# Inaccuracies in estimated grain size parameters and their implication on geological models

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## Abstract

The accuracy of the measurements of grain size parameters used in a geological model are crucial to the overall reliability of the constructed model. The main goal of the current research is to quantify the reliability of estimated grain size data and to determine the impact that these inaccuracies have on 3D geological models. Only comparisons of sand medians will be presented in this text. The analysis shows that the sand median is underestimated. The effect of the inaccuracies in estimated sand medians on the 3D interpolation of grain size data is evaluated, using two methods, first a visual check and, second the calculation of the Shields parameter. The conclusion is that the inaccuracies of the sand median do not lead to any significant changes in whether or not the sediment is transported or changes in river sedimentation patterns.

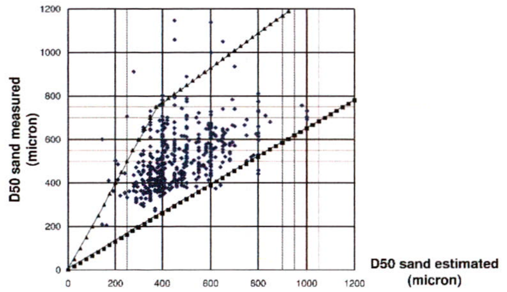

*Figure 1. Scatter plot of estimated D50 values and associated measured D50 values. In triangular points the upper boundary of the bandwidth and in squares the lower boundary of the bandwidth. From within this bandwidth the realizations of the sand median are taken.*

## Introduction

In geological modelling, geostatistics and geological knowledge are used to combine 1D and 2D measurements to a 3D frame of the different (geological) layers in the subsoil. Within these layers different parameters may be estimated. The accuracy of the measurements used in a geological model are crucial to the overall reliability of the constructed model. For morphological models, insight into the variation of the grain size parameters within a layer is dominant.

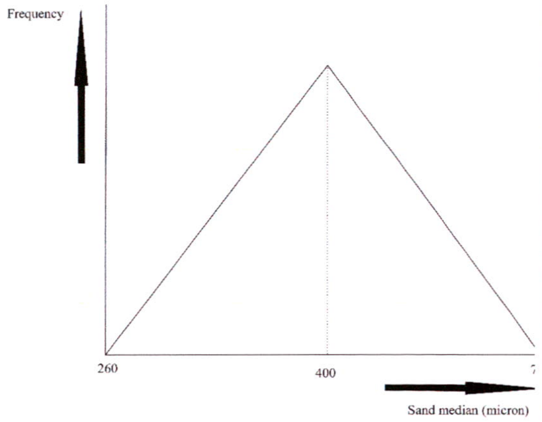

*Figure 2. Triangular probability curve of the sand median. In this example an estimated sand median of 400 µm and an associated bandwidth of 260-770 µm; 50 realizations of the sand median are taken from this frequency diagram*

The main goal of the current research is to quantify the reliability of the used grain size data and to determine the impact that these uncertainties have on geological models.The data used in this study originated from various projects carried out for the Institute for Inland Water Management and Waste Water Treatment (RIZA) during the years 2001-2003. Samples are taken from vibrocores and are analysed by Fugro B.V. using the sieve method (cf. NEN 2560). All of the samples were also described at TNO-NITG and these descriptions are all stored in the relational database DINO (Databank Informatie Nederlandse Ondergrond); the database of all subsoil data of the Netherlands used at TNO-NITG. In total, approximately 1500 samples have been used for analysis. First, the accuracy of estimated grain size parameters (median grain size of the sand fraction between 63 and 2000 µm (D50), silt content and gravel content) are compared with sieve results. Only comparisons of sand medians will be presented in this text. The effect of these inaccuracies in estimated sand medians on the 3D interpolation of grain size data is evaluated by using a Monte Carlo procedure for a set of samples in the IJsselkop bifurcation in the lower Rhine distributary system in the Netherlands. The inaccuracies in sand medians are transformed into triangular distributions. For every realization, the sand median, gravel and silt content are used to calculate a synthetic grain size distribution (GSD). Together with measured grain size distributions these data are interpolated using 3D kriging (cellsize 25x25x0.2 m).

## Results of the analysis of grain size parameters

Of the analysis of the three grain size parameters (sand median, silt and gravel content) only the results of the sand median are presented in this paper. The sand median is on average underestimated, which can be seen in Figure 1. More details of the inaccuracies and the way they are calculated can be found in Maljers & Gruijters (2004). In the next section the inaccuracies in the sand median are implemented in a geological model, and the implications on this model are discussed.

*Figure 3a. The calculated Shields parameter (θ) for two realizations (a and b), only minor differences can be seen, leading to no differences in whether or not the sediment is transported in this example ($\theta_{cr}$=0.06; threshold of motion).*

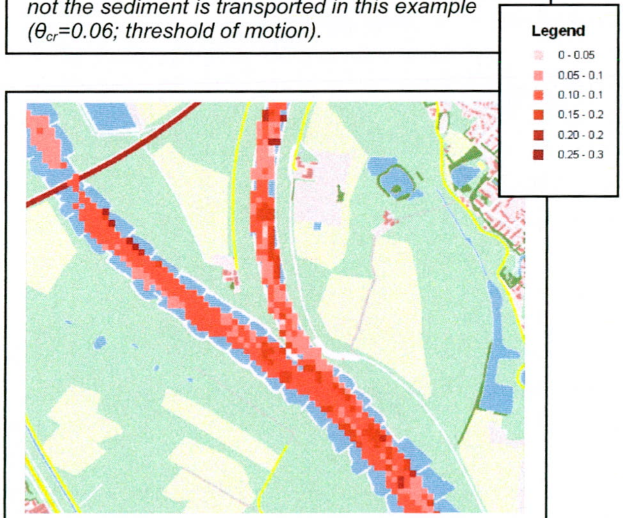

*Figure 3b. The calculated Shields parameter (θ) for two realizations (a and b), only minor differences can be seen, leading to no differences in whether or not the sediment is transported in this example ($\theta_{cr}$=0.06; threshold of motion).*

## Implementation and implications of inaccuracies

The implementation of the bandwidth found in analyzing the sand median will be presented below. In Fig. 1 the bandwidth of the sand median is shown (in triangular en square points). This bandwidth means that for samples of which the sand median is estimated as 400 μm, the actual sand median could be between 260 μm (lower boundary of the bandwidth) and 770 μm (upper boundary of the bandwidth).

For modelling, 50 realizations of the sand median are calculated. The values are within the bandwidth and follow a (random) triangular probability curve. This results in 50 different realizations of the sand median. For example the range of measured sand medians belonging to an estimated sand median of 400 μm, is between 260 μm and 770 μm. The main point will be around 400 μm (Fig. 2).

The other two grain size parameters of importance in constructing synthetic GSDs are the silt content and the gravel content. Silt content in this research has been set to 1% for all samples and during all realizations. The gravel content has been estimated for all samples and this value is used in modelling. This means that, during each realization the only parameter that differs is the sand median. Details about constructing synthetic GSDs are fully discussed in the NCR-publication 24 (Gruijters et al., 2004) and will therefore not be presented in this paper.

The implications on a geological model of the inaccuracies in the sand median can be shown in two ways. First of all a visual check has been made; second, the Shields parameter, which is a dimensionless measure for the transport of sediment, has been calculated for two realizations in combination with two river discharges (normal and high).

For the visual check, the median of the whole sample of the top of the Kreftenheye deposits has been used instead of the top of the mobile pavement, because the active layer was sampled in great detail and has therefore only measured GSDs. Therefore the bandwidth of the sand median and the associated synthetic GSDs will not affect the top of the mobile pavement. Only in details do the two realizations of the top of the Kreftenheye deposits differ in D50.

The implementation of the bandwidth of the estimated sand median does not seem to have any influence on river sedimentation patterns.

The calculated Shields parameter, has also been calculated for the top of the Kreftenheye deposits. When looked at the actual values for the Shields parameter for two realizations during normal discharges, minor differences can be seen (Fig. 3). Because the Kreftenheye sediment is not very coarse, the threshold of motion (which is set to 0.06) is already exceeded for most cells; therefore the minor differences in the Shields parameter do not lead to any difference in whether or not the sediment is transported.

## Conclusions

Based on the aforementioned methods it can be concluded that the applied bandwidth to the estimated sand median does not influence the sedimentation patterns or whether or not the sediment is transported. This means that the method described in Gruijters et al. (2004) to use synthetic grain size distributions based on estimated parameters is indeed a step forward in describing the natural variation in the subsoil of a riverbed.

## Acknowledgements

The authors would like to thank RIZA, Roy Frings and Maarten Kleinhans.

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# Supply-limited transport of bed-load sediment at the IJsselkop

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## Abstract

Bed-load transport calculations based on multibeam echo soundings suggest that the sediment transport at the IJsselkop is suppressed by a limited supply of transportable sediment. This probably results from bend sorting processes at the river bifurcations Pannerdensche kop and IJsselkop.

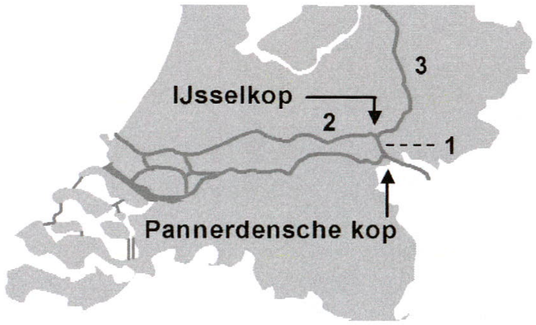

*Figure 1. Rhine branches at the IJsselkop bifurcation, the Netherlands: 1 = Pannerdensch Kanaal, 2 = Neder-Rijn, and 3 = IJssel*

## Introduction

The river bifurcations Pannerdensche Kop and IJsselkop determine the sediment and water distribution over the central part of the Netherlands (Fig. 1). A good understanding of this distribution process is crucial for operational river management. It is known that only a small part of the bed-load sediment that arrives at the Pannerdensche Kop bifurcation is directed towards the Pannerdensch Kanaal (Kleinhans, 2002). Therefore, the IJsselkop bifurcation, which is situated at the end of the Pannerdensch Kanaal, may be subject to a limited supply of transportable sediment. The purpose of this study was to determine whether the bed-load sediment transport at the IJsselkop is supply-limited.

## Methods

We used multibeam echo soundings in combination with a dune tracking technique to determine the bed-load transport rate at the IJsselkop. We thus assumed the sediment transport to be zero when dunes are absent, which is realistic according to direct transport measurements with a Delft Nile Sampler (Frings, unpublished data). The multibeam echo soundings that we used were conducted during discharge waves in November 2002 and January 2004. The resolution of these echo soundings enabled us to determine the temporal, longitudinal and lateral variation in sediment transport.

## Results

The lateral variation in sediment transport was large, especially in the Pannerdensch Kanaal. The absence of dunes near the edges of the river indicates that the sediment transport was limited to a clearly demarcated zone in the middle of the river (Fig. 2).

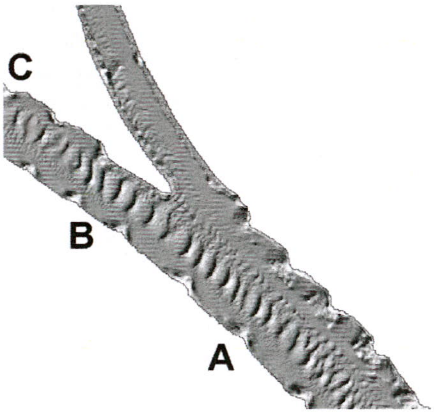

*Figure 2. Bed image of the IJsselkop bifurcation, November 2002. A,B,C refer to the sections described in Figure*

The temporal variability in sediment transport during the 2004 discharge wave is shown in Fig. 3, for three river sections. In the first section, the Pannerdensch Kanaal, the maximum sediment transport rate occurred well before the peak discharge. In the other sections, both in the Neder-Rijn, the maximum sediment transport rate occurred much later; in the downstream-most section even two weeks after the peak discharge. The long-term temporal variability is also pronounced: in 2002 the sediment transport was almost twice as high as in 2004 at the peak of the discharge wave, which was about 6500 $m^3/s$ at Lobith in both years.

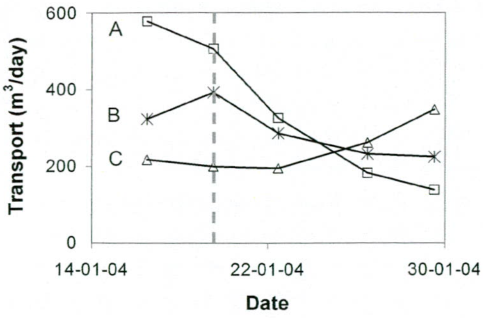

*Figure 3. Temporal variation in sediment transport during the January 2004 discharge wave in three river sections (A, B, C). The location of the three river sections is shown in Figure 2. The moment of the peak discharge is indicated with a dashed line*

Figure 4 shows the water and sediment distribution at the IJsselkop. The Neder-Rijn received about 90 % of the sediment load in the Pannerdensch Kanaal and 57 % of the water discharge, while the third river branch, the IJssel, received an almost equal amount of water (43 %), but only 10 % of the sediment load.

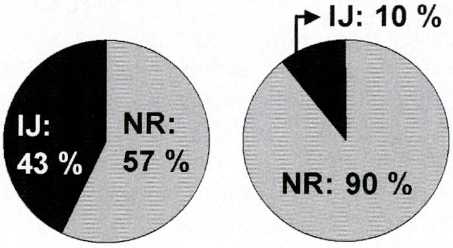

*Figure 4. Water distribution (left) and sediment distribution (right) at the IJsselkop during the 2002 and 2004 discharge waves. IJ: IJssel; NR: Neder-Rijn*

## Discussion

All results support the idea of supply-limited transport. Namely, in the case of redundant transportable sediment, dunes would have occurred over the entire river width, while the sediment transport rates in 2002 and 2004 would have been the same. Furthermore, the moment of maximum sediment transport would have been equal for the Pannerdensch Kanaal and the Neder-Rijn. The downstream shift in moment of maximum sediment transport points at a sediment wave that moved from the Pannerdensch Kanaal into the Neder-Rijn during the 2004 discharge wave. A sand wave has also been observed at the Pannerdensche Kop (Kleinhans, 2002), suggesting that sediment waves are a structural phenomenon at bifurcation points.

The supply-limitation is probably larger in the IJssel than in the other branches of the IJsselkop, because the IJssel receives hardly any sediment. We expect this to be the result of the interaction between sediment transport and bed grain size (Fig. 5), in the following way.

Bend sorting in the meander bend upstream of the IJsselkop causes the bed sediment at the entrance of the IJssel to be much coarser than the bed sediment at the entrance of the Neder-Rijn. This coarse material probably is only mobile at high discharges, leading to a limited supply of transportable sediment into the IJssel at low discharges and at intermediate discharges like those in 2002 and 2004. The same process can be held responsible for the limited supply of sediment into the Pannerdensch Kanaal at the Pannerdensche Kop.

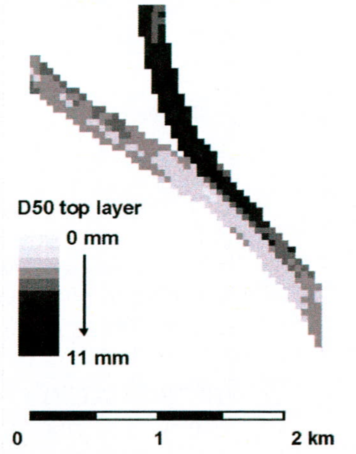

*Figure 5. Grain size of the top of the riverbed at the IJsselkop (after Gruijters et al., 2003).*

The supply limitation has severe implications for the prediction of sediment transport rates, as theoretical transport predictors all assume a redundant amount of transportable sediment. Empirical predictions are problematic too, because the supply-limitation is not constant in time. In 2002 the sediment transport was much higher than in 2004, while the discharge was the same. For reliable predictions of sediment transport at the IJsselkop, therefore, the sediment transport history needs to be taken into account.

## Acknowledgements

We would like to thank: Leonie Bolwidt (RIZA), Denise Maljers and Stephan Gruijters (NITG-TNO), and the members of the Meetdienst DON.

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# Morphological behaviour around bifurcation points; preliminary results of recent measurements

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## Abstract

To determine the morphological behaviour of bifurcation points two measurement campaigns were set up and performed in January 2004 and September 2004. A variety of measurements was performed successfully, resulting in a unique database on bifurcation points. These first results are promising.

## Introduction

The goal of this study is to determine the morphological behaviour of bifurcation points. Bifurcation points play a key role in the water and sediment movement of a river system. In the Netherlands there are three main bifurcation points: (1) Pannerdensche Kop (Boven-Rijn divides into Pannerdensch Kanaal and Waal); (2) Merwedekop (Boven-Merwede divides into Beneden-Merwede and Nieuwe Merwede); (3) IJsselkop (Pannerdensch kanaal divides into IJssel and Neder-Rijn).

## Measurement campaign

In January 2004 measurements on subsoil, sediment transport, active layer, water level, discharge, flow velocity and direction, were performed with 10 ships during high discharges at the IJsselkop and the Merwedekop. The campaigns lasted two weeks. In Fig. 1 the water levels can be seen together with the moments of measurements. Comparable measurements at the Pannerdensche Kop had been performed earlier, in 1998. In September 2004 measurements on sediment transport and discharges were performed at the IJsselkop during low discharges to determine the effect of the weirs in the Neder-Rijn. With the MEDUSA technique (Koomans, 2004) the variation of the natural radioactivity of the sediment was determined, by dragging a sensor that measures the natural radioactivity of the sediments, over the river bed. There is a relationship between the radioactivity and the grain size. Three surveys at different discharges were carried out to determine the change in grain size patterns of the top layer. In this way, it was possible to detect sorting processes and bed roughness patterns.

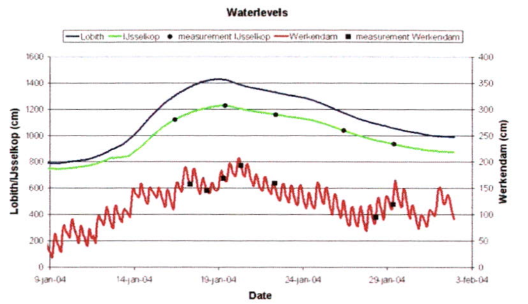

*Figure 1. The water levels during the measurement campaigns at the IJsselkop and at the Merwedekop (Werkendam).*

## Preliminary results

The data of the IJsselkop have been analysed first. In Fig. 2, the results of a MEDUSA scan of the grain size patterns after the peak discharge can be seen. In the Pannerdensch Kanaal the dune height reacts quickly on increasing discharge, with a maximum of ca 50 cm, reached at the peak discharge. Migration speed decreases with increasing dune height and vice versa. The highest and longest dunes were formed in the Pannerdensch Kanaal and the Neder-Rijn. More results are described in the paper of Frings & Kleinhans in this volume.

## Conclusions

A variety of measurements was performed successfully and the measurement results constitute a unique database on the morphological behaviour of bifurcation points. The results from the IJsselkop show a clear differences between the three branches, with respect to subsoil, sediment transport, grain size and dune characteristics. The Merwedekop measurement results are presently under study. Both studies will result in an integral report on the different data.

From the studies around the Pannerdensche Kop a clear view of the distribution of water and sediment on the bifurcation point Pannerdensche Kop has been obtained. The data helped to gain more insight into the morphological behaviour of this bifurcation point. Much of these data were used in two dissertations on the behaviour of sand dunes and sorting processes of sediments.

## Co-operation

This research has been performed under the authority of Rijkswaterstaat (DZH and DON), in co-operation with Utrecht University, TNO-NITG, MEDUSA Explorations BV and the Morphological Triangle of NCR.

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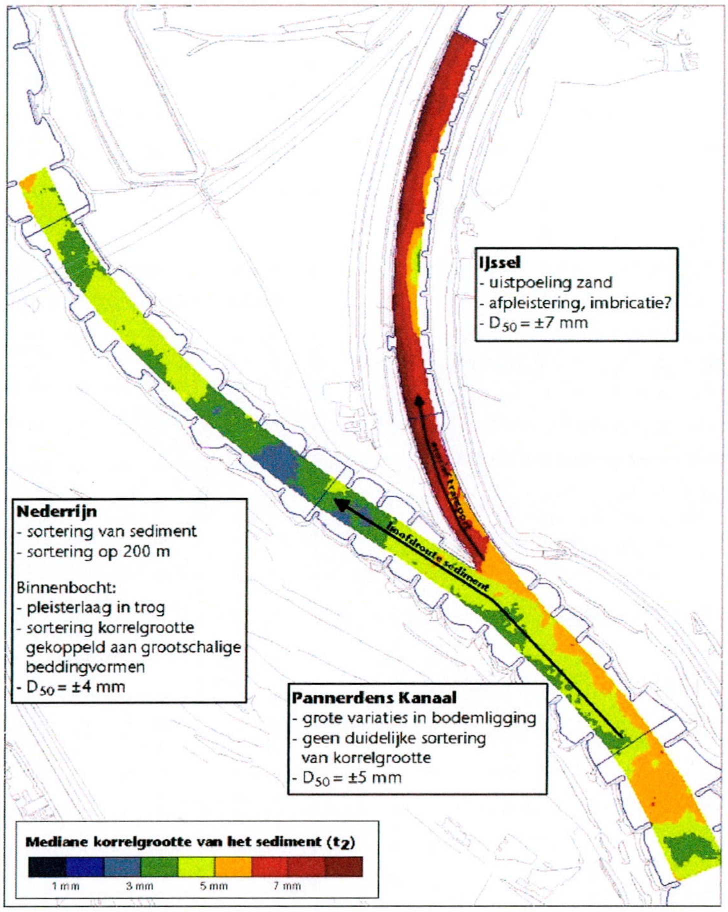

*Figure 2. Spatial distribution of bed material grain sizes near the IJsselkop after peak discharge. The main route of the sediment runs from the Pannerdensch Kanaal to the Neder-Rijn. During high discharges erosion takes place in the IJssel channel. The finer material is washed away leaving coarser material in the top layer.*

# Channel roughness in 1D steady uniform flow: Manning or Chézy?

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## Abstract

In river flow applications, consensus on the most appropriate roughness descriptor has yet to be found. A disturbing observation is that the coefficients and formulae of Chézy, Darcy-Weisbach, Manning, Strickler and White-Colebrook are used rather arbitrarily, and that a widely accepted scientific justification is lacking. The presented paper compares the most commonly used roughness parameters, and reflects on some arguments that are often used in favour of, or against, any of these. Some recent advances on the theoretical basis of different methods are put forward, and implications for commonly used hydraulic modelling packages are discussed.

## Commonly used roughness relations

For open channels, three different formulae are commonly used to describe the relation between the mean flow field and channel resistance in the steady uniform case. For completion, the Darcy-Weisbach equation is included, but will not be further reflected upon, because of its functional equivalence to the Chézy equation. The dates mentioned below for the Chézy, Darcy-Weisbach and Manning formulae are from a historical overview by Rouse & Ince (1957); the White-Colebrook equation was published by Colebrook (1939).

1. Chézy (1769): $U = C\sqrt{Ri}$, where $C = 18\log(12R/k_N)$ "White-Colebrook (1939)"
2. Darcy-Weisbach (1840's-50's): $U = \sqrt{8g/f}\sqrt{Ri}$
3. Manning (1889): $U = (1/n)R^{1/6}\sqrt{Ri}$, where $n = k_S^{1/6}/25$ "Strickler (1923)"

Where $k_N$ is the Nikuradse (1933) roughness height in the White-Colebrook equation (adapted for hydraulically rough flow), and $k_S$ the roughness height by Strickler. Originally, the equations above are all empirical in character, giving each of them validity in at least some specific situations. Since they are supposed to describe the mean flow velocity (U) as a function of roughness (C, n or f) and geometry (hydraulic radius R, slope i), the question arises which formula gives the simplest, yet most widely applicable, representation in a (natural) open channel?

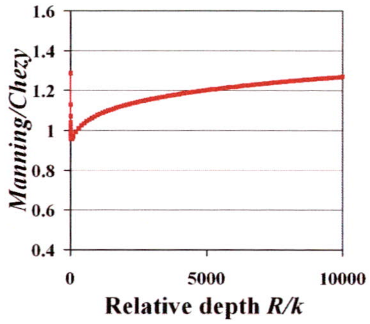

*Figure 1. Comparing the Manning equation (roughness height as in Strickler) with the Chézy equation (roughness height as in White-Colebrook): although the two formulae are clearly not equal, they show similar behaviour with respect to increasing relative roughness (hydraulic radius R/ roughness height k) because of the decreasing gradient of the fraction-function.*

Presently, there is no consensus on this matter and conflicting arguments remain to be heard. It is often argued that the Chézy equation (or Darcy-Weisbach for that matter) has a sound theoretical basis (see for derivation Jansen, 1979), which would justify its wider range of use. However, Gioia & Bombardelli (2002) have shown that similarity considerations of flow in the hydraulically rough regime lead to the Manning equation, where n is a measure of the absolute roughness height (as in Strickler's relation). In this respect it is not yet clear which theoretical foundation is most reliable, leaving the issue unresolved. In Fig. 1 the two formulations are compared with respect to changing relative depth. The figure shows that fundamental differences between the two approaches exist. How this affects performance of hydraulic models is discussed in the following sections.

## Calibration in a simple channel

According to their respective theoretical foundations, both the Chézy and the Manning equation are only expected to be valid in situations where sidewall effects are negligible. As a result, situations with wide channels (depth<<width of channel) have to be considered. Although Fig. 1 shows that both descriptions may differ significantly, still both can be made to fit data by allowing the respective roughness heights in each description to vary independently (assume different roughness height definitions). Consider for example a simple channel of width 100 m where a discharge of 300 $m^3/s$ results in a water depth of 2.5 m (slope i = 0.01 %). In this case the Chézy equation (with White-Colebrook) yields a roughness height of $k_N = 1.8$ mm, while the Manning equation (with Strickler) gives $k_S = 3.1$ mm. If these calibrated formulae are subjected to an increased discharge of 20% (360 $m^3/s$) a water level difference of 1.3 cm results (~3% of total water level rise, see Fig. 2). As could be expected, Fig. 2 shows that the two formulations follow a similar behaviour when made to fit each other at a specific (calibration) point.

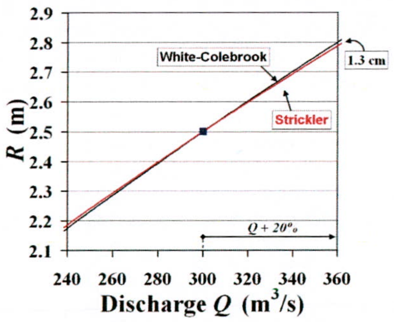

*Figure 2. Difference in predicted water levels (~hydraulic radius R) at 20% discharge increase when both the Manning and Chézy equation are calibrated on $Q = 300\ m^3/s$ and $R = 2.5$ m (see text for chosen geometry).*

## Calibration in a composite channel

In a composite channel the overall roughness may be computed by using a divided channel method (e.g. Chow, 1957; Jansen, 1979; Yen, 2002). This method assumes that flow in each of the subsections is independent from other section and that the total discharge equals the sum of section-discharges. The approach may be expected to be valid if the different sections are wide, such that possible lateral transfer mechanisms at the interfaces can be neglected. In Chow (1957) an indicative value of 10 for the relative width (as compared to water depth) is given for a channel to be considered wide. Consequently, for two-stage channel this argument is taken to count for each of the subsections. Fig. 3 shows a two-stage channel with overall flow conditions that resemble the previously mentioned case in a simple channel: the overall hydraulic radius is 2.5 m, the total width is 100 m and the discharge is 300 $m^3/s$.

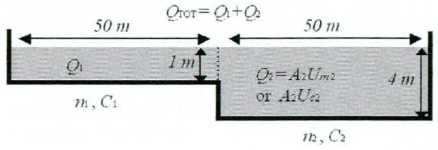

*Figure 3. Cross-section of a two-stage channel (A = cross-section subarea) showing relevant parameters for divided channel method. If the roughness parameters (n, C) in subsections are calibrated to fit a discharge of 300 $m^3/s$ (at a slope of 0.01%), then an increased discharge predicts different water levels between the 2 calibration sets (one based on addition of Chézy formulae $U_C$ in subsections, the other on Manning $U_m$).*

In order to calibrate the composite roughness equations on these specific conditions, roughness parameters for each of the subsections have to be determined. In the case considered here, the following roughness values are made to correspond to flow conditions: Manning's n = 0.05 $s/m^{1/3}$ in the floodplain and n= 0.017 $s/m^{1/3}$ in the main section (corresponding to C=20.0 $m^{1/2}/s$ and C=72.5 $m^{1/2}/s$, respectively). Note that calibration is not performed on roughness heights but on resistance coefficients C and n, as is often the case in practice. In real situations the roughness in the main sections may be determined independently at low water levels (flow below bankfull height). If, again, discharge is increased to 360 $m^3/s$ (+20%), different water levels result from using two different methods. In this case the water level difference turns out to be about 5 cm (~11% of total water level rise), significantly higher than in the simple channel case.

## Conclusions

For both the Manning and the Chézy equation, follow-up work provided a theoretical foundation. However, between the two, there remains a fundamental difference with regard to the dependence on the hydraulic radius (C depends on R, while n does not). In a simple channel the methods show equivalent behaviour when used for extrapolating beyond the level of the highest calibration point. In the specific case presented, a water level difference of 1.3 cm resulted after a 20% discharge increase. While theory shows that the Chézy parameter is a measure of relative roughness height, its value is often treated as a constant in methods to determine a composite roughness in more complex river geometries. This method seems more justified when using the Manning roughness parameter since theory predicts that this parameter is indeed a true measure of (absolute) wall roughness. Calibration of both Chézy and Manning on specific flow conditions consequently gives a much larger discrepancy in predicted water level than in the simple channel case, where calibration is performed on roughness height and thus theory is followed more closely. In the composite channel case presented, the discrepancy amounted to 5 cm (higher water level in Chézy approach) at a 20% discharge increase (equivalent composite characteristics as in the simple channel case).

In conclusion, for composite roughness methods it is advised to apply the Manning approach for roughness calibration, in order to give a theoretically correct weight to the roughness of each subsection.

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# 3D float tracking: in-situ floodplain roughness estimation

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## Abstract

3D float tracking is a new method to quantify hydrodynamic roughness of submerged floodplain vegetation. In this method a floating tripod is released on the inundated floodplain and tracked from shore by a tachymeter leading to a highly detailed representation of the water surface elevation along the flow path. Simultaneously, an Acoustic Doppler Current Profiler collects flow velocity profiles and water depth. Preliminary roughness values, based on backwater-curve modelling and the 1D equation for non-uniform flow, range less than one order of magnitude.

## Introduction

Hydrodynamic roughness of submerged vegetation is an important parameter for river flow models. Roughness values are mostly based on flume experiments, carried out with low water depths and high water surface slopes (Carollo et al., 2002; Wilson & Horrit, 2002). These experiments do not represent the hydrodynamic conditions of lower Rhine floodplains well. For roughness estimation from hydrodynamic parameters detailed information is needed on the water surface slope, water depth and depth averaged flow velocity (Van Rijn, 1994). The local water surface slope has always been a difficult parameter to measure due to the small differences in water level. Therefore the aim of this research is to collect accurate and detailed field measurements of water surface slope, water depth and depth-averaged flow velocity and to determine the hydrodynamic roughness of submerged floodplain vegetation. This can be done with a new method: 3D float tracking.

## Materials and methods

Measurements were taken with a tripod floating on the inundated floodplain (Fig. 1). An Acoustic Doppler Current Profiler (ADCP) mounted underneath the float measured (1) the float velocity using the bottom track option plus (2) deviations from the float velocity in the water column and (3) the water depth. Results were averaged over 5 seconds to decrease noise. Positioning is done with a shore-based tachymeter that automatically tracks the reflector on top of the float. Positioning was done with a 2 Hz frequency. Changes in position gave the float velocity and the water surface slope. This resulted in detailed information for roughness calculation.

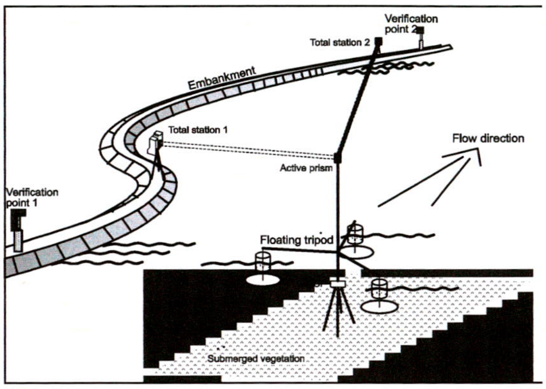

*Figure 1. 3D impression of the 3D float tracking method.*

## Results

In January 2004 the method was tested for the first time on two inundated floodplains: (1) the 'Groene rivier' in Arnhem (Fig. 2) and (2) a floodplain on the river Waal upstream of Zaltbommel. Figure 2 shows the float data of one run in the 'Groene rivier' floodplain in Arnhem. Table 1 shows the key hydrodynamic characteristics of the two floodplains. The water surface slope in Arnhem was measured more accurately than on the river Waal due to the absence of waves. ADCP measurements showed a fair amount of noise. However, averaged flow velocity profiles were consistent from one run to the other.

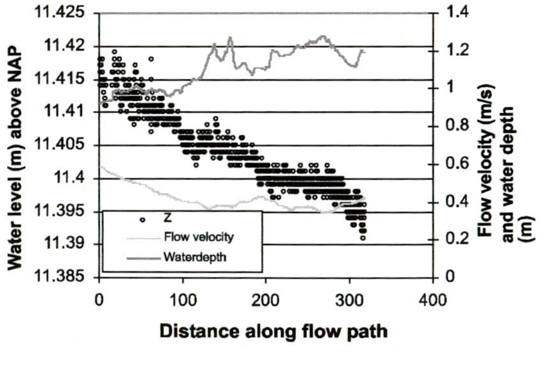

*Figure 2. Float data of one run in the 'Groene Rivier' floodplain in Arnhem. Open dots (Z) represent individual water-level measurements.*

Preliminary roughness values (Nikuradse's k) in Arnhem were 5 cm ± 3 cm, based on the predictor-corrector method and 8 cm ± 10 cm, based on the locally solved 1D equation for non-uniform flow. Roughness values for the Waal floodplain were not calculated due to the large amount of scatter in the local water surface slopes.

## Discussion and conclusions

3D float tracking can supply spatially distributed water surface elevations with unprecedented detail. Together with the flow-velocity and water-depth measurements of the ADCP it generates all necessary data for roughness calculations. However, the accuracy of the method is limited by wave activity.

Derived roughness values are within one order of magnitude. Further improvements are expected from more suitable window sizes for spatial filtering of water surface slope and local flow accelerations. The method can also be extended to roughness measurements of hedges, groynes or minor embankments.

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*Table 1. Key hydrodynamic characteristics of the two floodplains.*

| Floodplain | WSS (cm/km) | Water depth (m) | Flow velocity (m/s) |
|---|---|---|---|
| Arnhem | 5.9 | 1.2 | 0.41 |
| Waal | 8.8 | 2.5 | 0.74 |

WSS = water surface slope

# Variations in roughness predictions (flume experiments)

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## Abstract

Data of flume experiments with bed forms are used to analyze and compare different roughness predictors. In this study, the hydraulic roughness consists of grain roughness and form roughness. We predict the grain roughness by means of the size of the sediment. The form roughness is predicted by three approaches: Van Rijn (1984), Vanoni & Hwang (1967) and Engelund (1966). The total roughness values (friction factors) are compared with the roughness values according to the Darcy-Weisbach equation. Results show that the different methods predict different friction factors. In future research uncertainties in the hydraulic roughness will be taken into account to determine their influence on the computed water levels.

## Introduction

In the Netherlands, the heights and strengths of dikes and other flood defense systems are based on computed water levels which occur during a certain extreme discharge, i.e. the design discharge. The uncertainty in the hydraulic roughness of the river bed is one of the main sources of uncertainty in these computed water levels (Van der Klis, 2003). The purpose of the present research is to compare different state-of-the-art roughness predictors and examine the influence of the roughness predictor on water levels. We use the same approach as Julien et al. (2002). The overall aim of this study is to gain knowledge on the size and type of uncertainties in the hydraulic roughness and their influence on computed water levels.

## Material and methods

Flume experiments were conducted by Blom et al. (2003) in the sand flume facility at WL|Delft Hydraulics in the Netherlands (1997-2000). The experiments were performed under steady uniform flow conditions and sediment from the Waal River (near the Pannerdensche Kop) was used. The experiments were aimed at conditions with bed forms. Their heights ($\Delta$) and lengths ($\Lambda$) were measured, as well as the hydraulic radius (R), flow depth (h), flow velocity (u) and the energy slope (i). We derive the friction factors by means of two different methods. The first method gives the reference values. It uses flow data and the Darcy-Weisbach equation:

$$f = \frac{8gRi}{u^2} \qquad (1)$$

The second method for calculating the roughness is using a roughness predictor. In these experiments the only sources of roughness are grain roughness f ' (caused by the protrusion of grains from the bed into the flow) and form roughness f " (created by the pressure differences over bed forms). The sum of grain and form roughness gives the total roughness. To calculate the grain roughness we distinguish between a roughness height ($k'_s$) of $d_{90}$ and $3d_{90}$. The value of the roughness height can be converted to a value for f ' with the following relation (Van Rijn, 1993):

$$f' = 0.24\left(\log\frac{12R}{k'_s}\right)^{-2} \qquad (2)$$

For calculating the form roughness we study three models. For the Van Rijn (1984) approach (3), a value for $f''_R$ is obtained by applying equation (2) (using $k''_s$ instead of $k'_s$).

$$k''_s = 1.1\Delta\left(1 - e^{-25\frac{\Delta}{\Lambda}}\right) \qquad (3)$$

The other two models are the Vanoni & Hwang (1967) approach:

$$f''_{VH} = \left(3.3\log\frac{\Lambda R}{\Delta} - 2.3\right)^{-2} \qquad (4)$$

and the Engelund (1966) approach:

$$f''_E = 10\frac{\Delta^2}{R\Lambda}e^{-2.5\frac{\Delta}{h}} \qquad (5)$$

## Results and preliminary conclusions

Figure 1 shows some results of the calculations. The experiments T5, T7, T9 and T10 were conducted under different flow conditions, i.e. different discharges, velocities and slopes. All roughness predictors yield a larger friction factor than the Darcy-Weisbach reference value. From other calculations it appears that a difference of 0.05 in the friction factor (f) can lead to a 20 cm change in hydraulic radius (R), and thus a significant change in water levels. The results give a first impression of the uncertain hydraulic roughness and show that variations in friction factors influence calculated water levels.

## Further research

Plans for future research are first to choose the most appropriate roughness predictor (based on the flume experiments). Then, we want to include uncertainties and perform a Monte Carlo analysis to examine the influence of the uncertain hydraulic roughness on water levels. Furthermore, we will examine what the results of the flume experiments mean for field situations.

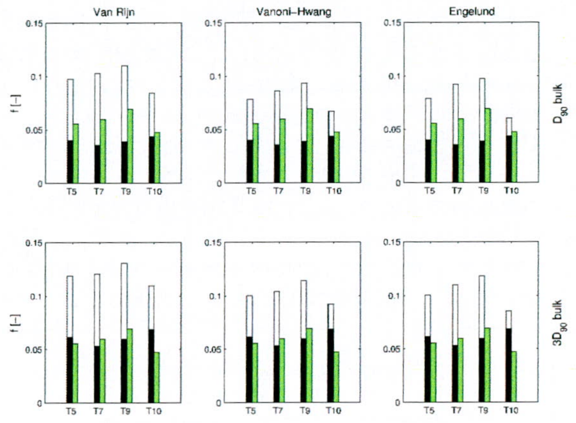

*Figure 1. Friction factors,*
*black = grain roughness,*
*white = form roughness,*
*green = Darcy-Weisbach roughness*

## Acknowledgements

This work is supported by the Dutch Technology Foundation STW, the applied science division of NWO, and the technology programme of the Dutch Ministry of Economic Affairs.

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# Effect of main channel roughness on water levels

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## Introduction

As a result of changing discharge during floods, dunes may develop in the main channel of a river (Fig. 1) leading to a changing bed roughness. This roughness is dynamic since dune dimensions and roughness lag the discharge. In current practice, hydraulic models are tuned using the roughness of the main channel as a calibration factor; the real dynamics of roughness are not taken into account. The aim of our research is to formulate an appropriate model concept for dynamic roughness during floods. This paper presents an analysis of the effect of main channel roughness on water levels. Not the variation of dunes is considered, but only the effect of variations of their roughness. It is also analyzed how the influence of main channel roughness depends on the geometry of the channel.

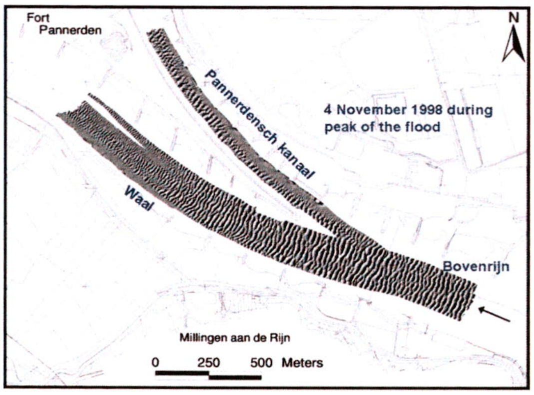

*Figure 1. Dunes at Pannerdensche Kop (Wilbers, 2004).*

## Method

The sensitivity analysis is performed in analogy to Huthoff & Augustijn (2004), using both a numerical and an analytical approach. Floodplain roughness is set constant at $k$ = 0.25 m (Van Velzen, 2003). Calculations are performed for a schematized channel of 100 km as shown in Figure 2a.

In the analytical approach, for simplicity no backwater effects are included. The composite channel roughness ($C_{comp}$) can be expressed as:

$$C_{comp} = \alpha \cdot (D_T)^{3/2} \cdot C_m + (1-\alpha) \cdot h^{3/2} \cdot C_f$$

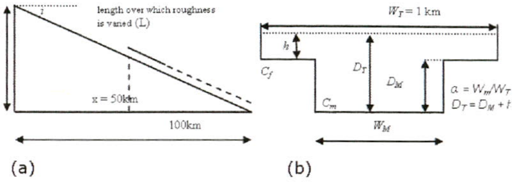

*Figure 2. Model schematization; (a) longitudinal, (b) cross-section.*

Varied parameters are (Fig. 2): discharge: $Q$ = 500 – 10500 m$^3$/s; depth of the main channel: $D_T$ = 3 – 5 m; ratio of main channel width to the total width of the cross-section: $\alpha = W_m/W_T$ = 0.2 – 1; main channel bed roughness: $k$ = 0.05 – 0.65 m, the latter is the maximum value observed during a large flood in the Rhine in 1995 (Julien et al., 2002); length of the area over which the roughness is changed (in the numerical approach): L = 0 – 50 km.

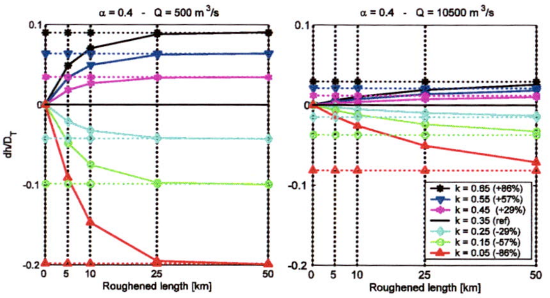

*Figure 3. Comparison of analytical (dashed) and numerical (solid) computations for two different discharges. Water-level differences are scaled with the total depth in the reference case (i.e. $k_m$ = 0.35 m).*

## Results

Results are presented by comparing with a reference case in which the roughness height $k$ of the main channel is 0.35 m. As expected, an increased main channel roughness leads to increased water levels (Fig. 3). At lower water levels (lower discharges), the influence of bed roughness is larger. For a floodplain width of 400 m ($\alpha$ = 0.4 in this case) and a discharge of 10500 m$^3$/s ($D_T \approx 9.9$ m), the influence of main channel roughness on absolute water-level difference varies between -80 and +30 cm. The influence of a changing main channel roughness on the relative water level increases, when the ratio of main channel width to total width (α) increases (Fig. 4), since more water flows through the main channel. For a floodplain width of 400 meter, the influence of main channel roughness on the water level varies between +40 and -100 cm.

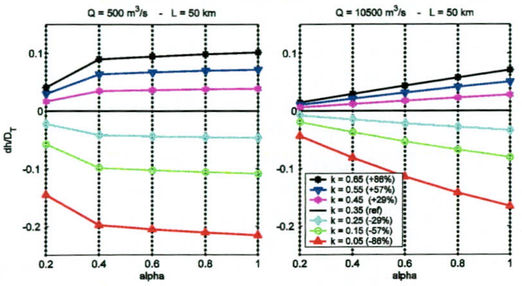

*Figure 4. Dependence of relative water-level difference on main channel roughness and on ratio of main channel width to total width.*

## Conclusions

- The influence of main channel roughness on absolute water levels is significant, and can be up to 100 cm for a typical Dutch situation (α = 0.4).
- The influence of main channel roughness on relative water-level difference ($dh/D_T$) decreases for increasing discharge and floodplain width.
- Dependence on main channel depth is small (not shown).

The numerical computations will be compared with 2D WAQUA computations in future research.

## Acknowledgements

This work is supported by the Dutch Technology Foundation STW, the applied science division of NWO, and the technology programme of the Dutch Ministry of Economic Affairs.

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# Effect of climate change on bedforms in the Rhine and consequences for navigation

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## Abstract

Navigation on the river Rhine is of great economic importance for the Netherlands. Low river discharges or the presence of river dunes on the bed may restrict the water depth available for navigation. River dunes are bedforms that develop at high discharges, as a result of the interaction between flow and sediment transport. Dunes might hinder navigation as their development shows a delayed response to changing flow conditions, because it takes time for a dune to form or to degrade. This means that the maximum dune height is reached when the water depth is already decreasing. Therefore, it is important to know if river dunes will restrict the water depth significantly and whether climate change influences the development of river dunes in the Rhine. From the research it can be concluded that dunes do not significantly influence the hindrance of navigation, neither now, nor in the future.

## Introduction

Many events, like more frequent flooding and extreme drought are addressed to a rising average global temperature. Several scenarios have been developed to forecast the possible consequences of this type of climate change for the Rhine basin. One of them is the UKHI scenario that is used to determine the expected discharges in the Rhine (Middelkoop et al., 2001). This scenario resulted in an increasing discharge in the Rhine in winter and a decreasing discharge in summer. As the development of river dunes depends on the discharge, it is important to know the effect on the dune development, in order to give insight in the effect of dunes on navigation.

Former research on river dunes in the Rhine showed that dunes of about 0.8 m are present at a discharge of 7,000 $m^3/s$. River dunes reach a height of about 1.6 m at an extreme discharge of 12,000 $m^3/s$ (Wilbers, 2004). As a result of climate change, these dune heights might increase.

Next to that, erosion and sedimentation processes (Fig. 1), which play an important role in the dune development, take time. Consequently, river dune development responds to changing flow conditions with a time lag. This means that the maximum dune height is reached a few days after the peak discharge, when the water depth is already decreasing. This time lag could be relevant in the hindrance of navigation by river dunes.

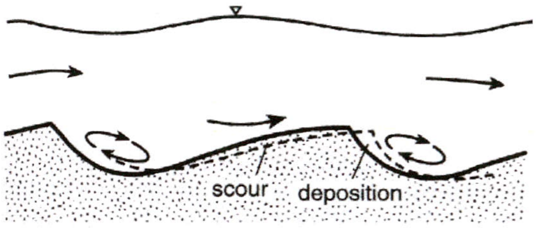

*Figure 1. Process of erosion and sedimentation (after Knighton, 1998)*

## Method

In order to determine the effect of climate change on river dune development, the climate change scenario UKHI (United Kingdom Meteorological Office High Resolution General Circulation Model) was used. This scenario results in factors that can be multiplied with recorded discharges. Recorded discharges of three different years ('base years') with different discharge patterns and peak discharges were selected. Recorded and predicted discharges are used as input for a 1D-hydraulic model for unsteady flows, to compute corresponding water depths. These water depths were used in the calculation of dune heights, with the method of Wilbers (2004). This method is based on the ideas of Allen (1976) and has been specifically developed for the Rhine branches. In order to quantify the hindrance of navigation, the loading capacity of vessels is calculated based on the calculated water depths. This is done for the water depth with dunes and without dunes present.

## Results

Climate change has a significant effect on the development of river dunes. Figure 2 presents the development of dune height in January for base year 2003 and for the expected dune

heights according to the UKHI scenarios for 2050 and 2100. A dune height of 1.2 m is calculated for a peak discharge of 9,000 $m^3/s$ in the base year.
The UKHI 2100 scenario leads to a peak discharge of 14,000 $m^3/s$ and the corresponding maximum dune height is 1.7 m.

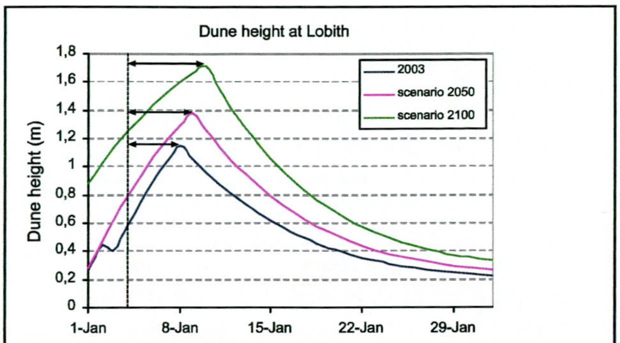

*Figure 2. River dune development in base year 2003 and for the UKHI scenarios (vertical line represents the day of the peak discharge).*

The time lag is clearly perceptible in the figure: the maximum dune height is reached later, while the peak discharge occurs on the same day. The maximum water depth at peak discharge is 14 m for the UKHI 2100 scenario. When the dune height is at its maximum, the water depth has decreased to 12 m, which is still large compared to the dune height of 1.7 m. Despite the delayed response of river dunes to changing discharges, the discharge does not decrease so quickly that the dune height becomes substantial compared to the water depth: after approximately one week the dune height is about 0.9 m, while the water depth is still about 9 m. It can be concluded that river dunes do not restrict the water depth for navigation during winter, because a low discharge does not occur very fast after a peak discharge. In other words: dunes get enough time to decay.

Figure 3 presents the loading capacity of vessels in time to get a better insight in the hindrance of navigation. It is clear that navigation is restricted by low flows in summer, when the discharge is about 900 $m^3/s$. The effect of climate change on the loading capacity is clearly perceptible; in September the loading capacity decreases from about 75% to about 55%. The influence of river dunes, however, appears to be insignificant: river dunes are calculated to be about 0.05 m high during summer. The restriction of the loading capacity by river dunes is only in the order of 1% (in Fig. 3 'with dunes' and 'without dunes') for the base year (1991) as well as for the UKHI 2100 scenario. Although climate change is expected to lead to higher river dunes, these higher river dunes do not further decrease the loading capacity of vessels.

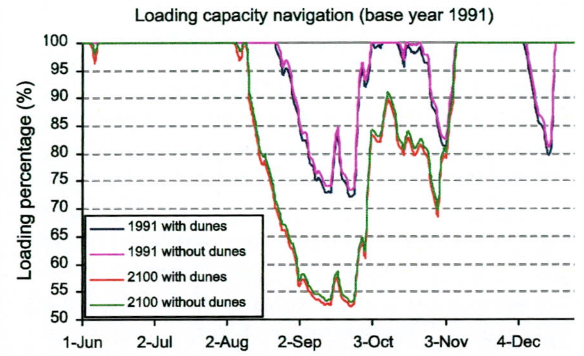

*Figure 3. The effect of climate change and dune height on loading capacity.*

## Conclusions

It can be concluded that river dune development in the Rhine is strongly influenced by changing discharges, as a result of climate change. Higher discharges cause higher dunes and a larger time lag between peak discharge and maximum dune height.

Climate change does influence the hindrance of navigation during summer, due to a decrease of low discharges. However, dune height compared to water depth will always remain such that river dunes have no significant influence on the hindrance of navigation and this effect is not enlarged by climate change.

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# Navigability of the Niederrhein and Waal in the Netherlands; a stochastic approach

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## Introduction

Half of the cargo transport between the port of Rotterdam and Germany goes via the Rhine. Safe, efficient and profitable inland shipping requires a deep and wide navigation channel, now and in the future. Navigability depends on morphological and hydraulic conditions in the river that exhibit spatial and temporal variations, such as bed level and water levels. Whenever the navigation depth is less than required, navigation is congested and/or ships may carry less cargo.

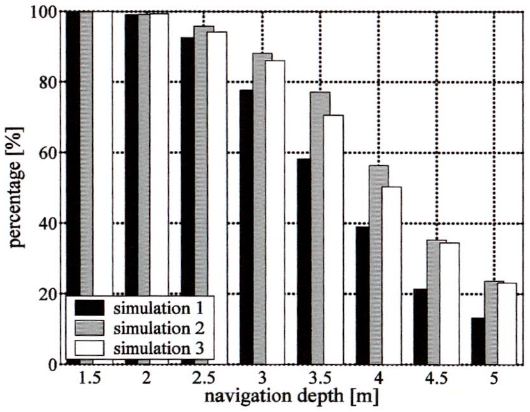
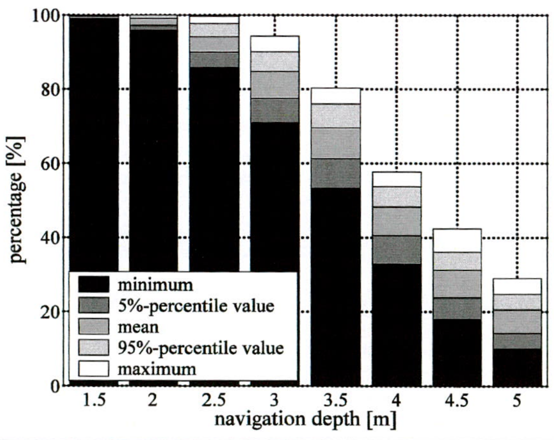


*Figure 1. Percentage of navigable time as a function of the ship draft for the Niederrhein and Waal in the period between 2002 and 2012. Narrow bars represent individual simulations, whereas thick bars in other graph represent aggregates of all simulations.*



## Methods

A 1-dimensional morphodynamic SOBEK model of the Rhine (Jesse & Kroekenstoel, 2001) is used for water-depth predictions. These predictions, in combination with a theoretical correction for the transversal slope in river bends, are used to assess the navigability of the Niederrhein and Waal for ships of various drafts and make it possible to indicate nautical bottlenecks (the reach of the Niederrhein that is situated within the Netherlands is also known as the 'Boven-Rijn'). The Rhine model is affected by various uncertainties, including the model schematisations, the specification of the model input (for example boundary conditions, initial conditions) and the model parameters. Van der Klis (2003) and Van Vuren et al. (2002) have shown that the future discharge hydrograph is one of the important sources of uncertainty.

Monte Carlo simulation, applied to the model, is utilised to quantify the uncertainty in the water depth and thus navigability, given an uncertain river discharge. Monte Carlo simulation (Hammersly & Handscomb, 1964) involves a large number of model runs - 400 runs - with statistically equivalent inputs. For each run a discharge time series of 10-years duration is randomly generated using the Bootstrap-re-sampling technique (Efron, 1982). The outputs of these model runs, can be expressed in terms of expected development and uncertainty of the navigability, which provides insight into the stochastic characteristics of the river's navigability.

The National Traffic and Transportation Plan gives guidelines with respect to the navigation channel requirements. According to this plan, during discharges above a threshold value of 1020 $m^3/s$ at Lobith (where the Rhine enters the Netherlands), the navigation channel in the Niederrhein and the Waal must have a guaranteed width of 170 m and a depth of 2.8 m, respectively. This threshold value is exceeded during 95 % of the time. The probability that these requirements are satisfied in the Dutch part of the Rhine for a period of 10 years is evaluated in this paper. The largest cargo ships in the Rhine have a draft of approximately 4 m.

Additionally, the navigability is assessed for drafts ranging from 1.5 to 5 m.

## Results

The navigability of the Niederrhein and the Waal for ships with a draft between 1.5 and 5 m is statistically assessed on the basis of 400 model runs. Each model run driven by one of the synthesised discharge time series results in one possible future morphological evolution. Figure 1 (left diagram) shows that the navigable percentage in the 10-year period between 2002 and 2012 differs for each synthesised discharge time series. Using the results of all model simulations, the statistical characteristics of the navigable percentage are derived (Fig. 1; right diagram). For example, the percentage of navigable time for ships with a draft of 3 m at the Niederrhein and the Waal, averaged over a period of 10 years, is 84%. Figure 1 (right diagram) also shows that for this draft there is a 90% probability that the percentage of navigable time lies between 78% and 91%.

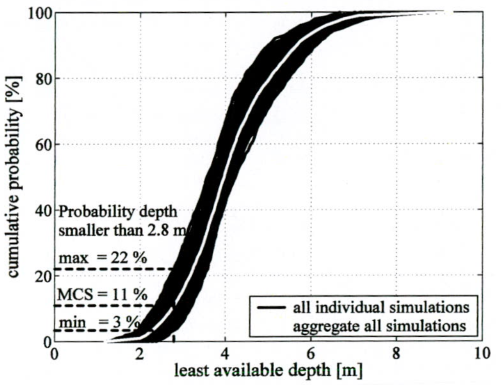


*Figure 2. Cumulative probability distribution of the least available depth for all simulations individually (black lines) and the aggregate line (white line) together.*

The probability of not fulfilling the navigation channel requirements of the National Traffic and Transportation Plan for ships at draft 2.8 m is of interest of both the river manager and the users of the inland waterway. This indicates how well the river manager maintains the required navigation condition. Figure 2 shows the cumulative distribution function of the least available navigation depth over a period of 10 years for all simulations individually and the aggregate line of all simulations together. The figure indicates the probability of meeting the requirements, i.e. ships can navigate at draft of at least 2.8 m, is 89.1 % (100 % minus 10.9 %) . The figure shows that this percentage of navigable time is at maximum 96.6 % and at minimum 77.9 % for the 400 model simulations considered herein.

Figure 3 illustrates the percentage of navigable time as a function of the river location for a draft of 2.8 and 4 m. This provides insight into which locations are critical to the navigability of the river. It appears that locations with strong spatial changes in geometry, such as the bifurcation Pannerdense Kop (km 867), the river bed protection in the river bend near Nijmegen (km 882 - 885), the variation in floodplain width in the Midden-Waal and the sharp bend at section Pannerdense Kop-Nijmegen and St.Andries, may evolve into navigation bottlenecks. Most of these bottlenecks become manifest in the dry period.

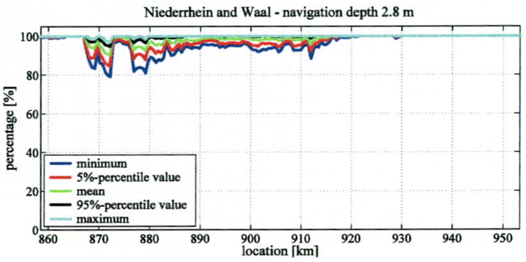
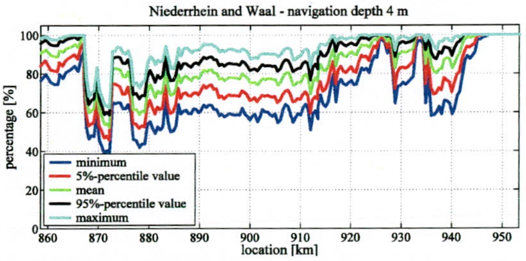


*Figure 3. Percentage of navigable time over a period of 10 years as a function of the river location for ships of 2.8 m and 4 m draft.*



## Conclusion and future research

The foregoing showed that the water depth in the Niederrhein and the Waal exhibit a strong spatial and temporal variation. The uncertainty in the future discharge hydrograph, in combination with strong spatial variations in the river geometry, leads to significant uncertainties in the predicted response. Some locations could develop into nautical bottlenecks, the removal of which may involve high costs.

The stochastic method that is proposed in this paper could be used to assess the impact of various human intervention measures on the river's navigability. In addition to the expected impacts, the change in maintenance requirements (and so costs) can be assessed. This is subject of further research. We have used a 1-dimenional model in this study. This means that the impact of two-dimensional features, such as alternate bars and transverse bed slopes in bends, are not considered. Neither is the fact that large floodplain areas along the Rhine branches are located alternately at the right and the left side of the river, which under flood conditions may lead to strong 3-D cross-flows over the main channel. It is therefore recommended to repeat this kind of analysis with a model capable of describing these phenomena.

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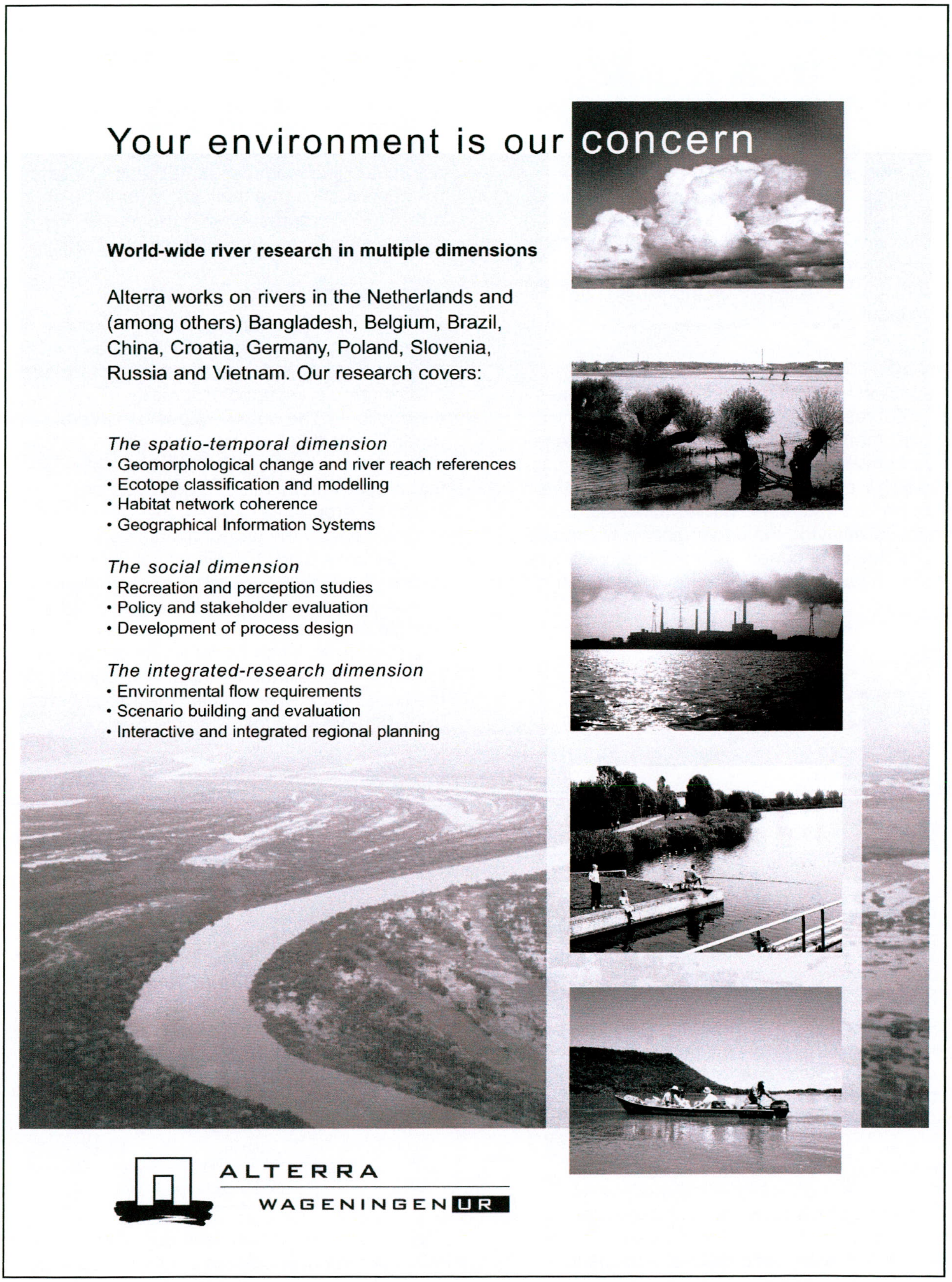

# Stochastic modelling of two-dimensional river morphology

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## Abstract

Uncertainty in the results of a 2-D river morphological model of the Upper Rhine due to the uncertainty in the river discharge is estimated by applying a Monte Carlo simulation. As a possible more efficient alternative, the applicability of the Quasi Monte Carlo method is studied. First results of the convergence rate of statistical quantities of the river bed are promising. Based on these results, further research on this subject has been planned.

## Introduction

The importance of knowledge of uncertainties in model results is more and more recognised by river managers. The quantification of these uncertainties, however, is often difficult. In this research we focus on a specific aspect of this problem, applied to a 2-D river morphological model: quantifying the uncertainty in the model results due to uncertain inputs. Thus, we leave uncertainties in the model structure and the modelling context out of consideration.

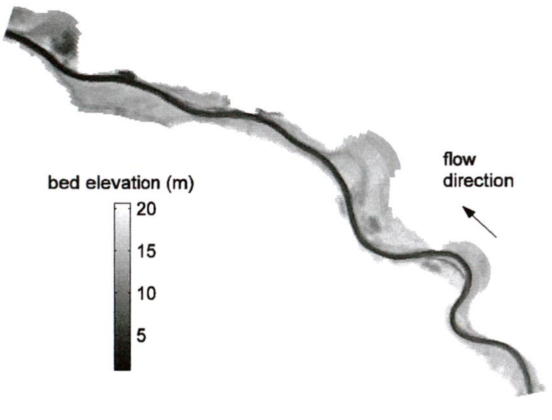

*Figure 1. Overview of the morphological Delft3D model.*

A (non-linear) river morphological model generally requires a Monte Carlo-like approach to estimate the uncertainty in the model results due to uncertain model input (Van der Klis, 2003). Such an approach is robust and much statistical information can be obtained from the results. An important disadvantage of the method, however, is the number of model simulations required. In case of the large, computational intensive models we talk about, a standard Monte Carlo (MC) simulation is practically impossible. Therefore, we search for alternative Monte Carlo approaches which lead to drastic decrease in the required number of simulations.

As a case study, we choose an existing 2-D river morphological model, namely a Delft3D model of a reach of the Upper Rhine (Baur et al., 2002). The model schematisation includes floodplains such that their effect during high discharges is represented in the model (Fig. 1). The model area has a length of 42 km.

## A Monte Carlo simulation

A MC simulation consists of a large number of deterministic simulations, of which the uncertain model input is randomly generated according to prescribed probability distributions. The output values constitute random samples from the probability distribution of the output. Standard statistical techniques can be used to estimate the statistical properties of the model output and the precision of the output distribution.

We carried out 100 simulations, each forced at the upper boundary by a randomly drawn 3-years discharge series. In order to sample discharge series, we derived a statistical description of the Rhine discharge at Lobith. We based the statistical description of the Rhine discharge on daily discharge data from 1946 to 2000 following the method previously applied by Van Vuren et al. (2002).

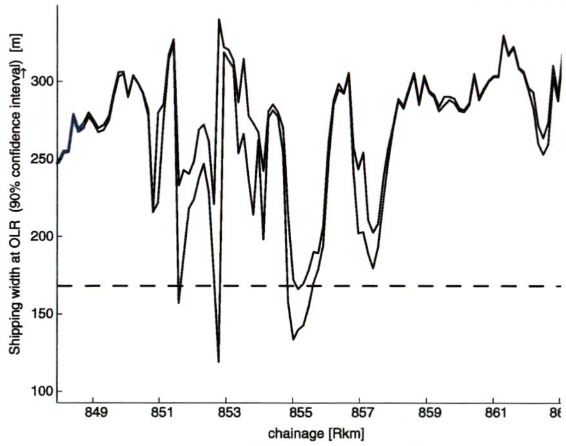

*Figure 2. The 90% confidence interval of the shipping width (area between the two lines) based on a minimum water depth of 2.8 m during OLR-conditions.*

The MC simulation results in an estimate of the effect of the uncertainty in the river discharge on the river bed changes in the model. Figure 2 shows an example of the information that can be obtained: the 90% confidence interval of the shipping width in a part of the modelled river reach. This type of information can help a river manager to decide where problems might occur for shipping.

## An alternative: Quasi Monte Carlo

The sampling method used in a standard MC simulation is rather inefficient: in order to double the accuracy of the estimate of the output uncertainty, four times the number of samples is required. Many alternative sampling methods have been developed to improve the efficiency of the Monte Carlo simulation. In this study, we examine the applicability of the so-called Quasi Monte Carlo (QMC) approach to our case study.

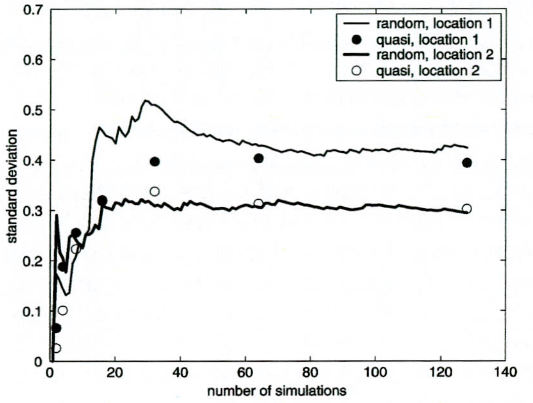

*Figure 3. Convergence rates of the standard deviation of the river bed level at two locations within the main channel, both for the random MC simulation and the quasi MC simulation.*

The QMC method is the deterministic version of the standard Monte Carlo method, in the sense that the random samples in an MC simulation are replaced by well-chosen deterministic points. Various methods have been developed to create sequences of those deterministic points. Often applied is the Sobol sequence, or $LP_\tau$ sequence. Homma & Saltelli (1995) compared the efficiency of various sampling methods and showed an evident advantage of using the $LP_\tau$ sequence.

This gives us sufficient ground for examining the applicability of the method to a river morphological model.

To test QMC to our case study, we simplified the description of the discharge series following Chapter 4 in Van der Klis (2003). With this simplified description we performed a QMC simulation and, for comparison, a standard MC simulation. Figure 3 illustrates the convergence rate of each of these methods. For two locations within the main channel of the modelled river reach the convergence of the standard deviation of the bed level is shown. This figure shows a relatively fast convergence of the QMC results.

## Conclusions and further research

The first results presented here are promising enough to further explore the possibilities of QMC in our model. In this, we will try to apply the method to more advanced descriptions of the river discharge. Furthermore, we will study whether QMC is applicable to other uncertain model parameters.

## Acknowledgement

The model of the Upper Rhine and the discharge measurements have been made available by RWS, DON and RIZA.

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# Sediment density stratification and river channel patterns in the lower Yellow River, China

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## Abstract

The lower Yellow River is characterised by a braiding channel pattern, which changes into a meandering pattern in the downstream direction. Although the bed-level gradient is important for this transition, the sediment concentration plays an additional role. At low sediment concentrations the flow is sub-saturated, leading to a meandering pattern. At higher concentrations the turbulent structure of the flow is suppressed and the sediment concentration profile collapses, leading to a braiding channel pattern. At even higher sediment concentrations, sediment is held in suspension by hindered settling, and the channel pattern becomes increasingly meandering.

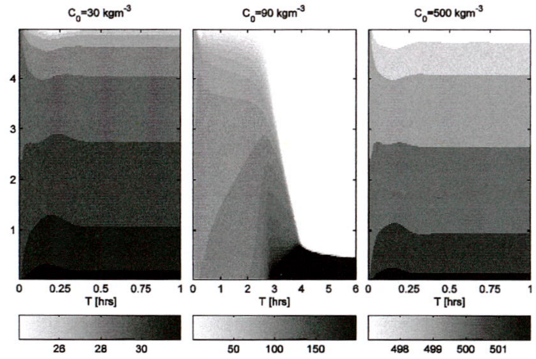

*Figure 1. Evolution of vertical sediment concentration profile during sub-saturated (C = 30 kg/m$^3$), supersaturated (C = 90 kg/m$^3$), and hyperconcentrated sub-saturated (C = 500 kg/m$^3$) flow conditions. The flow velocity is 2 m/s and the grain size 40 microns.*

## Introduction

The Chinese Yellow River carries huge amounts of suspended sediment, especially during hyperconcentrated floods when sediment concentrations exceed 100's kg/m$^3$. A large part of this sediment load is deposited in the lower Yellow River, leading to a rapidly rising floodplain and therefore increasing flood risks. In order to manage these siltation problems a 3D morphodynamic model is being developed for the lower Yellow River within Delft3D.

However, the sediment concentration in the Yellow River is so high that sediment density stratification is important for sediment transport mechanisms and river morphology. The relationship between these stratification processes and river channel patterns will be described here shortly.

## Vertical sediment density stratification

At low sediment concentrations, the downward motion of sediment particles is balanced by a net upward transport of sediment by turbulent motions, resulting in a typical Rouse sediment concentration profile. However, at high concentrations (order 1's to 10's kg/m$^3$, depending on the grain size) these turbulent motions are suppressed by the sediment concentration gradient (Winterwerp, 2001). Therefore the turbulent motions are no longer able to hold sediment in suspension and at a critical sediment concentration, the sediment concentration profile collapses: the flow changes from sub-saturated into super-saturated flow. At even higher sediment concentrations (order 10's to 100's kg/m$^3$), sediment is additionally held in suspension by hindered settling. Therefore the sediment concentration profile is re-established even though turbulence is low: sub-saturated hyperconcentrated flow (Winterwerp et al., 2003). This collapse and build-up of the vertical sediment concentration profile can be simulated with a 1DV version of Delft3D model that includes sediment density effects (Fig. 1). The sediment concentration at which the sediment concentration profile collapses or is re-established is the saturation concentration. This saturation concentration initially increases with the (dimensionless) flow strength [$u^3/ghw_0$ in which: u = depth-averaged velocity (m/s), g = gravitational acceleration (m/s$^2$), h = water depth (m), $w_0$ = sediment settling velocity in clear water (m/s)], but decreases when hindered settling effects become important. This means that at very high sediment concentrations, the flow strength required to keep sediment in suspension is low. Numerically modelled relations between flow strength and saturation concentration matches sediment concentrations observed in the Yellow River (Fig. 2).

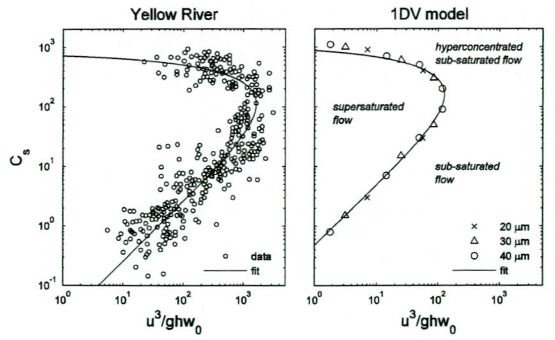

*Figure 2. Relationship between dimensionless flow strength $u^3/ghw_0$ (see text for explanation of symbols) and saturation concentration $C_s$ in the Yellow river (left panel) and based on 1DV results (right panel). Flow velocities during the 1DV simulations ranges from 0.5 to 3 m/s, and the $D_{50}$ between 20 and 40 microns.*

## River channel patterns

The upper part of the Yellow River is braiding, but changes into a meandering pattern in the downstream direction. And although this transition mainly results from a decreasing bed level gradient, field observations show that the Yellow River becomes increasingly meandering at concentrations below 30 kg/$m^3$ and above 200 kg/$m^3$, and braiding at intermediate concentrations (Xu, 2004). However, the reasons for this behaviour are not yet fully understood. The super-saturated flow conditions described in the previous section are characterised by deposition, whereas the sub-saturated flow conditions are characterised by bed erosion. This strongly suggests that the vertical sediment density effects discussed in the previous section are important for the morphology of the Yellow River. To verify this, 3D modelling experiments were carried out to identify the effect of hyperconcentration on the development of river morphology. Starting with an initially flat but randomly perturbed bed, a braiding river channel develops during relatively low sediment concentrations (upper panel in Fig. 3). However, when an additional wash load fraction of 100 kg/$m^3$ is included (hyperconcentrated flow), a meandering channel pattern begins to develop (lower panel in Fig. 3).

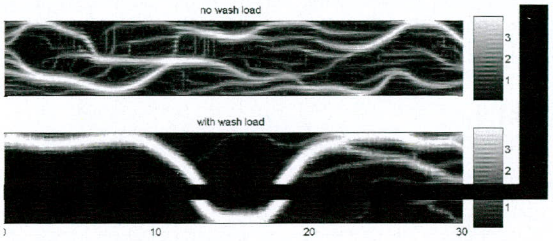

*Figure 3. Channel development after approximately one year in an initially flat bed using a slope of $2.2 \cdot 10^{-4}$, a discharge of 5000 $m^3$/s, and sediment with $D_{50}$ = 65 microns. X-axis and Y-axis are in km and the water depth in m. The simulations in the upper panel are without a wash load fraction whereas a wash load fraction with C=100 kg/$m^3$ and $D_{50}$ = 5 microns is included in the lower panel. Although this wash load fraction does not interact with the river bed, it results in hyperconcentrated flow and consequently a strongly incising meandering channel.*

## Conclusions

The Yellow River channel pattern partly depends on the sediment concentration with a meandering pattern at low and high sediment concentrations, but a braiding pattern at intermediate concentrations. This is caused by sediment density stratification, and can be numerically simulated with Delft3D.

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# Causal relationships between climate change and natural river behaviour in the Rhine delta during the last 15,000 years

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## Abstract

The Rhine delta in the Netherlands developed during the last 15,000 years under influence of tectonics, sea-level rise and, most importantly, the sediment flux from the hinterland. Changes in this sediment flux since the last Glacial-Interglacial transition, appear to be mainly related to changes in climate, land use and vegetation in the upstream part of the catchment, causing variations in sediment delivery from the German part of the basin (e.g. Berendsen et al., 1995, Vandenberghe, 1995). This Ph.D.-project (2004-2007) focuses on the relationship between upstream sediment delivery in the Rhine drainage basin and downstream sedimentation in the delta as a result of vegetation changes since the end of the Weichselian.

## Introduction

Natural river behaviour, i.e. sedimentation dynamics and fluvial style, in a delta area is controlled by several factors (Fig. 1). Allogenic influences on fluvial systems at drainage basin scale and on the time scales of 1,000 to 100,000 years are tectonics, climate and sea level-change. Although tectonics and sea level do have large effects on fluvial systems such as the river Rhine, climate (temperature and precipitation) ultimately controls river discharge and sediment supply and, thereby, the dynamics and size of sediment fluxes in a fluvial system. Beside climate change, land-use changes have a direct impact on the vegetation cover and hence, subsoil cohesion and effective runoff. All this results in variations in discharge and sediment load of the river Rhine in time, and eventually, in variations in downstream sedimentation dynamics and fluvial style.

Although a relationship between natural changes in climate, vegetation, and sediment delivery in the upstream area and the associated sedimentation downstream in the delta is obvious, this relationship has never been quantified.

With the current amount of data present in the whole drainage basin, it is now timely to make the link between upstream erosional phases and downstream sedimentation in order to obtain a better insight in the evolution of the delta and to determine causal relationships between climate change and natural river behaviour.

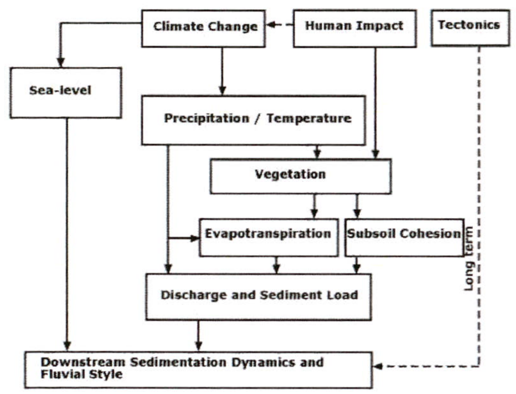

*Figure 1. Factors controlling sedimentation dynamics and fluvial style in a delta area*

## Approach

The Netherlands is in the unique position of having a near-complete capture of the Rhine sediment flux within the Rhine-Meuse delta for the later part of the Holocene (Berendsen & Stouthamer, 2001). Decades of palaeogeographic research makes the Rhine delta now one of the best-studied deltas in the world. This research enabled a detailed palaeogeographic reconstruction of the Dutch Rhine-Meuse delta during the last 15,000 years (Berendsen & Stouthamer, 2001). With the data from the extensive Rhine-Meuse delta database, three large north-south sections (length: ca. 20 - 50 km) were constructed. Together with two already existing sections (Törnqvist, 1993; Cohen, 2003) and complementary data, these sections will be used to obtain a detailed (100 m core spacing) Holocene stratigraphy, showing distinct differences in accumulation rates. A large number of radiocarbon dates provide time lines in these sections. Subsequently, deposited volumes of Rhine sediment during the last 15,000 years in the Rhine delta can be calculated per time slice, providing depositional rates.

During the last 15,000 years, climatic change induced large-scale changes in vegetation in the German part of the Rhine drainage basin. Several regional German studies show a distinct human impact on the landscape from 6400 cal yrs BP onward, leading to vegetation change and enhanced soil erosion (e.g. Lang et al., 2003). Because there is a strong link between vegetation and fluvial dynamics (e.g. Dambeck & Thiemeyer, 2002), vegetation changes are likely to have caused variations in upstream sediment delivery in the Rhine drainage basin. Phases of (increased) siliciclastic input in the German part of the basin will be determined and correlated with phases of downstream deposition, in the Rhine delta. It is expected that changes in sediment discharge during the last 15,000 years can be linked to climate change and, for the last 5000 years, increased human impact.

## Perspectives and relevance

Changes in land-use and climate are likely to affect rivers and their catchments during the next centuries, altering flows of water and sediment, which will have a large impact on the Rhine delta in the Netherlands. For the river Rhine, modelling results suggest a more frequent occurrence of abnormal low and high discharges in the near future. In addition, suspended sediment concentrations in the river are expected to increase due to an increased production of sediment by soil erosion (Middelkoop, 1997).

These future changes will be superimposed on changes triggered in the past. Therefore, a better understanding of (past) fluvial responses to land-use and climate change is needed. Until now, most research has been conducted in small catchments, while the response of large catchments has a more comprehensive impact, which is especially relevant for the Netherlands. The time period over which a large catchment responds to land-use or climate change is much longer than for small catchments and much longer than most instrumental time series. This means that palaeoenvironmental reconstructions are essential, because only these reconstructions provide a record of catchment responses on century to millennia time scales.

Quantifying the link between upstream erosional phases and downstream sedimentation in the Rhine drainage basin, will improve our insight in the evolution of the delta, including the causal relationships between (future) climate and land-use changes and natural river behaviour.

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