subsequently, k_n is converted to a Chézy coefficient C ($\mathrm{m}^{1/2}/\mathrm{s}$) for application in the shallow water equations. Its value is computed using the White-Colebrook resistance predictor (Van Rijn, 1994):

$$C = 18 \log\left(\frac{12h}{k_n}\right) \tag{2}$$

In the WAQUA model k_n and Chézy are calculated separately for each grid cell, because both depend on the water level in the grid cell. Therefore, the roughness is different for each cell, but the relation between water level and roughness is constant after calibration.

Waal model

In this study, the WAQUA model is used for the Waal branch only, so no bifurcation points are included in the model. Absence of bifurcation points makes the calibration procedure easier because no calibration on the discharge distribution at the bifurcation points is necessary. Fig. 1 shows the location of the Waal branch in the full WAQUA model of the Dutch distributaries. Water level measurements are conducted continuously at stations along the River Waal. The Waal model comprehends the measurement stations: Pannerdensche Kop, Nijmegen Haven, Tiel Waal, Zaltbommel, Vuren and Werkendam (Fig. 1). For the Waal branch, only the 1995 (model code: J95_4) and 1993 (J93_4) models were available with corresponding input data needed for calibration.

Bathymetries of the river bed for both models are measured in the period around the discharge peak using single-beam echo sounding. Input data for the WAQUA model consists of initial water levels, initial velocities and boundary conditions (Rijkswaterstaat, 2005). The roughness of the floodplain area in the modelled river stretch is estimated using the vegetation type method according to Van Velzen et al. (2003). This method comprises that every patch of vegetation in the floodplain area is classified by type. Every vegetation type is assigned a fixed roughness value, based on an empirical roughness predictor (Klopstra et al., 1997).

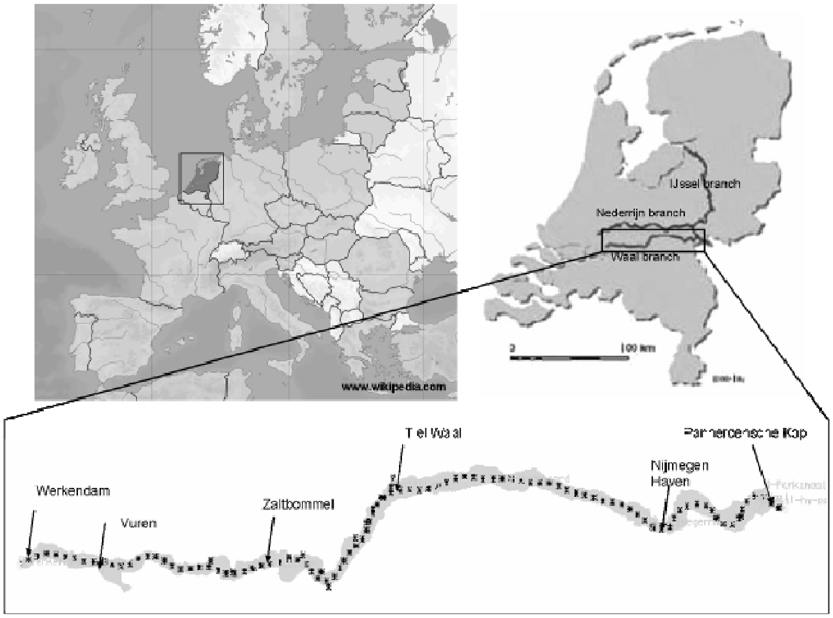

Fig. 1: Location of the WAQUA model schematization in the Netherlands (top right). The selected Waal model (bottom) with the measurement stations indicated by the arrows and the river kilometres as black crosses.

The full versions of these models are used in practise for calibration (1995) and verification (1993) of the roughness for the most recent determination of design water levels, in 2006 (Van den Brink et al., 2006). Comparison of the 1993 and 1995 models shows that similar vegetation and bathymetry measurements are used for the 1993 and 1995 model schematizations, because not all aspects which are schematized in the model are measured yearly in the field. Water level data with corresponding schematization were available from 11-01-1993 until 01-30-1994 for the 1993 peak discharge and from 12-01-1994 until 03-01-1995 for the 1995 peak discharge. This data comprehends the near flood events of 1993 (12-26-1993) and 1995 (02-01-1995), which both had a maximum discharge at Lobith of approximately 12000 m^3/s.

Boundary conditions

The boundary conditions of the Waal model consist of a discharge series upstream and measured water levels at the downstream end of the stretch. The upstream boundary condition is defined just upstream of Pannerdensche Kop at the left bank, at the Dutch-German border. This location is chosen, so the distance from Pannerdensche Kop to the upstream boundary condition ensures a stable model run at Pannerdensche Kop. The upstream boundary condition for this case study is computed by running the full WAQUA model including the Nederrijn and IJssel branches. The roughness for this model run is taken from the design water level computation and is calibrated on the 1995 peak discharge. The output of the full model at the location of the upstream boundary condition in the Waal model gives the boundary condition for the Waal model.

The downstream boundary condition is set at Werkendam and is determined by water level measurements during the peak discharges of 1995 and 1993. Initial water levels and velocities must be defined for every model run. These are calculated using a constant upstream discharge, which is equal to the discharge at the beginning of the calibration and verification run.

Calibration

The model is calibrated on the roughness of the main channel only by adapting α in equation 1 (Van den Brink et al., 2006). The river is divided into sections between measurement stations (Fig. 1). Calibration is conducted by optimizing the predicted water levels by adapting α for every section separately. Water levels around the maximum of the discharge peaks only are optimized. This maximum is determined for every station as the highest water level or when multiple equally large maxima exist, the central maximum or first of the two central maxima (Van den Brink et al., 2006).

The calibration "window" is defined as the 24 hour period around the top of the discharge. The average difference between the predicted water levels and the measured water level in the calibration window is minimized with a threshold value of 0.005 m (5 mm) after which calibration is defined successful.

Case study setup

For our experiment we redo the official calibration used for the design water level calculations. But we use eight different calibration discharges: three sub peaks in 1993 and five sub peaks in 1995, including both near flood events (Fig. 2). The roughness from the official calibration (Van den Brink et al., 2006) is used as initial roughness value for all calibration runs. These calibrations results in eight sets of roughness values for each of the six sections.

Subsequently the propagation of the uncertain roughness on the variability of predicted water levels is determined. Therefore eight model simulations are carried out using the roughness values from the eight calibrations as input for the 1995 model from 12-01-1994 to 03-01-1995. This gives eight different water level series along the River Waal for the simulated period.

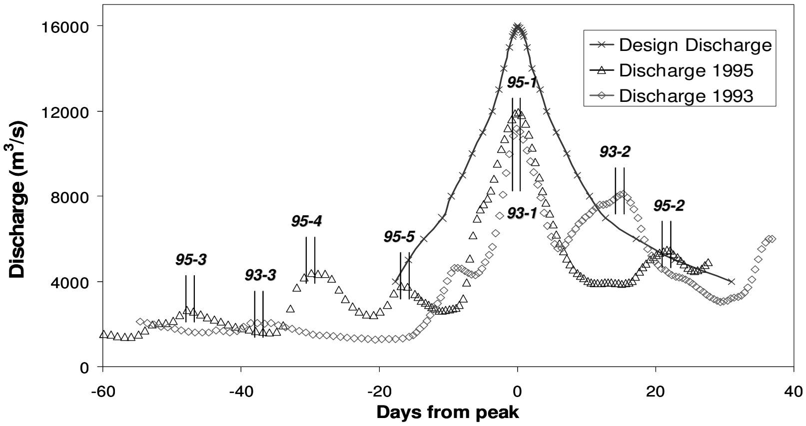

Fig. 2: The design discharge, used for calculation of dike safety compared to the discharge peaks of 1993 and 1995, used for model calibration and verification respectively. The calibration windows are shown between the two vertical lines and have a code referred to in the text. The peak discharges occurred on 26th of December 1993 and the first of February 1995.

To predict the effect of extrapolation of the calibrated roughness, first the roughness values are compared to the data used for calibration. Secondly, the errors in the predicted water levels using the 1993 and 1995 calibrated roughness values are compared to the measured water level. We focussed on the uncertainty in the roughness values and resulting water levels on high water levels only, because equation 1 is only valid at these water levels. We define high water levels as the water level resulting from a 5000 m^3/s at Lobith, which is approximately 3300 m^3/s for the Waal branch only. Around this discharge, the floodplain area becomes part of the flow section and floodplain roughness influences the total bed roughness.

Results

Uncertainty in the roughness coefficient

Fig. 3 shows the roughness sets expressed in Chézy values for the eight calibrations. The figure shows a general downstream trend of decreasing Chézy values (increasing roughness). This trend can be explained by the decreasing size of the sediment in downstream direction, which leads to increasing dune formation and therefore decreasing Chézy value (Juliën et al., 2002).

For calibration, two schematizations from 1993 and 1995 are used. Fig. 3 shows no dependence between the used schematization and calibration results. Calibration on the high peaks of the 1993 model gives similar results to calibration on the high peak 1995. Also, the range of the results from the 1993 calibration is comparable to the range of the results from the 1995 calibration. This can be explained, because the schematized period differs only one year and the schematizations are based on field measurements which are not conducted every year. Therefore, both schematization are mainly based on the same measurement data and show great similarity.

Table 2: Examples of roughness values from literature, formatted after (Shaw, 1999). Manning values are recalculated to Chézy values using the equation presented in Van Rijn (1994) with a water depth of 10 m.

Description	n	Chézy
Concrete lined channel	0.013	±100
Unlined earth channel	0.020	73
Straight stable deep natural channel	0.030	49
Winding natural streams	0.035	42
Variable rivers, vegetated banks	0.040	37
Mountainous streams (rocky bed)	0.050	29

Fig. 3 shows a large range of roughness values resulting from calibration on different discharge peaks. The variability of the roughness, for the three upstream sections, shows an average range of 10 $m^{1/2}s^{-1}$ depending on the river section. We observe that calibration on low discharge peaks (1993 peak 3 and 1995 peak 3, 4, 5) results in high Chézy values, while calibration on high discharge peaks (1993 peak 1, 2 and 1995 peak 1, 2) results in lower Chézy values. The Chézy values of the two most downstream sections however, have an unrealistic range. In these sections, calibrations on the lowest discharge peaks result in large Chézy values, whereas the high discharge peaks give more realistic results compared to literature. This can be explained by the presence of tidal influence in the downstream sections, which results in an oscillation in the measured data. Although the oscillation is imposed on the downstream boundary condition, a small shift in propagation time of the tidal wave upstream may cause large discrepancies between the measured and predicted data. This difference is then compensated, during calibration resulting in unrealistic Chézy values. At high discharges the tidal influence is suppressed by the high river discharge and therefore has less effect on calibration. At all discharges, the tidal wave does not propagate beyond Zaltbommel. So the large range in Chézy values in the three upstream sections is not influenced by tide.

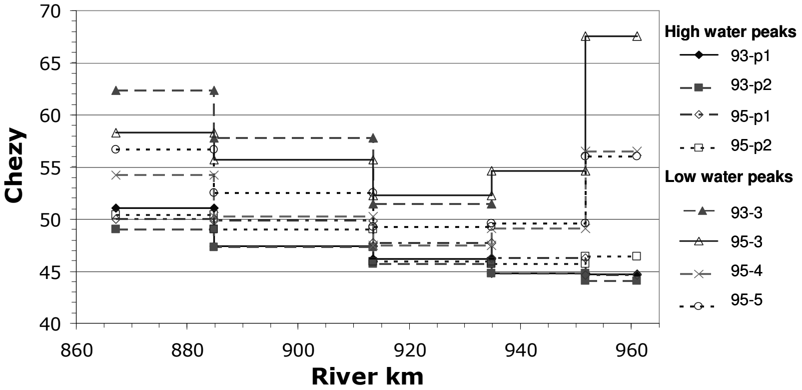

Fig. 3: Variation in Chézy values due to calibration on different water levels.

The calibrated roughness shows values between 63 and 44 $m^{1/2}/s$, with and average in the three upstream sections of 51 $m^{1/2}/s$. Values around 50 are quite high for natural channels. Table 2 shows average Chézy values presented in literature for different stream types. A Chézy value of 49 $m^{1/2}/s$ corresponds to “straight stable deep natural channels” (Shaw, 1999), which does not correspond to the Waal River (C = 44 $m^{1/2}/s$ according to Juliën et al (2002). This indicates that the combined other features and parameters in the model (e.g. floodplain roughness, energy losses due to spillways, etc.) have low Chézy values compared to the actual roughness, or model errors result in an overestimation of the resistance. The overall overestimation of roughness is compensated by calibration, resulting in relatively high Chézy values of the main channel. High values for the Chézy coefficient increase the flow velocity and the amount of discharge through the main channel compared to the floodplain area. Therefore, the flow velocity in the main channel is overestimated, while floodplain flow velocity is underestimated.

This uncertainty has significant influence when the results from the hydrodynamic model are used for morphological predictions.

In Fig. 4 the roughness values resulting from calibration are compared to the height of the peaks used for calibration. This comparison is conducted at the measurement station Nijmegen-Haven, because this location has least influence from the upstream and downstream boundary conditions. Fig. 4 shows a strong linear relation between the water level used for calibration and the resulting Chézy value ($R^2 = 0.82$). Consequently, calibration on a low water level only gives reliable results at that particular water level. Extrapolation of the roughness calibrated on low water levels is therefore, not valid. This conclusion is also drawn by Udo et al. (2007) who suggested a table with calibrated roughness values which depend on the discharge.

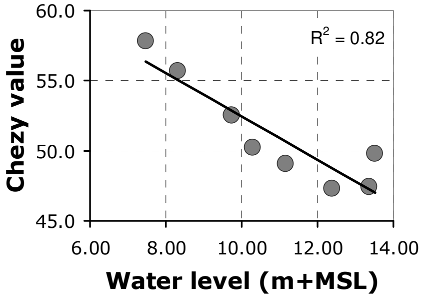

Fig. 4: Relation at Nijmegen Haven between the water level used for calibration and the resulting Chézy value.

Effect of uncertain roughness on water level predictions

We observe that the range in Chézy values has a significant effect on the predicted water levels (figure not shown). The differences between predicted and measured data range from 0.73 m for the simulations based on calibration on a high discharge peak to -0.60 m for simulations based on calibration on a low discharge peak. The difference between the highest and the lowest error is a constant 0.45 m. In consent with Fig. 3, the systematic increase of Chézy values over the river stretch with decreasing calibration water level, results in a systematic decrease of the water levels. We observe that water levels resulting from roughness calibrated at low water level, perform better at periods of low water levels and water levels resulting from roughness values calibrated at high water level perform better at periods of high water levels.

The hydraulic roughness in the model depends on equation 1. This equation however, is only valid under high water levels when subaqueous dunes are in equilibrium with the water depth. At low discharges, no dunes develop on the river bed and bed roughness is caused by smaller bed forms and grains which are not represented in equation 1. Therefore, the calibrated roughness does not represent the real roughness of the river bed under low flow conditions. For further analysis only the calibrations on high water levels are considered.

If the calibration on low water levels is not taken into account (Fig. 5), the range of roughness values decreases. Both 1993 peaks show comparable results, while calibration on the 1995 peaks results in slightly higher Chézy values. The small difference in Chézy values, however, has a large effect on the resulting water levels. Fig. 5 shows that the difference in Chézy values of 2.4 $m^{1/2}/s$ between the calibration on the 93-1 and 95-1 peaks results in a 10 cm error in water levels at Nijmegen Haven measurement station. Around the 1995 peak water level, the difference between measured and predicted water levels for the 1995 model is respectively 10 cm for the water level based on the 1995 calibrated roughness and 20 cm for the 1993 calibrated roughness.

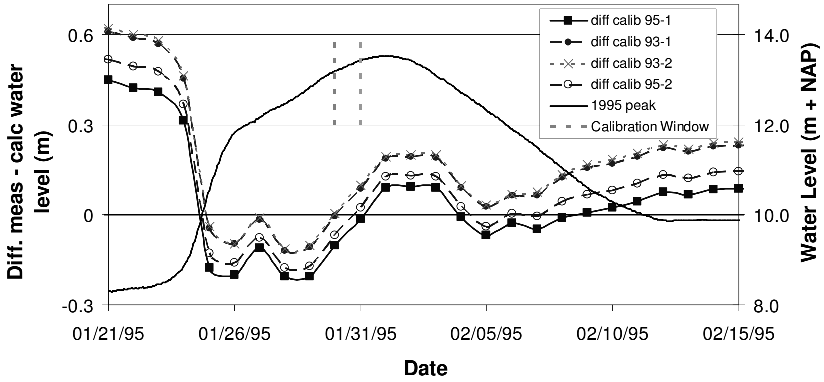

Fig. 5: Results of the verification run for the roughness values calibrated on high water levels at Nijmegen Haven measurement station. The marked lines represent the error in the verification water levels, the solid line represents the 1995 peak water levels used as input for the verification run.

Fig. 6 shows the water level used for calibration and the error of the verification run compared to the measured water level at Nijmegen Haven. The errors in water levels for the 1993 and 1995 peak water levels are shown, which range between -20 cm and +10 cm for the 1995 calibrated peak discharge. If Chézy is calibrated on the high water levels, the error increases with decreasing water levels, therefore the water level prediction of the model calibrated on the high discharge is uncertain for low water levels.

The design water level of 14.8 m + MSL at Nijmegen Haven is estimated based on the stage-discharge relation and is plotted in Fig. 6. This figure shows that the error at the design discharge is difficult to predict. If the water level deviates from the calibrated value, the error in the simulation becomes larger. This indicates that the calibrated model is only valid under the circumstances used for calibration. A deviation of one meter from calibration circumstances, which is smaller than the step from calibration to design conditions, results in errors up to 20 cm.

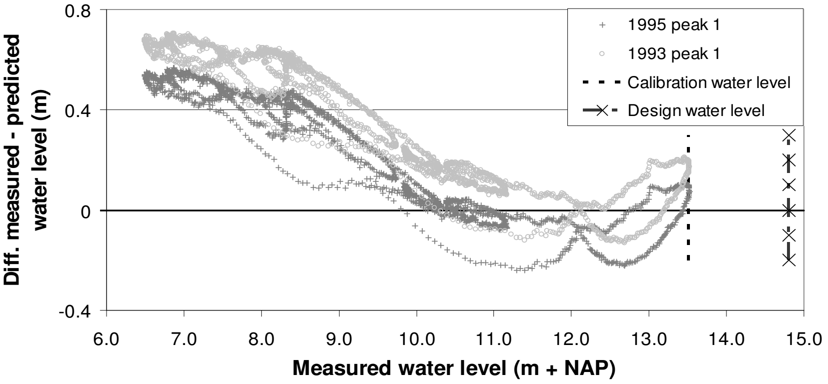

Fig. 6: Relation between the error in predicted water levels and the measured water level at Nijmegen Haven. The predicted water levels are computed using the 1995 model with the 1993 and 1995 calibrated roughness values. Subsequently, the difference between predicted water levels and the measured water levels are calculated and compared to the measured water levels to show the sensitivity for small deviations from the calibration water level. The vertical lines represent the water level used for calibration and the design discharge.

Discussion

This case study comprises an uncertainty analysis of the calibration of the WAQUA model. As a first step, the variability of the roughness is assessed which shows the average range in roughness values of 10 $m^{1/2}s^{-1}$ depending on the river section. For example, the roughness of the floodplain area does not influence the bed roughness at low water levels because the floodplain is no part of the flow. Therefore, errors in floodplain roughness parameterization are not compensated in the calibration coefficient at low flows. Because these errors are not accounted for during calibration, extrapolation of the calibration coefficient is not valid from low to high discharges. If only high discharges are taken into account (Fig. 5) the differences between calibrated Chézy values are less pronounced and result in smaller errors. However, also at high discharges empirical relations have different effects at different discharges. This variability can be explained by the relation between water level and roughness of subaqueous dunes which is different under various discharges. This variability, however, is not included in equation 1. After extrapolation this results in uncertainties in the predicted water levels.

Because no measurement data are available for calibration on the design discharge, extrapolation will always be needed. Therefore, the effect of extrapolation of the calibration coefficient on the uncertainty of the model results is inherently unknown. In practise, it is assumed that the calibrated roughness predictor is valid under design circumstances. However, Fig. 4 Fig. 6 shows that water level calculations are not stable around the period of the calibrated discharge, but show a large uncertainty band. Therefore, a small deviation in predicted water level from the water level used for calibration, results in significant errors in the predicted water level. Errors up to 20 cm are shown in Fig. 6 for deviations of 0.5 m from the calibration water level.

It is uncertain how the hydraulic roughness will be influenced under design conditions. When the regression line in Fig. 4 is extended to design conditions (approx. 14.8 m + MSL), the Chézy value will decrease. However, when the trend of the Chézy values resulting from calibration on the three highest water levels is extended, the Chézy value will increase with increasing water level. A similar visual extrapolation of the error in Fig. 6 indicates a range of possible errors between -30 cm up to + 50 cm.

It is shown that multi-objective calibration is not valid in this case, because roughness calibrated on lower discharge performs better for lower discharges and vice versa. Calibration causes all errors in the model, input and parameters, to be compensated. Errors are introduced by parameterization of hydraulic roughness in hydrodynamic river model. These errors have other sources for different circumstances, caused by limitations in the parameterization. At different discharges different physical processes are not (well) reflected. The errors are specifically compensated, but only for the circumstances during calibration. Therefore, calibration decreases the model uncertainty for calibration circumstances. However, if the calibrated roughness value is extrapolated, errors are again introduced because different physical processes are present at the extrapolation circumstances. This results in an increase of the model uncertainty (Stolker and Van Velzen, 2006). As a result, the uncertainty in the roughness parameter and water levels increases and the calibration coefficient introduces uncertainty.

The calibrated roughness is small compared to literature, indicating a structural overestimation of the roughness in other parts of the model. The location of this overestimation of roughness throughout the model is difficult to determine, because calibration compensates all model errors at the measurement locations, independent of the location of the uncertainty in the model. The challenge is to quantify these uncertainties and thereby decrease total model uncertainty after extrapolation.

Conclusions and ongoing research

It has been shown that roughness calibrated on different peaks shows a large variability up to differences in Chézy values of 11 $m^{1/2}/s$. When only the four high discharge peaks are considered, this range decreases to 2.4 $m^{1/2}/s$. The calibrated roughness value is low compared to literature, indicating an overestimation of the other combined resistance parameters or model errors.

The uncertainty in Chézy values results in a large uncertainty up to 65 cm for low water level predictions while the model is calibrated on high water level. However, the used roughness predictor is not valid for low water levels, therefore it is concluded that the WAQUA model calibrated on high water levels is not appropriate to calculate water levels during low flows without a separate calibration.

A strong linear relation exists between the water level used for calibration and the resulting roughness. This relation indicates that extrapolation of calibrated roughness introduces uncertainty in the model results. It is shown that computed water levels to calibration water levels have a relatively large uncertainty band. A difference of 0.5 m between the calibration water level and predicted water level results in significant errors up to 20 cm. The difference between the calibration water level and design water level is approximately 1.2 m. Therefore, the uncertainty in model results after extrapolation is significant.

In future research the source and location of uncertainties in the roughness predictor will be studied. This knowledge can decrease the uncertainty in the predicted water level due to extrapolation. With classification (Table 1) and quantification an indication is given of the total uncertainty which is inherent in numerical modelling. Quantification of uncertainty in models will give information on the reliability of the model results and will therefore be helpful in decision making processes.

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References

Bates PD, De Roo APJ (2000) A simple raster-based model for flood inundation simulation. Journal of Hydrology 236: 54

Chang CH, Yang JC, Tung YK (1993) Sensitivity and uncertainty analysis of a sediment transport model: a global approach. Stochastic Hydrological Hydraulics 7:299-314

Chow, VT (1959) Open channel hydraulics. McGraw-Hill Book Company New York USA

DHI (2007) MIKE FLOOD http://www.dhigroup.com/Software/WaterResources/MIKEFLOOD.aspx Horsholm

Duits MT, Wijbenga JHA (1998) Actualisatie WAQUA Rijntakken/Maas. Inregelen bij varierende afvoer (fase 2) HKV Lijn in Water (in Dutch)

Hoerner SF (1965) Fluid-dynamic drag Published by the Author

Janssen PHM, Slob W, Rotmans J (1990) Gevoeligheidsanalyse en onzekerheidsanalyse: een inventarisatie van ideeen, methoden en technieken. National Institute for Public Health and the Environment Bilthoven:119 (in Dutch)

Juliën PY, Klaassen GJ, Ten Brinke WBM, Wilbers AWE (2002) Case study: Bed resistance of Rhine River during 1998 flood. Journal of Hydraulic Engineering 128: 1042-1050

Klopstra D, Barneveld HJ, Van Noortwijk JM, Van Velzen EH (1997) Analytical model for hydraulic roughness of submerged vegetation. Proceedings of theme A, managing water: coping with scarcity and abundance

Noordam D (2006) Hydraulic resistance in rivers and related uncertainties - Literature review. University of Twente. ISSN 1568-4652

Rijkswaterstaat (2005) Gebruikershandleiding WAQUA Versie 10.41. (in Dutch)

RIVM/MNP (2003) Leidraad voor omgaan met onzekerheden, quickscan hints en acties-lijst. Project Onzekerheidsanalyse S/550002/01/OP National Institute for Public Health and the Environment Bilthoven (in Dutch)

Shaw EM (1999) Hydrology in practice Third Edition Stanley Thornes Ltd Cheltenham United Kingdom ISBN 0-7487-4448-7

Stolker C, Van Velzen EH (2006) Verschilanalyse toetspeilen HR2006 -- HR2001: bovenrivieren Maas. WL | Delft Hydraulics, RWS RIZA (in Dutch)

Van Asselt MB, Rotmans J (2002) Uncertainty in integrated assessment modelling. Climatic Change 54: 75-105

Van den Brink NGM, Beyer D, Scholten MJM, Van Velzen EH (2006) Onderbouwing hydraulische randvoorwaarden 2001 voor de Rijn en zijn takken. RWS RIZA (in Dutch)

Van der Klis H (2003) Uncertainty analysis applied to numerical models of bed morphology. PhD thesis at Delft University of Technology, Delft, the Netherlands.

Van Rijn LC (1984) Sediment transport, part III: Bed forms and alluvial roughness. Journal of Hydraulic Engineering 110: 1733-1754

Van Rijn LC (1994) Principles of fluid flow and surface waves in rivers, estuaries, seas and oceans. AQUA Publications, Amsterdam, the Netherlands

Van Velzen EH, Jesse P, Cornelissen P, Coops H (2003) Stromingsweerstand vegetatie in uiterwaarden: deel 1 handboek versie--1 2003. RWS RIZA (in Dutch)

Walker WEPH, Rotmans J, Van der Sluijs JP, Van Asselt MBA, Janssen P, Krayer von Kraus MP (2003) Defining uncertainty, a conceptual basis for uncertainty management in model-based decision support. Integrated Assessment 4: 5-17

Werner MGF (2004) Spatial flood extent modelling: A performance-based comparison. PhD thesis at Delft University of Technology, Delft, the Netherlands.

Wijbenga JHA, Duits MT (1998) Actualisatie WAQUA Rijntakken/Maas. Inregelen bij constante afvoer (fase 1). HKV Lijn in Water (in Dutch)

WL | Delft Hydraulics (2007) SOBEK. http://delftsoftware.wldelft.nl