THE MODELLING OF SUBMERGED VANES
A Means Of Fairway Improvement In River Bends

Part 1  Dynamic Bed Behaviour Under Influence Of Submerged Vanes
Part 2  Analysis Of Model Description Of Submerged Vanes

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering of Delft University of Technology.

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Preface

This report is the result of a study in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering of Delft University of Technology. It describes the results of a study of the dynamic behaviour of a river bed under influence of submerged vanes, focused on River Waal. Secondly it addresses the model description of the morphological impact of submerged vanes.

The study was conducted by F.E. Wiersma under the direct supervision of prof.dr.ir. H.J. de Vriend., professor in river morphology and river engineering. Furthermore ir. H.J. Opdam (Haskoning Consulting Engineers and Architects) and dr. Z.B. Wang (Delft Hydraulics) were members of the supervising committee.

The project was financed by Rijkswaterstaat Directie Oost-Nederland (part of the Dutch Ministry of Transport, Public Works and Water Management) and is related to the Waal Project, aiming at the realisation of a waterway improvement scheme for River Waal.

The author wishes to thank ir. E.S.P. Smit and ir. H. Havinga for their kind cooperation.

Furthermore Delft Hydraulics was closely involved, providing close support in the modelling work. The author gratefully acknowledges ir. N. Struiksma, ir. C. Flokstra and R. Witteveen for their kind assistance and lasting patience.

Finally this study project was initiated by ir. H.J. Opdam from Haskoning Consulting Engineers and Architects, who is gratefully thanked for his comments and enthusiastic support.

Summary

Recently submerged vanes have come to the attention of Rijkswaterstaat as an option to ensure a sufficient navigable width and depth for some bends in the main Dutch rivers. The submerged vanes can counteract the spiral flow, thus reducing the typical lateral bed slope in river bends. Model calculations by Delft Hydraulics and Haskoning Consulting Engineers and Architects, focusing on the bend in River Waal at Hulhuizen, indicate that an adequately designed vane field can make the river bend meet the norms for navigation. This work was based on a constant discharge, assumed to be representative for the river regime.

Part 1 Dynamic Bed Behaviour Under Influence Of Submerged Vanes
In the first part of this study the influence of discharge variations on the bed topography in this bend under the influence of submerged vanes is investigated, together with the resulting navigable width. For this purpose a calculation procedure was developed for the 2DH morphological model Rivcom, that allows varying boundary conditions.

With this model the equilibrium bed topography at various constant discharges was studied and compared for the situation with and without vanes. The most important changes in bed level occur in the downstream part of the bend, and these differences are reduced by the presence of submerged vanes.

Secondly the bed topography transition to a new equilibrium situation after the installation of the vanes was studied. The time needed for this transition is significantly shortened by floods.

Bed level variations, as a result of discharge fluctuations over the year, can result in important variations of the navigable width at the normative bed level of OLR -2.80 m. Especially during a long term dry period this width is reduced significantly, while at the same time there is a low water level in the river.

Finally the rise in the upstream water level caused by the vanes was estimated from the model results. Rivcom predicts a rise of approximately 2 mm at high discharges. During low discharge the most important factor in this rise turns out to be the change in the river bed topography as a result of the submerged vanes, whereas the direct influence of the vanes themselves is negligible. These do dominate the water level rise at high discharge.

Part 2 Analysis Of Model Description Of Submerged Vanes
In the second part of this thesis the problem is addressed of describing submerged vanes in a numerical model. For this purpose vane-induced near-bed velocity measurements were used from a large number of physical model tests.

An enhanced formula was sought, describing the generated transverse near-bed velocities as a function of the vane dimensions. This did not lead to a satisfying description, better results are expected to be achieved by studying the vane-induced transverse near-bed flow and its streamwise damping.

The placement of vanes in an array results in a significant reduction of the generated average near-bed velocities, and thus the morphological impact of a vane. This is caused by an increased damping of the vane vortices and counter-rotating vortices developing in between. The occurrence of these phenomena is strongly depending on the transverse vane spacing in relation to the vane height and the water depth.

For a valid numerical model description it is unavoidable to include this reduction of the vane efficiency. It was not possible to derive such a description from the physical model tests with vane arrays available.

Vane efficiency in the numerical Rivcom model was calibrated based on one mobile bed scale model test. Fortunately the vane spacing was comparable to the vane field design for River Waal, nonetheless it is appropriate to verify this calibration with other mobile bed tests.
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Lateral gradient in depth averaged inflow velocity at upstream model boundary

\( S \)  
Total sediment transport through river

\( T_{b} \)  
Adaptation time for river bed perturbation

\( T_{c} \)  
Torque of centripetal acceleration around river axis

\( T_{v} \)  
Torque of vane lift forces around river axis

\( T_{rel(axis)} \)  
Relaxation time of bed level indicator, defined in Paragraph 4.6

\( X \)  
Streamwise coordinate standardized with Rivcom vane equation

\( Y \)  
Peak transverse near-bed velocity standardized with Rivcom vane equation

\( \alpha \)  
Angle of attack of submerged vane

\( \beta \)  
Calibration coefficient in spiral flow intensity equation

\( \delta \)  
Transverse spacing of vanes in an array

\( \varepsilon \)  
Eddy viscosity

\( \zeta \)  
Position factor in relaxation time estimate defined in Paragraph 5.5

\( \theta \)  
Shields parameter

\( \theta_{cr} \)  
Critical shields parameter

\( \kappa \)  
Von Karman constant = 0.4

\( \lambda_{b} \)  
Adaptation length of river bed

\( \lambda_{s} \)  
Adaptation length of spiral flow

\( \lambda_{w} \)  
Damping length for vane-induced peak transverse near-bed velocity

\( \lambda_{o} \)  
Adaptation length of main flow

\( \mu \)  
Transverse coordinate of vane induced near-bed peak velocity

\( \mu_{fr} \)  
Fitted transverse coordinate of vane induced near-bed peak velocity

\( \mu_{m} \)  
Manual calibrated transverse coordinate of vane induced near-bed peak velocity

\( \xi \)  
Discharge factor in relaxation time estimate defined in Paragraph 5.5

\( \rho \)  
Density of water

\( \tau \)  
Shear stress

\( \tau_{b} \)  
Bed shear stress

\( \tau_{c} \)  
Vane-induced bed shear stress

\( \tau_{b} \)  
Bed shear stress

\( \psi \)  
Calibration coefficient in Rivcom averaging of vane-induced near-bed velocities

\( \Delta \)  
Relative density of sediment

\( \Delta b \)  
Width of Rivcom grid cell

\( \Delta h_{upstream} \)  
Rise in water level upstream of vane field

\( \Delta i \)  
Increase of streamwise water surface slope

\( \Delta l \)  
Length of Rivcom grid cell

\( \Delta s \)  
Streamwise interval

\( \Delta t \)  
Morphological time step in Rivcom model

\( \Delta M \)  
Vane-induced difference in momentum flux

**Subscripts**

- \( b \)  
River bed
- \( c \)  
River centerline / Rivcom cell
- \( end \)  
Final situation
- \( equi \)  
Equilibrium situation
- \( fit \)  
Value used in fitted function
- \( meas. \)  
Measured value
- \( p \)  
Peak value
- \( Riv \)  
Used in Rivcom
- \( Riv_{int} \)  
Rivcom transverse near-bed vane velocity integration/averaging
- \( s, n \)  
Direction along \( s \) and \( n \) coordinates
- \( v, vane \)  
Situation with vanes
- \( x, y \)  
Direction along \( x \) and \( y \) coordinates

---

ix
1. Introduction

1.1 Increase of Navigation Intensity on River Waal

The River Waal is a very busy navigation route from the Rotterdam Harbor to the German hinterland. Prognoses indicate a growth from 171,000 passing ships, carrying 140 million tons of cargo, in 1989 to 215,000 in 2010, carrying 160 to 220 million tons. Furthermore an increasing number of large push tugs is expected, beside river boats.

Plans from Rijkswaterstaat, part of the Dutch Ministry of Transport, Public Works and Water Management, give priority to assure that the River Waal will continue to have a high-quality transport function that allows the safe and quick passage of these ships. For this purpose the Waal Project was initiated, aiming at the realization of a waterway improvement scheme for the river.

Navigable Width Problems

The functionality of a river for navigation is predominantly determined by the available water depth and navigable width. Since 1947 international agreements for River Waal aim at maintaining an effective navigable width of 150 m at a bed level of OLR -2.50 m. OLR (Overeengekomen Lage Rivierwaterstand) is a sloping reference level, equivalent to the water surface along the river at a discharge that is exceeded during 95% of time on average.

An economic and navigation optimization study by Rijkswaterstaat [1993] resulted in the current norm for a minimum navigable width on River Waal of 170 m at a bed level of OLR -2.80 m.

Figure 1.1 indicates the stretches of River Waal that did not meet this norm at the onset of the Waal Project. As a consequence the safety for navigation is threatened in river bends and ships are forced to reduce their draught during dry periods, thus becoming less cost efficient. Rijkswaterstaat estimates that this leads to an increase of the annual transportation costs by /50 to /200 million.
Improvement Works

Rijkswaterstaat has made an inventory of the most important problems concerning the currently available navigable width. The river bends upstream of Nijmegen at Hulhuizen, Erlecom and Haalderen (depicted on the map in Figure 1.2) turned out to be important bottle necks, beside stretches between Nijmegen and Haalderen and near IJzendoorn. Some improvement works have been carried out, a fixed layer was deposited in the bend at Nijmegen in 1986 and underwater groins were built in the bend at Erlecom recently.

Rijkswaterstaat is now developing plans to deal with the remaining problems, starting with the river bend at Hulhuizen. Close to Pannerdensche Kop the River Waal bends gently to the right (Hulhuizen I), 2 kilometers further downstream a sharp left-hand bend (Hulhuizen II) creates a main navigation bottle neck. The minimum radius of this bend is 1100 m, while the currently available width at a bed level of OLR -2.80 m is 115 m only.

1.2 Lateral Bed Slope In River Bends

While in straight river stretches most problems occur as a result of shallowness in the entire cross section, in river bends often only a narrow lane in the outer bend has a sufficient depth. This is a result of a lateral bed slope, caused by a spiraling water motion.

In a river bend the water is flowing along a curved path and therefore undergoes a centripetal acceleration, which is supplied by a transverse water surface slope. This slope causes a uniform lateral acceleration in a vertical column of water, but as a result of bed friction the flow velocities are smaller near the bed than at the water surface.

As a consequence, water flowing near the bed follows a curve with a smaller radius than the river centerline and water flowing near the surface follows a curve with a greater radius. Thus there is a transverse velocity component towards the inner bend in the lower part of the water column and towards the outer bend in the upper part. Continuity requires water flowing downward in the outer bend and upward in the inner bend.

This phenomenon can be described as a transverse circulation of the water superimposed on the main flow following the river bend, yielding a spiral motion, as is depicted in Figure 1.3. Since most sediment is transported near the river bed, this would result in a transport of sediment from the outer bend towards the inner bend. Actually this transport potential is balanced by the transport potential resulting from gravity acting downwards along the lateral bed slope towards the outer bend.
Indicated areas are enlarged in Figures 3.7 and 3.10.

Figure 1.2  Topographical map of River Waal near Hulhuizen, original scale 1:50,000.
Possibilities For Increasing The Navigable Width

Rijkswaterstaat [1994] investigated several solutions to solve navigable width problems, particularly for river bends. The most promising options are:

1. Fixed layer in the deep outside bend.
2. Further constriction of the river channel with groins.
3. Underwater groins in the outer bend.
4. Submerged vanes in the outer bend.
5. Periodic dredging of the shallow inner bend.

The effects of the first three options are predominantly due to a reduction of the cross sectional area of the river. This increases the flow velocity in the inner bend, causing the bed level to erode there. However, it also leads to an increase of friction and as a consequence the upstream water level will rise. This influences the discharge distribution at an upstream bifurcation and is highly unfavorable in case of peak discharges. Therefore Dutch law requires compensatory measures, which can lead to a significant increase of the costs of river engineering projects. Furthermore the space available to realize these measures is often limited.

Submerged vanes and periodic dredging are considered the most promising options, since their influence on the upstream water level is expected to be small. However periodic dredging introduces a discontinuity in the sediment transport, with possible large scale and long term effects on the river bed level. The influence of submerged vanes on the streamwise sediment transport is minimal.

1.3 Submerged Vanes

Submerged vanes are vertical panels, placed in the river bed at an angle of $15^\circ - 25^\circ$ with the flow direction and a height of $0.2 - 0.4$ times the local water depth. The vanes exert normal and tangential forces (lift and drag) on the water and generate a horizontal vortex, with its axis in downstream direction. As a result a horizontal circulation is generated, influencing the near-bed velocity vector and thus the sediment transport vector.

If submerged vanes are placed deep enough in the outer bend they can be used to partly counteract the natural spiral flow, without hindering navigation.

Delft Hydraulics and Haskoning Consulting Engineers have conducted several studies, in order to assess the influence of submerged vanes on the flow pattern and their effectiveness in increasing the navigable width. Relevant results from this research are summarized in the introductions of Part 1 and Part 2 of this study.
1.4 Topics Of This Study

This study consists of two parts, in which two matters are addressed more or less separately.

**Part 1 Dynamic Bed Behaviour Under Influence Of Submerged Vanes**

The models used for calculating the influence of submerged vanes on the river bed are based on a constant morphologically representative discharge, assuming the influence of discharge variations on the bed level to be marginal. A 2DH model built by Delft Hydraulics for the bend in the River Waal at Hulhuizen was used as a tool to study the influence of varying discharges. Chapters 2 through 9 of this report concern Part 1 of this study. A problem definition and a statement of the aims for this part are presented in Chapter 2, Introduction Part 1.

**Part 2 Analysis Of Model Description Of Submerged Vanes**

Several models have been used to describe the influence of submerged vanes on the bed level in a river bend, but their reliability and the underlying assumptions are not always clear. A theoretical analysis could lead to a better understanding of these models. Furthermore, the results from a number of scale model tests can be used to verify these models. Chapters 10 through 14 concern Part 2. The problem definition and the statement of the aims are give in Chapter 10, Introduction Part 2.
Part 1

Dynamic Bed Behaviour Under Influence Of Submerged Vanes

An important question is whether or not River Waal is aware of the conclusions drawn in this thesis.

Based on prof. De Vries
2. **Introduction Part 1**
Dynamic Bed Behaviour Under The Influence Of Submerged Vanes

2.1 **Scope Of The Study Part 1**

Recently *Van Meerendonk and Struiksma* [1996] studied the effect of submerged vanes on the bed topography in the bend in River Waal at Hulhuizen, using the numerical 2DH model Rivcom. This was done based on the preliminary vane field design in the Hulhuizen II bend from *Haskoning* [1996a]. The results of these studies are summarized in Paragraph 2.3.

A constant Waal discharge of 1600 m$^3$/s was adopted, assuming the average bed topography in the bend to be determined by this discharge. Furthermore, variations in bed topography due to discharge variations were considered to be of minor importance only. These assumptions were studied in the first part of this study.

**Problem Definition**
The effect of discharge fluctuations on the bed topography under the influence of submerged vanes and the influence on the available navigable width have received little attention so far.

**Aims**
The aims identified for Part 1 of this study are:
1. To quantify the changes in bed level to be expected as a result of discharge fluctuations.
2. To estimate the time that the bed level needs to adapt to a change in river discharge.
3. To indicate the effect of changes of important river parameters on the preceding matters.
4. To indicate the influence on the available navigable width over the year.
5. To quantify the rise in water and bed level upstream, as a result of the vanes.

This study will primarily concern the bottleneck situation in the main bend Hulhuizen II. However it is intended to indicate general conclusions, whenever possible and useful.
2.2 Analytical Models For River Bends

Jansen [1979] summarizes basic theory describing the water motion in a river bend, which is included in Appendix 2A. Based on a scalar eddy viscosity model for the turbulence and a corresponding power law profile for the downstream velocity, an approximation of the vertical distribution of the transverse velocity is found, which is depicted in Figure 2.1.

Struikisma et al. [1985] studied the streamwise and lateral shape of steady state perturbations occurring in a straight river reach (summary in Appendix 2B). However, the results can also be used to understand the bed topography in river bends. A double harmonic perturbation was inserted in the linearized system of differential equations, describing the depth-averaged velocities, the spiral flow intensity, the sediment transport vector and the bed level. Essential are the longitudinal length scales for changes in the streamwise velocity \( \lambda_w \), spiral flow \( \lambda_m \) and the transverse bed slope \( \lambda_b \). The interaction parameter \( \lambda_b / \lambda_w \) strongly determines the wave length and the damping length of the bed perturbation and thus the bed topography in a river bend.

\[
\lambda_w = \frac{C^2 h}{2g} \tag{21}
\]

\[
\lambda_m = \beta \frac{Ch}{\sqrt{g}} \tag{22}
\]

\[
\lambda_b = \frac{1}{\pi^2} \left( \frac{B}{h} \right)^2 0.85 \sqrt{g}h \tag{23}
\]

Dynamic Bed Behaviour

Struikisma and Crouse [1989] continued in the same manner and included time dependent behaviour in the system of differential equations. This resulted in an expression for the propagation celerity and growth rate for river bed perturbations. Based on this growth rate a time scale \( T_b \) was derived for the adjustment of the river bed to disturbances.

\[
T_b = \frac{h \lambda_b}{s} \frac{1 + \left( \frac{\lambda_w}{\lambda_b} \right)^2}{1 - \left( \frac{U}{s \lambda_w} - \frac{2 \lambda_b}{\lambda_w} - 1 \right) \left( \frac{2 \pi}{\lambda_b} \right)^2} \tag{24}
\]

In which:
- \( L_b \) Wave length of bed perturbation [m]
- \( s \) Sediment transport [m^3/s/m]
2.3 Rivcom Model For The Bend In The River Waal Near Hulhuizen

*Van Meerendonk en Struiksma* [1996] modelled the stretch of River Waal near Hulhuizen using the 2DH morphological model Rivcom. In this model vanes were installed and their influence on the bed topography was studied. Rivcom solves the depth averaged momentum equations (rigid lid approximation) on a curvilinear grid. In addition, a spiral flow intensity is calculated, based on the streamline curvature. In this way the transverse near-bed velocities are taken into account in the sediment transport vector. The influence of submerged vanes is taken into account both in the momentum and in the spiral flow intensity equation. In the first place the lift and drag forces are calculated using the velocity vector averaged over the vane height. These are added to the bed friction. Secondly an extra term is added to the spiral flow intensity, corresponding to the vane-induced transverse near-bed velocities. A detailed mathematical model description, especially for the vanes, can be found in Part 2, Appendices 12A through 12C.

**Schematization Of Bend Near Hulhuizen**

A number of characteristics for the Rivcom model of the bend in River Waal near Hulhuizen are summarized in Table 2.1, further specifications of the vane field are included in Appendix 2A. Because of the fact that Rivcom is not able to handle varying boundary conditions, a dominant discharge was used, representing the river regime. For this purpose a Waal discharge of 1600 m³/s was selected, at which the sediment transport in the river equals the average yearly sediment transport. The model was run until an equilibrium bed topography was reached. The calibration of the model took place for the situation without vanes, based on the average of 4 similar bed level soundings between 1988 and 1991. A description of this calibration is included in Appendix 2D.
Description of River
1. Modelled stretch of River Waal near Hulhuizen bend km 867.550 - km 872.375.
2. Central part of river is described as a channel parallel to the river centerline, constant width between groin heads B = 240 m.
3. Curvilinear grid with intervals of 25 m streamwise by 5 m transverse on average, size primarily based on positioning of vanes in the model.
4. Grid dimensions 49 x 191 cells.
5. Upstream boundary fixed bed level and inflow velocities imposed parallel to river axis.
6. Side boundaries, parallel to the river centerline, closed.

Vane Field
1. Based on preliminary design by Harkasing [1996a].
2. Vanes located in outer bend between km 869.000 and km 871.525.
3. 296 Vanes in arrays of 7, 13, 15 and 10, with 10 m transverse spacing.
4. 25 Arrays, streamwise interval 100m.
5. Vane height to water depth ratio varies H/h = 0.0 - 0.4, level of top of vanes 3.50 m below OLR91 level.
6. Vane length L=10 m, angle with river centerline α=17.5°.

| Table 2.1 | Characteristics of Rivcom model for bend in River Waal at Hulhuizen. |

The following observations were made, comparing the resulting bed topography, in Appendix 2E, with the prototype bed level sounding from 1990 in Appendix 3B.
1. The bed level in middle 120 m of the river cross section predicted by the Rivcom model is close to the prototype situation. Near the river banks the predicted lateral slope is too steep.
2. In the bed level soundings significant disturbances are visible compared to the smooth equilibrium bed level in the model. These are a result of variations in the actual river width and the influence of groin heads.

Computational Results
Two computations were performed, the first resulting in the equilibrium bed level for the situation without vanes, the second for the situation with vanes. In Appendices 3D and 3E the equilibrium bed level for these situations are plotted. These clearly show the influence of the vanes. As a result of the fact that the spiral flow is counteracted by the vortices produced by the vanes, the average lateral bed slope in the river bend is reduced. A summary of quantified results is presented in Table 2.2.

![Equilibrium bed level](image)

**Figure 2.3** Equilibrium bed level 60 m from left and right bank and in river centerline for modelled stretch of River Waal, situations without and with submerged vanes.
Bed Level
1. Maximum bed level in the inner bend at 60 m from the left bank erodes from a level of approximately OLR -0.6 m to OLR -2.2 m.
2. Outer bend bed level at 60 m from the right bank rises from approximately OLR -4.2 m to OLR -5.1 m.
3. The bed level in the centre-line in the main bend erodes with a maximum of 1.0 m.

Navigable Width
The navigable width was calculated by looking at the width for which the river bed level was below OLR -3.08 m. The extra 0.28 m depth to the OLR -2.80 m norm were taken as a margin to account for bed forms in the prototype, which are not included in the Rivcom model concept. Furthermore a safety distance of 25 m was kept between the navigable area and the top of the groin heads, 10 m outside the modelled width at both sides.
1. The minimum navigable width in the main bend increases from 104 m to 141 m for the situation with vanes.
2. These changes take approximately 2 years to develop.

Table 2.2 Results from Rivcom computations with the model for the bend at Hulhuizen for the situation with and without vanes.

2.4 Outline Of Report Part 1

The first part of this study has the following outline:

Chapter 3 summarizes data for River Waal near Hulhuizen used in this study.
Chapter 4 describes the method that was developed for computing the dynamic bed behaviour of this stretch of River Waal, using the existing Rivcom model. Secondly the results of some tests with the developed procedure are presented.
Chapter 5 presents the results of a number of model runs with different constant discharges. This was done in order to have a first impression of the bed topography differences occurring as a result of discharge variations, both for the situation with and without vanes. Furthermore an estimate was made of the bed level changes over a long term wet or dry period.
Chapter 6 concerns the bed topography transition after the installation of the vane field. A comparison was made between the time needed for the bed adjustment with a constant and a varying discharge.
Chapter 7 quantifies the bed level differences that occur during discharge fluctuations in River Waal, both for the situation with and without vanes.
Chapter 8 addresses the problem of assessing the rise in water level in the stretch of river upstream of the vane field. An estimate was made based on Rivcom model output.
3. River Waal Bend Near Hulhuizen

3.1 Introduction

The data used for the stretch of the River Waal near the bend at Hulhuizen is described and analyzed briefly in this chapter. The same river data was used as much as possible in order to be able to compare the results with those from the previous model research by Van Meerdonk and Struijsma [1996].

Paragraph 3.2 summarizes basic river data.
Paragraph 3.3 presents results from a brief analysis of historical discharge records.
Paragraph 3.4 addresses the issue of the distribution of the River Waal discharge over its entire cross section during flood periods.
Paragraph 3.5 describes the sediment transport formulation used in this study.
Paragraph 3.6 gives an indication of the bed forms occurring on the river bed.

3.2 River Data

The following measured data is used:

2. Discharges (hydrological years October 1st-September 30th) from the 1990 stage-discharge relation.
   Note that all discharges used are Waal discharges, downstream of the bifurcation at Pannerdensche Kop (Appendix 3A).
3. Bed level soundings 1988 - 1995. 1990 sounding is plotted as an example in Appendix 3B.
Other parameters, in accordance with previous work by *Van Meerendonk and Straatsma* [1996]:

1. Discharge independent water surface slope along the river. Stage-discharge relation for the upstream model boundary (Appendix 3C) was derived from gauge at Pannerdensche Kop (330 m upstream).

   $i = 1.05 \times 10^{-4}$

2. Constant autonomous bed level subsidence by large scale processes.

   $20 \text{ mm/year}$

3. Chézy friction formula with a constant factor. *Brilhuis* [1988] found Chézy coefficients varying over 48 m$^{1/2}$/s - 52 m$^{1/2}$/s for the range of discharges from 1000 m$^3$/s to 8000 m$^3$/s.

   $C = 47 \text{ m}^{1/2}/\text{s}$

4. Characteristic sediment grain size, based on Rijkswaterstaat data. Relatively coarse grains, in order to account for the combination of sand and gravel in the river bed.

   $D = 2.0 \times 10^{-3} \text{ m}$

With this data a number of characteristic values for this stretch of the river were calculated, summarized in Table 3.1.

### 3.3 Analysis Of Discharge Records

The discharge measurements were analyzed. Figure 3.1 depicts the duration curve for the mean daily measurements, which can be used as a reference for the flood discharges used in this thesis. A statistical analysis indicated that in a hydrological year peak discharges between 4000 m$^3$/s and 5000 m$^3$/s are most likely to occur.

Furthermore from the discharge records in Appendix 3A it was concluded that significant trends in the discharges occur over more than one year. For instance the period from 1970 to 1977 was extremely dry compared to the other years, while in the years 1964 to 1967 relatively high discharges occurred.

<table>
<thead>
<tr>
<th></th>
<th>$900 \text{ m}^3/\text{s}$</th>
<th>$1600 \text{ m}^3/\text{s}$</th>
<th>$6500 \text{ m}^3/\text{s}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average velocity</td>
<td>$U$</td>
<td>0.9 m/s</td>
<td>1.1 m/s</td>
</tr>
<tr>
<td>Average depth</td>
<td>$h$</td>
<td>4.0 m</td>
<td>5.9 m</td>
</tr>
<tr>
<td>Froude number</td>
<td>$Fr = \frac{U}{\sqrt{gh}}$</td>
<td>0.14</td>
<td>0.14</td>
</tr>
<tr>
<td>Shields parameter</td>
<td>$\theta = \frac{u^2}{\Delta gD}$</td>
<td>0.11</td>
<td>0.16</td>
</tr>
</tbody>
</table>

#### Adaptation lengths

- **Main flow**
  
  $\lambda_w = \frac{C^2h}{2g}$
  
  460 m 660 m 1280 m

- **Spiral Flow**
  
  $\lambda_s = \frac{C^2h}{\sqrt{g}}$ (with $\beta=0.6$)
  
  40 m 60 m 100 m

- **Lateral bed slope**
  
  $\lambda_b = \frac{1}{\pi^2} \left( \frac{B}{h} \right)^2 \frac{0.85\sqrt{gh}}{h}$
  
  410 m 350 m 290 m

#### Adaptation time

$T_a = \frac{\lambda_b h}{U \Delta \theta}$

1950 days 990 days 120 days

(with $L = 4 \text{ km}$)

**Table 3.1** Characteristic values for River Waal near bend at Huiluizen for "minimum" 900 m$^3$/s, dominant 1600 m$^3$/s and "peak" discharge 6500 m$^3$/s.
Discharge Fluctuations
The type of discharge fluctuations that occur is important for the variations in bed level to be expected over a year, since a continuous flood period will cause greater bed topography changes compared to alternating periods with high and low discharge. This input chronology is difficult to characterize. Therefore another approach was followed, using particular periods from historical discharge records that were considered representative for a certain situation.

3.4 Discharge Distribution During High Discharge
During flood periods the high water level results in water flowing not only between the groins, but also over the groins and even over the foreland. The Rivcom model for the bend in River Waal at Hulhuizen covers only the central part of the river with a width of 240 m. In order to have a valid model during high discharge periods, it is important to determine which part of the total discharge flows through this modelled channel, indicated in Figure 3.2.

Figure 3.1 Cumulative probability function of mean daily discharges for River Waal, based on 1965 - 1991 records.

Figure 3.2 Modelled channel as part of the entire river cross section in a flood situation.
For this purpose model output was used from high discharge runs with the 2DH model Waqua, describing the entire area between the winter dykes for this stretch of the Waal. From this output the part of the discharge flowing through the modelled area was estimated. Moreover it was observed that this discharge is more or less constant along the bend at Hulhuizen, although there is some inflow and outflow from the main river channel into the forelands. This is visible in Figures 3.5 and 3.8.

Secondly from elevation data the water levels were estimated, at which water starts flowing over the groins and over the summer dykes. At these levels the cross sectional width of the water surface suddenly increases, changing the distribution of the marginal discharge.

With these results a relation was established between the total Waal discharge and the discharge flowing through the modelled channel, using linear interpolation. The resulting graph is presented in Figure 3.3.

### 3.5 Sediment Transport

In accordance with *Van Meerveld and Struijsma [1996]* the Engelund-Hansen formula was used to describe the sediment transport in this part of River Waal, multiplied by a correction factor. The Engelund-Hansen formula was selected arbitrarily, based on the expectation that the corresponding power 5 for the velocity would be a fair approximation of reality. This factor directly determines the magnitude of differences in sediment transport, at different flow velocities. Based on the sediment grain size, it is clear that bed load transport dominates in River Waal, suggesting the use of more appropriate formulae, such as Meyer-Peter-Müller.

Equating the calculated average annual sediment transport through the modelled channel with the 300,000 m³ annual transport in the upper part of River Waal estimated by *Barnes et al. [1994]*, a correction factor $\eta = 0.5$ was found. Note that a significant uncertainty exists in this total sediment transport and in the sediment transport formula.

$$ S_b = \eta \frac{0.084}{\sqrt{gC^2A^2D_{50}}} U^5 $$

(3.1)
A relation was established between the Waal discharge and the average sediment transport $s$ through the centerline of the modeled channel. For this purpose the average velocity in the river centerline was calculated from the discharge, using the stage-discharge relation and the distribution of discharge over the river cross section during floods. The result was plotted in Figure 3.4, a parabolic function that approximates this relation very roughly was added. The kinks in this graph are a result of the sudden change in the distribution of the marginal discharge flowing through the modeled channel, at times when the water starts overflowing the groins or the summer dykes.

Based on the Chézy equation and the increase of the water surface width during floods one would expect $s \sim Q^{2/3}$. The fact that $s \sim Q^2$ was found is a result of a decreasing Chézy coefficient with an increasing discharge, which influences the measured stage-discharge curve used here.

The resulting sediment transport over the years 1961 - 1995 was analyzed statistically, the average sediment transport in the river centerline is $4.1e-5 \text{ m}^3/\text{s/m}$, which corresponds to a constant discharge of $1585 \text{ m}^3/\text{s}$.

### 3.6 Bed Forms

Briënhuis [1988] showed that a number of prediction methods indicate that dunes occur in the bed of this part of River Waal and that these have been observed. Dunes have lengths of 4 - 8 water depths and their occurrence depends on the local water depth. Careful study of the graph of the bed level sounding in Appendix 3B shows that some dunes are visible indeed, these have heights of the order of 0.4 m. However, it should be kept in mind that this bed level was sounded from a boat constantly crossing the river. In order to measure the height of dunes a sounding in streamwise direction would be appropriate.

As mentioned in Paragraph 3.2 the influence of varying bed forms on the river bed roughness was not taken into account.
Figure 3.6 Map overlay indicating elevated areas, upstream part.
Figure 3.9 Map overlay indicating elevated areas, downstream part.
Figure 3.10 River map Rijkswaterstaat of River Waal downstream part of bend at Hulhuizen, original scale 1:5,000.
4. Dynamic Boundary Conditions

4.1 Introduction

The central question of the first part of this study is how the bed topography changes under the influence of a varying discharge. Therefore one would like to impose gradually changing inflow velocities at the upstream model boundary while changing the level of the rigid lid in order to let the water depth change, as the water level is rising and falling in reality. This concept was implemented in a computation procedure for the Rivcom model, which is described in this Chapter.

Paragraph 4.2 briefly describes the functioning of the procedure for dynamic computations.
Paragraph 4.3 indicates what morphological time steps are to be used in this procedure.
Paragraph 4.4 addresses the problem of establishing the upstream boundary conditions.
Paragraph 4.5 mentions the results of test runs with different discharge schematizations.
Paragraph 4.6 describes the way in which the output from the dynamic Rivcom computations was studied.
Paragraph 4.7 mentions the results of an attempt to validate the dynamic computations.

4.2 Procedure Outline

Rivcom can only handle fixed boundary conditions during one model run, but the model can be used in a series of subsequent runs with slightly different boundary conditions, thus approximating the real discharge variation in a stepwise manner. The final bed level from the previous run in the series should be used as the initial condition for the next run.

In this report one of the single Rivcom runs is called a run step, while the total series of single Rivcom runs is called one run.

During one run a central script file starts a secondary script file a number of times, for every run step. This secondary script file manages the adaptation of Rivcom input files and the start of the calculation, as is explained in Figure 4.1. Since every run step comprises in fact one Rivcom model run, each run step produces new output files, which are saved separately. The result is a number of labeled files that have to be read individually to retrieve the output from an entire run.
Handling Of Input Data
The Rivcom input files need to be adapted for each run step in order to fit the appropriate boundary conditions. Most of the data to be used in these input files is prepared before the entire run is started. This data is stored in a data file from which it is read and inserted in the Rivcom input files for every run step. In these files the following parameters are changed:
1. Morphological time step.
2. Number of time steps.
3. Inflow velocities on the upstream model boundary.
4. Distance of top of the vanes below water surface.
5. Initial water depth.
6. Times from which model output is desired.

A list of the relevant Rivcom files is included in Appendix 4A. In Appendix 4B an elaborate flow chart is included for the secondary script file runstep.scr, together with an explanation. Furthermore 4C through 4P contain checklists for the start and the end of a dynamic Rivcom run and listings of all non-Rivcom files used in this computation procedure.

Assumptions
The following assumptions underly the calculation procedure described in this paragraph:
1. The influence of discharge variations on the bed topography is represented sufficiently accurately by an acceptable number of stepwise changes in the boundary conditions. Within every step the discharges are averaged.
2. The discharge variations are gentle, thus the velocity derivative du/dt remains negligible compared to the other terms in the governing momentum equations.
3. Large-scale bed topography changes in River Waal are negligible compared to the bed topography variations that result from discharge variations over a few years.

Figure 4.1 Basic flow chart for a run, with functions of secondary script file Runstep.scr.
4.3 Morphological Time Step

It is important to ensure that every run step produces a stable Rivcom calculation. Because of the fact that the rate of the morphological changes is determined mainly by the magnitude of the sediment transport, an estimate was made for the required morphological time step at a certain discharge.

If the transport is expressed in volume of deposited bed material per unit time, the rate of bed level change is described by the continuity equation for sediment:

\[ \frac{\partial h}{\partial t} + \frac{\partial s}{\partial x} + \frac{\partial z}{\partial y} = 0 \]  \hspace{1cm} (4.1)

The sediment transport derivatives are proportional to the magnitude of the sediment transport \(|s|\). Stability of the time dependent morphological calculation requires that the magnitude of \(\Delta z\) over a time interval \(\Delta t\) is limited. If a calculation with a sediment transport \(|s_1|\) proves to be stable at a certain \(\Delta z_{01}\) over an interval \(\Delta t_{01}\), stability is assured if \(\Delta t_2 \leq \Delta t_{01}\). An equivalent relation is used for another calculation with a sediment transport \(|s_2|\) and a time step \(\Delta t_2\), with the same stability requirement for \(\Delta z_{01}\). Combining these equations results in:

\[ \Delta t_2 \leq \Delta t_{02} = \frac{|s_1|}{|s_2|} \Delta t_{01} \]  \hspace{1cm} (4.2)

This equation was used to estimate the allowable time step, based on a measure for the occurring sediment transport. In order to keep the results from calculations with different time steps comparable, it is chosen to use time steps multiplied by a factor \(2^n\) (\(n=-5,-4,...,5\)), taking as a basis a time step of \(\Delta t = 43200\) s at a discharge of \(Q_{\text{mod}} = 1600\) m³/s, used by Van Meern and Struiksma [1996]. The resulting time step \(\Delta t\), as a function of the Waal discharge, is plotted in Figure 4.2.

Test calculations confirmed this relation, as slightly greater time steps generally led to unstable computations.

![Figure 4.2](image)

**Figure 4.2** Relation between River Waal discharge and required morphological time step in Rivcom.
4.4 Upstream Boundary Conditions

Calibrating the constant discharge Rivcom model, Van Meerveld and Struik [1996] selected the inflow velocities at the upstream boundary to vary linearly from $U_{\text{max}} = 1.1 \ U_{\text{average}}$ on the left bank side to $U_{\text{min}} = 0.9 \ U_{\text{average}}$ on the right bank side. The same velocity distribution was used in this study, although Waqua velocity vector output indicates an opposite trend in the velocity distribution over the river cross section in this stretch. Considering the main flow adaptation length $\lambda_w = 0.5 \text{ km to 1.3 km}$, the influence of the inflow velocity distribution on the stretch of river in which the vanes are located will be minor.

Furthermore, the fixed bed level at this boundary was assumed to be a straight line from $z_b_{\text{minimum}} = 3.30 \text{ m+NAP}$ on the left bank side to $z_b_{\text{maximum}} = 4.34 \text{ m+NAP}$ on the right bank side, again in accordance with previous Rivcom research.

Integrating the assumed linear bed level across the river and the linearly distributed velocities over the upstream boundary a simple formula can be derived for the average velocity in this cross section as a function of the total discharge and the water level.

$$Q_{\text{mod el}} = \int_{-\frac{1}{2}B}^{\frac{1}{2}B} h(n)U_{\text{in}}(n)dn = \frac{1}{2}B \int (h_c + h'n)(U_{\text{in c}} + U_{\text{in c}}'n)dn$$

Thus the velocity distribution over the upstream boundary follows from:

$$U_{\text{in}}(n) = U_{\text{in c}} + U_{\text{in c}}'n$$

$$U_{\text{in c}} = \frac{Q_{\text{mod el}}}{0.2 \frac{h'B^2}{12} + h'B}$$

In which

- $h_c$: Water depth in river centerline, $\text{m}$
- $h'$: Lateral gradient in water depth, $\text{m}$
- $U_{\text{in c}}$: Inflow velocity in river centerline, $\text{m/s}$
- $U_{\text{in c}}'$: Lateral gradient in inflow velocity, $\text{1/s}$
4.5 Discharge Schematization

If the discharge variations are schematized with an increasing accuracy, the number of run steps increase, while each run step consists of a smaller number of time steps.

In two different runs, labeled B and C, the discharge variation in the year 1983-1984 were schematized in two different ways. Both schematizations are plotted in Figure 4.4, together with the actually measured discharge. In run B run steps occurred with a number of time steps between 16 and 112, while in run C the number of time steps varied between 48 and 218.

Although there are differences in the bed level variations, the magnitude of the changes, as well as the general trend are comparable for both runs. The bed level at a point in the inner bend is compared for both runs as an example in Figure 4.4.

Furthermore, in run steps with a very small number of time steps the results could be influenced by the somewhat inaccurate iteration of the velocity field by Rivcom during the first two iterations, in the first 8 time steps.

Finally the number of run steps determine the total calculation time of a run and thus the costs. This led to the conclusion that a very accurate schematization of the discharge variation is not very useful, whereas it does strongly increase the calculation costs. In the following a similarly coarse schematization is used, avoiding run steps with less than 40 time steps.
4.6 Study Of Results

Besides a qualitative description of the changes in bed topography it is useful to be able to describe the changes quantitatively. For this purpose the bed levels in 9 points in the river bend were used as indicators. These are located in cross sections at km 869.500, km 870.000 and 870.500, which are close to the cross section with the maximum lateral bed slope. In these cross sections the bed level in the river centerline and in points 60 m from the left and right bank are monitored. The bed level was averaged over an area of approximately 15 m width and 225 m length in order to reduce the effect of local numerical disturbances in Rivcom or bed forms in the soundings. Finally the minimum navigable width (VBB, Vaarbaanbreeuwe) at a level of OLR -3.08 m was monitored. The same margin to the OLR -2.80 m norm was used as applied Van Meervanzon and Straatsma [1996] to account for bed forms. Although bed forms with a greater height will develop during higher flow velocities it was assumed that these adjust rapidly to discharge variations. The same normative level was used during high discharge periods, because the discharge records indicate that a discharge reduction from a flood to less than 1000 m³/s can occur within one month. Over this period the bed level changes are small, thus navigation is confronted with a bed topography almost equal to the one that has developed during the previous flood period. In contrast to Van Meervanzon and Straatsma [1996] a navigation safety margin (schrikafstand) to the groin heads was not taken into account. This margin can easily be subtracted afterwards.

Relaxation Time

In order to measure the time the bed level needs to adjust to a new equilibrium situation a relaxation time was defined, which is clarified in Figure 4.6.

\[ \text{T}_{\text{relaxation}} \text{ is the time after which the indicator finally enters a range of } + \text{ or } -10\% \text{ of the total change surrounding the end value or a final average value.} \]
4.7 Procedure Validation

Since the Rivcom model and the stepwise procedure are approximations of the prototype situation with gradual discharge variations, the question arises how well the bed level variations calculated in a run approximate the bed level changes in the prototype. Moreover the Rivcom model was not calibrated for the bed level changes.

By studying the bed topography variations in the period between two bed level soundings and the equivalent model output, an idea of the validity of the model was obtained. The best opportunity for validation was offered by the 1994 and 1995 bed level soundings, in between which a significant flood period occurred (7700 m³/s peak discharge).

This flood is plotted in Figure 4.7, together with the times at which the bed level soundings were conducted. The bed levels were considered constant over the measurement periods.

The resulting run K is specified in Appendix 4R. In Appendices 4S through 4U graphs are included in which a comparison was made between the changes in bed level between the model output and the Rivcom results in three longitudinal sections and a cross section. Furthermore in Appendix 4V a quantitative comparison is made between the differences in the bed level indicators in the two soundings and in the corresponding model output.

Figures 4.8 through 4.10 depict the bed level changes for the inner bend indicators as an example.
Figure 4.7 Discharge during 1995 flood and times of 1994 and 1995 soundings.

The following observations were made:

**Comparison Of Longitudinal Sections**
1. There is reasonable agreement between the changes in bed level for the points in the centerline and the inner bend. The model tends to underestimate the inner bend bed level variations (factor 0.6 on average).
2. In the outer bend the model appears to overestimate the changes by a factor 1.4, except for the first point at km 869.500 where the model predicts a change in the wrong direction. This discrepancy could be a result of the fact that the river bed transition between the bends Hulhuizen I and Hulhuizen II is predicted by the model at a slightly differing chainage.

**Comparison Of Cross Sections**
1. The position of the center of rotation of the lateral bed slope according to the model coincides with the measured point, namely at approximately 20 m right of the river axis.
2. Since the navigable width is determined by the bed level in the inner bend, the changes in this navigable width are underestimated by a factor comparable to the underestimation of the bed level variations in the inner bend.

The differences in bed topography changes can be caused by the over- and underestimation of the magnitude of the bed level variations by the model or by an error made in the time needed for this adjustment. This latter factor is directly related to the sediment transport formulation in the model, which is an uncertain factor, as indicated earlier. Therefore no conclusions can be drawn about the correctness of the model representation of the time the bed level takes to adjust to new conditions. However the results do indicate that the procedure for modelling discharge variations can be used to estimate the occurring bed level variations along the bend.

1. This thesis is based on the assumption that the rate of bed change is adequately represented by the model, although it was not calibrated in this respect.
2. In general in the inner half of the bend these variations are underestimated with a factor 0.6, while in the outer bend the bed level changes are overestimated by a factor 1.4.
Figure 4.8  Bed level variation 60 m from left bank km 869.5, output run K compared to soundings before and after 1995 flood.

Figure 4.9  Bed level variation 60 m from left bank km 870.0, output run K compared to soundings before and after 1995 flood.

Figure 4.10  Bed level variation 60 m from left bank km 870.5, output run K compared to soundings before and after 1995 flood.
5. Constant Discharge Runs

5.1 Introduction

In order to have an impression of the changes in bed level that occur during periods with low or high discharge, the equilibrium bed levels at these discharges were studied. Furthermore, knowledge of the bed level changes can provide for an estimate of the time required for the bed topography to adjust to a change in the discharge. Runs were made with discharges of 1000 m$^3$/s, 1600 m$^3$/s, 2000 m$^3$/s, 3000 m$^3$/s and 4000 m$^3$/s and were continued until an equilibrium state for the bed topography was reached. Both the situation with and without vanes were investigated in order to find out the differences in behaviour.

This chapter has the following outline:

- Paragraph 5.3 describes the equilibrium bed topography found for the situation without vanes.
- Paragraph 5.4 summarizes the results for the situation with vanes.
- Paragraph 5.5 presents an analytical approximation method to predict bed level changes for a given series of discharges.
- Paragraph 5.6 describes the bed level changes expected over a long period with low discharges, derived from the constant discharge results.
- Paragraph 5.7 studies the bed level changes over some years with high discharge.
- Paragraph 5.8 summarizes the main conclusions of this chapter.

Specifications for the resulting runs E and G were included in Appendix 5A.
5.2 Equilibrium Bed Level For Situation Without Vanes

The equilibrium values for the bed level indicators were estimated for the situation without vanes, based on the bed level graphs in Appendices 5A to 5E. These were plotted in graphs in Appendices 5F. An impression of the bed topography differences is presented in Figures 5.1 through 5.4. In some cases an equilibrium situation was not reached completely, here the equilibrium value for the indicator was estimated, as a continued computation would have led to high calculation costs.

From these graphs the following conclusions were drawn:

Longitudinal Sections
1. Most important bed level changes occur as a result of the fact that at a higher discharge the stretch of river with a lateral bed slope extends further downstream. This can be explained by the significant increase of adaptation lengths of the main flow and the spiral flow as the water level rises.
2. The changes are caused by a combination of a gradual change in the river bed and sediment waves moving over the bed. This causes the irregular patterns visible in the bed adjustment, for example in Figure 5.1.

Cross Sections
3. The changing cross sections have one point in common. This center of rotation is always located at approximately 20 m to the right of the river axis.
4. The equilibrium lateral bed slope in the river centerline from the model and prototype were compared to a calculation with Rivcom theory applied for an infinitely long bend. The relevant equation was derived in Appendix 12B, average lateral bed slopes were calculated in Appendix 5H. A comparison with the maximum lateral bed slope in the River Waal model shows an overshoot by a factor 2.4-1.2, depending on the discharge. The decreasing overshoot with an increasing discharge is explained by the fact that the interaction parameter $\lambda_{bl}/\lambda_{wp}$ strongly reduces as a result of the higher water level. According to Struijsma et al. [1985] this comes with a significant decrease of the damping length for river bed perturbations. Thus at a higher discharge the lateral bed slope along the river bend is related more directly to the curvature of the river centerline.

Navigable Width
5. At a high discharge the navigable width decreases because of the increasing lateral bed slope, while the normative cross section shifts slightly in downstream direction. Differences are limited to 15 m over this range of discharges from 1000 m$^3$/s to 4000 m$^3$/s.

![Image of Figure 5.1: Minimum navigable width at OLR -2.80 m, equilibrium situation without vanes.](image-url)
**Figure 5.2** Equilibrium longitudinal sections at 60m from left and right bank and in centerline for situation without vanes.

**Figure 5.3** Equilibrium cross sections at km 870.000 for situation without vanes, bend Hulhuizen II.

**Figure 5.4** Equilibrium navigable width at OLR -2.80 m level, for situation without vanes, bends Hulhuizen I and II.
5.3 Equilibrium Bed Level For Situation With Vanes

The equilibrium bed level results from runs G for the situation with vanes were analyzed in the same way as in the previous paragraph. Appendices 5I to 5M contain graphs indicating the river bed topography, which are summarized by Figure 5.5 through 5.8.

Longitudinal Sections
1. The greatest bed level changes occur in the downstream part of the bend. Like in the situation without vanes the bend profile extends in downstream direction at a higher discharge, caused by the greater adaptation lengths for the flow.
2. The bed level in the outer bend where the vanes are located changes little. This “fixed” bed level has a significant influence on the bed topography changes in the inner bend. The type of bed behaviour in the inner bend is comparable to the behaviour found over the whole undisturbed cross section. This suggests that the longitudinal bed level distribution in the inner bend might comply with an adaptation length $\lambda_b$ for half the morphological width. The adaptation lengths for main flow and spiral flow are not affected.
   The resulting reduction of the wave and damping length of bed perturbations in the linear theory (Paragraph 2.2) appears to be confirmed by the fact that an equilibrium cross section is reached in the downstream part of the bend Hulhuizen II.
3. Downstream of km 871.000 the number of vanes in a cross section decreases. At a high discharge this results in a significant increase of the lateral bed slope in the downstream reach of the Hulhuizen II bend.

Cross Sections
4. In contrast to the situation without vanes a center of rotation is located 50 m to the left of the river axis. At this point the bed level is OLR -2.75 m. Only small differences in navigable width are therefore expected with varying discharges.
5. The bed level in the outer bend slightly decreases if the discharge increases, therefore the effective vane height increases.

Navigable Width
6. In contrast to the situation without vanes, the navigable width increases during a flood period. However, this is very sensitive to the model representation of the vertical position of the center of rotation of the bed level in a cross section in the bend.
7. Differences in equilibrium navigable width are limited to 15 m over the range of constant discharges of 1000 m$^3$/s to 4000 m$^3$/s.
8. Compared to the situation without vanes the occurring bed level differences are small, however this is not the case for the navigable width, because of the smaller lateral bed slope.

![Figure 5.5 Minimum navigable width at OLR -2.80 m, equilibrium situation with vanes.](image-url)
Figure 5.6  Equilibrium longitudinal sections at 60m from left and right bank and in centerline for situation with vanes.

Figure 5.7  Equilibrium cross section at km 870.000 for situation with vanes in bend Hulhuizen II.

Figure 5.8  Equilibrium navigable width at OLR -2.80 m level, for situation with vanes, bends Hulhuizen I and II.
5.4 Prediction Of Relaxation Time

Whether or not an equilibrium bed topography is reached depends on the time available for this change. It is expected that the time needed is proportional to the river bed adaptation time $T_b$ as indicated by basic theory in Paragraph 2.2.

The relaxation time for each indicator was measured and included in the Appendices 5G and 5N for the situation without and with vanes. These relaxation times were assumed to be a function of two independent factors that depend on the discharge and the position in the river bed, and a relaxation time standard.

$$T_{	ext{relaxation for a certain indicator at a certain discharge}} = \zeta \xi T_{\text{relaxation std inner bend km 870.000 at Q=1000 m}^3/\text{s}} \quad (5.1)$$

In which 

$$\zeta = f(\text{position}) = \frac{T_{\text{relaxation indicator}}}{T_{\text{relaxation inner bend km 870.000}}}$$

$$\xi = f(Q) = \frac{T_{\text{relaxation indicator}}}{T_{\text{relaxation indicator at Q=1000 m}^3/\text{s}}}$$

**Estimate of Position Factor**

Based on the relative relaxation times in the second column of the Appendices 5G and 5M estimates were made for the position factors. Following the hypothesis above, two different values were calculated for the situation with and without vanes, because of the different shape of the cross sections. There is a significant scatter with the discharge, which is caused by the definition of the relaxation time, combined with the fact that the indicators change along irregular curves of different shapes. Corrected averages were calculated for every indicator, eliminating the strongly deviating values. These are presented in Table 5.1.

The following conclusions can be drawn:

1. Ratios $\zeta$ are comparable for either situation, which indicates that the relative rate of bed level change is constant throughout the area.
2. This is not the case for the navigable width, as a result of the effect of the vanes on the center of rotation of the river bed cross section.
3. The relaxation time for the navigable width is smaller than the relaxation time for the inner bend points. This is explained by the fact that the changes of this minimum width have two causes, namely the changes of the cross-section and the fact that the normative cross section moves along the bend.

<table>
<thead>
<tr>
<th>Indicator</th>
<th>Situation without vanes</th>
<th>Situation with vanes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\zeta = T_{\text{rel}} / T_{\text{rel centerline km 870.000}}$</td>
<td>$\zeta = T_{\text{rel}} / T_{\text{rel centerline km 870.000}}$</td>
</tr>
<tr>
<td>Bed level 60 m from left bank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>0.70</td>
<td>0.84</td>
</tr>
<tr>
<td>km 870.000</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>km 870.500</td>
<td>0.79</td>
<td>1.14</td>
</tr>
<tr>
<td>Bed level in centerline</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>0.74</td>
<td>0.67</td>
</tr>
<tr>
<td>km 870.000</td>
<td>0.68</td>
<td>0.66</td>
</tr>
<tr>
<td>km 870.500</td>
<td>0.66</td>
<td>0.69</td>
</tr>
<tr>
<td>Bed level 60 m from right bank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>0.94</td>
<td>0.84</td>
</tr>
<tr>
<td>km 870.000</td>
<td>1.07</td>
<td>1.07</td>
</tr>
<tr>
<td>km 870.500</td>
<td>0.81</td>
<td>0.84</td>
</tr>
<tr>
<td>Minimum navigable width at OLR -2.80 m</td>
<td>0.62</td>
<td>0.89</td>
</tr>
</tbody>
</table>

**Table 5.1** Position factor $\zeta$, estimated from measured relative relaxation times $T_{\text{relaxation indicator}} / T_{\text{relaxation inner bend km 870.000}}$, for situations without and with vanes.
Estimate Of Discharge Factor

In the same manner a relation between the relaxation time and the discharge was established, relating these to the equivalent ones at a discharge of 1000 m³/s. The estimates for the discharge factor $\xi$ are plotted in Figure 5.9 for run E and run G. Furthermore, the similar results of the constant discharge transition runs M were included, which are described in Chapter 6. In the scatter in this graph no pattern for the position in the river bed was recognized. A corrected average curve was drawn through the points.

Again it is clear that this relation is highly approximative, therefore the margins of error are assessed in the following.

Estimate Of Base Relaxation Time

The base relaxation time at a discharge of 1000 m³/s for the bed level in the inner bend at km 870.000 can be estimated from the measured relaxation times by reversing equation (5.1). By doing so for all measured relaxation times, an average value results that is more reliable than a single measured base relaxation time. Furthermore an estimate was made for the inaccuracies in the predictions that can now be made for the relaxation time.

Situation without vanes: $T_{relaxation std \, \text{inner bend km 870.000 at } Q = 1000 \text{ m}^3/\text{s without vanes}} = 2050 \text{ days}$,

leading to predictions of the relaxation time with a standard deviation of approximately 25%.

Situation with vanes: $T_{relaxation std \, \text{inner bend km 870.000 at } Q = 1000 \text{ m}^3/\text{s with vanes}} = 1460 \text{ days}$,

leading to predictions of the relaxation time with a standard deviation of approximately 30%.

Note that for the situation without vanes the relaxation time is approximately twice as long as the theoretical bed level adaptation time calculated in Paragraph 3.3.

Based on the hypothesis for the vane-induced reduction of the effective morphological width of the river, a reduction of the relaxation time for the situation with vanes is expected indeed. Using the theoretical equation (2.4) for $T_0$ and a morphological width of $B_{morph} = 120 \text{ m}$, a reduction of the bed level adaptation time was found with a factor 0.2, compared to the original situation with $B = 240 \text{ m}$.

The hypothesis concerning the morphological width gives a plausible explanation for the reduction of the relaxation time for the situation with vanes. Considering the measured relaxation times the effective morphological width appears to reduce less than the presumed half.
Comparison To Theoretical Bed Topography Adaptation Time

In Figure 5.10 the relaxation time deduced from model results was plotted versus the theoretical adaptation time scale $T_b$, described by equation (2.4). Both parameters were normalized with the value at a discharge of 1000 m$^3$/s.

Furthermore a similar comparison was made between the measured relaxation time and the inverse of the sediment transport. This was done because of the morphological time step theory in Paragraph 4.3.

The following conclusions were drawn:

1. At a high discharge the adjustment of the bed topography takes significantly longer than expected based on the linear theory.
2. The relaxation time can be considered proportional to the inverse of the sediment transport at discharges less than 2000 m$^3$/s. At higher discharges the bed level adaptation takes place slower than expected on the basis of the occurring sediment transport.

5.5 Bed Level Changes Over Long Term Dry Period

The discharge records include periods of 5 - 6 years that are significantly drier than average. It is expected that during such a period the bed level evolves more or less like in the constant discharge runs, since few discharge peaks occur. Using the results of run G an estimate was made for the changes in the bed topography that develop over a persisting dry period for the situation with vanes. The sediment transport was averaged over the driest 4 year period of 70-71 to 73-74, resulting in a corresponding discharge of approximately 1200 m$^3$/s.

Relaxation Time

Firstly the relaxation time was estimated at the 1200 m$^3$/s discharge:

$$T_{rel\ inner\ bend\ km\ 870.000} = \xi T_{rel\ standard} = 0.7 \times 1460 \text{ days} = 1020 \text{ days} = 2.8 \text{ years}.$$
Bed Level
An estimate was made for the corresponding differences in bed level compared to the 1600 m$^3$/s equilibrium bed topography, using the graphs for the equilibrium situation in Appendix 5M. These differences can be used to estimate the bed level changes from an initial situation.
1. The average lateral bed slope reduces mostly in the downstream part of the bend, bed level differences up to 0.3 m occur.
2. The minimum navigable width at OLR -2.80 m will reduce by 7 m.

5.6 Bed Level Changes Over Long Term Wet Period.

Like long term dry periods also periods of 3 to 4 consecutive years occur with relatively high discharge peaks, and therefore relatively high average discharges. For the three maximum seasons, 64-65 to 67-68, the sediment transport was averaged, resulting in an average constant discharge of 2100 m$^3$/s. It is expected that over these years the bed topography will shift towards an equilibrium state corresponding to this discharge.

Relaxation Time
Again the relaxation time was estimated for the average discharge of 2100 m$^3$/s.

$$T_{rel \text{ inner bend km 870.000}} = \xi T_{rel \text{ standard}} = 0.32 \times 1460 \text{ days} = 467 \text{ days} = 1.3 \text{ years.}$$

Including the margin of error and the location factor it is expected that the bed level will have evolved within a time span of 0.5 to 2 years.

Bed Level Changes
The corresponding changes in the average bed level were estimated:
1. In the downstream part the lateral bed slope increases significantly. Maximum bed level differences occur of 0.2 m.
2. The minimum navigable width at OLR -2.80 m will increase with about 3 m.

5.7 Conclusions

Over the range of discharges of 1000 m$^3$/s to 4000 m$^3$/s the minimum navigable width for the equilibrium bed topography differs about 15 m, both for the situation with and without vanes. The variations found with a fluctuating discharge are expected to be less.
In contrast to the situation with vanes the minimum navigable width at a level of OLR -2.80 m increases during a period of high discharge. Note that this phenomenon is very sensitive to the model representation of the center of rotation of the cross section.

Over a wet or dry period lasting a number of years the average bed topography will evolve to a situation close to the equilibrium corresponding to the average of these high or low discharges. In case of a dry period lasting approximately 3 years this leads to a reduction of the navigable width of up to 7 m for the situation with vanes.
6. Bed Topography Transition

6.1 Introduction

After the installation of vanes in the outer bend, the river bed topography changes from a situation near an average without vanes, to a situation that varies around an average for the situation with vanes. In this chapter this transition of the shape of the river bed is studied, the emphasis lies on the time required for this adjustment. The structure of this chapter is as follows:

Paragraph 6.2 briefly describes the runs made to study the river bed transition.
Paragraph 6.3 studies the transition assuming a constant river discharge.
Paragraph 6.4 presents the bed topography transition over some years with average discharge variations, as well as over a period with significant floods.
Paragraph 6.5 summarizes the conclusions drawn in this chapter.

6.2 Transition Runs

In order to investigate the sensitivity of the adaptation time to the selected dominant discharge, three runs M were conducted with constant discharges of 1000 m$^3$/s, 1600 m$^3$/s, 2000 m$^3$/s. Although the bed level varies around an average value, these variations are small compared to this transition. Therefore the equilibrium bed level at the assumed dominant discharge of 1600 m$^3$/s is used as an initial situation.

The bed level transition is likely to accelerate during relatively short periods of high discharge. Whether this has a significant influence was studied using two series of varying discharges. In the first series the year 1983-1984 is used repeatedly in run H. This season was considered an average, as explained in the following chapter. Furthermore a series with the years 1965-1966, 1969-1970, 1974-1975, 1970-1971 and 1971-1972 was used in run P. In this case right after the installation of the vanes three years with relatively high discharges occur.

Further specifications of these runs are presented in Appendices 6A, 6E and 6H.
6.3 Transition At Constant Discharge

The bed topography transition at different constant discharges was studied using the changes in bed level indicators presented in Appendices 6B and 6C for run M. The first depicts the transition in a longitudinal and a cross section through time at a discharge of 1600 m$^3$/s. The latter includes graphs presenting the changes in bed level indicators in time for 1000 m$^3$/s, 1600 m$^3$/s and 2000 m$^3$/s discharge.

Longitudinal Sections
1. In the inner and outer bend the greatest bed level changes occur in the downstream part of the bend. This development is rapid compared to the upstream part.
2. The development of the bed level is more or less following an exponential curve, after a slow start.

Navigable Width
3. The transition of the minimum navigable width occurs at approximately the same rate as the transition of the inner bend bed level.

Relaxation Time
Using the same method as in Chapter 5 the relative relaxation times were measured, the results are included in Appendix 6D. However, the bed level changes in this transition are much greater than those occurring in the equilibrium runs.

The position factors $\zeta$ do not match those calculated for the constant discharge runs, the discharge factors $\xi$ were included in Figure 5.7 and do coincide with those found earlier. Again using the method of reversing equation (5.1), an average for the base relaxation time in the inner bend at km 870.000 at a discharge of 1000 m$^3$/s was calculated. Secondly the relaxation time for the inner bend bed level at a discharge of 1600 m$^3$/s was estimated.

\[
T_{\text{rel base inner bend km 870.000 Q=1000 m}^3/\text{s transition}} = 1710 \text{ days},
\]

the standard deviation of the prediction at other constant discharges is less than 10%.

\[
T_{\text{rel inner bend km 870.000 Q=1600 m}^3/\text{s trans}} = \frac{\xi}{\xi_{1600 m}^3/\text{s}} T_{\text{relaxation base trans}} = 0.42 \cdot 1710 \text{ days} = 720 \text{ days}
\]

This agrees with the adaptation time of 2 years found by Van Meerendonk and Struijsma [1996].

![Diagram](image-url)

**Figure 6.1** Transition of minimum navigable width at OLR -2.80 m after installation of vanes, at a constant discharge. Initial bed topography: equilibrium without vanes.
6.4 Transition With Discharge Variations

Based on runs H and P the bed topography transition was studied during a year with average discharge fluctuations and with significant flood discharges. Results for run H were included in Appendix 6F, the measured relaxation times in Appendix 6G. Secondly Appendices 6H and 6J contain the results of run P.

From this data the following observations were made:

Average Season 1983 - 1984

1. On average, the transition curve for most indicators is much like the one at the constant discharge of 1600 m$^3$/s, including some of the observed overshoots in the bed level transition. However the differences in discharge cause significant variations around this trend, predominantly in the inner bend.

2. These variations result in a slightly shorter relaxation time, compared to the constant discharge situation.

3. The period with high discharges causes a rapid change of the bed level in some cases. However, in other cases there is a slow start and the most important changes occur only during the subsequent period with lower discharges.

4. In Appendix 6G a corrected average discharge factor $\xi = 0.36$ was found, indicating a bed topography development that is approximately 10% faster than at the 1600 m$^3$/s constant discharge. A constant discharge of 1850 m$^3$/s would result in a similar relaxation time.

Flood Discharge Seasons

1. Although the general trend is comparable, the bed level transition in run P is significantly faster than in the 1600 m$^3$/s case. Furthermore, overshoots in bed level curves are greater.

2. The relatively long period of high discharge in the first year results in most indicators making a significant step towards the average value for the new situation. The entire 35 m increase in navigable width is achieved within the first half year after the installation of the vanes.

3. Averaging the relative relaxation times in Appendix 6J resulted in a discharge factor of $\xi = 0.24$. This indicates a bed topography development that is approximately 40% faster compared to the 1600 m$^3$/s constant discharge. A constant discharge of over 4000 m$^3$/s would be required to achieve a comparable relaxation time.

![Figure 6.2](image_url) Transition of minimum navigable width at OLR -2.80 m after the installation of vanes, varying discharge season 1983-1984 run H, compared to constant 1600 m$^3$/s discharge.
6.5 Conclusions

The fact that the discharge varies results in a reduction of the time needed for the bed topography transition after the installation of the vanes, even if compared to a constant discharge situation with the same average sediment transport. This is caused by the non-linear relation between flow velocity and sediment transport and by the constant presence of small sediment waves moving over the bed, generated by the constantly unstable bed situation.

The time the bed level needs to adjust after the installation of the vanes strongly depends on the occurring discharges. For an average situation it is expected that the transition takes place over a 1.7 year period, however, if a year with high discharges occurs it can be as fast as 1.2 years. Finally it is remarkable that the relaxation time for the minimum navigable width is shorter than the bed level at a fixed point, because of the fact that the normative cross section moves along the bend.
7. Bed Level Variation

7.1 Introduction

Once a new bed topography has developed after the installation of the vanes, it is important to know what variations in the bed level can be expected, and directly related to that, what variations in the navigable width. For the analysis of the yearly bed topography variations a series of computations with historical discharge records was conducted. This was done both for the situation with and without vanes, so that these could be compared. Secondly the validity of the dominant 1600 m³/s discharge was tested, looking at the average bed topography during runs with average discharge series.

This Chapter has the following outline:
Paragraph 7.2 briefly describes the dynamic Rivcom runs used in this chapter.
Paragraph 7.3 analyzes the bed variations occurring for the situation without vanes.
Paragraph 7.4 describes the results from similar runs with vanes.
Paragraph 7.5 presents a simple analytical method with which the variation of the bed level indicators can be predicted.
Paragraph 7.6 verifies the 1600 m³/s dominant discharge used in previous Rivcom research.
Paragraph 7.7 summarizes the conclusions drawn in this chapter.

7.2 Variation Runs

Three seasons were selected from the discharge records presented in Appendix 3A. These were used both for the situation with and without vanes, taking the equilibrium bed level at a discharge of Q = 1600 m³/s as an initial situation. Based on the peak discharge (5700 m³/s, flood period lasting 1 month) and the average sediment transport, the year 1983 - 1984 is considered an average. It was used in runs C and I. Furthermore two years with greater floods with different shapes were selected. In the first place the season 1965-1966 was used in runs N and J (peak discharge 5000 m³/s, total flood period lasting 3 months). Secondly the hydrological year 1994-1995 was used in runs K and L (peak discharge 7700 m³/s, total flood period lasting one month).
The average bed level was assessed based on runs B and I with the average year 1983-1984. Secondly run O was conducted, consisting of 5 years with longer periods with high and low discharges, that as a whole has an average sediment transport. In this series two years with higher discharges are followed by two years with somewhat lower discharges. The following years were selected: 1974-1975, 1979-1980, 1976-1977, 1973-1974, 1964-1965. Specifications of these runs were included in Appendices 7A, 7D, 7F, 7I, 7L, 7O and 7T.

7.3 Situation Without Vanes

The occurring ranges of variation of the bed level were measured for the runs without vanes. The results were summarized in Table 7.1.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Bed level 60 m from left bank</td>
<td>0.2 m</td>
<td>0.8 m</td>
<td>0.8 m</td>
</tr>
<tr>
<td>Bed level in centerline</td>
<td>0.2 m</td>
<td>0.4 m</td>
<td>0.3 m</td>
</tr>
<tr>
<td>Bed level 60 m from right bank</td>
<td>0.1 m</td>
<td>0.7 m</td>
<td>0.6 m</td>
</tr>
<tr>
<td>Minimum navigable width at OLR-2.80 m</td>
<td>3 m</td>
<td>6 m</td>
<td>6 m</td>
</tr>
</tbody>
</table>

Table 7.1 Range of variation of the bed level and navigable width in years 1983-1984 (average) and 1965-1966 and 1994-1995 (high discharge peaks). Without vanes.

The following observation were made:
1. In most cases a rapid change occurs in a period of high discharge, while the net change is nullified during the following period of lower discharge. Since the initial bed level was selected arbitrarily in relation to the start of the discharge series, a change of the bed level over these ranges of variation is possible in either direction.
2. The bed level variations show the same type of trend with the discharge variations in all three cases. Of course the differences are more pronounced in the latter runs, due to the higher floods.
3. In general the tendency of the changes is towards the equilibrium state which corresponds with the discharge at that moment.

The navigable width variation in the year 1965-1966 was plotted as an example in Figure 7.1.

![Figure 7.1 Variation of minimum navigable width at OLR -2.80m during season 1965-1966, situation without vanes, run N.](image-url)
7.4 Situation With Vanes

Bed level variations found in the runs for the situation with vanes are presented in Table 7.2.

<table>
<thead>
<tr>
<th>Range of variation of indicator</th>
<th>Season 83-84</th>
<th>Season 65-66</th>
<th>Season 94-95</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bed level 60 m from left bank</td>
<td>0.3 m</td>
<td>0.4 m</td>
<td>0.3 m</td>
</tr>
<tr>
<td>Bed level in centerline</td>
<td>0.3 m</td>
<td>0.5 m</td>
<td>0.4 m</td>
</tr>
<tr>
<td>Bed level 60 m from right bank</td>
<td>0.1 m</td>
<td>0.2 m</td>
<td>0.2 m</td>
</tr>
<tr>
<td>Minimum navigable width at OLR-2.80 m</td>
<td>9 m</td>
<td>10 m</td>
<td>13 m</td>
</tr>
</tbody>
</table>

Table 7.2 Range of variation of the bed level indicators and navigable width in a number of seasons, situation with vanes.

The following conclusions were drawn:
1. Again the rapid bed level changes during flood periods are generally nullified in the following period with lower discharges, while the tendency of the changes is towards the equilibrium state.
2. The range of variation of the bed level is less than in the situation without vanes.
3. The differences in the navigable width are significantly greater than those in the situation without vanes. This can be explained by the smaller lateral bed slope at the normative level.
4. In contrast to the situation without vanes, the navigable width increases during periods of high discharge. This is a result of the fact that the center of rotation of the cross section is located on the left bank side of the navigable fairway, as indicated in Figure 5.7.

![Figure 7.2 Variation of minimum navigable width at OLR-2.80m during season 1965-1966, situation with vanes, run J.](image)
7.5 Analytical Approximation Method

Given the equilibrium bed levels and the relaxation times found in the constant discharge runs of Chapter 6, it was attempted to approximate the variations in the bed level indicators by a simple analytical function. This idea results from the fact that in general the bed level variations appear to be in accordance with a shift towards the equilibrium bed level at the prevailing discharge. For this purpose the following steps were taken:

1. It was assumed that during these variations gradual bed level changes are dominant, while the time is lacking for the development of significant sediment waves moving over the river bed. Normal discharge variations are rapid compared to the relaxation times found in the constant discharge runs, of the order of 1500 days.
2. The trend of the bed development was assumed to be towards the equilibrium state.
3. Exponential curves were used within each run step, starting at the end value of the bed level of the previous run step and with an asymptotic value equal to the equilibrium level at the prevailing discharge.
4. The relaxation times for the exponential functions were assumed to be proportional (factor $\phi$) to the relaxation time found from the constant discharge runs in Paragraph 5.5.

Relaxation Time Factors

The relaxation time factor was obtained by fitting a series of consecutive exponential functions to the variation of the bed level indicator in the Rivcom results. Each describes a function $z_{b \text{approx}}(t)$ that approximates the bed level from the Rivcom results for one run step. A factor $1/2.3$ was added to the relaxation time, in order to account for its shape and the $10\%$ deviation used in the relaxation time definition in Paragraph 4.6, since $e^{2.3} \approx 0.1$.

$$
z_{b \text{approx}}(t) = z_{b \text{equil}}(Q) + (z_{b \text{end previous step}} - z_{b \text{equil}}(Q)) \exp \left( \frac{(t - t_{\text{end previous run step}})}{(\phi T_{\text{relaxation}}(Q) / 2.3)} \right)
$$  (7.1)

A useful method is obtained if the predicted range of bed level variation over a year is close to the one found in the Rivcom output. This criterion was used to find an optimal relaxation time factor $\phi$ in the three runs including vanes, for the indicator points in the cross section at km 870.000 for the situation with vanes. The resulting factors $\phi$ were averaged for each indicator and presented in Table 7.3.

![Figure 7.3](image)

Figure 7.3 Parameters defining exponential function that describes bed level indicator during one run step.
<table>
<thead>
<tr>
<th>Bed Level Indicators</th>
<th>Relaxation Time Factor $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>km 870.000</td>
<td></td>
</tr>
<tr>
<td>Bed level 60 m from left bank</td>
<td>0.19</td>
</tr>
<tr>
<td>Bed level in centerline</td>
<td>0.31</td>
</tr>
<tr>
<td>Bed level 60 m from right bank</td>
<td>0.22</td>
</tr>
<tr>
<td>Navigable width at OLR -2.80 m</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 7.3 Relaxation time factors $\phi$ for approximation method, derived from ranges of variation in bed level indicators in runs I, J and L. Situation with vanes only.

Apparently the relaxation times for the short-term bed level changes due to rapid discharge variations, are in the order of 20% of those for the long-term bed topography development.

Approximating the bed level variations with these averages resulted in ranges of variation within 15% of the Rivcom results. As an example the approximation function for the navigable width was plotted in Figure 7.4.

A precise representation of the changes was not found, especially not for the bed level in the outer bend points. This is a result of the fact that changes in the bed topography cannot be explained by local processes only. Sediment waves caused by previous events and in other places move along the river. Their effect on the bed topography cannot be predicted easily.

The presence of sediment waves generated by the constantly changing boundary condition results in a "restless" river bed with all sorts of small local irregularities that change relatively fast. This might explain the relatively rapid short-term bed level changes.

The method cannot be used to describe the absolute bed level over the year, since it was not possible to achieve an accurate representation of the actual bed level. A good representation of a long term trend in the bed topography is not expected to be found either, since in this case the much longer relaxation times from the constant discharge runs are appropriate.

However this approximation method can be used for a statistical analysis of the bed level variations occurring within a year, using historical discharge data.

![Figure 7.4](image)

Figure 7.4 Approximation of navigable width variations in season 65-66, run J, situation with vanes.
7.6 Verification Of Dominant Discharge

Besides the fact that the use of a dominant discharge concept implies that no results are obtained for bed level variations, the resulting bed topography should approximate the average of the actual varying bed topography. For this purpose the average value of all bed level indicators was determined, and a corresponding constant discharge was sought, based on the constant discharge results.

The best approach would be to run a large number of years from historical discharge records through the model and to calculate the time average bed level in all points. This is practically impossible considering the computation time involved.

**Situation Without Vanes**

The bed topography at the end of the first year in run B is approximately equal to the one at the end of the second year. Therefore the variations in the second year are around the actual average value and initial deviations from this average have disappeared.

An average in time for each bed level indicators, calculated over this second year, is included in Appendix 7R, together with equivalent constant discharges. The following observations were made comparing these averages with the values found with the dominant discharge:

1. In the downstream part of the bend the average lateral bed slope is greater, resulting in bed level differences of the order of 0.2 m.
2. Equivalent constant discharges were found between 1300 m³/s and 1800 m³/s, with an average value of approximately 1650 m³/s.
3. The average navigable width suggests an equivalent discharge that is much smaller. Since the differences are 1 m only, this anomaly could be the result of very small bed level changes.

![Variation of navigable width in run O, calculated average in run O and situation for 1600 m³/s constant discharge.](image)

**Figure 7.5** Variation of navigable width in run O, calculated average in run O and situation for 1600 m³/s constant discharge.
Situation With Vanes
An identical procedure was applied to the results of run I and O, for the situation with vanes.
1. In the inner and outer bend differences of the order of 0.04 m occur, however no systematic deviation was found.
2. The minimum navigable width is somewhat greater (of the order of 1 m) compared to the constant discharge situation.
3. Average equivalent discharges were found of 1670 m³/s and 1650 m³/s respectively, no significant deviations occurred.

7.7 Conclusions
For the situation with vanes bed level variations are limited to 0.5 m in the longitudinal sections at 60 m from the left and right bank, even in the case of significant periods with a high discharge. Subsequent changes in the navigable width are limited to 13 m. Note that this is within the differences in the equilibrium values found in the constant discharge runs (Chapter 5). Bed level variations are significantly less compared to the situation without vanes, however the variations in the navigable width are greater as a result of the smaller lateral bed slope.

For bed level changes as a result of normal discharge fluctuations in a year, the short term adaptation time is of the order of 20% of the long-term relaxation time for bed topography adjustment to trends in the average discharge.

Based on calculations with representative discharge variations it was concluded that a dominant discharge of approximately 1650 m³/s results in a good approximation of the average bed topography. The 1600 m³/s dominant discharge used by Van Meerendonk and Struiksma [1996] results in bed levels close to this average.
8. Rise In Water Level Caused By Vanes

8.1 Introduction

A field of submerged vanes is expected to cause a small rise in the upstream water level. This rise is caused by the drag forces exerted by the vanes, as well as by the changes in bed level in the bend. Such a rise in upstream water level is unfavorable in case of floods and Dutch law requires that compensating measures be taken. These can lead to a significant increase of the costs involved in river engineering projects. Therefore an accurate prediction is required beforehand. Paragraph 8.2 describes some of the phenomena causing the river slope to change from an axi-symmetric point of view.

Paragraph 8.3 studies the backwater curves in River Waal just after the installation of vanes.

Paragraph 8.4 addresses the problem of obtaining water levels from the results of the rigid lid Rivcom model.

Paragraph 8.5 indicates the rise in water level caused by the vanes in the equilibrium situation.

8.2 Axi-Symmetric Estimate Of Rise In Water Surface Slope

Odgaard and Mosoni [1987] calculated an "upper limit" estimate for the influence of submerged vanes on the water slope, using the sum of drag forces exerted by the vanes. They did not take into account the influence of the changing shape of the river bed cross section.

\[ \Delta i = \sum \frac{F_d - \Delta M}{\Delta b p g h} \approx \frac{0.5 \alpha}{(1 + \frac{H}{L})} \frac{u^2}{g R} \]  

(8.1)

In which

- \( \Delta i \) Increase in stream wise water surface slope
- \( \Delta M \) Vane-induced difference in momentum flux between the upstream and downstream end of the reach (neglected in upper limit estimate).
The drag forces can be taken into account by a reduced Chézy coefficient.

\[ C_{\text{vane}} = \frac{1}{\sqrt{1 + \frac{F_D}{C_0^2 \cdot \Delta s \cdot \rho \cdot g \cdot U^2}}} \]  

(8.1)

Prototype measurements in the East Nishnabotna River (average discharge 10.6 m³/s) did not indicate a difference in energy slope before and after the installation of 77 vanes. Calculations based on velocity profile measurements indicated that the change in momentum flux induced by the vanes was of the same order as the sum of drag forces.

Rivcom output includes the drag forces exerted by the vanes. From these an estimate was made for the river slope and the resulting rise in upstream water level. Results are presented in Table 8.1. The rise in upstream water level caused by the vane drag forces proved to cause a rise of a few millimeters only. The calculation was based on the equilibrium bed level for 1600 m³/s dominant discharge.

<table>
<thead>
<tr>
<th>Q (m³/s)</th>
<th>h (m)</th>
<th>( \Sigma F_D ) (kN)</th>
<th>( \Delta h_{\text{upstream}} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>4.35</td>
<td>33.9</td>
<td>3.3</td>
</tr>
<tr>
<td>1600</td>
<td>5.92</td>
<td>35.6</td>
<td>2.6</td>
</tr>
<tr>
<td>2000</td>
<td>6.77</td>
<td>43.5</td>
<td>2.7</td>
</tr>
<tr>
<td>3000</td>
<td>8.46</td>
<td>50.5</td>
<td>2.5</td>
</tr>
<tr>
<td>4000</td>
<td>9.60</td>
<td>60.1</td>
<td>2.7</td>
</tr>
</tbody>
</table>

Table 8.1 Upper limit estimate of rise in upstream water level by vane drag forces, based on Ref. [1987].

Rise In Water Level Resulting From Change In Lateral Bed Slope

A straight stretch of river with a horizontal lateral bed level results in a greater downstream water surface slope compared to a similar stretch with a constant transverse bed slope. This is a result of the fact that a relatively great part of the discharge will flow through the deep side of this channel, thus experiencing less friction per unit volume.

This can be deduced by integrating the Chézy equation across a hypothetical river cross section, using an average lateral bed level slope \( \frac{\partial z_b}{\partial n} \) as a variable and \( Q, C \) and the depth \( h_c \) in the river centerline as known values.

\[ Q = \int \left( h_c + \frac{\partial z_b}{\partial n} \right)^2 C \sqrt{\frac{2}{g}} \frac{1}{\left( \frac{\partial z_b}{\partial n} \right)^2} \left( h_c + \frac{\partial z_b}{\partial n} \right)^{\frac{1}{2}} \left( \frac{\partial z_b}{\partial n} \right)^{\frac{1}{2}} \]  

(8.2)

\[ i = \int \left( \frac{1}{Q C^2} \left( h_c + \frac{\partial z_b}{\partial n} \right)^{\frac{1}{2}} \right)^2 \left( \frac{1}{2} \right) \left( h_c + \frac{\partial z_b}{\partial n} \right)^{\frac{1}{2}} \left( h_c + \frac{\partial z_b}{\partial n} \right)^{\frac{1}{2}} 

(8.3)

With the resulting equation (8.3) the relation between the lateral bed slope and the downstream water surface slope was computed for several River Waal discharges, plotted in Figure 8.1.
1. The maximum average lateral bed for the situations without and with vanes are roughly 1.9\% and 3.5\% respectively. At a discharge of 1600 m$^3$/s the vanes lead to a 7\% increase in water surface slope, but this effect strongly decreases as the discharge increases.

2. Note that the average lateral bed slope over the river bends Hulhuizen I and II is much less than this maximum. Therefore, the change in the average water surface slope will be significantly less.

Considering the fact that this phenomenon cannot be described by averaging the water depth over the river cross-section, it is not appropriate to express the effect in terms of a reduction of the Chézy coefficient.

### 8.3 Backwater Curve

Just after construction of the vanes the river bed topography will have hardly changed and a backwater curve will develop. Assuming that the influence of the vanes on the stretch of river can be described simply by an increased friction, or reduction of the Chézy coefficient, an estimate of the maximum water level difference in this backwater curve was made.

In the final equilibrium situation, the rise of the water level upstream of the stretch in which the vanes are placed is described by equation (8.4). The water surface slope was used for the downstream part of the stretch of river with vanes, applying the Bélanger equation. As the damping length of the backwater curve is of the order of 20 km for this part of River Waal, a straight line is a good approximation of the backwater curve over the 2.5 km stretch of the vane field. The method is depicted in Figure 8.2, the resulting upstream water level change is described by equation (8.5).

\[
\Delta h_{\text{upstream vane final equal}} = \Delta s (i_{\text{vane final equal}} - i_0) = \Delta s i_0 \left( \frac{C_0}{C_{\text{vane}}} - 1 \right)
\]  

(8.4)

\[
\Delta h_{\text{upstream vane backwater curve}} = -\Delta h_0 - h_{\text{equi}} - h_{\text{fr}}
\]  

(8.5)
Figure 8.2 Rise in water level shortly after installation of vanes, directly upstream of vane field, determined by backwater curve. Approximating tangential line to backwater curve on downstream end of vane field was used.

Dividing equations (8.4) and (8.5) resulted in the simple equation (8.6). In the case of River Waal the right hand side factor is almost equal to 1.0. Therefore it was concluded that the maximum initial rise in water level, as a result of the installation of the vane field, is of the same order as the eventual rise in the upstream stretch of river, once the river bed has adjusted on a large scale to the local increase of friction.

\[
\Delta h_{\text{upstream vane backwater curve}} = \frac{C_0^2 i_0}{g} \left(1 - \frac{C_{\text{vane}}^2 i_{\text{vane final equal}}}{g}\right)
\]  

In which  
- \(C_0\) Chézy coefficient for undisturbed river \(m^{1/2}/s\)  
- \(C_{\text{vane}}\) Chézy coefficient for stretch of river with vanes \(m^{1/2}/s\)  
- \(i_0\) Water surface slope of undisturbed river  
- \(i_{\text{vane final equal}}\) Equilibrium surface slope for stretch of river with vanes
8.4 Method For Calculating Water Surface Slope From Rivcom Output

Pressure gradients are used in the rigid lid formulation of the 2DH momentum equations in Rivcom. These pressure gradients are related to the actual water surface slope, via equation (8.7).

$$i = \frac{1}{\rho g} \frac{dp}{ds}$$

(8.7)

In which

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$p$</td>
<td>Pressure on rigid lid</td>
<td>N/m²</td>
</tr>
<tr>
<td>$i$</td>
<td>Water surface slope</td>
<td></td>
</tr>
</tbody>
</table>

Rivcom solves the system of momentum and continuity equations, using a stream function/vorticity formulation, in which the pressure is eliminated. However, it is possible to recalculate the pressure gradient from the resulting velocity field, using the original momentum equations.

The resulting water surface slope was integrated over the modelled stretch of river along each longitudinal grid line. In this way a water level relative to the downstream boundary was obtained for all 48 grid point in each cross section.

From this pressure gradient the water surface slope was calculated, Appendix 8A contains a detailed description of this calculation method. The water level in two different cross sections (km 867.550 and km 868.750) was used, relative to the level at the downstream model boundary, as is indicated in Figure 8.3. Between these two cross sections the influence of the vanes on the flow pattern is negligible, hence the same rise in water level should be found theoretically.

The water surface slope was calculated in this way for the situation with and without vanes, using the equilibrium bed levels from the 1600 m³/s constant discharge runs. Constant discharges of 1000 m³/s, 1600 m³/s, 2000 m³/s, 3000 m³/s and 4000 m³/s were used, while bed level variations were assumed not to have a significant effect on the water surface slope.

Since the terms in the momentum equations are not approximated numerically in the same way as in Rivcom, it is unlikely that the calculated water surface elevation is evenly distributed over the cross section. The deviations occurring in a cross-section do give an impression of the inaccuracies of the calculation method. Note that small lateral head differences do occur in river bends, in the Hulhuizen II bend differences of the order of 30 mm are expected.
8.5 Rivcom Prediction Of Rise In Upstream Water Level

Upstream water levels calculated for the cross section at km 868.750 were plotted in Appendix 8B. From these average water levels over both cross sections were calculated and are included in Appendix 8C, together with the range over which the calculated water level varies within the cross section. The resulting rise in the water level upstream of the vane field was plotted in Figure 8.4.

The main conclusion of this graph is that the rise expected at a high discharge is of the order of 2 mm, while at discharges between 1000 m$^3$/s and 2000 m$^3$/s it varies between 16 mm and 4 mm. Comparing this to the phenomena mentioned in Paragraph 8.2 it was concluded that the lateral bed slope is an important factor at a low discharge, while the vane drag forces determine the minor rise in water level at a high discharge. As a result, again, it is not appropriate to derive a reduced Chezy coefficient from the resulting rise in water level.

Study of the results for every term in the numerical approximation separately learnt that:
1. The friction term is by far the most important factor. It yields a head difference of the order of 0.45 m over the modelled stretch of river. Errors in the calculation of this term are expected to be minimal, as no velocity derivatives are used.
2. The convection term predominantly results in a redistribution of the head difference over the cross section. It causes differences of the order of 0.03 m.
3. Finally the vane drag forces exerted by all vanes result in an average rise over the cross section of the order of 0.002 m only, in accordance with the axi-symmetric approximation.

The vanes have little direct effect on the head drop over the bend. The change in the cross-sectional shape due to the vanes is the dominating cause of the water level rise upstream of the vane field. This effect is significant during low discharges.

![Graph](image)

**Figure 8.4** Rise in water level upstream from Hulhuizen bend in River Waal as a result of vane field in preliminary design.
Remarks On Results

1. The water level in the 5 points closest to either bank often show significant deviations from the points closer to the river axis. This is a result of the inaccurate numerical approximation near the model boundaries. These points were therefore neglected.

2. Most numerical inaccuracies were caused by the approximation of the velocity derivatives in the convective term of the momentum equation. It was assumed that, on average, the approximations give good results for the mean slope along the river.

3. The differences in water level are almost equal for both cross sections, this confirms the reliability of the averaging procedure.

4. Peaks in the calculated water level are due to the drag forces of the vanes. In reality such water level irregularities do not occur upstream of the vane field. This is a result of the fact that these forces are compensated locally by the convective term in the momentum equation, as the water slows down slightly just upstream of a vane. Numerical calculation of the vane drag forces exerted on the flow is more accurate than the convective term approximation. It can therefore be assumed that a local error is made in the convective term approximation, because of the significant gradients in this term near the vanes. Reduction of this error should therefore eliminate the peaks, thus the water level rise due the vanes is reduced even further.

5. The differences in the calculated water level within the cross section are of the same order as the differences between the situations with and without vanes. This casts doubt upon the accuracy of the calculated head differences.

6. As a result of a change in the water level, a redistribution of discharges over the entire width of the river will occur during flood periods, causing a greater part of the discharge to flow outside the modelled channel. This effect was not taken into account in Rivcom input, as the water level data was based on the current situation. The actual rise in water level will therefore be slightly less than calculated.

7. In case more submerged vanes are to be applied the resulting rise in upstream water level will be approximately proportional to the number of vanes. This is an upper limit estimate, as the marginal changes in lateral bed slope are less.
9. Conclusions Part 1
Dynamic Bed Behaviour Under Influence Of Submerged Vanes

Based on the Rivcom calculations with varying boundary conditions, the following main conclusions can be drawn concerning the effects of submerged vanes on the bed topography in the Hulhuizen bend in River Waal:

Equilibrium Cross-Sectional Topography
1.1 At an increasing Waal discharge the most important changes in bed level occur in the downstream part of the bend. The typical bed profile in the bend extends in downstream direction, the maximum lateral bed slope increases significantly less than in the situation without vanes.
1.2 Over the range of discharges from 1000 to 4000 m³/s the minimum navigable width at OLR -2.80 m varies over only 15 m in the equilibrium situation. In contrast to the situation without vanes the minimum navigable width increases with an increasing discharge.
1.3 The time the bed topography needs to adapt to a new discharge strongly depends on this discharge.
1.4 Long wet or dry periods lasting a number of years result in the shift of the average bed levels. The average bed topography will change to the equilibrium state that corresponds with the prevailing average high or low discharge.
1.5 In the case of a dry period lasting approximately 3 years, this can lead to a reduction of the minimum navigable width at OLR -2.80 m of 7 m for the situation with vanes.

Bed Topography Transition After Installation Of Vanes
1.6 The discharges occurring just after the installation of the vanes strongly determine the time taken by the bed topography to reach a new equilibrium.
1.7 Relatively short periods with high discharge speed up this process significantly.
1.8 For a moderately high fluctuating discharge a transition time was found that is slightly shorter than the 2 - 3 years estimated by Van Meerdonk and Struijsma [1996] on the basis of a 1600 m³/s dominant discharge.
1.9 A significant period of high discharge can result in the total increase of the minimum navigable width at OLR -2.80 m of 35 m within half a year.
Bed Level Variations
1.10 The bed level variations through the year, as a result of the normal short-term discharge fluctuations, are reduced significantly by the presence of the vanes.
1.11 During a year with high flood discharges the bed level variations are of the order of 0.3 m, while variations in the minimum navigable width of up to 13 m can occur.
1.12 These ranges of variation apply both in positive and in negative direction with respect to the average bed topography.
1.13 The bed topography resulting from a dominant discharge of 1600 m³/s is a sufficiently accurate estimate of the average bed topography.

Rise In Upstream Water Level Caused By Vanes
1.14 The rise in the upstream water level as a result of the installation of submerged vanes in the Hulhuizen II bend in River Waal, according to the preliminary Haskoning design, is less than 16 mm, for a discharge above 1000 m³/s in the long term equilibrium situation.
1.15 At a discharge higher than 4000 m³/s the rise in water level is less than 2 mm.
1.16 Shortly after the installation of the vanes the maximum water level rise due to backwater effects is of the same order of magnitude.
1.17 At a low discharge the increase of the total bed friction is predominant, which is a result of a reduction of the lateral bed slope caused by the vanes.
1.18 The drag forces exerted by the vanes are dominating at a higher discharge, however the resulting rise in upstream water level is limited to 2 mm.
Part 2

Analysis Of Model Description Of Submerged Vanes

The main purpose of calibrating a model is to able to sleep well afterwards.

*based on Struiksmo*
10. Introduction Part 2
Analysis Of Model Description Of Submerged Vanes

10.1 Scope Of Study Part 2

In recent years some model studies were conducted to describe the influence of submerged vanes on river bed topography. The underlying assumptions, and the way in which the submerged vanes are included in these models are not always clear. 
A theoretical analysis could lead to a better understanding of these models. Furthermore results are available from a number of physical model tests, which can be used to verify these models.

Problem Definition
The reliability of the models describing the influence of submerged vanes on the flow and the bed topography in a river is uncertain.

Aims
The following aims are identified for the second part of this study:
1. To indicate the reliability of the model proposed by Wang, and to propose improvements.
2. To indicate the reliability of the way in which submerged vanes are modeled in Rivcom, and to propose improvements.
3. To propose how a better morphological model description for the vanes can be achieved.

10.2 Physical Model Research On Submerged Vanes

Scale model measurements were conducted in a number of studies on submerged vanes, but often insufficient data is presented in the publications to be able to compare the results. In some of the projects the emphasis has been on the measurement of the flow pattern induced by the vanes, while in others the induced changes in bed level have been studied primarily.
Data on the scale model tests used in this study are included in Appendices 10A through 10D.
Figure 10.1 Transverse velocity profiles measured downstream of a single vane. Flow depth \( h = 0.30 \) m, vane dimensions \( H = 0.1 \) m, \( L = 0.40 \) m, \( \alpha = 20^\circ \). *Van Meerendonk* [1995]

**Fixed Bed Tests**

*Odgaard and Spaljaric* [1986] studied the influence of a single vane on the flow pattern, measuring both the longitudinal and the transverse velocities downstream of a vane. Data regarding the measuring circumstances is scarce.

*Wang* [1991] conducted a number of scale model tests to underpin the theory described in Paragraph 11.3. Both flat plates and foil-shaped vanes were used in several configurations. In test series Q single flat-plate vanes of different height were placed in a flume with a flow depth of 0.15 m.

*Van Meerendonk* [1995] carried out an extensive physical model study for Delft Hydraulics in cooperation with the Egyptian Hydraulics Research Institute. Three test series were conducted (water depth 0.30 m):

1. Single vane test series \( T_1 \): several types of vanes, sizes and angles of attack.
2. Single transverse row tests \( T_r \): vanes placed in arrays with a different spacing.
3. Multiple transverse row tests \( T_m \): varying intervals between the arrays.

*Straathofma and De Groot* [1996 and 1997] investigated the effect of the dimensions of sheet-pile vanes in the Zandgoot flume of Delft Hydraulics (ZG series, 0.50 m flow depth).

In these scale model tests the near-bed velocities were measured at different heights above the bed. Figure 10.1 depicts the vertical distribution of the transverse velocities for natural spiral flow, as derived in Appendix 2B. It is similar to vertical distribution of the transverse velocity in the vortex trailing a vane, once it fills the entire water depth. In this graph the levels at which the near-bed velocities were measured are indicated. These levels are in the near-bed range where large gradients occur in the transverse velocity.

Furthermore, very coarse sediment was used on the bed of these models in order to generate sufficient roughness. This can result in a thicker layer in which these gradients occur. It is imaginable that "near-bed" velocities were measured at different relative heights in this layer. Therefore, the velocity readings might not be equivalent.
Mobile Bed Tests

The importance of mobile bed scale model tests lies in the fact that a description of the transverse velocities at a certain small height above the bed does not directly lead to a transverse bed shear stress.

Transverse bed shear stresses are related to the transverse velocities near the river bed generated by the vane. At a certain small height above the river bed, the velocity vector is assumed to be parallel to the bed shear stress vector. However this height is not obvious as there are great velocity gradients close to the river bed.

Odgaard and Kennedy [1983] conducted mobile-bed tests in a curved flume with transverse rows of two vanes only. These proved to be surprisingly effective in improving the uniformity of depth and velocities across the channel.

Odgaard and Spoljaric [1986] studied the bed topography changes induced by a single vane and arrays of four vanes in a straight flume (0.15 m flow depth).

In the single vane test little effect on the bed was found beyond about 6 channel depths, much faster than the measured decay of secondary current in flat-bed tests (of the order of 25 channel depths). It was concluded that this was due to the increased roughness as a result of the bed forms.

Deet Hydraulics [1987] conducted two test with arrays of four vanes in the De Voorst curved flume (average flow depth 0.11 m). The arrays were placed at longitudinal intervals of 7 and 14 water depths. With both configurations a promising increase of the navigable width was achieved, of the same order as in a similar test where the bed in the outer bend was fixed.

Bed level measurements from one of these tests (Q98 T7) were used to calibrate the morphological effect of the submerged vanes in the numerical Rivcom model, by means of a factor $\psi$ in the vane-induced spiral stream intensity. This is explained in Chapter 12. Appendices 10E and 10F contain depth profiles measured in cross sections and longitudinal sections in these two tests.

Odgaard and Wang [1991b] briefly mention the results of further scale model research in a curved and a straight flume with multiple transverse rows of vanes.
10.3 Prototype Trials On Submerged Vanes

Odgaard and Mosconi [1987] describe the results achieved in the East Nishnabotna River (average discharge 10.6 m³/s) in an eroding river bend with 110 m radius. A satisfactory improvement of the outer bend bed level was achieved initially, after the installation of 77 vanes. Later on the river started to change its course upstream of the vane field, thus reducing the effect. There was no indication that the vane system had increased the slope of the energy head.

Odgaard and Wang [1991b] were involved in solving shoaling problems in the West Fork Cedar River (average discharge 14 m³/s). Twelve submerged vanes maintain a deep main channel in the center of the river cross section, thus preventing erosion of the banks. Furthermore they mention other prototype field tests by Fukuoka and Watanabe in Japan.

Lambeck [1994] analyzed measurements by Rijkswaterstaat from the bend at Fortmond in River IJssel (bank full discharge 450 m³/s), in which 33 submerged vanes had been installed. Significant changes in the river bed topography were not found, neither were important changes in the transverse velocities. Finally, no significant rise in the water level upstream of this river bend was measured.

In the first place this lack of effect of the vanes was explained by the fact that important deviations in the positioning had occurred. Secondly, recent Haskoning calculations have shown that only a minor effect could be expected from this vane field design.
10.4 Outline Report Part 2

The general approach followed in the second part of this study is explained by the contents of the following chapters:

Chapter 11 describes the axi-symmetric models for the morphological effect of submerged vanes. The way in which submerged vanes are included in the model developed by Wang [1991] is tested, against measurements.

Chapter 12 deals with the Rivcom model. The complex way in which submerged vanes are included in this model is explained and tested against measurements. Furthermore, a summary is given of the most important factors in this model description.

Chapter 13 analyzes the measured transverse near-bed velocities generated by submerged vanes. An enhanced description of these velocities is sought and some conclusions on the vane-induced flow pattern are drawn.

Chapter 14 summarizes the most important conclusions and recommendations of Part 2 of this study.
11. Vanes In Axi-Symmetric Models

11.1 Introduction

A first estimate for a vane field design can be obtained using an axi-symmetric model. This describes the equilibrium bed level in a cross section of a hypothetical infinitely long bend of constant curvature. Wang [1991] introduced submerged vanes in such a model, which was implemented by Hekkoping [1996a] and applied for the Hulhuizen II bend in the River Waal. In this chapter this way of modelling submerged vanes is studied.

Paragraph 11.2 describes work by Odgaard et al., predecessors of the Wang model.
Paragraph 11.3 studies the axi-symmetric model developed by Wang [1991].
Paragraph 11.4 investigates the lateral bed slope predicted by this model.
Paragraph 11.5 compares the results from physical model research to the vane implementation in the Wang model.

11.2 Early Models

Odgaard and Kennedy [1983] developed a design method for a vane field, based on a balance of torque about the river axis, as explained in Figure 11.1. The driving torque is a result of non-uniform vertical distribution of the centripetal acceleration. It is counteracted by torque of the lateral “lift” forces exerted by the vanes. The total vane surface required for a balance between these influences is described by:

\[
\frac{NLH}{\Delta S_B} = \frac{2h}{c_L R} \left[ (m + 1) - (m + 2) \frac{H}{h} \right]^{1/2} \frac{h^2}{H^m} \quad (11.1)
\]
Most important assumptions:
1. The vane lift forces can be averaged over the entire width and length of the stretch of river.
2. The global balance of torque yields a globally horizontal bed level.

From scale tests in a curved recirculating sediment flume they concluded that the vane field following from this formula is effective in nullifying the secondary currents in the channel bend and the consequent lateral bed slope.

Description Of Bed Topography Downstream Of Individual Vanes

*Odgard and Spojuric* [1986] described the influence of a submerged vane on the river bed topography in detail. For this purpose a cosine-shaped lateral distribution of the transverse near-bed velocities behind each vane was assumed. They adopted a linear relation between this transverse velocity and the resulting lateral bed slope, which led to equation (11.2) for the changes in bed topography \( \Delta h \) behind each vane:

\[
\frac{\Delta h}{h} = \frac{\text{av}_{vp}(x = 0)}{\pi} \sin(\pi \frac{y}{h}) \exp\left(-\frac{2\pi}{8 + \frac{1}{h}} \frac{x}{\kappa}\right) \quad (11.2)
\]

In which

\[
a = \frac{\partial^2 h}{\partial n \partial v_b}
\]

Other assumptions:
1. Exponential damping of the transverse near-bed velocities in downstream direction, based on an eddy viscosity model for undisturbed open channel flow.
2. Simple superposition of the bed level changes in case of transverse arrays of vanes.

Measurements indicated the damping of the transverse velocities to be slightly faster than predicted, which could be a result of the extra turbulence induced by the vane. The observed changes in bed level with several vane configurations showed a pattern close to the predicted values.

*Odgard and Mosconi* [1987] followed an approach similar to *Odgard and Kennedy* [1983]. Furthermore an upper limit estimate was made for the influence of submerged vanes on the local water level slope, applied in Chapter 8.
11.3 Detailed Axi-Symmetric Description Of Vanes

Wong [1991] developed a complete design procedure for a field of submerged vanes, which describes the influence of the vanes on the bed level in a cross section in detail. Using an iterative procedure he found an equilibrium bed level for the axi-symmetric situation, using a finite difference scheme in transverse direction. The lateral bed slope was described by:

\[
\frac{dz_B}{dn} = \frac{m}{\rho g U R \sqrt{g_c \Delta z_D}} \tau_{b_n} \tag{11.3}
\]

The influence of the vanes was taken into account by means of the generated transverse bed shear stresses, superposed on those induced by the spiral flow in river bends. The first term in equation (11.4) introduces the influence of the natural spiral motion, note that it is similar to the equation derived in Appendix 2B. Differences are a result of different assumptions regarding the bed shear stress vector and the longitudinal and transverse near-bed velocity components.

The transverse bed shear stress generated by the vanes \( \tau_{v_{bn}} \) was calculated based on the induced transverse near-bed velocities, using equation (11.5).

\[
\tau_{v_{bn}} = \frac{w(2m + 1)(m + 1)}{R_c m^2 (2m^2 + w(m + 1))} \tau_{v_{bn}} + \tau_{v_{bn}}
\]

\[
\tau_{v_{bn}} = \frac{w}{u_B} \tau_{v_{bn}} \tag{11.5}
\]

In which \( w = U/u_B \) calibration coefficient
Iteration Procedure
An equilibrium bed level is found by assuming an inner bend bed level and calculating the bed level along the lateral slope, using equation (11.3). An iterative procedure results in such an inner bend bed level that the total discharge through the cross section equals the actual discharge. This theory is further explained in Appendix 11A, it was implemented by Harkoning [1996a] in the computer program Bocht1, focused on the bend Hulhuizen II in River Waal.

Most Important Assumptions
1. Infinitely long river bend of constant curvature.
2. Linear vertical distribution of transverse velocity due to spiral flow.
3. The transverse velocities induced by vanes can be described by vortices generated at the tip of each vane and its mirror images below the river bed and above the water surface.
4. In a cross section the vortex can be described by potential flow theory, which highly idealizes the actual flow pattern close to the vane.
5. Vortices damp out exponentially in downstream direction behind each vane, independently of the presence of neighboring vanes.
6. The reduction of the effectiveness of vanes placed in a lateral array can be described using biplane theory. It takes into account the mutual influence of vanes in the generation of the vortex only. An example of this interaction is depicted in Figure 11.4.

Figure 11.3 Tip vortex generated by each vane and its mirror images as applied in the theory developed by Wang [1991].

Figure 11.4 Distribution of vane-induced transverse near-bed velocities, with and without considering interaction of vanes. Wang [1991]
11.4 Axi-Symmetric Prediction Of Lateral Bed Slope

In order to test equation (11.3) for the transverse bed slope, for the circumstances in River Waal, a comparison is made between the bed level predicted by the Wang [1991] theory and the actual situation in the bend at Hulhuizen. Note however, that the length scales from Paragraph 3.2 and the Rivcom calculations in Part 1 suggest that an axi-symmetric equilibrium cross section is not reached in this bend. It is therefore expected that the maximum lateral bed slope is determined by the overshoot phenomenon, which is known to occur in the upstream part of river bends.

In Figure 11.5 the bed level in river cross section is plotted for:
1. Actual cross section with maximum lateral bed slope in bend Hulhuizen II km 869.700.
2. Cross-section predicted by Rivcom theory applied to an axi-symmetric situation.
3. Bed level predicted by the axi-symmetric Wang theory.

The calibration factor $w = 4$, as proposed by Odgaard, gives results that agree with the actual situation in this cross section. However, this agreement is deceptive because of the fact that the bed level in this cross section is probably determined by an overshoot.

A calibration factor $w = 5.5$ results in a shape of the cross section in accordance with the axi-symmetric situation predicted by Rivcom. The resulting lateral bed slope is plotted in Figure 11.5. Note that the water depth is greater on average, as a result of the sloping walls introduced in the implementation in the Flasikoning [1996a] program Bocht1.

Taking Account Of Overshoot In Vane Field
The spiral flow intensity included in an axi-symmetric approximation is based on an infinitely long bend with a constant radius and cross section. In the upstream part of a long river bend a combination of the spiral flow intensity and the transverse distribution of the main flow velocities result in an overshoot of the transverse bed slope.
The slope \( \frac{dz}{dn} \) is approximately proportional to the spiral flow intensity \( I \), which is directly influenced by the vanes (Appendix 11A, equation (A3). The following design procedure, based on an axi-symmetric model, is therefore suggested as a first approximation:

1. Calculate the equilibrium transverse bed slope with the axi-symmetric model for a bend with a representative radius and find the number of vanes necessary to achieve the required fairway improvement.

2. Multiply this number of vanes with the ratio of the observed transverse bed slope in a cross section and the equilibrium transverse bed slope. This results in the number of vanes to be applied in each cross section.

Of course the changing transverse distribution of the main flow velocity is not taken into account here. In order to achieve this a 2D model description is required. However this method could give better results compared to the splitting up of the bend in small stretches for each of which an axi-symmetric model is used, based on the local radius.

11.5 Comparison With Single-Vane Scale Model Tests

The transverse near-bed velocities predicted by \( \mathcal{W}ang [1991] \) were compared to the data available from physical model tests. For this purpose equation (11.6), describing the transverse near-bed velocities, was used to predict the velocities in the measuring points downstream of the vanes.

\[
 v_{vb} = \frac{\Gamma_i}{\pi \rho u_i H} \sum_{j=1}^{n} \left[ 1 - \exp \left( \frac{u}{4 \epsilon r_j^2} \right) \right] z_j
\]

(11.6)

In which:
- \( r_j \) Distance to the core of vortex (image) \( j \) (m)
- \( z_j \) Vertical distance to the core of vortex (image) \( j \) (m)

Figure 11.6 Transverse near-bed velocities induced by the vortex trailing the tip of a vane. \( \mathcal{W}ang [1991] \)
A quantitative comparison was made in two ways:

1. The vane-induced transverse near-bed flow per unit length \( j_{\text{vbn}} \) is estimated in each measurement cross section, both for the measurements and from the theoretical equation.

\[
j_{\text{vbn}} = \int_{\text{measured width}} v_{\text{vbn}} \, \mathrm{dn}
\]

(11.7)

This flow is estimated from the measured transverse velocities by multiplying these with the transverse distances between the measuring points and summation in each cross section. The method for integrating the transverse velocities in a cross section is explained in Figure 11.7.

2. The total vane-induced transverse near-bed flow \( J_{\text{vbn}} \) is estimated by multiplying \( j_{\text{vbn}} \) with the distances between the measurement cross sections and summing these values. This results in an integration of the transverse velocities over the bed area downstream of the vane, instead of just in a cross section.

\[
J_{\text{vbn}} = \iint_{\text{measured bed area}} v_{\text{vbn}} \, \mathrm{dn} \, \mathrm{ds}
\]

(11.8)

This flow is a measure for the morphological impact of a vane. This follows from the adopted linear relation between transverse near-bed velocities and transverse bed slope. Furthermore, the bed level changes are clearly a result of the set of vanes, while the direct local influence of a single vane is minor.

The \( j_{\text{vbn}} \) and \( J_{\text{vbn}} \) from the theoretical velocities were calculated relative to those obtained from measured transverse near-bed velocities. These relative values are included in Appendix 11C for the physical model tests available. Table 11.1 summarizes this data, presenting the transverse flow \( J_{\text{vbn}} \). A value >1 indicates that the equations overestimate the measured vane-induced near-bed flow.

<table>
<thead>
<tr>
<th>Flat Plate Tests</th>
<th>( J_{\text{vbn , Wang}} / J_{\text{vbn , meas.}} )</th>
<th>Sheet Pile Tests</th>
<th>Array Tests</th>
<th>( J_{\text{vbn , Wang}} / J_{\text{vbn , meas.}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_{1,1} )</td>
<td>1.05</td>
<td>( T_{1,16} )</td>
<td>Vane ( T_{1,14} )</td>
<td>1.11</td>
</tr>
<tr>
<td>( T_{1,2} )</td>
<td>0.77</td>
<td>( T_{1,17} )</td>
<td>spacing</td>
<td>1.07</td>
</tr>
<tr>
<td>( T_{1,4} )</td>
<td>0.88</td>
<td>( T_{1,18} )</td>
<td></td>
<td>0.79</td>
</tr>
<tr>
<td>( T_{1,5} )</td>
<td>0.93</td>
<td>( T_{2,} )</td>
<td>0.20 m</td>
<td>2.02</td>
</tr>
<tr>
<td>( T_{1,13} )</td>
<td>0.92</td>
<td>( ZG_{1} )</td>
<td>0.30 m</td>
<td>2.64</td>
</tr>
<tr>
<td>( T_{1,14} )</td>
<td>0.84</td>
<td>( ZG_{2} )</td>
<td>0.40 m</td>
<td>1.33</td>
</tr>
<tr>
<td>( T_{1,15} )</td>
<td>0.80</td>
<td>( ZG_{3} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( Q_{1} )</td>
<td>1.77</td>
<td></td>
<td>Vane ( T_{1,4} )</td>
<td>1.01</td>
</tr>
<tr>
<td>( Q_{2} )</td>
<td>2.01</td>
<td></td>
<td>spacing</td>
<td></td>
</tr>
<tr>
<td>( Q_{3} )</td>
<td>1.54</td>
<td>( d, ) 0.30 m</td>
<td></td>
<td>1.55</td>
</tr>
</tbody>
</table>

Table 11.1 Transverse near-bed flow \( J_{\text{vbn}} \) obtained by integration of velocities predicted by equation (11.6), relative to those obtained by integration of measured velocities in physical model tests.
Single Vane Tests
The following observations were made.
1. The initially generated transverse flow is underestimated in the case of the T series measurements.
2. The measurements from Wang [1991] in series Q are overestimated.
3. The streamwise damping is underestimated in all cases.

Vane Array Tests
4. In comparison to the single vane test with an identical vane, a significant overestimation occurs of the transverse near-bed flow.

An explanation for the observed phenomena is presented in Paragraph 12.3 and Chapter 13. The interaction of vanes in an array is studied specifically in Paragraph 13.5
12. Vanes In Rivcom Model

12.1 Introduction

For the design of the vane field for the bend in the River Waal near Hulhuizen a number of runs with the 2DH Rivcom model were conducted. In this Chapter the way in which vanes were introduced in this model is analyzed. Furthermore this part of the model was tested against data from physical model tests.

Paragraph 12.2 describes the way in which the morphological impact of submerged vanes was modelled in Rivcom.

Paragraph 12.3 tests the applied equation for the vane-induced transverse near-bed velocities.

Paragraph 12.4 analyzes the way in which this equation is utilized by Rivcom.

Paragraph 12.5 addresses the calibration of the Rivcom model.

Paragraph 12.6 summarizes conclusions.

12.2 Rivcom Implementation Of Vanes

The influence of submerged vanes is included in Rivcom at two points.

Firstly, the lift and drag forces exerted by the vanes are introduced in the 2D depth averaged equations of motion. For the River Waal situation, the downstream bed shear stress equivalent to the vane drag forces is of the order of 2 N/m², while bed friction is of the order of 8 N/m². In transverse direction the equivalent bed shear stress is of the order of 8 N/m².

Secondly, a spiral flow intensity is calculated, on the basis of the river bend curvature. Transverse near-bed velocities generated by the vanes are added as a corresponding spiral flow intensity. This latter influence is dominant for the bed topography, as it directly influences the transverse bed slope in the area downstream of a vane.

In Appendix 12C a detailed description of submerged vanes in Rivcom is presented and the calculation of the transverse bed slope is explained in Appendix 12B.
Crucial Factors In Implementation Of Vanes
Based on Appendix 12B and Rivcom results from Part 1 the following observations were made:
1. No influence on the bed level by individual vanes is visible.
2. The transverse bed shear stress is related linearly to the resulting transverse bed slope.
3. At a certain small height above the bed the transverse bed shear stress induced by a vane is assumed to depend linearly on the transverse near-bed velocity generated by the vane.
4. Therefore, the total vane-induced transverse near-bed flow determines the morphological impact of a vane.
5. The exact transverse position of the vane vortex in the river bed is of minor importance.

As a result two crucial factors in the description of submerged vanes in Rivcom can be identified:

A. Description of transverse near-bed flow $J_{v,be}$ generated by a vane.
   $J_{v,be}$ is directly determined by:
   1. Initial transverse near-bed peak velocity generated by a vane.
   2. Longitudinal damping of this peak velocity.
   3. Width of the transverse velocity profile in a cross section.
   This part is comprised by the transverse velocity equation used by Struikisma and De Groot [1996, 1997]. It is studied in Paragraph 12.3.

B. Relation between transverse near-bed velocities or flow and the resulting transverse bed slope generated by a vane.
   In Rivcom the transverse near-bed velocity generated by a vane is transformed to an equivalent contribution to the spiral flow intensity, which determines the direction of the sediment transport vector. This transformation consists of two components:
   1. Averaging of the transverse near-bed velocity over a Rivcom cell. This is investigated in Paragraph 12.4.
   2. Relation between this averaged velocity and a corresponding spiral flow intensity. The relation between the spiral flow intensity and “the” transverse near-bed velocity is not obvious, as explained in Paragraph 10.2. This point is not investigated in this study. The relation used in Rivcom is described in Appendix 12B.

12.3 Test Of Rivcom Equation For Near-Bed Velocities

Struikisma and De Groot [1996, 1997] used equation (12.1) as a basis for the description of the transverse near-bed velocities generated by a vane in Rivcom. It was tested versus measured near-bed velocities.

This comparison was made by predicting the transverse near-bed velocity in the measurement cross sections for the single vane test $T_{1,14}$ and for the same vane placed in an array with a spacing of $0.67 \times h$ and $1.33 \times h$ (tests $T_{12}, T_{77}$ and $T_{1,3}$). Furthermore, this was done for a smaller vane height in tests $T_{1,4}$ and $T_{86}$.

These graphs are included in Appendices 12D to 12I, an example was plotted in Figure 12.1.

\[
\nu = \frac{U \left( \frac{H}{h} \right)^{1/m} \tan \alpha}{1 + \frac{H}{L} \delta} \left( 1 - \frac{1}{1 + \left( \frac{y - y_l}{\delta H} \right)^2} \right) \left( 1 - \exp \left( - \frac{3 \frac{C_h}{x} \left( \frac{H}{h} \right)^2}{2x} \right) \right)
\]  

(12.1)
Near-Bed Flow
The transverse near-bed flow per unit length $j_{vbn}$ and the total transverse near-bed flow $J_{vbn}$ predicted by equation (12.1) were compared quantitatively with the values from physical model tests. Again relative values were calculated, these are included in Appendix 12. As a summary of these figures the values for $J_{vbn}$ are given in Table 12.1 on the following page.

Single Vane Tests
Based on the relative values for $j_{vbn}$ and $J_{vbn}$ from Appendix 12D through 12F the following conclusions were drawn regarding equation (12.1):
1. Prediction of initial transverse flow is on the safe side in general, except for the Q series measurements from Wang [1991].
2. For sheet pile shaped vanes in the ZG series measurements the initial transverse flow is predicted significantly higher compared to the T series. Apparently the influence of the water depth and the vane height are not included in the equation correctly.
3. Damping in streamwise direction is to weak generally, except for the ZG series measurements, for which a different scale was used. Again this suggests that the influence of the water depth and the vane height are not included in the damping length correctly.
### Table 12.1

<table>
<thead>
<tr>
<th>Flat Plate Tests</th>
<th>John Rev. (/ J_{\text{vbm, meas.}})</th>
<th>Sheet Pile Tests</th>
<th>John Rev. (/ J_{\text{vbm, meas.}})</th>
<th>Array Tests</th>
<th>John Rev. (/ J_{\text{vbm, meas.}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{1,1}$</td>
<td>1.05</td>
<td>$T_{1,1,16}$</td>
<td>1.12</td>
<td>$T_{1,14}$</td>
<td>spacing</td>
</tr>
<tr>
<td>$T_{1,2}$</td>
<td>0.77</td>
<td>$T_{1,1,17}$</td>
<td>1.07</td>
<td>$T_{1,18}$</td>
<td>0.79</td>
</tr>
<tr>
<td>$T_{1,4}$</td>
<td>0.88</td>
<td>$T_{1,1,16}$</td>
<td>1.22</td>
<td>$T_{1,14}$</td>
<td>0.93</td>
</tr>
<tr>
<td>$T_{1,5}$</td>
<td>0.93</td>
<td>$T_{1,1,17}$</td>
<td>1.25</td>
<td>$T_{1,18}$</td>
<td>0.92</td>
</tr>
<tr>
<td>$T_{1,10}$</td>
<td>0.97</td>
<td>$T_{1,1,16}$</td>
<td>1.23</td>
<td>$T_{1,14}$</td>
<td>0.84</td>
</tr>
<tr>
<td>$T_{1,14}$</td>
<td>0.84</td>
<td>$T_{1,1,17}$</td>
<td>1.25</td>
<td>$T_{1,18}$</td>
<td>0.80</td>
</tr>
<tr>
<td>$T_{1,15}$</td>
<td>0.80</td>
<td>$T_{1,1,16}$</td>
<td>1.45</td>
<td>$T_{1,14}$</td>
<td>0.86</td>
</tr>
<tr>
<td>$Q_1$</td>
<td>1.74</td>
<td></td>
<td></td>
<td>$T_{1,14}$</td>
<td>0.80</td>
</tr>
<tr>
<td>$Q_2$</td>
<td>2.11</td>
<td></td>
<td></td>
<td>0.30 m</td>
<td>0.94</td>
</tr>
<tr>
<td>$Q_3$</td>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Observe from Vane Array Tests

*Odegard and Wang [1991a]* predicted the generation of a coherent circulation in case the vane spacing is less than 2 to 3 vane heights. In case the vane spacing is greater, a less efficient system of individual vortices is expected. The actual measurements ($H/h = 0.33$ and 0.16) indicate more complex interaction phenomena.

1. Measurements indicate that indeed a consistent circulation is generated in the test with a vane spacing of 0.67h (2 H). However, the transverse velocities in this circulation are significantly less than expected based on the interaction theory presented by *Wang [1991]*, resulting from an increased streamwise damping. With a greater spacing, individual vortices persist downstream.

2. The test with a lateral vane spacing of 0.20 m (0.67 h, 2 H) shows a significant overestimation of the transverse near-bed flow close to the vane, and a longitudinal damping that is much too weak. The vortices appear to hinder each other during their generation and further downstream.

3. In the test with a vane spacing of 0.30 m (1 h, 3 H) the total overestimation of near-bed velocities is much smaller, however there is still a distinct difference compared to the test with a single vane.

4. In case of the lateral spacing of 0.40 m (1.33 h, 4 H) clearly counter-rotating vortices appear at some distance downstream from the vanes in between the vane-generated vortices. These lead to negative near-bed velocities, thus strongly reducing the net transverse near-bed flow.

5. Counter-rotating vortices occur as well in the test with a smaller vane height and 0.30 m transverse vane spacing (1.33 h, 6 H) counter rotating vortices occur as well.

These observations are further analyzed in Paragraph 13.5.

### 12.4 Calculation Of Average Transverse Near-Bed Velocity In Rivcom Cell

The vortex generated by a vane is assumed to be contained by the chain of curvilinear grid cells downstream of the one in which the vane is located. For a cell in this chain the peak value of the transverse near-bed velocity is calculated with equation (12.1) in the cross section through the cell center. A corresponding average transverse near-bed velocity over the entire Rivcom cell $v_{vbc}$ is calculated based on this peak velocity, using equation (12.2).

$$
\bar{\psi}_{vbc} = \psi_{vbc} \frac{A_z}{A_c} = \psi_{vbc} \frac{\psi \Delta \lambda}{\Delta \delta} = \psi \frac{\psi_{vbc}}{\psi_{vbc}} \frac{h}{\Delta b}
$$

(12.2)
Figure 12.2 Calculation of average transverse velocity $v_{ve}$ in a Rivcom cell, by means of the transverse near-bed flow $j_{ve}$ and based on peak transverse velocity from equation (12.1).

In which

- $\Delta e$: Plan surface of a Rivcom grid cell
- $\Delta v$: Surface over which a vane is assumed to induce transverse near-bed velocities
- $\Delta b$: Transverse width of a Rivcom grid cell
- $\Delta l$: Streamwise length of a Rivcom grid cell
- $v_{ve}$: Vane-induced cell-averaged transverse near-bed velocity
- $v_{ve}$: Peak value of transverse near-bed velocity induced by a vane in cross section through the cell center.
- $\psi$: Calibration coefficient for averaging

This equation can be understood as the peak velocity, multiplied by a factor $\psi$ to obtain an average, multiplied by an effective width equal to the water depth $h$. In this way a transverse flow per unit length $j_{ve}$ is obtained. This flow is then distributed over the width of the entire Rivcom cell in order to obtain a corresponding average transverse velocity. This is explained in Figure 12.2.

As a result the influence of vanes on the river bed topography is directly proportional to the averaging factor $\psi$. Furthermore Rivcom only utilizes the peak-velocities in a cross section predicted by equation (12.1).

In equation (12.2) the local water depth is used as a measure for the width of the velocity profile, in contrast to equation (12.1) in which the vane height is used as a measure for the width. It is indicated in Figure 12.3 that this leads to a mayor overestimation of the morphological effect of a vane in case of small ratios of vane height to water depth. It should however be kept in mind that in this case the effect of the vanes is limited anyhow.
Figure 12.3  Transverse near-bed flow resulting from the averaged velocity in equation (12.2) (including the calibration coefficient \( \psi = 1.6/\pi \)) relative to the base equation (12.1).

Using the velocity averaging method in Rivcom, again the relative transverse near-bed flow was calculated. The results can be found in Appendix 12K and are summarized in Table 12.2. Comparing Tables 12.1 and 12.2 it seems that the Rivcom averaging procedure is on the safe side in general. However, in prototype circumstances vanes will be buried partly, which leads to the overestimation of the morphological impact described above.

<table>
<thead>
<tr>
<th>Flat Plate Tests</th>
<th>( \text{J}<em>{\text{Riv int.}} / J</em>{\text{Riv meas.}} )</th>
<th>Sheet Pile Tests</th>
<th>( \text{J}<em>{\text{Riv int.}} / J</em>{\text{Riv meas.}} )</th>
<th>Array Tests</th>
<th>( \text{J}<em>{\text{Riv int.}} / J</em>{\text{Riv meas.}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_{1,1} )</td>
<td>0.78</td>
<td>( T_{1,16} )</td>
<td>0.84</td>
<td>( T_{1,14} )</td>
<td>Vane ( T_{1,14} )</td>
</tr>
<tr>
<td>( T_{1,2} )</td>
<td>0.60</td>
<td>( T_{1,17} )</td>
<td>0.81</td>
<td>( T_{1,18} )</td>
<td>spacing</td>
</tr>
<tr>
<td>( T_{1,3} )</td>
<td>0.83</td>
<td>( T_{1,18} )</td>
<td>0.60</td>
<td>( T_{1,2} )</td>
<td>0.20 m            1.52</td>
</tr>
<tr>
<td>( T_{1,4} )</td>
<td>0.91</td>
<td>( ZG_{1} )</td>
<td>1.29</td>
<td>( T_{1,7} )</td>
<td>0.30 m            1.85</td>
</tr>
<tr>
<td>( T_{1,5} )</td>
<td>0.71</td>
<td>( ZG_{2} )</td>
<td>0.43</td>
<td>( T_{7} )</td>
<td>0.40 m            1.34</td>
</tr>
<tr>
<td>( T_{1,6} )</td>
<td>0.59</td>
<td>( ZG_{3} )</td>
<td>0.50</td>
<td>( Vane T_{14} )</td>
<td>spacing</td>
</tr>
<tr>
<td>( Q_{1} )</td>
<td>2.37</td>
<td>( ZG_{3} )</td>
<td>0.45</td>
<td>( T_{14} )</td>
<td>0.30 m            1.71</td>
</tr>
<tr>
<td>( Q_{2} )</td>
<td>1.89</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( Q_{3} )</td>
<td>1.19</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 12.2  Transverse near-bed flow \( J_{\text{Riv int.}} / J_{\text{Riv meas.}} \) obtained by integration of the cell averaged velocities predicted by the Rivcom averaging equation (12.2), relative to those obtained by integration of measured velocities in scale model tests \( J_{\text{measured}} \).

12.5  Calibration On Mobile-Bed Scale Model Tests

Equation (12.2) was calibrated by Flinkstra and Van Zanten [1995] using one mobile bed scale model test Q98 T7, mentioned in Paragraph 10.2. This led to a value for \( \psi = 1.6/\pi = 0.51 \), which is in the range of what is expected from an integration of equation (12.1). However this is contradictory to the significant decrease of vane efficiency, observed for vanes placed in an array.

Considering the uncertainties involved in these physical model tests and the dominant influence of factor \( \psi \) it is appropriate to test this calibration versus the results of other mobile bed tests with vanes.
Mobile-bed scale model tests with a different vane spacing could be used to test if the river bed changes are indeed as sensitive to the vane spacing as expected based on the measured transverse near-bed flow.

**Vane Spacing**

The critical influence on the vane efficiency of transverse spacing of the vanes placed in an array, in relation to the water depth and the vane height, is currently not included in Rivcom as the effects of single vanes are simply added.

In Figure 12.4 the vane height and vane spacing combinations in the Tᵢ series test are plotted, together with the values occurring in the preliminary vane field design for the Hulhuizen II bend in River Waal. Finally those in some other mobile bed scale tests, mentioned in Appendix 10D, were plotted. From this graph two important conclusions were drawn:

1. In the preliminary design for the vane field for the bend at Huilhuizen counter-rotating vortices are likely to occur. A smaller transverse spacing of the vanes would be beneficial.
2. In the scale model test Q98-T7, used for calibrating the vane efficiency in Rivcom, vane heights and spacings occur close to those in the design for the bend at Hulhuizen. Therefore the calibration is valid. Other mobile-bed tests in the same range are available and should be used for a verification of this calibration.

Assuming that the current calibration of the vane efficiency in Rivcom is valid for this vane field design, it is strongly emphasized that changing the applied transverse vane spacing and vane height leads to an invalid model.

This would require a formulation for the negative effect of nearby vanes based on this transverse spacing in relation to the vane height and water depth. In Paragraph 13.5 a first indication of this efficiency reduction is given, based on the physical model test series Tᵢ, in which several layouts for vanes placed in an array were utilized.
12.6 Conclusions

The equation describing the vane-induced near-bed velocity in Rivcom gives adequate results for the single vane situation, when looking at the generated transverse near-bed flow. However, this flow is significantly overestimated for the situation with arrays of vanes. This is a result of the fact that the transverse spacing has a very critical effect on the generated near-bed flow, because of counter-rotating vortices and increased turbulence. The subsequent reduction of the morphological impact of the vanes should be included in the Rivcom model, if these parameters are to be changed from the calibration circumstances.

The current calibration of the vane efficiency in the Rivcom model for River Waal is valid in this respect. However, further testing of this calibration of the vane efficiency by several physical mobile-bed model tests is appropriate. It is of utmost importance that the vane height, vane spacing and water depth combinations occurring in these models are close to the prototype situation.
13. Analysis Of Vane Scale Tests

13.1 Introduction

Earlier studies of physical scale model tests with vanes resulted in a useful indication about the validity of equation (12.1), describing the vane-induced transverse near-bed velocities behind the vane. However, some issues remained unanswered and recently new physical model research by Struijsma and De Groot [1997] brought about new questions. Therefore, the vane-induced transverse near-bed velocities are studied in this chapter thoroughly from scratch.

Paragraph 13.2 summarizes the results from the previous analyses.
Paragraph 13.3 concerns the quest for an enhanced description of the vane-induced transverse near-bed velocities by single vanes.
Paragraph 13.4 tests this equation versus measurements.
Paragraph 13.5 addresses the problems occurring in case the vanes are placed in arrays.
Paragraph 13.6 summarizes the main conclusions from this Chapter.

13.2 Previous Analysis Of Scale Model Measurements

Struijsma and De Groot [1996 and 1997] tested the Rivcom equation (12.1), describing the transverse near-bed velocities downstream from a submerged vane, using the T and ZG series measurements. This analysis was based on the transverse peak velocities, found by fitting equation (13.1) to the near-bed measurements.

\[ v_{vb} = \frac{v_{vbp}}{1 + \left( \frac{y - \mu_c}{0.8H} \right)^2} \quad (13.1) \]

The peak value \( v_{vbp} \) was used as the only parameter, while the transverse position of this peak \( \mu_c \) was inserted manually. A velocity profile width was assumed, equal to 0.8H. The resulting transverse peak velocities were standardized with those predicted by the Rivcom equation (12.1). For this purpose two parameters were introduced: the standardized near-bed transverse peak velocity \( Y \), and the standardized distance downstream from the vane center \( X \).
Figure 13.1 Vane-induced transverse near-bed peak velocities $v_{\text{vbp}}$ from ZG series measurements (0.50 m water depth) plotted standardized with prediction by Rivcom equation (12.1). *Struiksma and De Greef* [1997]

\[
Y = \frac{v_{\text{vbp measurement}}}{v_{\text{vbp Rivcom, } x = 0}} = \frac{0.8 \left( 1 + \frac{H}{L} \right)}{U \left( \frac{H}{h} \right)^{1/3} \tan \alpha} 
\]

(13.2)

\[
X = \frac{x_{\text{measurement cross section}}}{cA_{\text{v damping}}} = \frac{x_{\text{measurement cross section}}}{\frac{Ch}{\sqrt{g}} \left( \frac{H}{h} \right)^2} 
\]

(13.3)

For the single vane T series measurements it was concluded that the equation used in Rivcom leads to an underestimation of the actual peak transverse velocities generally, thus being on the safe side. However a comparison with the ZG series measurements showed that with relatively high vanes it results in a significant overestimation of the occurring transverse velocities, as is depicted in Figure 13.1. Finally the Rivcom equation overestimated the measurements presented in literature by *Wang* [1991] and *Odgard and Spójnicz* [1989].

**Position Of Vortex Core**

Using transverse velocities measured in vertical profiles an estimate was made of the vertical position of the vortex core behind the vane. This was done by looking at the level at which the transverse velocities are equal to 0. Because of the fact that the absolute transverse velocities are very small in this region, this procedure is rather inaccurate. Nonetheless, it was confirmed that the vortex core tends to be at a level of 0.3 to 0.5 water depths above the bed, beyond about 8 water depths downstream of the vane. Close to the vane the vortex core is at the same level as the top of the vane, as is clearly seen in Figure 13.2.

This indicates that the vortex grows in downstream direction, to fill the entire water depth.
Secondly the transverse position of the vortex core was studied, using the manually calibrated transverse coordinate $\mu_s$ in equation (13.1). It was concluded that the vane size and especially the angle of attack of the vane determine this position, as is seen in Figure 13.3. The maximum transverse displacement of the vortex core was found to be of the order of 0.4 water depths, which corresponds to less than 3.3 m, scaled to River Waal. As this is significantly less than the width of a Rivcom grid cell, the transverse displacement is of minor importance only and will not be studied any further.
13.3 Enhanced Description Of Transverse Near-Bed Velocities

Based on the scale model measurements it was investigated if an enhanced formulation for the vane-induced transverse near-bed velocities could be found. For this purpose first a function was fitted to the transverse near-bed velocities measured in each cross section for all measurement data. For this purpose a transverse velocity distribution function is adopted from Struckmeier and De Groot [1996], the shape of which has proven to be close to the measured values.

\[
v_{vb} = \frac{v_{vbp \, fit}}{1 + \left( \frac{y - \mu_{fit}}{b_{fit}} \right)^2}
\]

(13.4)

In which
- \( v_{vbp \, fit} \) Transverse near-bed peak velocity \( \text{m/s} \)
- \( b_{fit} \) Representative width, at which \( v_{vb} = 0.5 \) \( v_{vbp} \) \( \text{m} \)
- \( \mu_{fit} \) Transverse position of velocity peak \( \text{m} \)

However, this time all three parameters were fitted simultaneously, minimizing the sum of squared velocity differences between this function and measured values. In this process the small negative transverse velocities, occurring in some cases as a result of counter-rotating vortices, were neglected. The occurrence of these counter-rotating vortices is highly depending on the presence of nearby vanes in an array or a flume wall. This will be accounted for in the analysis of the tests of vanes placed in an array.

The analysis of the resulting figures was split up in three parts: the damping, the transverse width of the velocity profile and the initially generated near-bed peak velocity.

Damping Of Peak Velocity

In the first place the damping of the peak velocities from the fitted function in every cross section was studied. For this purpose the peak values were standardized with the values from the second measured cross section behind the vane, multiplied by the local value of the adopted damping function. In this way the initial value is restandardized as 1. The measured initial peak velocities were not used for two reasons.

1. The transverse velocities measured close to the vane are disturbed by local flow patterns.
2. In the first cross section the number of measuring points is often limited while the velocity peak is steep. This leads to an inaccurate fitting process.

![Figure 13.4](image-url)

Figure 13.4 Enhanced damping function for flat plate vanes, with transverse near-bed peak velocities from measurements, standardized in second measurement cross section.
The data from tests with flat plate vanes and sheet pile shaped vanes were treated separately. Firstly the factor $\frac{Ch}{\sqrt{g}}$ was adopted from the damping length of spiral flow intensity, as the influence of turbulence is expected to have a comparable damping effect on the vortices. Furthermore a relation with the vane height was clearly observed. The damping functions plotted in Figures (4.4) and (4.5) were selected for flat plate vanes and sheet pile shaped vanes. Note that the axis of this graph differ from the factors X and Y used by Van Meerendonk and Straatsma [1996].

$$v_{vpb,pl} = v_{vpb,pl}(x = 0) \left[ 1 - \exp \left( -\frac{0.74}{x} \right) \right]$$ \hspace{1cm} (13.5)

$$v_{vpb,sp} = v_{vpb,sp}(x = 0) \left[ 1 - \exp \left( -\frac{0.44}{x} \right) \right]$$ \hspace{1cm} (13.6)

With a damping length $\lambda_v = \frac{Ch}{\sqrt{g}} \sqrt{\frac{H}{h}}$, and $x$, distance downstream from backside vane.

From these figures the following conclusions were drawn.

1. T series measurements lead to the conclusion that the influence of the relative vane height is included in the damping length correctly for the range $H/h = 0.16$ to 0.33.
2. Series T and ZG coincide, therefore the influence of the water depth appears to be correct.
3. The validity of the damping functions is limited to the range for the angle of attack $\alpha = 15^\circ$ to $20^\circ$. Note that at vane angles greater than $\alpha = 20^\circ$ flow separation was observed.
4. In the case of sheet pile shaped vanes, the damping appears to be significantly faster initially, compared to flat plate vanes. This is probably a result of the fact that the generated vortex initially includes a higher turbulence intensity because of the irregular shape of the vane.
5. For measurements ZG1 and ZG4 the damping behaviour is significantly different compared to the other measurements. In these tests the vanes are relatively high ($H/h=0.4$), thus the initial vortex induces relatively small near-bed velocities. In contrast to other tests the vortex core does not rise downstream, resulting in a smaller reduction of the near-bed velocities.
6. The damping in the Q series measurements can not be brought in agreement with the other tests, probably due to the small scale and the small measuring height above the bed.

![Graph](image.png)

Figure 13.5 Enhanced damping function for sheet pile shaped vanes, with transverse near-bed velocities from measurements, standardized in second measurement cross section.
Width Of Transverse Velocity Profile
Next the values $b_{th}$, describing the width of the transverse near-bed velocity profile were studied. The dominating factors in the width are the water depth $h$ and the vane height $H$. Based on the measurements at different vane heights a linear proportion to $H^{0.5}$ was obtained and for reasons of dimension also with $h^{0.5}$. The fitted widths, standardized with these proportions are plotted in Figures 13.6 and 13.7.

From these graphs the following conclusions were drawn concerning the width of the transverse velocity distribution.
1. Only measurements in the ZG series show a significant increase of the velocity profile width in downstream direction, no explanation was found for this phenomenon. Nonetheless the average width is equal to the T series measurements, confirming the proportion to $h^{0.5}$.
2. A minor increasing trend in downstream direction occurs generally. This confirms the existence of a coherent vortex, no distinct differences are visible between the flat plate vanes and the sheet pile shaped vanes. As no significant general trend was found a constant average width was adopted.
3. Subsequently it was assumed that the damping in downstream direction of the entire transverse velocity profile is approximately equal to the damping of the peak transverse velocity. This was more or less confirmed by damping trends of the average transverse velocity found by Van Meerendonk [1995].
4. Measurements at different vanes sizes confirm the proportion of the width to $H^{0.5}$. No clear trend with other vane parameters was apparent.
5. The level of the vortex core suggests that close to the vane the width of the velocity profile is dominated by the vane height, while further downstream it is determined by the water depth. Careful study of the figures learned that no confirmation can be found for this effect.

For the cases with flat plate vanes and sheet pile shaped vanes an approximately equal average factor $0.45H^{0.5}h^{0.5}$ was found. This led to the following function describing the transverse distribution of the transverse near-bed velocities.

\[ v_{th} = \frac{v_{vbp}}{1 + \left(\frac{y - H^{0.5}}{0.45H^{0.5}h^{0.5}}\right)^2} \]  (13.7)

---

**Figure 13.6**  Width of transverse near-bed velocity profile in measurement cross sections for flat plate vanes, standardized with $H^{0.5}h^{0.5}$.
Initial Peak Transverse Near-Bed Velocity

The remaining factor was the initial transverse peak velocity. However as can be seen in Figures 4.4 and 4.5 the measured initial peak velocities contain a lot of irregularities that have disappeared in the second cross section. Therefore an initial peak transverse velocities $v_{\text{ebp}}(x=0)$ corresponding to the velocity measurements in the cross sections further downstream was used for the prediction of the total effect of a vane. For this purpose the equation resulting from a combination of (13.5)/(13.6) and (13.7) was fitted to all velocity measuring points for each test. The first cross section was not taken into account, as the relatively high velocities here would have significant influence in the fitting process.

<table>
<thead>
<tr>
<th>Flat Plate Tests</th>
<th>$H$, $L$, $\alpha$</th>
<th>$v_{\text{ebp}}(x=0)$</th>
<th>Sheet Pile Tests</th>
<th>$H$, $L$, $\alpha$</th>
<th>$v_{\text{ebp}}(x=0)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{1,14}$</td>
<td>$h=0.30 \text{ m}, U=0.22 \text{ m/s}$</td>
<td>0.077</td>
<td>$T_{1,16}$</td>
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<td>0.077</td>
</tr>
<tr>
<td>$T_{1,2}$</td>
<td>0.1, 0.4, 18</td>
<td>0.063</td>
<td>$T_{1,17}$</td>
<td>0.1, 0.4, 18</td>
<td>0.092</td>
</tr>
<tr>
<td>$T_{1,4}$</td>
<td>0.05, 0.4, 18</td>
<td>0.048</td>
<td>$T_{1,18}$</td>
<td>0.1, 0.39, 18</td>
<td>0.092</td>
</tr>
<tr>
<td>$T_{1,5}$</td>
<td>0.05, 0.2, 18</td>
<td>0.031</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$T_{1,3}$</td>
<td>$h=0.30 \text{ m}, U=0.22 \text{ m/s}$</td>
<td></td>
<td>$T_{1,1}$</td>
<td>$h=0.50 \text{ m}, U=0.36 \text{ m/s}$</td>
<td></td>
</tr>
<tr>
<td>$T_{1,3}$</td>
<td>0.1, 0.4, 15</td>
<td>0.062</td>
<td>$ZG_1$</td>
<td>0.1, 0.67, 17.5</td>
<td>0.100</td>
</tr>
<tr>
<td>$T_{1,14}$</td>
<td>0.1, 0.4, 18</td>
<td>0.077</td>
<td>$ZG_1$</td>
<td>0.2, 0.67, 17.5</td>
<td>0.094</td>
</tr>
<tr>
<td>$T_{1,13}$</td>
<td>0.1, 0.4, 20</td>
<td>0.082</td>
<td>$ZG_1$</td>
<td>0.2, 0.36, 17.5</td>
<td>0.049</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$ZG_3$</td>
<td>0.1, 0.36, 17.5</td>
<td>0.080</td>
</tr>
<tr>
<td>$Q_1$</td>
<td>$h=0.15 \text{ m}, U = 0.24 \text{ m/s}$</td>
<td>0.046</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_2$</td>
<td>0.04, 0.15, 20</td>
<td>0.046</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_3$</td>
<td>0.05, 0.15, 20</td>
<td>0.048</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_4$</td>
<td>0.07, 0.15, 20</td>
<td>0.051</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 13.1 Initial peak transverse near-bed velocities $v_{\text{ebp}}(x=0)$, obtained by fitting combined equations (13.5)/(13.6) and (13.7) to the velocity measurements in all cross sections, except for the first just downstream of the vane.
This resulted in transverse near-bed velocities $v_{vbp}$ at $x_w=0$ presented in Table 13.1. Next an equation was selected that describes the influence of the vane characteristics on these initial transverse peak velocities. In this process the following steps were taken:

1. A linear relation with the average main flow velocity $U$ was assumed.
2. The proportions are assumed to be equal for the cases with flat plate vanes and sheet pile shaped vanes, however an independent proportionality factor was used.
3. Based on $T_{1,13}$ and $T_{1,14}$ tests a linear proportion to $\tan \alpha$ was confirmed (range $15^\circ$-$20^\circ$).
4. Using tests ZG1, ZG2, T1,4, T1,2 and T1,6, T1,5 it followed that the velocity is proportional to the vane length $L_0,4$. Measurements were used over the range $L/h = 0.67 - 1.33$.
5. Test $T_{1,14}$, $T_{1,4}$ and $T_{1,2}$, $T_{1,5}$ suggested the velocity to be proportional to the vane height $H_0,8$. In these measurements the vane height varied over the range $H/h = 0.17 - 0.33$.
6. It remains unexplained why the results from the Q series measurements indicate a much more weak relation to the vane height.
7. For reasons of dimension a factor $h^{-1,2}$ is added. Comparing T series and the ZG series measurements with different scales suggests the validity of this proportion.
8. A simultaneous fit was conducted for measurements from both all flat plate vanes and from all sheet pile shaped vanes. This resulted in the respective proportionality factors included in the resulting equations (13.8) and (13.9).

The complete enhanced equation for the near-bed velocities induced by a single vane now reads:

For flat plate vanes

$$v_{vbp, pl} = 2.3 \, U \tan \alpha \frac{H^{0.8} \, L^{0.4}}{h^{1.2}} \left[ 1 + \left( \frac{y_v - \mu}{0.45 \, H^{0.3} \, h^{0.5}} \right)^2 \right] \left[ 1 - \exp \left( - \frac{0.7 \lambda_v}{x_v} \right)^{1.5} \right]$$  \hfill (13.8)

For sheet pile vanes

$$v_{vbp, sp} = 2.8 \, U \tan \alpha \frac{H^{0.8} \, L^{0.4}}{h^{1.2}} \left[ 1 + \left( \frac{y_v - \mu}{0.45 \, H^{0.3} \, h^{0.5}} \right)^2 \right] \left[ 1 - \exp \left( - \frac{0.4 \lambda_v}{x_v} \right)^{1.0} \right]$$  \hfill (13.9)

With

$$\lambda_v = \frac{C_h}{\sqrt{8}} \frac{H}{h}$$

Valid for the ranges

<table>
<thead>
<tr>
<th>$H/h$</th>
<th>$0.17 - 0.33$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L/h$</td>
<td>$0.17 - 0.33$</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>$15^\circ$ - $20^\circ$</td>
</tr>
</tbody>
</table>

It is remarkable that the constant factor is highest for the case of sheet pile shaped vanes. This might be explained by the increased turbulence in and nearby this vortex as a result of the irregularly shaped vane. This turbulence can cause an increased mixing in the vortex, thus forcing water with a higher transverse velocity, from close to the vortex core, downwards towards to bed. The turbulence also increases the initial damping of the vortices.

### 13.4 Comparison Of Enhanced Equation With Scale Model Tests

The transverse near-bed velocities predicted by the enhanced equations (13.8) and (13.9) were tested versus the transverse near-bed velocities from scale model tests. Again this was done comparing the transverse flow per unit length $j_{vbn}$ in each measurement cross section and the total transverse near-bed flow generated by the vane $j_{vbn}$.

The results were included in Appendix 13A, the total transverse near-bed flows are summarized in Table 13.2.

Comparing this data to those for the Rivcom equation (Table 12.1), the results are disappointing.

1. A significantly better approximation of the measured damping in streamwise direction is achieved for the flat plate single vane situation.
2. The transverse flows measured in the Q series tests by Wang are seriously overestimated.
### Table 13.2

<table>
<thead>
<tr>
<th>Flat Plate Tests</th>
<th>$J_{\text{enh}}$ / $J_{\text{enh \ meas.}}$</th>
<th>Sheet Pile Tests</th>
<th>$J_{\text{enh}}$ / $J_{\text{enh \ meas.}}$</th>
<th>Array Tests</th>
<th>$J_{\text{enh}}$ / $J_{\text{enh \ meas.}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_{1,1}$</td>
<td>1.26</td>
<td>$T_{1,16}$</td>
<td>1.26</td>
<td>Vane $T_{1,14}$</td>
<td></td>
</tr>
<tr>
<td>$T_{1,2}$</td>
<td>0.83</td>
<td>$T_{1,17}$</td>
<td>1.21</td>
<td>spacing</td>
<td></td>
</tr>
<tr>
<td>$T_{1,4}$</td>
<td>0.96</td>
<td>$T_{1,18}$</td>
<td>0.90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$T_{1,5}$</td>
<td>0.85</td>
<td>$ZG_{1}$</td>
<td>1.76</td>
<td>$T_{1,2}$ 0.20 m</td>
<td>2.39</td>
</tr>
<tr>
<td>$T_{1,13}$</td>
<td>1.13</td>
<td>$ZG_{2}$</td>
<td>1.43</td>
<td>$T_{1,7}$ 0.30 m</td>
<td>1.97</td>
</tr>
<tr>
<td>$T_{1,14}$</td>
<td>1.01</td>
<td>$ZG_{3}$</td>
<td>1.58</td>
<td>$T_{1,3}$ 0.40 m</td>
<td>3.55</td>
</tr>
<tr>
<td>$T_{1,15}$</td>
<td>0.95</td>
<td>$ZG_{4}$</td>
<td>1.01</td>
<td>Vane $T_{1,4}$</td>
<td></td>
</tr>
<tr>
<td>$Q_{1}$</td>
<td>2.07</td>
<td>$Q_{2}$</td>
<td>2.45</td>
<td>spacing</td>
<td></td>
</tr>
<tr>
<td>$Q_{2}$</td>
<td>2.56</td>
<td>$T_{5,8}$ 0.30 m</td>
<td>2.68</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3. Unfortunately transverse flows in the case of sheet pile shaped vanes are significantly overestimated in most cases. Firstly it must be kept in mind that the tests $ZG_{3}$ and $ZG_{4}$ are outside the range of validity of the enhanced equations. Secondly in test $ZG_{1}$ the overestimation is partly due to negative near-bed velocities measured in the points close to the flume wall, as a result of counter-rotating vortices.

4. For the test with arrays of vanes the vane-induced transverse flow is overestimated significantly, which is not surprising considering earlier observations. Attention is focused on these tests in Paragraph 13.5.

### Recommended Further Analysis

The most important conclusion for this test is that the approach followed does not lead to a satisfying description of the near-bed velocities induced by vanes. Nevertheless the relations found in several parts of the previous analysis appear to be useful.

A further analysis is required and should be based, from the start, on the vane-induced transverse near-bed flow in the measurement cross-sections.

A description of the streamwise damping of this transverse flow will prove to be more useful than attempting to accurately describe the vane-induced near-bed velocities.

1. The number of phenomena involved is reduced, just an initially generated transverse near-bed flow per unit length $J_{\text{enh}}(x=0)$ and its damping need to be studied.

2. The vortex contains a certain amount of energy, damped in downstream direction. An inverse relation is therefore expected between the profile width and the near-bed peak velocity.

3. Averaging over the cross sections leads to a reduction of the influence of measuring errors.

4. Negative near-bed velocities due to counter-rotating vortices can be included in the vane-induced near-bed flow. Careful attention is required, as the measurements indicate that the presence of nearby vanes or a flume wall can lead to these negative velocities. This is also related to the transverse position at which measurements took place.

5. The assumed transverse velocity profile has relatively long "tails" in a lateral region where no measurements are available. 20% Of $J_{\text{enh}}$ is situated outside 3 $b_{\text{v}}$ from the peak velocity. Simply integrating this profile would lead to a significant overestimation of the vane impact. In this respect the transverse flows established in measurements should be used, while not extrapolating the transverse velocity profile.

6. The vane-induced transverse flow is what is actually being used in the Rivcom model.
13.5 Array Of Vanes

The positioning of vanes in an array can result in a significant reduction in the vane-induced transverse velocities. If the vane spacing is too small the initially generated vortices strongly hinder each other. On the other hand if the vane spacing is too great counter-rotating vortices occur, reducing the net morphological impact of the vanes.

The vane interaction theory presented by Wang [1991] was based on the vortex patterns surrounding a vane and is included in Appendix 11B. The same reduction factors would apply to the vane-induced transverse near-bed flow. In Table 13.3 the predicted reduction factors are compared to those obtained from the scale model test series Tn related to the corresponding single vane tests. In the measurements a much greater reduction of the near-bed flow occurs. This can be explained by the following phenomena:

1. If the vanes are close to each other greater velocity gradients occur near the cross sectional boundary of the vortices. Strongly increased turbulence penetrates the vortices, hindering the generation and increasing the damping downstream. Counter-rotating vortices develop between the main vane generated vortices if the vane spacing is too great. It results in a major reduction of the net vane-induced transverse near-bed flow. This was also observed in test T1,8 with a relatively small vane height.

<table>
<thead>
<tr>
<th>Test</th>
<th>Transverse spac.</th>
<th>Vane height</th>
<th>Wang Red.</th>
<th>Red. of J_{vbn}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>δ/L - δ/h</td>
<td>H/L - H/h</td>
<td>factor</td>
<td>measured</td>
</tr>
<tr>
<td>T_{1,6}</td>
<td>0.50 0.67</td>
<td>0.25 0.33</td>
<td>0.84</td>
<td>0.42</td>
</tr>
<tr>
<td>T_{1,7}</td>
<td>0.75 1.00</td>
<td>0.25 0.33</td>
<td>0.91</td>
<td>0.51</td>
</tr>
<tr>
<td>T_{1,9}</td>
<td>1.00 1.33</td>
<td>0.25 0.33</td>
<td>0.95</td>
<td>0.29</td>
</tr>
<tr>
<td>T_{1,8}</td>
<td>0.75 1.00</td>
<td>0.125 0.17</td>
<td>0.96</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Table 13.3 Reducion factors for initially generated vortex intensity and thus transverse near-bed flow, predicted by Wang [1991] versus the actual reduction of generated transverse flow J_{vbn} based on measurements.

Figure 13.8 gives an impression of the vane sizes and spacing occurring in the test series Tn, together with the presumed phenomena reducing the vane efficiency. Furthermore, the initial and downstream vortex size is plotted, based on the presumptions in Paragraph 13.2.

The transverse vane spacing, in relation to the vane height and the water depth, is a very important factor in the morphological impact of the vanes. It should therefore be included in the numerical models.

In Paragraph 12.5 it was indicated that the vane height will not be constant in prototype situation, as a result of the changing local bed level. In the scale model test series Tn only a limited number of combinations of vane spacing and vane height occur. Moreover, the occurring phenomena and thus the reduction factor strongly differ between these scale model tests. The currently available data is insufficient to produce an adequate description of a reduction factor in the transverse near-bed flow generated by a vane, based on the local vane spacing and vane height to water depth ratio. Reduction factors should therefore be chosen on the safe side.

Van Meervonden and Struijsma [1996] investigated the generated near-bed velocities by multiple arrays of vanes. They did not find an additional negative effect on the generated transverse near-bed velocities.
Factors In Enhanced Formula

In a first attempt to describe the reduction of the vane efficiency, it was investigated how the enhanced equation should be modified in order to fit the streamwise distribution of the transverse near-bed flow for the vane arrays.

For this purpose three constant reduction factors were inserted in the magnitude of the transverse velocity, the damping length and transverse width of the velocity profile. Optimal results were obtained with the reduction factors presented in Table 13.4. Note that the enhanced equation gave accurate results generally for the tests with similar single vanes (T_{1,14} and T_{1,4}).

The following observations were made:

1. In the tests T_{s5} and T_{s8}, in which counter-rotating vortices were present, the damping length had to be reduced using a factor of the order 0.25, in order to fit streamwise distribution of the transverse near-bed flow.

2. A reduction factor 0.6 is required in all cases for the initially generated transverse velocities.

<table>
<thead>
<tr>
<th>Vane Array Tests</th>
<th>Reduction factors for</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vane T_{1,14} spacing</td>
<td>v_{dep}(x=0)</td>
<td>b</td>
</tr>
<tr>
<td>T_{s2} 0.7 h</td>
<td>0.6</td>
<td>1.0</td>
</tr>
<tr>
<td>T_{s7} 1.0 h</td>
<td>0.7</td>
<td>1.0</td>
</tr>
<tr>
<td>T_{s3} 1.3 h</td>
<td>0.7</td>
<td>0.9</td>
</tr>
<tr>
<td>Vane T_{1,4} spacing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>T_{s8} 1.0 h</td>
<td>1.2</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Table 13.4 Reduction factors added to the enhanced equation (13.8)/(13.9) in the initial transverse near-bed peak velocity, the width of the transverse velocity distribution and the damping length. Tests T_{25}, T_{s5} and T_{s7} (H=0.10 m, L=0.40 m, \alpha=18^\circ) like T_{1,14}. Test T_{s8} (H=0.05 m, L=0.40 m, \alpha=18^\circ) like T_{1,4}.
13.6 Conclusions

A thorough analysis of the near-bed velocities downstream of a single vane did not lead to a satisfactory prediction formula based on the vane dimensions. This is due to the fact that a large number of phenomena are involved, such as the vertical position of the vortex core and vane induced turbulence. Furthermore, the different measuring heights in the physical model tests could play a role.

It is proposed to pursue a further analysis of these measurements based on the vane-induced transverse near-bed flow in every cross section. This reduces the number of phenomena considered and the influence of errors in the velocity measurements. Furthermore, the total morphological impact of a vane is directly related to this transverse flow.

Although the occurring phenomena can be understood, it is not easy to describe the reduced morphological efficiency of vanes placed in an array. In case counter-rotating vortices occur, the net morphological efficiency of the vanes strongly damps in downstream direction.
14. Conclusions Part 2
Analysis Of Model Description Of Submerged Vanes

Based on a comparison of the model description for the vane-induced transverse near-bed velocities the following conclusions were drawn:

Axi-Symmetric Model
2.1 The description of these transverse velocities in the axi-symmetric model developed by Wang [1991] is on the safe side for single vanes.
2.2 However, in the case of vanes placed in arrays the vane efficiency strongly reduces, much more than expected on the basis of the biplane interaction theory.
2.3 The calibration of the lateral bed slope in the axi-symmetric model should be based on an equilibrium cross-section for the Hulhuizen bend in the River Waal. The actual overshoot of the lateral bed slope is to be taken into account afterwards.

Rivcom Model
2.4 The description of vanes in Rivcom consists of two important components.
   a. Firstly the description of the vane-induced transverse near-bed velocity. The equation applied has been tested earlier using a large number of physical model tests.
   b. Secondly the relation between this velocity and a corresponding spiral flow intensity, directly determining the transverse bed slope. This part of the vane efficiency was calibrated using one mobile-bed test only.
2.5 The latter calibration is considered scarce and it is recommended to verify it using more than one mobile bed test with vanes.
2.6 The averaging of vane-induced transverse near-bed velocity over a Rivcom grid cell should be brought in accordance with the equation tested versus measurements. The formulations of the width of the transverse velocity profile do not correspond, which leads to differences in the resulting transverse near-bed flow.
2.7 The lateral spacing of vanes placed in an array has a critical influence on the net morphological impact. This is not taken into account in the Rivcom model.
2.8 In order to obtain a valid model description of the vanes two steps are required:
a. Calibration of the morphological impact of the vanes against mobile-bed scale model tests, as carried out by Flokstra and Van Zanten [1995]. This should preferably be done with the same type of vanes and vane-spacing as in the prototype situation. The current calibration of Rivcom is valid in this respect.
b. In case the vane size and spacing are to be varied, a formulation of the reduction of the vane-efficiency by the neighboring vanes should be introduced.

Analysis Of Scale Model Tests
2.9 An analysis of the measurements from physical model tests did not lead to a satisfactory description of the vane-induced transverse near-bed velocity.
2.10 It is recommended to pursue a further analysis based on the vane-induced transverse near-bed flow, as this is expected to be normative for the net morphological impact of the vanes.
2.11 Considering the fact that the occurring phenomena strongly vary with the transverse vane-spacing, a sufficiently accurate description of the reduction of the morphological impact of the vanes cannot be obtained from the physical tests so far.
2.12 In case counter-rotating vortices occur, the morphological effect of vanes is expected to be damped strongly in downstream direction.
2.13 Counter-rotating vortices are expected to occur in the preliminary vane field design for the Hulhuizen II bend in River Waal. Reducing the transverse vane-spacing would be beneficial.
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THE MODELLING OF SUBMERGED VANES
A Means Of Fairway Improvement In River Bends

Appendices With Part 1 And Part 2

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering of Delft University of Technology.

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Hydraulic and Geotechnical Engineering Division

F.E. Wiersma

Delft, March 1997
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<tr>
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<tr>
<td>6j</td>
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An important question is whether or not River Waal is aware of the conclusions drawn in this thesis.

*Based on prof. De Vries*
APPENDIX 2A    Basic Theory On Flow in River Bends

Lateral Distribution Of Streamwise Velocity
If the bed friction in a river bend is neglected over such a small stretch of river, the flow can be described by potential theory. As the velocity is now constant in a vertical column of water the bed level will be horizontal.

Using an axis system along a stream line (s, n, z axis and u, v, w velocities) the momentum equations can be written as:

\[ \frac{\partial u}{\partial t} + u\frac{\partial u}{\partial s} = -\frac{1}{\rho} \frac{\partial p}{\partial s} \]  \hspace{1cm} (A 1)

\[ \frac{\partial v}{\partial t} + u^2 \frac{1}{r} = -\frac{1}{\rho} \frac{\partial p}{\partial n} \]  \hspace{1cm} (A 2)

\[ \frac{\partial w}{\partial t} = -\frac{1}{\rho} \frac{\partial p}{\partial z} \]  \hspace{1cm} (A 3)

Concentrating on a steady flow situation, all derivatives in time are 0. Equation ( A 3 ) is integrated vertically, describing a hydrostatic pressure distribution.

\[ p = \rho g (h - z) \]  \hspace{1cm} (A 4)

For a constant level z equation ( A 2 ) can now be written in the following way:

\[ \frac{u^2}{r} = -\frac{1}{\rho} \frac{\partial}{\partial n} \rho g (h - z) = -g \frac{\partial h}{\partial n} \]  \hspace{1cm} (A 5)

Furthermore Bernoulli's law can be applied with one constant γ over the entire flow field.

\[ \frac{u^2}{2g} + h = \gamma \]  \hspace{1cm} (A 6)

Differentiating this equation in radial direction results in:

\[ \frac{u}{g} \frac{\partial u}{\partial t} + \frac{\partial u}{\partial r} = 0 \]  \hspace{1cm} (A 7)

Realizing that \( \frac{\partial}{\partial r} = -\frac{\partial}{\partial n} \), the equations ( A 5 ) and ( A 7 ) can be combined to read:

\[ \frac{u}{r} \frac{\partial u}{\partial r} + \frac{u^2}{r} = 0 \]  \hspace{1cm} or \( \frac{\partial u}{\partial r} = -\frac{u}{r} \)  \hspace{1cm} (A 8)

The solution for this differential equation describes the lateral distribution of the longitudinal velocity in a river bend:

\[ u = -\frac{c}{r} \]  \hspace{1cm} (A 9)

Measurements indicate that this is valid only in the most upstream part of a bend. As a result of bed friction, the velocity at the water surface is greater than the vertically averaged velocity, while near the river bed it is smaller. Therefore:

\[ \frac{u^2}{r} > -g \frac{\partial h}{\partial n} \]  near to the surface \hspace{1cm} (A 10)

\[ \frac{u^2}{r} < -g \frac{\partial h}{\partial n} \]  near to the bed \hspace{1cm} (A 11)

As a result the water near to the surface is heading slightly towards the outer bend, while the water flowing near to the river bed is diverted slightly towards the inner bend.
Vertical Distribution Of Transverse Velocities

Again using a coordinate system parallel to the a streamline and neglecting convective inertia the equations of motion for horizontal steady flow can be written as:

\[
\frac{\partial u}{\partial s} + \frac{1}{\rho} \frac{\partial \tau_{s\theta}}{\partial \theta} = 0 \tag{A 12}
\]

\[
\frac{u^2}{r} + \frac{\partial u}{\partial \theta} + \frac{1}{\rho} \frac{\partial \tau_{\theta\theta}}{\partial \theta} = 0 \tag{A 13}
\]

Equation (A 12) can be integrated vertically, resulting in a linear vertical distribution of the longitudinal shear stress:

\[
\tau_{s\theta} = \tau_{s\theta} \left(1 - \frac{z}{h}\right) = -\rho gh \frac{\partial u}{\partial s} \left(1 - \frac{z}{h}\right) \tag{A 14}
\]

According to mixing length theory, this shear stress can be related to gradients in the longitudinal velocity with:

\[
\tau_{s\theta} = \rho \ell^2 \frac{\partial u}{\partial s} \frac{\partial u}{\partial \theta} \tag{A 15}
\]

Adopting the power-law profile \( u = \frac{m+1}{m} \left( \frac{z}{h} \right)^{1/m} \bar{u} \) for the vertical distribution of the longitudinal velocity leads to the following expression for this mixing length:

\[
l = \kappa h \left( \frac{z}{h} \right)^{1-1/m} \sqrt{1 - \frac{z}{h}} \tag{A 16}
\]

Assuming heterogeneity this mixing length is used to relate the gradients in the transverse velocity to the transverse shear stress:

\[
\tau_{\theta\theta} = \rho^2 \frac{\partial v}{\partial \theta} \frac{\partial v}{\partial \theta} = \kappa h \left( \frac{z}{h} \right)^{2-2/m} \left(1 - \frac{z}{h}\right) \frac{\partial v}{\partial \theta} \frac{\partial v}{\partial \theta} \tag{A 17}
\]

Entering the power law profile and equation (A 17) in equation (A 11) and leads to an expression for the transverse velocity \( v \), with the lateral water surface slope \( \frac{\partial u}{\partial \theta} \) as an unknown value. However if the resulting equation is combined with the condition that there can be no net radial flow \( q = \int_0^h u \, dz = 0 \) the following expression results:

\[
\frac{\partial u}{\partial \theta} = \frac{m+1}{m^2(m+3)} \frac{\bar{u}^2}{gr} \tag{A 18}
\]

Now this extra equation lead to a description of the vertical distribution of the transverse velocity:

\[
\frac{rv}{hu} = \kappa \left[ \frac{m(m+1)^2}{m+3} \left( \frac{z}{h} \right)^{1/m} + \frac{m^2(m+1)}{m+2} \int_0^{1/z} \frac{\bar{u}^2}{1-z'^m} \, dz' \right] \tag{A 19}
\]

This theoretical profile corresponds reasonably well with measured profiles, as can be seen in Figure 2.1 in Paragraph 2.2.

The transverse component of the bottom shear stress results from combining equations (A 17) and (A 19):

\[
\tau_{br} = -\rho \frac{\bar{u}^2}{r} \left[ \frac{2(m+1)^2}{m^2(m+2)(m+3)} \right] \tag{A 20}
\]
APPENDIX 2B  Linear Model For Perturbations In River Bed

Smit[1989] presents an analytical model for steady state perturbations in the river bed, based on a linear analysis of the governing equations. Starting point are the steady state depth averaged equations of motion in an x, y, z coordinate system:

\[ \begin{align*}
&u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + g \frac{\partial}{\partial x} (h + z_b) + \frac{g}{C_s h} u \sqrt{u^2 + v^2} = 0 \\
&u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + g \frac{\partial}{\partial y} (h + z_b) + \frac{g}{C_s h} u \sqrt{u^2 + v^2} = 0 \\
&\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0
\end{align*} \]  

(A 21)

(A 22)

(A 23)

Furthermore the sediment transport is described with:

\[ \frac{\partial z_b}{\partial t} + \frac{\partial z_b}{\partial x} + \frac{\partial z_b}{\partial y} = 0 \]

(A 24)

The sediment transport in x and y direction is calculated from the effective sediment transport \( S_e \) and the heading \( \alpha \) of this sediment transport vector. This heading is determined by the direction \( \delta \) of the bed shear stress and the river bed slope:

\[ \begin{align*}
&\sin \delta = \frac{1}{f(\delta)} \frac{\partial z_b}{\partial y} \\
&\tan \alpha = \frac{\cos \delta - \frac{1}{f(\delta)} \frac{\partial z_b}{\partial x}}{\cos \delta - \frac{1}{f(\delta)} \frac{\partial z_b}{\partial x}}
\end{align*} \]

(A 25)

In which the Shields parameter is defined as \( \delta = \frac{u^2 + v^2}{C_s \Delta D_{50}} \), and an weighing function is used \( f(\delta) = \frac{0.85}{E} \sqrt{\delta} \), where E is a calibration coefficient.

The lateral near bed velocities induced by the spiral flow is taken into account in the heading of the bed shear stress vector via:

\[ \delta = \arctan \left( \frac{v}{u} \right) - \arctan \left( \frac{\Lambda h}{R_s} \right) \]

(A 26)

Where the effective local radius of a near bed stream line:

\[ R_s = \frac{h \sqrt{u^2 + v^2}}{I} \]

(A 27)

The spiral flow intensity is described by:

\[ \lambda_u \frac{\partial I}{\partial I} + I = \frac{h}{R} \sqrt{u^2 + v^2} \]

(A 28)

with a damping length \( \lambda_u = \frac{Ch}{\sqrt{g}} \). As this concerns the transverse bed shear stress a coefficient \( \beta = 0.6 \) is adopted in accordance with De Vriend[1981].

The above described system of equations is linearized and a double harmonic perturbation is introduced:

\[ h' = h \exp \left( k x + \frac{\pi}{B} y - \phi t \right) \]

(A 29)
The resulting equations deliver two important results. In the first place a length scale for the bed perturbations is found:

\[ \lambda_s = \frac{1}{\pi^2} \left( \frac{B}{h} \right)^2 \]  \hspace{1cm} (A 30)

Secondly a time scale for the movement of such a large scale river bed perturbation is obtained, which reads in strongly simplified form:

\[ T_s = -\frac{h \lambda_s}{s} \frac{1+\left( \frac{\lambda_w}{L} \frac{2\pi}{L} \right)^2}{1-\left( \frac{b-2}{2} \frac{\lambda_s}{\lambda_w} - 1 \right) \left( \frac{\lambda_w}{L} \frac{2\pi}{L} \right)^2} \]  \hspace{1cm} (A 31)

With the adaptation length of the main flow \( \lambda_w = \frac{C^2 h}{2g} \).
### APPENDIX 2C  Position of vanes

<table>
<thead>
<tr>
<th>Cross Section km</th>
<th>Number of Vanes</th>
<th>Angle of vane ° to North</th>
<th>Distance from centerline to inner vane m</th>
<th>Distance from centerline to outer vane m</th>
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Table A 1  Preliminary vane field design for bend Hulhuizen II in River Waal, as used in Rivcom model.

For a further specification of this vane field the reader is referred to van Meerdonk and Straaksma [1996].
APPENDIX 2D Calibration of Rivcom Model

Calibration Parameters

For the calibration of the model use was made of the bed level measurements from 1988, 1989, 1990 and 1991, which were averaged in order to eliminate bed forms. In general these soundings are very much alike, no large scale differences are visible, which could have been caused by discharge variations. The model was started from an equal water depth over the entire area and after a number of time steps equivalent to approximately 6 years an equilibrium bed level was reached. Initially no vanes were installed and calibration took place by varying the following parameters.

- $E_1$ factor in the sediment transport direction formula in Appendix 3A.
- $E_2$ factor in the sediment transport direction formula in Appendix 3A.
- Distribution in lateral direction of the velocity at the upstream model boundary.
- Bed level along the upstream boundary.

The influence of the submerged vanes was neither calibrated, nor compared to measurements.

Criteria for this calibration were especially the bed level in the centerline and 60m from the left and right bank, since this is the area that is determining the available width for navigation. In particular attention was paid to maximum lateral slope in the river bend and the position of the transition between the main bend at Hulhuizen and the slight bend near the upstream model boundary.

Resulting Values

The following parameter values resulted from the calibration:

1. $E_1 = 1.33$
2. $E_2 = 1.0$
3. The distribution of the velocity at the upstream boundary is assumed to be linear with a maximum of $U_{max} = 1.1 U_{average}$ on the left bank side and a minimum of $U_{min} = 0.9 U_{average}$ on the right bank side.
4. The bed level at the upstream boundary is also assumed to be a straight line from $z_b_{minimum} = 0.34 \text{ m+NAP}$ on the left bank side and $z_b_{maximum} = -0.70 \text{ m+NAP}$ on the right bank side.

The velocity field calculation is repeated every 4 morphological time steps of $\Delta t = 43200 \text{ s}$.
Figure A: Equilibrium bed level from X-recording station without values (1600 m/s).

Equilibrium Bed Level Without Values

Appendix 2E

Part I Appendix 2E Equilibrium Bed Level Without Values
Figure A 9  Equilibrium bed level from Rivcom output, situation with vanes 1600 m$^3$/s.
Figure A.7  Discharge-discharge curve for model upstream boundary km 870.550, 1990.
Figure A 8  Discharge duration curve for mean daily discharge in River Waal near Hulhuizen.

Figure A 9  Probability density function of mean daily discharges in River Waal near Hulhuizen.
APPENDIX 3E  Sediment Transport Statistics

Figure A 10  Duration curve for daily sediment transport in River Waal near Hulhuizen.

Figure A 11  Probability density function of sediment transport in River Waal near Hulhuizen.
## APPENDIX 4A  Rivcom Input Files To Be Changed

<table>
<thead>
<tr>
<th>Input File</th>
<th>Parameters to be changed</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOTMIN.DAT</td>
<td>Time step for morphological calculation</td>
<td>The flow velocity varies and therefore the time steps should vary to in order to ensure stability of the calculation.</td>
</tr>
<tr>
<td>NDYNAMcase.INP</td>
<td>Inflow velocities over cross section at upstream boundary</td>
<td>A varying discharge brings about varying inflow velocities at the upstream boundary.</td>
</tr>
<tr>
<td>NCASE1case.INP</td>
<td>Time step for morphological calculation , Chézy coefficient</td>
<td>In general the Chézy coefficient varies with the river discharge due to difference in bed forms.</td>
</tr>
<tr>
<td></td>
<td>Number of time steps</td>
<td>Run steps have different lengths in time.</td>
</tr>
<tr>
<td></td>
<td>Output specification</td>
<td>Time steps at which output is to be saved.</td>
</tr>
<tr>
<td>GRIDFLcase.label.INP</td>
<td>Initial water depth for the entire modelled area.</td>
<td>Obtained from the end result of the former run step.</td>
</tr>
<tr>
<td>RIVCOM.DAT</td>
<td>Distance below the water level of the top of the vanes</td>
<td>Together with the discharge the water level will rise or sink.</td>
</tr>
</tbody>
</table>
Figure A12  Main flowchart for a Rivcom run with dynamic boundary conditions.
# APPENDIX 4C  List Of Files And Functions

## File | Task
--- | ---

### Script Files
- chnbod.scr: Put initial water depths on calculation grid.
- chndep.scr: Obtains water depth file from output former run step, write in wd.dat file.
- copyout.scr: Renames the RIVCOM output files with a label for a particular runstep.
- coretest.scr: Checks if a calculation failed and a coredump took place.
- crenef.scr: Creates new NEFIS output file for current run step.
- gti.scr: Initializes calculation grid.
- qstatchk.scr: Help file to start qstatchk.exe.
- qwait.scr: Waits for end of former command in queue.
- rivcom.scr: Starts actual morphological calculation.
- run[lab].scr: Main script file that starts each run step.
- runstep.scr: Commands for each next run step of the run.
- 1ststep.scr: Commands for the first run step of the run.

### Programs
- bminmod.exe: Reads relevant data for this run step from RUNSTEP.DAT file and writes in BOTMIN.DAT input file.
- ndynmod.exe: Reads relevant data for this run step from RUNSTEP.DAT file and writes in NDYNAM[case].DAT input file.
- ncasemod.exe: Reads relevant data for this run step from RUNSTEP.DAT file and writes in NCASE1[case].DAT input file.
- qstatchk.exe: Reads qstat.txt file to check if submitted program is still in queue.
- rcommod.exe: Reads relevant data for this run step from RUNSTEP.DAT file and writes in RIVCOM.DAT input file.
- rsscreen.exe: Reads label for this run step from RUNSTEP.DAT file and prints on screen.
- scrmod.exe: Reads label for former and current run step from RUNSTEP.DAT file and replaces former labels in all relevant script files.
- wdout.in: Reads water depths from end former runstep in wd.dat file, reads water levels from RUNSTEP.DAT file and writes new water depths in GRIDFL[case][lab].INP file.

### Other Files
- qempty: Dummy file generated when submitted program is no longer in queue.
- qstat.txt: Contains the queue status, to be read by qstatchk.exe program.
- runstep.dat: Contains the data for every run step to be written into relevant RIVCOM input files.
- shwd.dat: Contains possible shadow water depths.
- wd.dat: Contains water depth over all grid points, obtained from NEFIS output file.
START of runstep.scr

Print label of runstep on screen

Modify script files for current run step

Extract water depths out of NEFIS output file from former runstep

Wait until chnnef has ended

rscreen.exd reads label from runstep.dat

sermod.exd reads labels from runstep.dat and writes new label to crenef.scr chndept.scr grl.scr chnbot.scr rivcom.scr copyout.scr

chndept.scr starts haklib chnnef with USPLUSdept.f subroutine, wd.dat file is created

qwait.scr waits in loop

sleep 10 s

qstatchk.exd checks if chnnef in queue

yes

bmimod.exd reads time step data from runstep.dat and writes to BOTMIN.DAT

ndynmod.exd reads inflow velocity data from runstep.dat and writes to NDYNAM[label].INP

ncasemod.exd reads various data from runstep.dat and writes to NCASE1[label].INP

wdoutin.exd reads old water depts from wd.dat and water levels from runstep.dat and writes new water depths in GRIDFL[label][case].INP

recommod.exd reads water levels from runstep.dat and writes relative vane height to RIVCOM.DAT

crenef.scr starts haklib crenef
NEFIS file is created

qwait.scr waits in loop

sleep 10 s

qstatchk.exd checks if crenef is in queue

yes

gri.scr starts haklib grini
input file GRIDFL[case][label].INP is inserted into NEFIS file

qwait.scr waits in loop

sleep 10 s

qstatchk.exd checks if griini is in queue

yes

CONTINUED ON NEXT PAGE
APPENDIX 4D Checklist For Starting A Dynamic Calculation

1. Check subdirectory structure
   NEFDAT - RIVCOM<case><label> present
   NEFDEF - RIVCOM<case><label> present
   GRIDFL - empty
   OUT - empty
   stdout - empty
   stderr - empty
   err - empty

2. Check contents of directory
   PRESENT
   executables:
   bminmod.exd
   ncaemod.exd
   ndynmod.exd
   qtestchk.exd
   reconstmod.exd
   rscreen.exd
   rdouin.exd
   script files:
   chnbsrscr 22 bytes
   chndelprscr 73
   copyout.scr 1537
   crenef.scr 45
   firstrun.scr 551
   gr.out 194
   qtestchk.scr 31
   qwait.scr 166
   rvecom.scr 66
   run<case>.scr
   runstep.scr 1892
   chnbsr
   rvecom
   input files:
   BOUNDY<case>.INP
   GRIDFL<case><label>1.INP
   NCASE<case>.INP
   NDFIN<case>.INP
   ULYSSEX1A.LINP
   BOTMIN.DAT
   RIVCOM.DAT
   TRANSFPH.DAT
   USBSUP files:
   USP SUBbed.f
   USPSUBbod.f
   USPSUBboco.f
   USPSUBbdept.f
   USPSUBbwpr.f
   USPSUBbwv.f
   USPSUBwgs.f
   USPSUBwce.f
   USPSUBwta.f
   USPSUBWspa.f
   USP SUBLIST

3. Create run<case>.scr file
   firstrun.scr
   runstep.scr

4. Check runstep.scr file
   Waiting cycle after each HAKLIB task

5. Create runstep.dat file
   1. Mark 1 for first runstep record.
   2. Mark line empty for all other runstep records.
   3. Correct labels and discharge rates.
6. Adapt input file for first run step

7. Adapt script files for first run step

```
chnbod.scr  haklib chnref $1 <<++++ 2>error
          RIVCOM
          WBHHH
          _A01
          <label1>
          bod
          0
          4
          217
          218
          2118
          2124
          e
          ++++

chndept.scr  haklib chnref $1 <<++++ 2>error
          RIVCOM
          WBHHH
          ____________
          dept
          0
          1
          0 0 0
          e
          ++++

copyout.scr  22 times <label1>

crenf.scr  haklib crenf $1 <<++++ 2>error
          y
          RIVCOM
          WBHHH
          _A01
          e
          0
          1
          0
          1
          1
          y
          2
          192 50
          n
          23 lines with c
          1
          1
          2
          192 50
          y
          ++++
```
if test -s "GRIDFLWBHH____.INP"
  then mv GRIDFLWBHH____.INP GRIDFL/.
fi
cp GRIDFLWBHH_A01.INP GRIDFLWBHH.INP
cp GRIDFLWBHH_A01.INP GRIDFL/.
haklib print $1 <<++++ 2>error

RIVCOM
WBHH
_A01
++++

rivcom.scr
haklib rivcom $1 <<++++ 2>error

WBHH
_A01
n
nh

n

y
fw
++++

8. Check contents of GRIDFL<case><labell>.INP

9. Start run

1. SIXTEEN block correct water depths for first run step.
2. cp GRIDFL<case><labell>.INF GRIDFL<case>.INF

1. Check that no own request remains in queue.
2. Start run so that results both on screen and in log file.
   run_A.scr | tee run_A.log
APPENDIX 4E  Checklist For Checking A Dynamic Calculation

1. Check queues
   No own request in queues any more

2. Check error files
   1. Check *.stderr files for all run steps and tasks.
   2. Check *.stdout files for all run steps and tasks.
   3. Check that no *.err files from all runsteps present.

3. Check contents directory
   Files ABSENT
   *NEW*
   *new*
   *.stderr
   *.stdout
   *.OUT
   wd.dat
   qerrmpy
   core
   tmp*
   scratch*

4. Check contents directories
   Files PRESENT for all runstep
   GRIDPL
   NEFDEF
   NEFDAT
   OUT

5. Check contents run<label>.log file
   No irregularities

6. Check numerical convergence
   Check in RIVCOM<case>><label>.OUT file from all runsteps:
   1. Number of iterations for velocity an spiral flow intensity fields.
   2. Check criteria for ending iteration after first 2 velocity field calculations.
APPENDIX 4F  
Listing Of Script Files

chnbod.scr

```bash
haklib chnbf $1 <++++> 2>error
RIVCOM
WHHH
_005
bd
0
4
2 1 7
2 1 8
2 1 9
2 1 24
e
++++
```

chndept.scr

```bash
haklib chnbf $1 <++++> 2>error
RIVCOM
WHHH
___
dep
0
1
0 0 0
e
++++
```

copyout.scr

```bash
echo "Moving chnbf dept *.OUT, *.stdout, *.stderr files with label to dirs."
mv CHNBFOUT.OUT OUT/CHNBFOUT_G05.OUT
mv CHNBFOUT.stdout stdout/CHNBFOUT_G05.stdout
mv CHNBFOUT.stderr stderr/CHNBFOUT_G05.stderr
echo "Moving crenef *.OUT, *.stdout, *.stderr files with label to dirs."
mv CRENEFOUT.OUT/CRENEFOUT_G05.OUT
mv CRENEF.stdout stdout/CRENEF_G05.stdout
mv CRENEF.stderr stderr/CRENEF_G05.stderr
echo "Moving grnini *.OUT, *.stdout, *.stderr files with label to dirs."
mv GRNINOUT.OUT/GRNINOUT_G05.OUT
mv GRNINI.stdout stdout/GRNINI_G05.stdout
mv GRNINI.stderr stderr/GRNINI_G05.stderr
echo "Moving chnbfout *.OUT, *.stdout, *.stderr files with label to dirs."
mv CHNBFOUTOUT.OUT/CHNBFOUTOUT_G05.OUT
mv chnbfout.stdout stdout/chnbfout_G05.stdout
mv chnbfout.stderr stderr/chnbfout_G05.stderr
echo "Moving rivcom *.OUT, *.stdout, *.stderr files with label to dirs."
mv RIVCOMOUT.OUT/OUT/RIVCOMOUT_G05.OUT
mv rivcom.stdout stdout/rivcom_G05.stdout
mv rivcom.stderr stderr/rivcom_G05.stderr
echo "Moving possible *.src files to directory."
if test -s "rsvscreen.err"
then mv cscscreen.err err/cscscreen_G05.err
fi
if test -s "scrmmod.err"
then mv scrmmod.err err/scrmmod_G05.err
fi
if test -s "minmod.err"
then mv minmod.err err/minmod_G05.err
fi
if test -s "ndymod.err"
then mv ndymod.err err/ndymod_G05.err
fi
if test -s "ncasmod.err"
then mv ncasmod.err err/ncasmod_G05.err
fi
if test -s "wdsout.err"
then mv wdsout.err err/wdsout_G05.err
fi
if test -s "rcasmod.err"
then mv rncasmod.err err/rcasmod_G05.err
fi
if test -s "scrmmod.err"
then mv scrmmod.err err/scrmmod_G05.err
fi
fi
```

coretest.scr

```bash
while test -s "core"
do
echo "ERROR! An executable was terminated prematurely, coredump took place."
echo "PROCESS PERMANENTLY ASLEEP, ABORT RUN, PRESS <Ctrl>+<c>!"
sleep 604800
done
```

crenef.scr

```bash
haklib crenef $1 <++++> 2>error
y
RIVCOM
WHHH
_005
e
0
1
0
1
1
Y
2
192 50
n
c
```

A 26
gri.scr
if test -s "GRIDFLOWNH.INF"
  then mv GRIDFLOWNH.INF GRIDFLW/.
  fi
  cp GRIDFLOWNH.005 INF GRIDFLOWNH.INF
  cp GRIDFLOWNH.005,INF GRIDFLW/.
  haklib grini $1 <<<<< 2>&error
  RIVCOM
  WSHS
  _005
  >>>>
qstatck.scr
  qstatck.csk <<<<<
  $1
  $2
  >>>>
qwait.scr
  echo "Waiting for all dutrex queues to be empty..."
  until test -z "qempty"
  do
    sleep $3
    qstat -a >qstatus.txt
    qstatck.scr $1 $2
    done
  rm qstatus.txt
  rm qempty
rivcom.scr
  haklib rivcom $1 <<<<< 2>&error
  WSHS
  _005
  n
  n
  sh
  n
  n
  y
  fw
  >>>>
run([label]).scr
runstep.scr
runstep.scr
runstep.scr

runstep.scr

echo "Begin of new runstep."
date
case runstepnew.dat runstep.dat
echo "Modify script files for current runstep."
screen.mod
echo "Modify BOTMIN.DAT for current runstep."


1ststep.scr

echo "Begin of new run."
date
echo ""
echo ""
echo ""
echo ""
echo ""
echo ""
echo ""
echo ""
echo ""


echo ""
echo ""
echo ""
echo ""
echo ""

echo "First runstep of run! ="

echo ""
echo "Create Mesh file for current runstep."
cranef.scr 0
qwait.scr wwkfen cranef 10
echo "Insert new GRIDFLW slab.INP in Mesh file for current runstep."
gpi.scr 0
qwait.scr wwkfen grini 10
echo "Modify new Mesh file concerning 2 1 with 7 water dep, 8 bottom, 10 z-nonaxod and 24 beta."
cnhbod.scr 0
qwait.scr wwkfen chnhf 10
mv CRNNEP.OUT CRNNEPbod.OUT
mv chnhf.stdout chnhbod.stdout
mv chnhf.stderr chnhbod.stderr
echo "Start RIVCOM calculation for current runstep."
rivcom.scr 51
qwait.scr wwkfen rivcom 120
echo "Copy output files with current label to relevant directories."
copyout.scr
echo "Check if coredump took place."
coretest.scr
### APPENDIX 4G  Listing Of Runstep.Dat File

<table>
<thead>
<tr>
<th>Line</th>
<th>Description</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>BOTMIN.DAT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>NDYNAWBHH.INP</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>discharge m3/s</td>
<td>3380.000</td>
<td>10800.000</td>
</tr>
<tr>
<td>5</td>
<td>time step s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>upstream</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>u v m/s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>-1.214</td>
<td>0.527</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>-1.220</td>
<td>0.530</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>-1.226</td>
<td>0.532</td>
</tr>
<tr>
<td>11</td>
<td></td>
<td>-1.231</td>
<td>0.534</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>-1.237</td>
<td>0.537</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td>-1.242</td>
<td>0.539</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td>-1.248</td>
<td>0.542</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>-1.254</td>
<td>0.544</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>-1.259</td>
<td>0.547</td>
</tr>
<tr>
<td>17</td>
<td></td>
<td>-1.265</td>
<td>0.549</td>
</tr>
<tr>
<td>18</td>
<td></td>
<td>-1.270</td>
<td>0.551</td>
</tr>
<tr>
<td>19</td>
<td></td>
<td>-1.276</td>
<td>0.554</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td>-1.282</td>
<td>0.556</td>
</tr>
<tr>
<td>21</td>
<td></td>
<td>-1.287</td>
<td>0.559</td>
</tr>
<tr>
<td>22</td>
<td></td>
<td>-1.293</td>
<td>0.561</td>
</tr>
<tr>
<td>23</td>
<td></td>
<td>-1.298</td>
<td>0.564</td>
</tr>
<tr>
<td>24</td>
<td></td>
<td>-1.304</td>
<td>0.566</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>-1.310</td>
<td>0.568</td>
</tr>
<tr>
<td>26</td>
<td></td>
<td>-1.315</td>
<td>0.571</td>
</tr>
<tr>
<td>27</td>
<td></td>
<td>-1.321</td>
<td>0.573</td>
</tr>
<tr>
<td>28</td>
<td></td>
<td>-1.326</td>
<td>0.576</td>
</tr>
<tr>
<td>29</td>
<td></td>
<td>-1.332</td>
<td>0.578</td>
</tr>
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<td></td>
<td>-1.338</td>
<td>0.581</td>
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<td></td>
<td>-1.360</td>
<td>0.590</td>
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<td></td>
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<td>0.595</td>
</tr>
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<td></td>
<td>-1.377</td>
<td>0.598</td>
</tr>
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<td></td>
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<tr>
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<td>0.612</td>
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<td>-1.416</td>
<td>0.615</td>
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<td></td>
<td>-1.421</td>
<td>0.617</td>
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<td>46</td>
<td></td>
<td>-1.427</td>
<td>0.619</td>
</tr>
<tr>
<td>47</td>
<td>NCASE1WBHH.INP</td>
<td></td>
<td></td>
</tr>
<tr>
<td>48</td>
<td>time step s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>49</td>
<td>number of time steps</td>
<td>3000</td>
<td>47000</td>
</tr>
<tr>
<td>50</td>
<td>chezy coef ml/2s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>51</td>
<td>maptim start tstep</td>
<td></td>
<td>12</td>
</tr>
<tr>
<td>52</td>
<td>maptim increment</td>
<td></td>
<td>249</td>
</tr>
<tr>
<td>53</td>
<td>GRIDFLWBHH_G01.INP</td>
<td></td>
<td></td>
</tr>
<tr>
<td>54</td>
<td>and RIVCOM.DAT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td>max num od iteratie</td>
<td></td>
<td>40</td>
</tr>
<tr>
<td>56</td>
<td></td>
<td>10800.000</td>
<td></td>
</tr>
<tr>
<td>57</td>
<td>G02</td>
<td></td>
<td></td>
</tr>
<tr>
<td>58</td>
<td>BOTMIN.DAT</td>
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<td>59</td>
<td>NDYNAWBHH.INP</td>
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<tr>
<td>60</td>
<td>discharge m3/s</td>
<td>3380.000</td>
<td>10800.000</td>
</tr>
<tr>
<td>61</td>
<td>time step s</td>
<td></td>
<td></td>
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<td>62</td>
<td>upstream</td>
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<td>63</td>
<td>u v m/s</td>
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<td>0.564</td>
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<td>-1.321</td>
<td>0.573</td>
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<td>0.576</td>
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<td>-1.332</td>
<td>0.578</td>
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<td></td>
<td></td>
<td>-1.343</td>
<td>0.583</td>
</tr>
<tr>
<td></td>
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APPENDIX 4H  Listing Of Fortran Program Bminmod.F

PROGRAM Bminmod

CHARACTER*80 ratepline,bmline
CHARACTER*1 mark
REAL timestapel
INTEGER runstep,runstepnew,botmin,botminnew,error
INTEGER i,ilog,1,1

runstep = 22
runstepnew = 33
botmin = 44
botminnew = 55
error = 66

runstep file lezen tot en met eerste record zonder mark 2

OPEN (runstep, FILE='runstep.dat', STATUS = 'old')
OPEN (runstepnew, FILE='runstepnew.dat', STATUS = 'new')

luz voor iedere runstep record

READ (runstep,'(A)',END=200) ratepline
l=ilog(ratepline,80)
IF (ratepline(1:6).eq.'133456') THEN
WRITE (runstepnew, '(A)') ratepline(1:1)
mark=2
ELSEIF (ratepline(1:6).eq.'1') THEN
WRITE (runstepnew, '(A)') ratepline(1:1)//'2'
mark=1
ELSE
GOTO 100
ENDIF

runstep record doorlopen en kopieren tot relevante regel

READ (runstep,'(A)',END=200) ratepline
l=ilog(ratepline,80)
IF (ratepline(1:6).eq.'BOTMIN') THEN
1 = ilog(ratepline,80)
WRITE (runstepnew, '(A)') ratepline(1:1)
GOTO 30
ENDIF

inlezen nieuwe tijdstap uit betreffende runstep record

READ (ratepline(1:52),'(F12.3)') timestapel
kopiëren laatste regel van runstep record

DO 45 j=1,60
READ (runstep,'(A)',END=200) ratepline
l = ilog(ratepline,80)
WRITE (runstepnew, '(A)') ratepline(1:1)
45 CONTINUE

rasterende deel runstep file kopieren

READ (runstep,'(A)',END=60) ratepline
1 = ilog(ratepline,80)
WRITE (runstepnew, '(A)') ratepline(1:1)
GOTO 55

afsluiten runstep files

CLOSE (runstep)
CLOSE (runstepnew)

bewerken botmin.dat file

WRITE ('(A)') 'Modifying BOTMIN.DAT -> BOTMINNEW.DAT'
OPEN (botmin, FILE='BOTMIN.DAT', STATUS='old')
OPEN (botminnew, FILE='BOTMINNEW.DAT', STATUS='new')

doornemen en kopieren botmin.dat tot aan relevante regel

READ (botmin,'(A)',END=200) bmline
1 = ilog(bmline,80)
WRITE (botminnew, '(A)') bmline(1:1)
IF (bmline(2:12).ne.'TIDESTAMP IN SECONDEN') GOTO 60
READ (botmin,'(A)',END=200) bmline
WRITE (botminnew, '(A,10,F15.6)') ' ',timestapel
WRITE (botminnew, '(A)') ' ' ,timestapel
Sparen restant van botmin.dat file

READ (botmin,'(A)',END=80) bmline
1 = ilog(bmline,80)
WRITE (botminnew, '(A)') bmline(1:1)
GOTO 70

afsluiten programma

CLOSE (botmin)
CLOSE (botminnew)

OPEN (error, FILE='bminmod.err', STATUS = 'new')
WRITE (error, '(A)') 'ERROR! RUSTEP.DAT file was not used // & in right order.'
STOP

OPEN (error, FILE='bminmod.err', STATUS = 'new')
WRITE (error, '(A)') 'ERROR! Unexpected end of input file.'
STOP

END
APPENDIX 4I  Listing Of Fortran Program Ndynmod.F

PROGRAM ndynmod
CHARACTER*80 rateplane, ndynline, ndynlinenew
CHARACTER*1 mark
REAL u(3:49), v(3:49)
INTEGER runstep, runstepnew, ndynam, ndynamnew, error
INTEGER itlog, li
runstep = 22
runstepnew = 33
ndynam = 44
ndynamnew = 55
error = 66

c runstep file lezen tot en met eerste record zonder mark 3
 c kopieren naar vervangende file runstepnew
 OPEN (runstep, FILE='runstep.dat', STATUS = 'old')
 OPEN (runstepnew, FILE='runstepnew.dat', STATUS = 'new')
 c lees voor telkens een runstep
 c check en bijwakten markering LI=3
 READ (runstep, '(A, I)', END=200) rateplane
 IF (rateplane(1:6), eq., '123456') THEN
 WRITE (runstepnew, ' (A)') rateplane(1:11)
 mark = '3'
 ELSEIF (rateplane(1:6), eq., '12') THEN
 WRITE (runstepnew, ' (A)') rateplane(1:11)// '3'
 mark = '1'
 ELSE
 GOTO 100
 ENDIF

 c runstep record doornemen en kopieren tot relevante regel
 READ (runstep, '(A, I)', END=200) rateplane
 IF (rateplane(1:11), eq., 'NDYNAM') THEN
 li = ilog(rateplane, 80)
 WRITE (runstepnew, ' (A)') rateplane(1:11)
 GOTO 35
 ENDIF

 li = ilog(rateplane, 80)
 WRITE (runstepnew, ' (A)') rateplane(1:11)

 c lezen van ieder volgende regel u en v uit runstep record
 READ(rateplane(4:52), '(F12.3)', u(3))
 READ(rateplane(53:64), '(F12.3)', v(3))
 DO 40 j=4, 49
 li = ilog(rateplane, 80)
 READ(runstep, '(A, I)', END=200) rateplane
 READ(rateplane(4:52), '(F12.3)', u(3))
 READ(rateplane(53:64), '(F12.3)', v(3))
 WRITE (runstepnew, ' (A)') rateplane(1:11)
 40 CONTINUE

 c doornemen en kopieren laatste regel betreffende runstep record
 DO 45 j=51, 69
 READ (runstep, '(A, I)', END=200) rateplane
 li = ilog(rateplane, 80)
 WRITE (runstepnew, ' (A)') rateplane(1:11)
 45 CONTINUE

 IF (mark, eq., '3') GOTO 30
 c resterende deel runstep file kopieren
 READ (runstep, '(A, I)', END=40) rateplane
 li = ilog(rateplane, 80)
 WRITE (runstepnew, ' (A)') rateplane(1:11)
 GOTO 55

 c afsluiten runstep files
 CLOSE (runstep)
 CLOSE (runstepnew)

 c bewerken NDYNAMNEW. INF file
 WRITE (*, ' (A)') 'Modifying NDYNAMNEW. INF -> NDYNAMNEWNEW. INF'
 OPEN (ndynam, FILE='NDYNAMNEW. INF', STATUS='old')
 OPEN (ndynamnew, FILE='NDYNAMNEWNEW. INF', STATUS='new')

 c doornemen en kopieren ndynam file tot aan relevante regel
 READ (ndynam, '(A, I)', END=200) ndynline
 li = ilog(ndynline, 80)
 WRITE (ndynamnew, '(A)') ndynline(1:11)
 IF (ndynline(57:72), ne., '2') GOTO 65

 c inschrijven nieuwe snelheden in ndynam file
 DO 70 j=1, 49
 li = ilog(ndynline, 80)
 READ (ndynam, '(A, I)', END=200) ndynline
 ndynlinenew(1:li)=ndynline(1:li)
 WRITE(ndynlinenew(12:19), '(F12.3)') u(11)
 ndynlinenew(19:19)= '0'
 WRITE(ndynlinenew(20:26), '(F7.3)') v(11)
 ndynlinenew(27:27)=ndynline(27:27)
 WRITE (ndynamnew, '(A)') ndynlinenew(1:11)
 70 CONTINUE

 c kopieren rest van ndynam file
 READ (ndynam, '(A, I)', END=80) ndynline
 li = ilog(ndynline, 80)
 WRITE (ndynamnew, '(A)') ndynline(1:11)
 GOTO 75

 c afsluiten programma
 CLOSE(ndynam)
 CLOSE(ndynamnew)
 GOTO 500

 OPEN (error, FILE='ndynmod.erc', STATUS='new')
 WRITE (error, '(A)') 'ERROR! RUNSTEP. DAT file was not used' //
 ' in right order.'
 STOP
200 OPEN (error, FILE='ndynmod.err', STATUS='new')
WRITE (error,'(A)') 'ERROR! Unexpected end of input file.'
STOP

500 END

WRITE (ncasenew, '(A)') ncaseline(4:1)
GOTO 68
ENDIF
1 = ilog(ncaseline,80)
WRITE (ncasefline, '(a)') ncaseline(5:1)
WRITE (ncasenew, '(A)') ncaseline(6:1)
WRITE (ncasenew, '(A)') ncaseline(7:1)
WRITE (ncasenew, '(A)') ncaseline(8:1)
WRITE (ncasenew, '(A)') ncaseline(9:1)
WRITE (ncasenew, '(A)') ncaseline(10:1)
READ incase, '(A)', END=200) ncaseline
1 = ilog(ncaseline,80)
WRITE (ncasenew, '(A)') ncaseline(11:1)
IF (ncaseline(11:1).ne.'----- OUTPUT') GOTO 70
checken of tijdstappen voor mptim file beginnen
IF (chkstep*MOD(numberts,chkstep),mptimincr) GOTO 171
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APPENDIX 4K  Listing Of Fortran Program Qstatchk.F

PROGRAM qstatchk
  CHARACTER*80 reqname, user
  INTEGER qstatfile, qempty
  INTEGER itlog, i, luser, i, l, i2
  qstatfile = 11
  qempty = 22
  READ (*, '(A)') user
  READ (*, '(A)') reqname
  luser = itlog(user, 80)
  i = itlog(reqname, 80)
  c beïjken file qstata txt waarin reply van qst 'a commando
  c is geplaatst
  OPEN (qstatfile, FILE='qstata.txt', STATUS='OLD')
  c regel voor regel file doornemen
  10 READ (qstatfile, '(A)'), END=80) qstatline
  c checken ieders positie op regel voor user name
  1 luser, i = 1
  DO 20 i, l = 1, luser
    IF (luser, i) THEN
      IF (qstatline(luser, i) eq ' /user(/luser/)' ' ') GOTO 30
    ELSE
      IF (qstatline(luser, i) eq 'user(/luser/)' ' ') GOTO 30
    ENDIF
  20 CONTINUE
  c user name is gevonden in deze regel, nu checken ieders
  c positie op regel voor request name
  30 DO 40 j = 1, luser
    IF (j, i2 = 1) THEN
      IF (qstatline(juser, i2) eq ' /reqname(/luser/)' ' ') GOTO 50
    ELSE
      IF (qstatline(juser, i2) eq 'reqname(/luser/)' ' ') GOTO 50
    ENDIF
  40 CONTINUE
  GOTO 10
  c user en request name komen voor in qstata_txt file
  c er wordt dus geen markeringsfile qempty aangemaakt
  50 WRITE (*, '(A)') 'REQUEST / reqname(/luser/)' ' of USER '/'
  4 /user(/luser/)' ' is still in nxtree queue.'
  GOTO 500
  c blijkaar staat deze request en user namen in qstata_txt
  c mask dan empty file aan
  80 WRITE (*, '(A)') 'REQUEST / reqname(/luser/)' ' of USER '/'
  4 /user(/luser/)' ' is no longer in nxtree queue.'
  OPEN (qempty, FILE='qempty', STATUS='NEW')
  WRITE (qempty, '(A)') 'Request of user is no longer in queue.'
  CLOSE (qempty)
  GOTO 500

500 END
APPENDIX 4L  Listing Of Fortran Program Rcomod.F

PROGRAM rcomod
CHARACTER*80 rspanline,rcoline
CHARACTER*1 mack
REAL hold, bnew, afstandv, afstandvnew
INTEGER runstep, runstepnew, rivcom, rivcomnew, error
INTEGER iktstod, it*log, i, 1
runstep = 22
runstepnew = 33
rivcom = 46
rivcomnew = 55
error = 66
hnew = 999999

RUNFILE lesen met eerste record zonder mark 6
kopieren runstep naar vervangende file runstepnew
OPEN (runstep, FILE='runstep.dat', STATUS='OLD')
OPEN (runstepnew, FILE='runstepnew.dat', STATUS='NEW')
for alle runstep record
check en bijwerken markering 12345+6
READ (runstep, 'A', END=200) rspanline
  = it*log(rspanline, 80)
  IF (rspanline(1:6).EQ. '12345 ') THEN
    WRITE (runstepnew, 'A') rspanline(1:i)
    mark = 6
  ELSE IF (rspanline(1:6).EQ. '12345 ' ) THEN
    WRITE (runstepnew, 'A') rspanline(1:i)//'6'
    mark = '6'
  FOR het geval dit eerste record is
    ELSE IF (rspanline(1:6).EQ. '1234 ' AND hnew .EQ. 999999) THEN
      WRITE (runstepnew, 'A') rspanline(1:i)//'55'
      mark = '55'
    ELSE
      GOTO 100
    ENDIF
    hold = hnew
ranstep record doornemen en kopieren tot relevante regel
READ (runstep, 'A', END=200) rspanline
  IF (rspanline(21:30).NE. 'waterlevel') THEN
    i = it*log(rspanline, 80)
    WRITE (runstepnew, 'A') rspanline(1:i)
    GOTO 35
  ENDIF
  c lezen waterniveau uit betreffende runstep record
  READ (rspanline(41:51), '(F12.3)') hnew
  c voor het geval dat dit eerste record is
  IF (hold.EQ.999999) hold=hnew
  c lezen en kopieren volgende regel uit runstep file
  READ (runstep, 'A', END=100) rspanline
  s = it*log(rspanline, 80)
  WRITE (runstepnew, 'A') rspanline(1:s)
  c lezen maximaal aantal iteraties per dyssaea stap
  READ (rspanline(41:52), ' (i2) ') maxitcoln
  c doornemen en kopieren laatste regels van
  betreffende runstep record
  DD 45 1+58,60
  READ (runstep, 'A', END=200) rspanline
  1 = it*log(rspanline, 80)
  WRITE (runstepnew, 'A') rspanline(1:i)
  GOTO 30
  CONTINUE
50 IF (.not. mark.EQ. '6') GOTO 30
  c opvolgende deel file kopieren
  55 READ (runstep, 'A', END=60) rspanline
  1 = it*log(rspanline, 80)
  WRITE (runstepnew, 'A') rspanline(1:i)
  GOTO 55
  c sluiten runstep files
  close (runstep)
  close (runstepnew)
  c bewerken rivcom files
  WRITE ('rcomline,80') 'Modifying RIVCOM.DAT -> RIVCOMM.DAT'
  OPEN (rivcom, FILE='RIVCOM.DAT', STATUS='OLD')
  OPEN (rivcomnew, FILE='RIVCOMM.DAT', STATUS='NEW')
  c doornemen rivcom file en schrijven naar rivcom file
  voor volgende tijdstap tot aan maximaal aantal iteraties
  65 READ (rivcom, 'A', END=200) rcomline
  IF (rcomline(1:22).NE. 'MAXIMAL AANTAL ITERATIES') THEN
    1 = it*log(rcomline, 80)
    WRITE (rivcomnew, 'A') rcomline(1:i)
    GOTO 65
  ENDIF
  c inlog (rcomline, 80)
  WRITE (rivcomnew, 'A') rcomline(1:i)
  c bovengenoemde nieuw maximaal aantal iteraties
  READ (rivcom, 'A', END=200) rcomline
  WRITE (rivcomnew, 'A') rcomline(1:i)
  c doornemen rivcom file en schrijven naar rivcom file
  voor volgende tijdstap tot aan vane niveau
  70 READ (rivcom, 'A', END=200) rcomline
  IF (rcomline(1:22).NE. 'AFSTAND BOVENNEM VANE') THEN
    1 = it*log(rcomline, 80)
    WRITE (rivcomnew, 'A') rcomline(1:i)
    GOTO 70
  ENDIF
  c bovengenoemde nieuw afstand van vane tot waterlep
  READ (rivcom, 'A', END=200) rcomline
  READ (rcomline(1:5), '(E5.2) ') afstandv
  afstandvnew=afstandv+hnew
  hold
WRITE (rivcomnew,'(F5.2)') afstandnew
Kopieren rest van rivcom file
75 READ (rivcom,'(A)') rcomline
   1 = itlog(rcomline,80)
   WRITE(rivcomnew,'(A)') rcomline(1:1)
GOTO 75
C Afsluiten programma
80 CLOSE(rivcom)
   CLOSE(rivcomnew)
   GOTO 500
100 OPEN (error, FILE = 'rcommod.err', STATUS = 'new')
   WRITE (error,'(A)') 'ERROR: RUNSTEP.DAT file was not used ' //
   & 'in right order.'
   STOP
200 OPEN (error, FILE = 'rcommod.err', STATUS = 'new')
   WRITE (error,'(A)') 'ERROR: Unexpected end of input file.'
   STOP
500 END
APPENDIX 4M  Listing Of Fortran Program Rsscreen.F

PROGRAM rsscreen

CHARACTER*80 ratepline
CHARACTER*4 mark
CHARACTER*4 label
INTEGER runstep,runstepnew,error

runstep = 22
runstepnew = 33
error = 66

label = 'RUNSTEP FILE LETEN TOT EN MET EERSTE RECORD ZONDER MARK 123456

OPEN (runstep, FILE='RUNSTEP.DAT', STATUS = 'OLD')
OPEN (runstepnew, FILE='RUNSTEPNEW.DAT', STATUS = 'NEW')
lir voor iedere runstep record

READ (runstep, '(A,10X, 5I5)', ERR=.4) runstep, runstepnew, mark
ELSEIF (runstepnew(1:6).EQ.'123456') THEN
    mark = '123456'
ELSE
    GOTO 150
ENDIF
WRITE (runstepnew, '(A)') runstepnew(1:1)

READ (runstep, '(A, 5I5)') runstep, runstepnew, mark
ELSEIF (runstepnew(1:6).EQ.'123456') THEN
    mark = '123456'
ELSE
    GOTO 150
ENDIF
WRITE (runstepnew, '(A)') runstepnew(1:1)

READ (runstep, '(A)') runstepnew, (A)
WRITE (runstep, '(A)') runstepnew(1:1)

CONTINUE

DO 45 = 1,90
READ (runstep, '(A,5I5)') runstep, runstepnew, mark
WRITE (runstepnew, '(A)') runstepnew(1:1)

CONTINUE

STOP

END
PROGRAM scrmod
CHARACTER*80 rstatline, scrline, scrlineew
CHARACTER*1 mark
CHARACTER*4 labelveryd, labeloid, labelnew, labelnext
CHARACTER*6 groupoweryd, groupoid, groupnew
INTEGER scr, scrnew, runstep, runstepnew, error
INTEGER i1, i, i, i, i
INTEGER naptmcellnew, naptmcellold, naptmcellnew
INTEGER naptmcell1new, naptmcell1old, naptmcell1new
INTEGER groupnew, igroupnew, chktsp, labelcheck
runstep = 22
runstepnew = 33
scr = 44
scrnew = 55
error = 66

if groupnew = '0 0 0' groupoid = '0 0 0'
labelveryd = '0 0 0'
labeloid = '
labelnew = 35
naptmcellnew = 0
naptmcellnew = 0
naptmcellnew = 0
c runstep file laten lezen tot en met eerste record zonder mark 1
c kopieren naar vervangende file runstepnew
c open runstep, FILE=*, STATUS = 'old'
c open runstepnew, FILE=*, STATUS = 'new'
c lus voor iedere runstep record
c check of bijwerken markeren +1
30 READ (runstep, 'A'), END=200) rstatline
  i = i + 1
  IF (rstatline(1:6).eq. '123456') THEN
    WRITE (runstepnew, 'A') (rstatline(1:i-1))
    mark + 1
    ELSE IF (rstatline(1:6).eq. '1') THEN
      WRITE (runstepnew, 'A') (rstatline(1:i-1))
      mark + 1
      ELSE
    GOTO 150
  ENDIF
groupoweryd = groupoid
    groupnew = groupnew
    labelveryd = labeloid
    labeloid = labelnew
    naptmcell1wyrd = naptmcelloid
    naptmcell1old = naptmcellold
    naptmcell1new = naptmcellnew
c runstep record doorlopen tot en met relevante regel
35 READ (runstep, 'A'), END=200) rstatline
  IF (rstatline(1:34).ne. 'discharge m3/s') THEN
    l = i
    ELSE WRITE (runstepnew, 'A') rstatline(1:i)
    GOTO 35
  ENDIF
    l = i
    WRITE (runstepnew, 'A') rstatline(1:i)
  c lezen nieuwe label uit betreffende runstep record
  READ (rstatline(1:i), 'A') labelnew
c doorgaan en kopieren runstep record tot relevante regel
  c met tijdstap informatie
37 READ (runstep, 'A'), END=200) rstatline
  IF (rstatline(1:34).ne. 'number of time') THEN
    l = i
    ELSE WRITE (runstepnew, 'A') rstatline(1:i)
    GOTO 37
  ENDIF
  c kopieren en uitlezen regel met aantal tijdstappen
c l = i
  WRITE (runstepnew, 'A') rstatline(1:i)
  READ (rstatline(41:52), 'I12') nuptmstasnew
c kopieren van een regel
39 READ (runstep, 'A'), END=200) rstatline
  l = i
  WRITE (runstepnew, 'A') rstatline(1:i)
c kopieren en uitlezen regels met maplin gegevens
  READ (runstep, 'A'), END=200) rstatline
  l = i
  WRITE (runstepnew, 'A') rstatline(1:i)
c kopieren en uitlezen regels met maplin gegevens
  READ (runstep, 'A'), END=200) rstatline
  l = i
  WRITE (runstepnew, 'A') rstatline(1:i)
c kopieren en uitlezen regels met maplin gegevens
  DO 45 i = 46, 60
    READ (runstep, 'A'), END=200) rstatline
    l = i
    WRITE (runstepnew, 'A') rstatline(1:i)
  45 CONTINUE
    c hktspMOD() numberstasnew = naptmitasnew
    IF (chktsp.ne.0) GOTO 175
    naptmcellnew = (nuptmstasnew + naptmitasnew)/4
    c maken groepsname string
groupnew(1:2) = '+'
    IF (naptmcellnew.1.10) THEN
      WRITE (groupnew(3:3), 'I1') naptmcellnew
groupnew(4:6) = '1'
    ENDIF
    IF (naptmcellnew.1.10) THEN
      WRITE (groupnew(3:3), 'I1') naptmcellnew

A 40
ENDIF
50 IF (exit.eq.'1') GOTO 30

voor het geval dat dit laatste runstep record was
labelnext = labelnew

volgend runstep record doorlopen tot en met relevante regel
52 READ (runstep,'(A1, END=60)') ratepline
   IF (ratepline(1:3).eq."discharge a3") THEN
      l = ilog(ratepline, 80)
      WRITE (runstepnew, '(A)') ratepline(l:1)
   GOTO 52
   ENDIF
   l = ilog(ratepline, 80)
   WRITE (runstepnew, '(A)') ratepline(l:1)
   inlezen nieuwe label uit dit volgend runstep record
   READ (ratepline(1:4), '(ai)') labelnext
   kopieren laatste regels van dit volgend runstep record
   DO 54 j=l,60
      READ (runstep,'(A)') END=100 ratepline
      l = ilog(ratepline, 80)
      WRITE (runstepnew, '(A)') ratepline(l:1)
   54 CONTINUE

55 afsluiten runstep file
   CLOSE (runstep)
   CLOSE (runstepnew)
   bewerken script files
   aanpassen crenef.scr
   file
   WRITE (*,'(A1)') 'Modifying crenof.scr -> crenofnew.scr'
   OPEN (scr, FILE='crenef.scr', STATUS='old')
   OPEN (new, FILE='crenefnew.scr', STATUS='new')
   labelcheck=0
   doornemen en kopieren script file zaak regels met mark
   70 READ (scr, '(A)') END=90 scrline
      l = ilog(scrline, 90)
   scrline=scln=scrline
   checken iedere positie op regel voor label
   DO 75 i=1,l
      IF (scrline(i:1:i+3).eq.labelversion) THEN
         scrline(new)(i:1:i+3)=labelold
         labelcheck=labelcheck+10
      ENDIF
      IF (scrline(i:1:i+3).eq.labelold) THEN
         scrline(new)(i:1:i+3)=labelnew
         labelcheck=labelcheck+1
      ENDIF
   75 CONTINUE

   inschrijven nieuwe scrline in file
   WRITE (scrnew, '(A)') scrline(new)(1:1)
   GOTO 70
   afsluiten script file
   CLOSE(scr)
   CLOSe(scrnew)
   IF (labelcheck ne a) GOTO 250
   aanpassen chdept.scr
   file
   WRITE (*,'(A1)') 'Modifying chdept.scr -> chdeptnew.scr'
   OPEN (scr, FILE='chdept.scr', STATUS='old')
   OPEN (new, FILE='chdeptnew.scr', STATUS='new')
   labelcheck=0
   doornemen en kopieren script file zaak regels met mark
   80 READ (scr, '(A)') END=90 scrline
      l = ilog(scrline, 90)
   scrline=new=scrline
   checken iedere positie op regel voor label
   DO 83 i=1,l
      IF (scrline(i:1:i+3).eq.labelversion) THEN
         scrline(new)(i:1:i+3)=labelold
         labelcheck=labelcheck+10
         WRITE (scrnew, '(A)') scrline(new)(1:1)
      ENDIF
      IF (scrline(i:1:i+3).eq.labelold) THEN
         scrline(new)(i:1:i+3)=labelnew
         labelcheck=labelcheck+1
      ENDIF
   83 CONTINUE
   inschrijven nieuwe scrline in file
   WRITE (scrnew, '(A)') scrline(new)(1:1)
   GOTO 80
   doornemen en kopieren script file zaak regels met c groupcevyold
   c rook optieneeold string en kopieren regels
   85 IF (groupcevon.eq.ilog(groupcevyold, 6))
      IF (groupcevon.eq.ilog(groupcevonid),1:1) THEN
         labelcheck=labelcheck+10
      ENDIF
      ELSE
         labelcheck=labelcheck+1
      ENDIF
A 41
c inschrijven regel in nieuwe scrline
1 = itlog('scrline.new', 80)
WRITE (scrnew, '(A)', scrline = '1:1')
GOTO 85
c afsluiten script file
85 CLOSE (scr)
CLOSE (scrnew)
IF (labelcheck.ne.100) GOTO 250
c aanpassen gti scr file
WRITE ('-', '(A)'), 'Modifying gti.scr => grinew.scr'
OPEN (scr, FILE='gti.scr', STATUS='old')
OPEN (scrnew, FILE='grinew.scr', STATUS='new')
labelcheck=0
doornemen en kopieren script file zoeken regels met mark
90 READ (scr, '(A)', END=98) scrline
1 = itlog(scrline, 80)
scrline = scrline
c checken iedere positie op regel voor label
DO 95 I=1,1:3
IF (scrline((I:1:3)) .eq. labelold) THEN
scrline((I:1:3)) = labelold
labelcheck = labelcheck + 100
ENDIF
IF (scrline((I:1:3)) .eq. labelnew) THEN
scrline((I:1:3)) = labelnew
labelcheck = labelcheck + 1
ENDIF
IF (scrline((I:1:3)) .eq. labelnext) THEN
scrline((I:1:3)) = labelnext
labelcheck = labelcheck + 10
ENDIF
95 CONTINUE
c inschrijven nieuwe scrline in file
WRITE (scrnew, '(A)', scrline = '(I:1:1:1)')
GOTO 90
c afsluiten script file
98 CLOSE (scr)
CLOSE (scrnew)
IF (labelcheck.ne.200) GOTO 250
c aanpassen chnbd scr file
WRITE ('-', '(A)'), 'Modifying chnbd.scr => chnbdnew.scr'
OPEN (scr, FILE='chnbd.scr', STATUS='old')
OPEN (scrnew, FILE='chnbdnew.scr', STATUS='new')
labelcheck=0
doornemen en kopieren script file zoeken regels met mark
100 READ (scr, '(A)', END=108) scrline
1 = itlog(scrline, 80)
scrline = scrline
c checken iedere positie op regel voor label
DO 105 I=1,1:3
IF (scrline((I:1:3)) .eq. labelold) THEN
scrline((I:1:3)) = labelold
labelcheck = labelcheck + 100
ENDIF
IF (scrline((I:1:3)) .eq. labelnew) THEN
scrline((I:1:3)) = labelnew
labelcheck = labelcheck + 1
ENDIF
IF (scrline((I:1:3)) .eq. labelnext) THEN
scrline((I:1:3)) = labelnext
labelcheck = labelcheck + 10
ENDIF
105 CONTINUE
c inschrijven nieuwe scrline in file
WRITE (scrnew, '(A)', scrline = '(I:1:1)')
GOTO 100
c afsluiten script file
108 CLOSE (scr)
CLOSE (scrnew)
IF (labelcheck.ne.1) GOTO 250
c aanpassen rivcom scr file
WRITE ('-', '(A)'), 'Modifying rivcom.scr => rivcomnew.scr'
OPEN (scr, FILE='rivcom.scr', STATUS='old')
OPEN (scrnew, FILE='rivcomnew.scr', STATUS='new')
labelcheck=0
doornemen en kopieren script file zoeken regels met mark
110 READ (scr, '(A)', END=118) scrline
1 = itlog(scrline, 80)
scrline = scrline
c checken iedere positie op regel voor label
DO 115 I=1,1:3
IF (scrline((I:1:3)) .eq. labelold) THEN
scrline((I:1:3)) = labelold
labelcheck = labelcheck + 100
ENDIF
IF (scrline((I:1:3)) .eq. labelnew) THEN
scrline((I:1:3)) = labelnew
labelcheck = labelcheck + 1
ENDIF
IF (scrline((I:1:3)) .eq. labelnext) THEN
scrline((I:1:3)) = labelnext
labelcheck = labelcheck + 10
ENDIF
115 CONTINUE
c inschrijven nieuwe scrline in file
WRITE (scrnew, '(A)', scrline = '(I:1:1)')
GOTO 110
c afsluiten script file
118 CLOSE (scr)
CLOSE (scrnew)
IF (labelcheck.ne.1) GOTO 250
c aanpassen copyout scr file
WRITE ('-', '(A)'), 'Modifying copyout.scr => copyoutnew.scr'
OPEN (scr, FILE='copyout.scr', STATUS='old')
OPEN (scrnew, FILE='copyoutnew.scr', STATUS='new')

labelcheck=0

doornemen en kopieren script file zoeken regels met mark
120 READ (scr, '(A)') scrline
   1 = itlog(scrline,80)
   scrline=newscrline
   checken iedere positie op regel voor label
   DO 125 (=1,13)
      IF (scrline(i:1:3).eq.labelvergoed) THEN
         scrline(new(i:1:3))=labelold
         labelcheck=labelcheck+100
      ENDIF
      IF (scrline(i:1:3).eq.labelold) THEN
         scrline(new(i:1:3))=labelnew
         labelcheck=labelcheck+1
      ENDIF
   IF (scrline(i:1:3).eq.labelnew) THEN
      scrline(new(i:1:3))=labelnext
      labelcheck=labelcheck+10
   ENDIF
   CONTINUE
125 WRITE (scrnew, '(A)') scrline(new(1:1))
   GOTO 120
   afdichten script file
128 CLOSE(scr)
   CLOSE(scrnew)
   IF (labelcheck.ne.22) GOTO 250
   GOTO 500
150 OPEN (error, FILE='scrmid.err', STATUS='new')
   WRITE (error, '(A)') 'ERROR! RUNSTEP.DAT file was not used '//
   & 'in right order.'
   STOP
175 OPEN (error, FILE='scrmid.err', STATUS='new')
   WRITE (error, '(A)') 'ERROR! Number of cells in net is not a whole number.'
   STOP
200 OPEN (error, FILE='scrmid.err', STATUS='new')
   WRITE (error, '(A)') 'ERROR! Unexpected end of input file.'
   STOP
250 OPEN (error, FILE='scrmid.err', STATUS='new')
   WRITE (error, '(A)') 'ERROR! Former label or runstep info ' //
   & 'was not correct in a scr file.'
   STOP
500 END
APPENDIX 40  Listing Of Fortran Program Wdoutin.F

PROGRAM wdoutin

  CHARACTER*40  ratepline, qfline, shdoldline, shdnewline
  CHARACTER*10  qffilename, qffilenameold
  CHARACTER*1  mark, dummy
  REAL          wdot, rdin(6), wdn(6), hold, hnew, minw
  REAL          shdold
  INTEGER       shdoldfile, shdnewfile
  INTEGER       wd, runstep, runstepnew, gridfl, gridflnew, error
  INTEGER       ilog, il, counthold, count

  wdot = 11
  runstep = 22
  runstepnew = 33
  gridfl = 44
  gridflnew = 55
  error = 66
  shdoldfile = 77
  shdnewfile = 88
  hnew = 0.0
  counthold = 0
  countnew = 0
  minw = 0.5
  qffilename = ' '

  CONTINUE

  runstep file lezen tot en met eerste record zonder mark 5
  OPEN (runstep, FILE='runstep.dat', STATUS = 'old')
  OPEN (runstepnew, FILE='runstepnew.dat', STATUS = 'new')
  lus voor elke runstep record

  READ (runstep,'(A)', END=200) ratapline
  l = ilog(ratepline,80)
  IF (ratepline(1:6).eq.'123456') THEN
    WRITE (runstepnew,'(A)') ratapline(1:11)
    mark = 5
    ELSE IF (ratepline(1:6).eq.'1234 ') THEN
    WRITE (runstepnew,'(A)') ratepline(1:11)//'5'
    mark = 5
    ELSE
    GOTO 100
  ENDIF

  hold = hnew
  qffilename = qffilenameold

  runstep record doorlopen en kopieren tot relevante regel
  READ (runstep,'(A)', END=200) ratapline
  IF (ratapline(1:6).ne.'GRIDFL') THEN
    l = ilog(ratepline,80)
    WRITE (runstepnew, '(A)') ratapline(1:11)
  ELSE IF (ratapline(1:6).eq.'GRIDFL') THEN
    OPEN (gridfl, FILE='gridfl.dat', STATUS = 'old')
    OPEN (gridflnew, FILE='gridflnew.dat', STATUS = 'new')
    dopen gridfl new en aanbrengen in relevante regel
  ENDIF

  runstep record doornemen en kopieren tot relevante regel
  READ (runstep,'(A)', END=200) ratapline
  IF (ratapline(1:6).eq.'GRIDFL') THEN
    l = ilog(ratepline,80)
    WRITE (runstepnew, '(A)') ratapline(1:11)
  ELSE IF (ratapline(1:6).eq.'GRIDFL') THEN
    OPEN (gridfl, FILE='gridfl.dat', STATUS = 'old')
    OPEN (gridflnew, FILE='gridflnew.dat', STATUS = 'new')
    dopen gridfl new en aanbrengen in relevante regel
  ENDIF

  IF (mark.eq.'5') THEN GOTO 30
  else voor volgende tijdstap

  READ (runstep,'(A)', END=60) ratapline
  l = ilog(ratepline,80)
  WRITE (runstepnew, '(A)') ratapline(1:11)
  GOTO 35

  CONTINUE

  IF (mark.eq.'5') THEN GOTO 30
  CLOSE (runstep)
  CLOSE (runstepnew)

  bewerken gridfl files
  WRITE (*,'(A)') 'Modifying ', //qffilenameold// ' --> ', //qffilename
  OPEN (gridfl, FILE='gridfl.dat', STATUS = 'old')
  OPEN (gridflnew, FILE='gridflnew.dat', STATUS = 'new')
  dopen gridfl new en aanbrengen in relevante regel
  voor volgende tijdstap

  READ (gridfl,'(A)', END=200) gfline
  IF (gfline(1:7).eq.'SHADOW') THEN
    l = ilog(gfline,80)
    WRITE (gridflnew,'(A)') gfline(1:11)
  ENDIF

  GOTO 65

  CONTINUE

  l = ilog(gfline,80)
  WRITE (gridflnew,'(A)') gfline(1:11)

  openen file met waterdiepten resultaten vorige runstep
  OPEN (wd, FILE='wd.dat', STATUS = 'old')
  READ (wd,'(A)', END=200) dummy

  openen vorige file met eventuele shadow waterdiepten
  OPEN (shdoldfile, FILE='shdold.dat', STATUS = 'old')
  READ (shdoldfile,'(A)', END=200) shdoldline

  lezen eventuele eerste shadow waterdiepte
  READ (shdoldline,'(A)', END=67) shdoldline
  READ (shdoldline(1:12),'(I2*1)') counthold
  READ (shdoldline(13:24),'(F13.5)') shdold

  openen nieuwe file met eventuele shadow waterdiepten
  OPEN (shdnewfile, FILE='shdnew.dat', STATUS = 'ew')

A 44
c inlezen rijtje van 6 waterdiepten uit vorige runstep
70 READ (ud, '(6F12.3)') wdout
     berekening rijtje van 6 nieuwe waterdiepten
     DO 75 i = 1, 6
         count = count + 1
         IF ((count .ge. 192) .AND. MOD((count - 1), 192) .NE. 0) THEN
             win(i) = wdout(i) + (newhold)
         ELSE
             win(i) = wdold(i) + (newhold)
         ENDIF
      ENDIF
      IF ((count .lt. 192) .AND. MOD((count - 1), 192) .NE. 0) THEN
         IF (win(i) .lt. winold(i)) THEN
             win(i) = winold(i)
         ENDIF
      ELSE
         IF (win(i) .lt. winold(i)) THEN
             win(i) = winold(i)
         ENDIF
      ENDIF
    ENDIF
    ELSE
       win(1) = 0.
    ENDIF
    CONTINUE
75 CONTINUE
     check 6 nieuwe waterdiepten op waarden kleiner dan minimum
     count = count - 6
     DO 77 i = 1, 6
         IF ((count .ge. 192) .AND. MOD((count - 1), 192) .NE. 0) THEN
             IF (win(i) .lt. min(new)) THEN
                 write waterdiepte in shadow file
                 WRITE (shadowline(1:24), '(12,1Z,2I2)') count
                 WRITE (shadowfile, '(12,1Z,2I2)') win(i)
                 WRITE (shadownewfile, '(12,1Z,2I2)') win(i)
                 ENDIF
         ELSE
             IF (win(i) .lt. min(new)) THEN
                 write waterdiepte in gridfline
                 WRITE (gridfline, '(6F12.3)') win
             ENDIF
         ENDIF
    ENDIF
    CONTINUE
    CONTINUE
    CONTINUE
80 CONTINUE
    CONTINUE
    abort programma
    CLOSE(ud)
    CLOSE(shadow)
    CLOSE(shadowold)
    CLOSE(gridfline)
    GOTO 506
100 OPEN (error, FILE='wdoutin.err', STATUS='NEW')
    WRITE (error, '(A)') 'ERROR! RUNSTEP.DAT file was not used '/
    WRIT BS 'in right order.'/'
    STOP
200 OPEN (error, FILE='wdoutin.err', STATUS='NEW')
    WRITE (error, '(A)') 'ERROR! Unexpected end of input file.'
    STOP
500 END
INTEGER FUNCTION itlog(string, lengte)

  INTEGER lengte
  CHARACTER*1 string(lengte)

  INTEGER k

  Verwerk character buffer

  itlog = 0
  DO 190 k = lengte, 1, -1

   Character is ongelijk aan spatie
   IF (string(k).NE. ' ') THEN
    Werkelijke lengte is gevonden
    itlog = k
    GOTO 200
  ENDIF
190 CONTINUE
200 RETURN
END
Figure A 14  Calculation time for run steps without vanes.

Figure A 15  Calculation time for run steps with vanes.
APPENDIX 4R  Specification Run K

Objective
To compare the bed topography variations before and after a period of high discharges with bed level measurements.

Discharge
Measurement from 1-10-1994 to 30-9-1995

Initial bed level
Equilibrium bed level in model at Q = 1600 m³/s without vanes, end situation from run T00

Vanes
No

<table>
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<th>Run step</th>
<th>Begin</th>
<th>Interval</th>
<th>End</th>
<th>Discharge</th>
<th>Discharge</th>
<th>Water Level</th>
<th>Velocity</th>
<th>Sed. Transp.</th>
<th>Time step</th>
<th>Number</th>
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<td>day</td>
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<td>day</td>
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<td>boundary</td>
<td>centerline</td>
<td>centerline</td>
<td>s</td>
<td></td>
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<td>1</td>
<td>1</td>
<td>90</td>
<td>90</td>
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<td>1200</td>
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<td>1.03</td>
<td>2.68E-05</td>
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<td>180</td>
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<td>128</td>
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<td>192</td>
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<td>2440</td>
<td>11.82</td>
<td>1.27</td>
<td>7.94E-04</td>
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<td>269</td>
<td>2100</td>
<td>1990</td>
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<td>1400</td>
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<td>1.08</td>
<td>3.43E-05</td>
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<td>192</td>
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</tbody>
</table>

Figure A 16  Schematization of discharges for run K.
**APPENDIX 4S**  Longitudinal Sections Soundings And Run K

![Graph]

**Figure A 17**  Bed level in long sections of soundings in 1994 and 1995.

![Graph]

**Figure A 18**  Bed level in long sections of Rivcom output run K, equivalent to soundings.

A 49
Figure A 19  Differences in bed level in inner bend.

Figure A 20  Differences in bed level in centerline.
Figure A 21  Differences in bed level in outer bend.
Figure A 22  Differences in bed level in cross section in soundings 1994 and 1995.

Figure A 23  Differences in cross section in Rivcom output of run K, equivalent to soundings.
APPENDIX 4U  Indicator Variation Run K

Figure A.24  Bed level in inner bend.

Figure A.25  Bed level in centerline.
Figure A 26  Bed level in outer bend.

Figure A 27  Minimum navigable width at OLR -2.80 m.
## APPENDIX 4V  Comparison Of Indicator Variation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Measured 24-11-1994</th>
<th>Measured 22-4-1995</th>
<th>Δ&lt;sub&gt;meas&lt;/sub&gt;</th>
<th>Model run K&lt;sub&gt;e&lt;/sub&gt; day 54 24-11-1994</th>
<th>Model run K&lt;sub&gt;e&lt;/sub&gt; day 204 22-4-1995</th>
<th>Δ&lt;sub&gt;mod&lt;/sub&gt;</th>
<th>Δ&lt;sub&gt;mod&lt;/sub&gt;/Δ&lt;sub&gt;meas&lt;/sub&gt;</th>
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</thead>
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<tr>
<td>Bed level 60m from left bank</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>-2.25</td>
<td>-1.86</td>
<td>+0.39</td>
<td>-1.60</td>
<td>-1.40</td>
<td>+0.20</td>
<td>0.50</td>
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<tr>
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<td>-1.42</td>
<td>-0.88</td>
<td>+0.54</td>
<td>-0.76</td>
<td>-0.53</td>
<td>+0.23</td>
<td>0.42</td>
</tr>
<tr>
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<td>-2.11</td>
<td>-1.04</td>
<td>+1.07</td>
<td>-1.40</td>
<td>-0.63</td>
<td>+0.77</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>-3.38</td>
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<tr>
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<td>-3.24</td>
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<td>-3.22</td>
<td>-3.15</td>
<td>+0.07</td>
<td>0.35</td>
</tr>
<tr>
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<td>+0.44</td>
<td>-3.24</td>
<td>-2.95</td>
<td>+0.29</td>
<td>0.66</td>
</tr>
<tr>
<td>Bed level 60m from right bank</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
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<td>-5.81</td>
<td>+0.16</td>
<td>-5.71</td>
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<td>-1.31</td>
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<td>-5.66</td>
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<td>-0.35</td>
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<tr>
<td>Minimum navigable width at OLR-2.80m</td>
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<td>m 146.1</td>
<td>m -13.5</td>
<td>m 128.5</td>
<td>m 122.5</td>
<td>m -6.0</td>
<td>0.44</td>
</tr>
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</table>

Table A 2  Comparison of bed level differences between 1994 and 1995 soundings, and as predicted by Rivcom
### APPENDIX 5A  Specification Runs E And G

**Objective**

To study the time the bed topography needs to adapt itself to an equilibrium situation at a different discharge and the magnitude of the differences for the situation without vanes.

**Discharge**

Constant 1600 m³/s, 2000 m³/s, 1000 m³/s, 3000 m³/s, 4000 m³/s

**Initial bed level**

Equilibrium bed level in model at Q = 1600 m³/s without vanes, end situation from run T00

**Vanes**

No

<table>
<thead>
<tr>
<th>Run label</th>
<th>Begin day</th>
<th>Interval days</th>
<th>End day</th>
<th>Discharge Waal m³/s</th>
<th>Discharge model m³/s</th>
<th>Water Level boundary m + NAP</th>
<th>Velocity centerline m/s</th>
<th>Sed. Transp. centerline m³/s/m</th>
<th>Time step s</th>
<th>Number</th>
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<td>100</td>
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<td>1600</td>
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<td>1.13</td>
<td>4.35E-05</td>
<td>43200</td>
<td>200</td>
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<tr>
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<td>500</td>
<td>500</td>
<td>2000</td>
<td>1920</td>
<td>10.59</td>
<td>1.18</td>
<td>5.52E-05</td>
<td>21600</td>
<td>2000</td>
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<td>1000</td>
<td>8.17</td>
<td>0.96</td>
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<td>1.47</td>
<td>1.62E-04</td>
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</table>

**Objective**

To study the time the bed topography needs to adapt itself to an equilibrium situation at a different discharge and the magnitude of the differences for the situation with vanes.

**Discharge**

Constant 1600 m³/s, 2000 m³/s, 1000 m³/s, 3000 m³/s, 4000 m³/s

**Initial bed level**

Equilibrium bed level in model at Q = 1600 m³/s with vanes, end situation from run T10

**Vanes**

Yes

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<th>Discharge model m³/s</th>
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<th>Velocity centerline m/s</th>
<th>Sed. Transp. centerline m³/s/m</th>
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<th>Number</th>
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<td>250</td>
<td>1600</td>
<td>1600</td>
<td>9.74</td>
<td>1.13</td>
<td>4.35E-05</td>
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<td>500</td>
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<td>G02</td>
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<td>500</td>
<td>500</td>
<td>2000</td>
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<td>1.18</td>
<td>5.52E-05</td>
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<td>1000</td>
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<td>8.17</td>
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<td>1.94E-05</td>
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<td>3000</td>
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APPENDIX 5B  Development Longitudinal Sections Run E

Figure A.28  Development of longitudinal sections at 1000 m³/s without vanes.

Figure A.29  Development of longitudinal sections at 3000 m³/s without vanes.
APPENDIX 5C  Equilibrium Sections Run E

Figure A.30  Equilibrium longitudinal sections for situation without vanes.

Figure A.31  Equilibrium cross section for situation without vanes.
Figure A.32 Equilibrium navigable width at OLR -2.80 m for situation without vanes.
APPENDIX 5E  Indicator Variation Run E

Figure A 33  Bed level in inner bend at km 869.500.

Figure A 34  Bed level in centerline at km 869.500.
Figure A 35 Bed level in outer bend at km 869.500.

Figure A 36 Bed level in inner bend at km 870.000.
Figure A.37 Bed level in centerline at km 870,000.

Figure A.38 Bed level in outer bend at km 870,000.
Figure A 39  Bed level in inner bend at km 870.500.

Figure A 40  Bed level in centerline at km 870.500.
Figure A 41 Bed level in outer bend at km 870.500.

Figure A 42 Minimum navigable width at OLR -2.80m.
APPENDIX 5F  
Equilibrium Situation Without Vanes

Figure A 43  Equilibrium bed level in inner bend for situation without vanes.

Figure A 44  Equilibrium bed level in centerline for situation without vanes.
Figure A 45 Equilibrium bed level in outer bend for situation without vanes.

Figure A 46 Equilibrium minimum navigable width at OLR -2.80m.
## APPENDIX 5G  Relaxation Time Without Vanes Run E

<table>
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<th>Parameter</th>
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<th>$T_r / T_{r,1000m^3/s}$</th>
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<td></td>
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</tr>
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<td>1.00</td>
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<td>0.69</td>
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<td>0.34</td>
</tr>
<tr>
<td>km 870.500</td>
<td>380</td>
<td>0.68</td>
<td>0.24</td>
</tr>
<tr>
<td>Minimum navigable width at OLR - 2.80m</td>
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<td>0.63</td>
<td>0.31</td>
</tr>
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<tr>
<td>km 869.500</td>
<td>400</td>
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<td>0.33</td>
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<td>1.00</td>
<td>0.31</td>
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<td>km 870.500</td>
<td>420</td>
<td>0.78</td>
<td>0.30</td>
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<tr>
<td>Bed level in centerline</td>
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<td></td>
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<td>km 870.500</td>
<td>330</td>
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<td>0.20</td>
</tr>
<tr>
<td>Bed level 60m from right bank</td>
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<td></td>
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</tr>
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<td>km 869.500</td>
<td>520</td>
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<td>0.49</td>
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<td>220</td>
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<td>0.20</td>
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Table A.3  Relaxation times for situation without vanes, measured from results Run E
APPENDIX 5H  Axial Symmetric Bed Level Theory

An axial symmetric approximation of the bed level was derived from the equations underlying the Rivcom model in Appendix 12B.

\[
\frac{\partial z_b}{\partial n} = 0.85 \frac{E_i}{E_s} \frac{2}{\kappa C} \left(1 - \sqrt{\frac{g}{\kappa C}}\right) \sqrt{\frac{h}{R}}
\]  
(A 32)

At the cross section at km 870,000 the local radius of curvature of the river axis is approximately R = 1100 m. The lateral slope in the river centerline was calculated from the near equilibrium situations of the 2DH RIVCOM results. A comparison is made in the following table.

<table>
<thead>
<tr>
<th>Discharge ( m^3/s )</th>
<th>Lateral Slope in Centerline RIVCOM result</th>
<th>Lateral Slope in Centerline axial-symmetric theory</th>
<th>Overshoot</th>
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<tr>
<td>1000</td>
<td>0.038</td>
<td>0.016</td>
<td>2.4</td>
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<td>1600</td>
<td>0.047</td>
<td>0.026</td>
<td>1.8</td>
</tr>
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<td>2000</td>
<td>0.050</td>
<td>0.032</td>
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<td>0.054</td>
<td>0.044</td>
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</tr>
<tr>
<td>4000</td>
<td>0.065</td>
<td>0.056</td>
<td>1.2</td>
</tr>
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APPENDIX 51  Development Longitudinal Sections Run G

Figure A 47  Development of longitudinal sections at 1000 m³/s with vanes.

Figure A 48  Development of longitudinal sections at 3000 m³/s with vanes.
APPENDIX 5J  Equilibrium Sections Run G

Figure A 49  Equilibrium longitudinal sections for situation with vanes.

Figure A 50  Equilibrium cross sections for situation with vanes.
Figure A 51  Equilibrium navigable width at OLR -2.80 m for situation with vanes.
Figure A 52  Bed level in inner bend at km 869.500.

Figure A 53  Bed level in centerline at km 869.500.
Figure A 54  Bed level in outer bend at km 869.500.

Figure A 55  Bed level in inner bend at km 870.000.
Figure A 56  Bed level in centerline at km 870.000.

Figure A 57  Bed level in outer bend at km 870.000.
Figure A.58 Bed level in inner bend at km 870.500.

Figure A.59 Bed level in centerline at km 870.500.
Figure A 60  Bed level in outer bend at km 870.500.

Figure A 61  Minimum navigable width at OLR -2.80m.
APPENDIX 5M  Equilibrium Situation With Vanes

Figure A 62  Equilibrium bed level in inner bend for situation with vanes.

Figure A 63  Equilibrium bed level in centerline for situation with vanes.
Figure A 64  Equilibrium bed level in outer bend for situation with vanes.

Figure A 65  Equilibrium navigable width at OLR -2.80m.
## APPENDIX 5N  
Relaxation Time With Vanes Run G

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<th>Relaxation Time $T_r$</th>
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<th>$T_r / T_{r, 000 m^3/s}$</th>
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<td>0.23</td>
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<td>0.29</td>
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<td>0.42</td>
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Table A4  
Relaxation times for situation with vanes, measured from results Run G
APPENDIX 6A  Specification Runs M

Objective  
To estimate the time the bed topography needs to adapt itself once the vanes are installed at various constant discharges.

Discharge  
Constant 1600 m$^3$/s, 2000 m$^3$/s, 1000 m$^3$/s

Initial bed level  
Equilibrium bed level in model at Q = 1600 m$^3$/s without vanes, end situation from run T00

Vanes  
Yes

<table>
<thead>
<tr>
<th>Run label</th>
<th>Begin</th>
<th>Interval</th>
<th>End</th>
<th>Discharge</th>
<th>Discharge</th>
<th>Water Level</th>
<th>Velocity</th>
<th>Sed. Transp.</th>
<th>Time step</th>
<th>Number</th>
</tr>
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<tr>
<td></td>
<td>day</td>
<td>days</td>
<td>day</td>
<td>m3/s</td>
<td>m3/s</td>
<td>m+NAP</td>
<td>m/s</td>
<td>m/s</td>
<td>s</td>
<td></td>
</tr>
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<td>M01</td>
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<td>1460</td>
<td>1600</td>
<td>1600</td>
<td>9.74</td>
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<td>4.35E-05</td>
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<td>2920</td>
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<td>1095</td>
<td>2000</td>
<td>1920</td>
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<td>1.18</td>
<td>5.52E-05</td>
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Appendix 6B
Development Sections Run M
APPENDIX 6C  Indicator Variation Run M

Figure A 68  Bed level in inner bend at km 869.500.

Figure A 69  Bed level in centerline at km 869.500.
Figure A 70  Bed level in outer bend at km 869.500.

Figure A 71  Bed level in inner bend at km 870.000.
Figure A 72  Bed level in centerline at km 870.000.

Figure A 73  Bed level in outer bend at km 870.000.
Figure A 74  Bed level in inner bend at km 870.500.

Figure A 75  Bed level in centerline at km 870.500.
Figure A 76 Bed level in outer bend at km 870.500.

Figure A 77 Minimum navigable width at OLR -2.80 m.
# APPENDIX 6D  
Relaxation Time Run M

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<th>Tₜ / Tₕ 1000 m³/s</th>
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Table A5  Relaxation times measured in runs M.
APPENDIX 6E  Specification Run H

Objective
To study the time the bed topography needs to adapt itself once the vanes are installed, during the representative year 1983-1984.

Discharge
Measurement from hydrological year 83-84, 3 consecutive times.

Initial bed level
Equilibrium bed level in model at Q = 1600 m³/s without vanes, end situation from run T00

Vanes
Yes

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![Graph](image-url) 

**Graph Note:**
- Measurement
- Schematization

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Figure A.80 Bed level in contact at km 869.500

Figure A.79 Bed level in contact at km 869.500

Appendix 6F Indicator Variation Run H

Appendix 6F Indicator Variation Run H
Figure A 81  Bed level in outer bend at km 869.500.

Figure A 82  Bed level in inner bend at km 870.000.
Figure A 83  Bed level in centerline at km 870.000.

Figure A 84  Bed level in outer bend at km 870.000.
Figure A 85  Bed level in inner bend at km 870.500.

Figure A 86  Bed level in centerline at km 870.500.
Figure A 87  Bed level in outer bend at km 870.500.

Figure A 88  Minimum navigable width at -2.80 m OLR.
APPENDIX 6H  Specification Run P

Objective  
To estimate the time the bed topography needs to adapt itself to the new situation when the vannes are installed, in case of some subsequent years with high discharge periods.

Discharge  
Measurement from hydrological years 65-66, 69-70, 74-75, 70-71, 71-72  
Initial bed level  
Equilibrium bed level in model at $Q = 1600$ m$^3$/s without vannes, end situation from run T00  
Vannes  
Yes

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**Graph**

- Measurement
- Schematization

---

A97
## APPENDIX 6G  Relaxation Time Run H

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*Table A 6* Relaxation times Run H relative to inner bend bed level at km 870.000 and corresponding constant discharge.

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*Table A 7* Relaxation times Run H relative to 1000 m$^3$/s and 1600 m$^3$/s constant discharge runs M.
Figure A.90  Bed level in inner bend at km 869.500 (1/2).

Figure A.91  Bed level in inner bend at km 869.500 (2/2).
Figure A 89  Schematization of discharges for tun P.
Figure A 92  Bed level in centerline at km 869.500 (1/2).

Figure A 93  Bed level in centerline at km 869.500 (2/2).
Figure A 94  Bed level in outer bend at km 869.500 (1/2).

Figure A 95  Bed level in outer bend at km 869.500 (2/2).
Figure A.96  Bed level in inner bend at km 870.000 (1/2).

Figure A.97  Bed level in inner bend at km 870.000 (2/2).
Figure A.98 Bed level in centerline at km 870.000 (1/2).

Figure A.99 Bed level in centerline at km 870.000 (2/2).
Figure A 100 Bed level in outer bend at km 870.000 (1/2).

Figure A 101 Bed level in outer bend at km 870.000 (2/2).
Figure A 102 Bed level in inner bend at km 870.500 (1/2).

Figure A 103 Bed level in inner bend at km 870.500 (2/2).
Figure A 104 Bed level in centerline at km 870.500 (1/2).

Figure A 105 Bed level in centerline at km 870.500 (2/2).
Figure A.106 Bed level in outer bend at km 870.500 (1/2).

Figure A.107 Bed level in outer bend at km 870.500 (2/2).
Figure A 108 Minimum navigable width at OLR -2.80 m (1/2).

Figure A 109 Minimum navigable width at OLR -2.80 m (2/2).
### APPENDIX 6J  Relaxation Time Run P

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<td>Minimum navigable width at OLR -2.80 m</td>
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**Table A8**  Relaxation times run P relative to inner bend bed level at km 870.000.

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<th>Parameter</th>
<th>$T_r / T_{r 1000 \text{ m}^3/\text{s M}}$</th>
<th>$T_r / T_{r 1600 \text{ m}^3/\text{s M}}$</th>
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<tbody>
<tr>
<td>Bed level 60m from left bank</td>
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<td>0.76</td>
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<td>Bed level in centerline</td>
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<td>Bed level 60m from right bank</td>
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**Table A9**  Relaxation times run P relative to 1000 m³/s and 1600 m³/s constant discharge runs.
APPENDIX 7A  Specification Run B

Objective
To test the procedure for the situation without vanes, using a fine schematization of the discharge variations.

Discharge
Measurement from hydrological year 83-84.

Initial bed level
Equilibrium bed level in model at Q = 1600 m³/s without vanes, end situation from run T00

Vanes
No

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<th>Discharge (model) m³/s</th>
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<th>Velocity m/s</th>
<th>Sed. Transp.</th>
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Figure A 110 Schematization of discharges in run B.
Figure A.111 Variations in longitudinal sections before and after flood period.

Figure A.112 Variations in cross section before and after flood period.
Figure A 113 Bed level in inner bend.

Figure A 114 Bed level in centerline.
Figure A 115 Bed level in outer bend.

Figure A 116 Minimum width for navigation at OLR -2.80 m.
APPENDIX 7D Specification Run C

Objective
To compare the results for a somewhat coarser schematization of the discharge variations with the results for the fine schematization of run B for the situation without vanes.

Discharge
Measurement from hydrological year 83-84.

Initial bed level
Equilibrium bed level in model at Q = 1600 m³/s without vanes, end situation from run T00.

Vaness
No

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<th>Water Level centerline m/s</th>
<th>Velocity centerline m/s</th>
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Figure A 117 Schematization of discharges for run C.
APPENDIX 7E  Indicator Variation Run C

Figure A 118 Bed level in inner bend.

Figure A 119 Bed level in centerline.
Figure A 120 Bed level in outer bend.

Figure A 121 Minimum navigable width at OLR -2.80m.
APPENDIX 7F  Specification Run I

Objective: To compare the average bed topography during the representative year 1983-1984, with the bed level at the constant discharge of 1600 m$^3$/s.

Discharge: Measurement from hydrological year 83-84, 2 consecutive times.

Initial bed level: Equilibrium bed level in model at Q = 1600 m$^3$/s with vanes, end situation from run T10.

Yves: Yes

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<th>End</th>
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Figure A 122 Schematization of discharges for run I.
APPENDIX 7G  Variation In Sections Run I

Figure A 123 Variations in longitudinal sections before and after flood period.

Figure A 124 Variations in cross section before and after flood period.
APPENDIX 7H  Indicator Variation Run I

Figure A 125 Bed level in inner bend.

Figure A 126 Bed level in centerline.
Figure A.127 Bed level in outer bend.

Figure A.128 Minimum navigable width at OLR -2.80 m.
APPENDIX 7I  Specification Run N

Objective
To compare the variations in bed topography for the situation without vanes and with vanes (run 70) during a year with a longer period of moderately high discharges.

Discharge
Measurement from 1-10-1965 to 30-6-1966

Initial bed level
Equilibrium bed level in model at Q = 1600 m³/s without vanes, end situation from run T00

Vanes
No

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Figure A 129  Schematization of discharges for run N.
APPENDIX 7J  Variation In Sections Run N

Figure A130 Variations in longitudinal section before and after flood period.

Figure A131 Variations in cross section before and after flood period.
APPENDIX 7K Indicator Variation Run N

Figure A 132 Bed level in inner bend.

Figure A 133 Bed level in centerline.
Figure A 134 Bed level in outer bend.

Figure A 135 Minimum navigable width at OLR 2.80 m.
APPENDIX 7L  Specification Run J

Objective
To study the variations in bed topography during a year with a longer period of moderately high discharges.

Discharge
Measurement from 1-10-1965 to 30-9-1966

Initial bed level
Equilibrium bed level in model at Q = 1600 m³/s with vanes, end situation from run T10

Vaness
Yes

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**Figure A.36** Schematic of discharges for run J.
APPENDIX 7M Variation In Sections Run J

Figure A 137 Variations in longitudinal sections before and after period of high discharge.

Figure A 138 Variations in cross section before and after period of high discharge.
APPENDIX 7N  Indicator Variation Run J

Figure A 139 Bed level in inner bend.

Figure A 140 Bed level in centerline.
Figure A 141 Bed level in outer bend.

Figure A 142 Minimum navigable width at OLR -2.80 m.
APPENDIX 7O  Specification Run L

Objective  To study the variations in bed topography during a year with a high discharge peak.

Discharge  Measurement from 1-10-1994 to 30-9-1995
Initial bed level  Equilibrium bed level in model at Q = 1600 m³/s with vanes, end situation from run T10
Vanes  Yes

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<th>Discharge model m³/s</th>
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Figure A 143 Schematization of discharges for run L.
APPENDIX 7P  Variation In Sections Run L

Figure A 144 Variations in longitudinal sections before and after period of high discharge.

Figure A 145 Variations in cross section before and after period of high discharge.

A131
Figure A 146 Bed level in inner bend.

Figure A 147 Bed level in centerline.
Figure A 148 Bed level in outer bend.

Figure A 149 Minimum navigable width at OLR -2.80 m.
APPENDIX 7R  Average Bed Level Indicators Run B

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*Table A 10  Average bed levels over second year run B, with corresponding constant discharge.*
### Average Bed Level Indicators Run I

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**Table A1.1** Average bed levels over second year run I, with corresponding constant discharge.
**Part 1 Appendix 7T  Specification Run O**

**APPENDIX 7T  Specification Run O**

**Objective**
To compare the average bed topography during some years with representative discharges with the bed level at the constant discharge of 1600 m$^3$/s.

**Discharge**
Measurement from hydrological years 74-75, 79-80, 76-77, 73-74, 64-65

**Initial bed level**
Equilibrium bed level in model at $Q = 1600$ m$^3$/s with vanes, end situation from run T10

**Vaness**
Yes

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<th>End day</th>
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**Diagram:**
- Measurement
- Schematization

A136
Figure A 150  Schematization of discharges for run O.
Figure A 151 Bed level in inner bend (1/2).

Figure A 152 Bed level in inner bend (2/2).
Figure A 153 Bed level in centerline (1/2).

Figure A 154 Bed level in centerline (2/2).
Figure A 155 Bed level in outer bend (1/2).

Figure A 156 Bed level in outer bend (2/2).
Figure A 157 Minimum navigable width at OLR -2.80 m (1/2).

Figure A 158 Minimum navigable width at OLR -2.80 m (2/2).
## APPENDIX 7V  Average Bed Level Indicators Run O

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</tr>
<tr>
<td><strong>Minimum navigable width at OLR</strong></td>
<td>155.3</td>
<td>+0.3</td>
<td></td>
<td>1600</td>
</tr>
</tbody>
</table>

**Table A12**  Average bed levels in run O and corresponding constant discharge.
## APPENDIX 7W  Average Bed Level After Dry Years

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Equilibrium bed level at 1300 m³/s</th>
<th>Difference with Equilibrium 1600 m³/s</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>m+OLR</strong></td>
<td><strong>m</strong></td>
<td></td>
</tr>
<tr>
<td>Bed level 60m from left bank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>-2.27</td>
<td>+0.04</td>
</tr>
<tr>
<td>km 870.000</td>
<td>-2.50</td>
<td>-0.11</td>
</tr>
<tr>
<td>km 870.500</td>
<td>-2.94</td>
<td>-0.29</td>
</tr>
<tr>
<td>Bed level in centerline</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>-4.02</td>
<td>+0.21</td>
</tr>
<tr>
<td>km 870.000</td>
<td>-4.26</td>
<td>+0.11</td>
</tr>
<tr>
<td>km 870.500</td>
<td>-3.90</td>
<td>+0.22</td>
</tr>
<tr>
<td>Bed level 60 m from right bank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>-5.10</td>
<td>-0.01</td>
</tr>
<tr>
<td>km 870.000</td>
<td>-4.68</td>
<td>+0.08</td>
</tr>
<tr>
<td>km 870.500</td>
<td>-4.34</td>
<td>+0.23</td>
</tr>
<tr>
<td>Minimum navigable width at OLR</td>
<td>148.7</td>
<td>-6.5</td>
</tr>
<tr>
<td>-2.80 m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Table A 13  Average bed levels after long term dry period.*
### APPENDIX 7X  Average Bed Level After Wet Years

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Equilibrium bed level at 2100 m³/s</th>
<th>Difference with Equilibrium 1600 m³/s</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m+OLR</td>
<td>m</td>
</tr>
<tr>
<td>Bed level 60m from left bank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>-2.33</td>
<td>-0.02</td>
</tr>
<tr>
<td>km 870.000</td>
<td>-2.36</td>
<td>+0.03</td>
</tr>
<tr>
<td>km 870.500</td>
<td>-2.47</td>
<td>+0.18</td>
</tr>
<tr>
<td>Bed level in centerline</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>-4.62</td>
<td>-0.39</td>
</tr>
<tr>
<td>km 870.000</td>
<td>-4.45</td>
<td>-0.08</td>
</tr>
<tr>
<td>km 870.500</td>
<td>-4.29</td>
<td>-0.17</td>
</tr>
<tr>
<td>Bed level 60 m from right bank</td>
<td></td>
<td></td>
</tr>
<tr>
<td>km 869.500</td>
<td>-5.05</td>
<td>+0.04</td>
</tr>
<tr>
<td>km 870.000</td>
<td>-4.79</td>
<td>-0.03</td>
</tr>
<tr>
<td>km 870.500</td>
<td>-4.65</td>
<td>-0.08</td>
</tr>
<tr>
<td>Minimum navigable width at OLR</td>
<td>158.5</td>
<td>+3.3</td>
</tr>
<tr>
<td>-2.80 m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Table A14  Average bed levels after long term wet period.*
APPENDIX 8A  Calculation Of Head Drop In Rivcom

Rivcom Model Output
Pressure gradients are used in the basic 2DH momentum equations in the rigid lid model formulation in Rivcom. In reality these pressure gradients are a result of the water surface slope, their relation is described by:

\[ i_x = \frac{1}{g} \frac{dp}{dx} \quad (A\ 32) \]

If this equation is integrated along the x axis, the water level drop over an interval follows from:

\[ \Delta z_x = \frac{1}{g} \Delta p \quad (A\ 33) \]

In which

- \( p \) Pressure on rigid lid \( N/m^2 \)
- \( i_x \) Water surface slope along the river -
- \( z_x \) Water surface level \( m \)
- \( g \) Acceleration of gravity \( m/s^2 \)
- \( x \) Spatial coordinates -

The applied scheme of Navier Stokes and continuity equations is solved by Rivcom using the stream function \( \psi \), the velocities \( u \) and \( v \) in x and y direction are calculated from the gradients in \( \psi \). In this formulation the pressure \( p \) is eliminated and therefor no longer available as output.

Pressure Gradient Calculation
In order to deduce the pressure the original 2DH Navier Stokes equations are used, as mentioned in Appendix 3A. Since for a single Rivcom run the boundary conditions are constant it can be assumed that \( \frac{\partial u}{\partial x} \) is negligible in comparison to the other terms in the equation, as it is caused by the changes in bed level only. Furthermore the coriolis term, the viscosity term and influence of the spiral flow intensity on the velocity field are neglected.

The equations for the remaining terms are then rewritten to

\[ \frac{\partial p}{\partial x} = -(u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y}) - \frac{g}{C^2 h} \sqrt{u^2 + v^2} + \chi_x \quad (A\ 34) \]

\[ \frac{\partial p}{\partial y} = -(u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y}) - \frac{g}{C^2 h} \sqrt{u^2 + v^2} + \chi_y \quad (A\ 35) \]

In which

- \( h \) Water depth \( m \)
- \( u \) Velocity in x direction \( m/s \)
- \( v \) Velocity in y direction \( m/s \)
- \( C \) Chézy friction coefficient \( m^{1/2}/s \)
- \( \chi_x \) Influence of the vane forces on the velocity field in x direction \( m/s^2 \)
- \( \chi_y \) Influence of the vane forces on the velocity field in y direction \( m/s^2 \)

The velocity field \( u \) and \( v \), the forces exerted by the vanes and the water depth are available from the model output, therefor the pressure gradients \( \frac{\partial p}{\partial x} \) and \( \frac{\partial p}{\partial y} \) can be calculated. From \( \frac{\partial p}{\partial x} \) the water surface slope \( i_x \) can be calculated using Equation \( (A\ 32) \).
Numerical Approximation

Of course the Rivcom model numerically approximates the solution of the equations, thus values for \( u \) and \( v \) and \( h \) are calculated on a limited number of points on the grid only. From these data the terms in Equation (A.10) are numerically approximated again.

The most important simplification made, is that the curvilinear grid, with which the Rivcom model works, is approximated by a simple quadrangle grid for every grid cell. Of course the differences in cell dimensions between the inner and outer bend are taken into account, but within one cell the flow is assumed to be along a straight line. This approximation is explained in Figure A 159.

In this local approximation a new coordinate system is used with \( i \) and \( j \) coordinates. The \( i \)-axis is parallel to the model grid, positive in upstream direction, while the \( j \)-axis is perpendicular to this axis pointing towards the outside of the main bend. The velocity vectors are transformed to these axis for every grid cell.

It is expected that the errors introduced by neglecting curvilinear terms in the momentum equation are small, especially considering the large number of grid cells and thus the small contortion within one grid cell (maximum \( 6^\circ \), mostly less than \( 2^\circ \)). Furthermore the error introduced by the difference in length between a straight and a curved stream line within a cell is of the order of 0.01%.

Figure A 159 Approximation of curvilinear grid, curvature extremely exaggerated.

Convective Term

In Figure A 157 the numerical approximation of the convective term is explained in the staggered \( u, v \) and \( p \) grid.

Figure A 160 Numerical approximation of convective term.
APPENDIX 8B  Water Level Relative To Downstream Boundary

Figure A 161  Water level drop from downstream model boundary calculated in cross section at km 868.750 at 1000 m³/s

Figure A 162  Water level drop from downstream model boundary calculated in cross section at km 868.750 at 1600 m³/s
Each p-point is located in between four u, v-points. In the convective term \( \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} \), the two gradients are approximated by averaging the gradients on both sides of the p-point, thus for p-point 5

\[
(\frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y})_{5} \approx (u \frac{\partial u}{\partial x})_{5,6} + (v \frac{\partial u}{\partial y})_{5,6} \approx \frac{(u \frac{\partial u}{\partial x})_{5,6} + (u \frac{\partial u}{\partial x})_{5,9}}{2} + \frac{(v \frac{\partial u}{\partial y})_{5,9} + (v \frac{\partial u}{\partial y})_{6,10}}{2}
\]  

(A 36)

**Friction Term**

The friction term is approximated by averaging the terms with velocity and water depth from the four surrounding u, v-points. For p-point 5 the following equation is used.

\[
\left( -\frac{g}{C^2h} u v \sqrt{u^2 + v^2} \right)_{5} \approx \frac{u_5 \sqrt{u_5^2 + v_5^2} + u_6 \sqrt{u_6^2 + v_6^2} + u_{10} \sqrt{u_{10}^2 + v_{10}^2} + u_{11} \sqrt{u_{11}^2 + v_{11}^2}}{4}
\]  

(A 37)

**Vane Forces**

The drag and lift forces exerted by the vane within a cell are first transformed to forces in i and j direction using the coordinates of the grid and the effective angle of attack of the vane. These forces are then distributed over a grid cell.

\[
\chi_{Si} = \frac{F_{Si}}{\Delta x \Delta y h} = \frac{F_{Si}}{2} \left( \frac{x_5 - x_3}{2} + (x_{10} - x_9) \right) + \frac{x_9 - y_5}{2} + (y_{10} - y_6) \frac{h_2 + h_6 + h_9 + h_10}{4}
\]  

(A 38)

**Pressure Drop**

With the convection, friction and vane term the pressure drop between the p-points 5 and 6 in i direction is now calculated as follows.

\[
\Delta p_{56} = (\frac{\partial p}{\partial i})_{x} \Delta i_{56} + (\frac{\partial p}{\partial i})_{y} \Delta i_{56} = (-u_i \frac{\partial u_i}{\partial i} + u_i \frac{\partial u_i}{\partial j})_{56} - \left( \frac{g}{C^2h} u_1 \sqrt{u_1^2 + u_j^2} + \chi_{Si} \Delta i_{56} + \ldots \right)_{56}
\]  

(A 39)

The distances \( \Delta i_{56} \) and \( \Delta i_{56} \) are calculated as explained in Figure A 159.
Figure A 163  Water level drop from downstream model boundary calculated in cross section at km 868.750 at 2000 m³/s

Figure A 164  Water level drop from downstream model boundary calculated in cross section at km 868.750 at 3000 m³/s
Figure A 165 Water level drop from downstream model boundary calculated in cross section at km 868.750 at 4000 m³/s
### APPENDIX 8C  Head Drop Comparison

<table>
<thead>
<tr>
<th>Discharge (m³/s)</th>
<th>Water level relative to downstream model boundary at km 868.750</th>
<th>Increase (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without vanes average range</td>
<td>With Vanes average range</td>
</tr>
<tr>
<td></td>
<td>m</td>
<td>m</td>
</tr>
<tr>
<td>1000</td>
<td>0.3182</td>
<td>0.0160</td>
</tr>
<tr>
<td>1600</td>
<td>0.3364</td>
<td>0.0133</td>
</tr>
<tr>
<td>2000</td>
<td>0.3271</td>
<td>0.0111</td>
</tr>
<tr>
<td>3000</td>
<td>0.3247</td>
<td>0.0056</td>
</tr>
<tr>
<td>4000</td>
<td>0.3606</td>
<td>0.0021</td>
</tr>
</tbody>
</table>

Table A 15  Head drop in cross section at km 868.750, relative to downstream model boundary. Equilibrium situation a long time after the installation of the vanes.

<table>
<thead>
<tr>
<th>Discharge (m³/s)</th>
<th>Water level relative to downstream model boundary at km 867.550</th>
<th>Increase (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without vanes average range</td>
<td>With Vanes average range</td>
</tr>
<tr>
<td></td>
<td>m</td>
<td>m</td>
</tr>
<tr>
<td>1000</td>
<td>0.4303</td>
<td>0.0155</td>
</tr>
<tr>
<td>1600</td>
<td>0.4512</td>
<td>0.0181</td>
</tr>
<tr>
<td>2000</td>
<td>0.4377</td>
<td>0.0192</td>
</tr>
<tr>
<td>3000</td>
<td>0.4334</td>
<td>0.0187</td>
</tr>
<tr>
<td>4000</td>
<td>0.4810</td>
<td>0.0182</td>
</tr>
</tbody>
</table>

Table A 16  Head drop in cross section at km 867.500, relative to downstream model boundary. Equilibrium situation a long time after the installation of the vanes.
APPENDIX 10A  
Fixed Bed Test Series Q

The Q test series conducted by Wang[1991] took place in a 0.9 m wide, 24 m long glass flume. The flume bed was horizontal and covered with coarse sand (4 mm diameter). Transverse near bed velocities were measured at 5 mm above this bed, using an electromagnetic velocity meter, for which the error in reading is less than 0.003 m/s.

Data from test series:
- Geometrical scale related River Waal 1 : 50
- Flow depth \( h = 0.15 \) m
- Mean flow velocity \( u = 0.24 \) m/s
- Chézy value \( C = 55 \) m\(^{1/2}\)/s

<table>
<thead>
<tr>
<th>Parameter Varied</th>
<th>Test Number</th>
<th>Height ( H/H )</th>
<th>Length ( L/H )</th>
<th>Angle ( \text{deg} )</th>
<th>Thickn. ( m )</th>
<th>Shape</th>
<th>Spacing ( \delta/h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height</td>
<td>( Q_1 )</td>
<td>0.25</td>
<td>1.00</td>
<td>20</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>( Q_2 )</td>
<td>0.33</td>
<td>1.00</td>
<td>20</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>( Q_3 )</td>
<td>0.30</td>
<td>1.00</td>
<td>20</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
</tbody>
</table>

Table A17  Summary of data on Q series measurements used in this study.
APPENDIX 10B  Fixed Bed Test Series T

Measurements took place in an 6.0 m wide, 30 m long flume, with a 2 m wide horizontal middle part and gently sloping sides. The bed was covered with uniform gravel, resulting in a roughness height of approximately 15 mm. Velocities were measured using an electromagnetic flow meter, giving readings as close as 20 mm above the bed.

Important data from test series:
- Geometrical scale related River Waal 1 : 25
- Velocity scale 1 : 5
- Maximum flow depth \( h \) = 0.3 m
- Mean flow velocity \( u \) = 0.22 m/s
- Chézy value \( C \) = 47.3 m\(^{1/2} /s\)

<table>
<thead>
<tr>
<th>Parameter Varied</th>
<th>Test Number</th>
<th>Height H/h</th>
<th>Length L/h</th>
<th>Angle deg</th>
<th>Thickness m</th>
<th>Shape</th>
<th>Spacing ( \delta /h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Vane Tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Angle</td>
<td>T1,15</td>
<td>0.33</td>
<td>1.33</td>
<td>10</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,1</td>
<td>0.33</td>
<td>1.33</td>
<td>15</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,14</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,13</td>
<td>0.33</td>
<td>1.33</td>
<td>20</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td>Sizes</td>
<td>T1,13</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,2</td>
<td>0.33</td>
<td>0.66</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,4</td>
<td>0.17</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,5</td>
<td>0.17</td>
<td>0.66</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td>Thickness</td>
<td>T1,14</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,8</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.03</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td>Shape</td>
<td>T1,14</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,16</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Sheet pile 1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,17</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Sheet pile 2</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>T1,18</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Sheet pile 3</td>
<td>-</td>
</tr>
<tr>
<td>Single Transverse Row</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing</td>
<td>T2,2</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>T2,7</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>T2,5</td>
<td>0.33</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td>T2,8</td>
<td>0.17</td>
<td>1.33</td>
<td>18</td>
<td>0.01</td>
<td>Flat plate</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table A 18  Summary of data on T series measurements used in this study.
APPENDIX 10C  Fixed Bed Test Series ZG

The Zandgoot is a long 1.5 m wide flume. In this series again measurements were made using an electromagnetic flow velocity meter. The most important data used are the near-bed velocities taken at 20 mm above the bed, which was covered with gravel with a grain diameter between 10 and 20 mm.

Important data from test series:
- Geometrical scale related River Waal 1 : 15
- Velocity scale 1 : 3.9
- Flow depth h = 0.5 m
- Mean flow velocity u = 0.36 m/s
- Chézy value C = 45 m$^{1/2}$/s

<table>
<thead>
<tr>
<th>Parameter Varied</th>
<th>Test Number</th>
<th>Height H/h</th>
<th>Length L/h</th>
<th>Angle deg</th>
<th>Thickness m</th>
<th>Shape</th>
<th>Spacing δ/h</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sizes</td>
<td>ZG₁</td>
<td>0.20</td>
<td>1.33</td>
<td>17.5</td>
<td>0.024</td>
<td>Sheet pile</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>ZG₂</td>
<td>0.40</td>
<td>1.33</td>
<td>17.5</td>
<td>0.024</td>
<td>Sheet pile</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>ZG₃</td>
<td>0.40</td>
<td>0.71</td>
<td>17.5</td>
<td>0.024</td>
<td>Sheet pile</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>ZG₄</td>
<td>0.20</td>
<td>0.71</td>
<td>17.5</td>
<td>0.024</td>
<td>Sheet pile</td>
<td>-</td>
</tr>
<tr>
<td>Top plate</td>
<td>ZG₂</td>
<td>0.20</td>
<td>1.33</td>
<td>17.5</td>
<td>0.024</td>
<td>Sheet pile</td>
<td>top plate</td>
</tr>
</tbody>
</table>

Table A 19  Summary of data on ZG series measurements used in this study.
## APPENDIX 10D  Index Of Mobile Bed Tests With Arrays

<table>
<thead>
<tr>
<th>Test</th>
<th>Depth</th>
<th>Vel.</th>
<th>Sed. Diam.</th>
<th>h-H</th>
<th>L</th>
<th>α</th>
<th>Int. δ</th>
<th>n</th>
<th>Int. γ</th>
<th>n</th>
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<tr>
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<td>m</td>
<td>m/s</td>
<td>mm</td>
<td>m</td>
<td>m</td>
<td>deg</td>
<td>m</td>
<td></td>
<td>m</td>
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<td>0.425</td>
<td>0.78</td>
<td>0.106</td>
<td>0.16</td>
<td>15</td>
<td>0.20</td>
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<tr>
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<td>0.78</td>
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<td>0.18</td>
<td>4</td>
<td>0.92</td>
<td>2</td>
</tr>
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<td>0.41</td>
<td>0.104</td>
<td>0.152</td>
<td>15</td>
<td>0.18</td>
<td>3</td>
<td>0.95</td>
<td>23</td>
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<tr>
<td>Wa91b S</td>
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<td>0.405</td>
<td>0.41</td>
<td>0.108</td>
<td>0.152</td>
<td>20</td>
<td>0.18</td>
<td>3</td>
<td>0.95</td>
<td>14</td>
</tr>
</tbody>
</table>

**Table A 20**: Summary of data on mobile bed tests with arrays of vanes.
Figure A.166  Bed profile measured in cross sections in mobile bed scale model tests Q98 T8 (Deft Hydraulics[1987]). T2 situation without vanes, T8 arrays with 4 vanes interval 0.75 m.

Figure A.167  Bed profile measured in cross sections in mobile bed scale model tests Q98 T7 (Deft Hydraulics[1987]). T2 situation without vanes, T8 arrays with 4 vanes interval 0.75 m.
APPENDIX 10F  Longitudinal Sections In Q98 T7 And T8

Figure A 168  Longitudinal sections at 0.15 B, 0.5 B and 0.85 B, measured in reference test T2 without vanes *(Delft Hydraulics [1987]).

Figure A 169  Longitudinal sections at 0.15 B, 0.5 B and 0.85 B, measured in test T7 with vanes *(Delft Hydraulics [1987]).

Figure A 170  Layout of test T7, test T8 is identical except for a double distance of the vane arrays *(Delft Hydraulics [1987]).

Figure A 171  Longitudinal sections at 0.15 B, 0.5 B and 0.85 B, measured in test T8 with vanes, double interval *(Delft Hydraulics [1987]).
APPENDIX 11A  Wang Theory Basic Equations

Equations of Motion
Assuming an axial symmetric \( \frac{\partial}{\partial z} = 0 \) and steady state \( \frac{\partial}{\partial t} = 0 \) situation, the equations of motion in longitudinal and lateral direction for a horizontal layer of water can be reduced to:

\[
\rho g \frac{\partial z_w}{\partial t} + \frac{\partial z_w}{\partial x} - \frac{\partial z_n}{\partial n} + f_{vn} = 0 \quad \text{with} \quad i_v = -\frac{\partial z_w}{\partial n} \tag{A 40}
\]

\[
\rho g \frac{\partial z_w}{\partial n} - \frac{\partial z_n}{\partial z} - \rho \frac{U^2}{r} + f_{vn} = 0 \quad \text{with} \quad i_w = \frac{\partial z_w}{\partial n} \tag{A 41}
\]

Here the terms \( f_{vn} \) and \( f_{vn} \) introduce the forces exerted by the submerged vanes in longitudinal and lateral direction.

These equations are integrated vertically to:

\[
\rho g h_i = \tau_{xi} + \tau_{yi} \tag{A 42}
\]

\[
\rho g h_n = \tau_{xn} - \tau_{yn} - \rho \frac{hU^2}{r} \tag{A 43}
\]

If Equation (A2) is evaluated at the water surface and subtracted from Equation (A4), the lateral water surface slope is eliminated:

\[
\rho \frac{h}{r} \left( u_{2, h}^2 - U^2 \right) - \tau_{yn} + \tau_{vn} + \left( \frac{\partial z_n}{\partial n} \right)_{z=h} = 0 \tag{A 44}
\]

Velocity Profiles
The vertical profile for the longitudinal velocities is approximated with:

\[
u = U \left( \frac{z}{h} \right)^{m+1/m} \tag{A 45}
\]

A linear vertical distribution of the longitudinal shear stress leads to an expression for the vertical distribution of the eddy viscosity using \( \tau_e = \rho e \frac{\partial u}{\partial z} \):

\[
\varepsilon = \frac{m^2}{m+1} \frac{h}{\rho U} \tau_{sh} \left( \frac{z}{h} \right)^{m+1/m} \left( \frac{z}{h} \right)^{1-1/m} \tag{A 46}
\]

This eddy viscosity is assumed to be isotropic for the longitudinal and lateral direction.

The linear profile for the transverse velocities is a good approximation of the theoretical profile that would be consistent with the longitudinal velocity profile.

\[
\nu = 2(\nu_b - \nu) \left( \frac{z}{h} - \frac{1}{2} \right) \tag{A 47}
\]

It is important to keep in mind that in the theoretical profile the transverse near-bed velocities are reduced because of the bed friction.

Bed Shear Stresses
If Equations (A 46) and (A 47) are used in \( \tau_n = \rho e \frac{\partial v}{\partial z} \) the following expression is obtained for the vertical distribution of the lateral shear stress:

\[
\tau_n = 2 \frac{m^2}{m+1} \left( \frac{\nu_b - \nu}{U} \right) \tau_{sh} \left( \frac{z}{h} \right) \left( \frac{z}{h} \right)^{1-1/m} \left( \frac{z}{h} \right)^{2-1/m} \tag{A 48}
\]

This function is differentiated with respect to \( z \) and evaluated at the surface, using

\[
\tau_{bs} = \rho U^2 \kappa^2 \left( \frac{m+1}{m} \right)^2 \tag{A 48}
\]
\[
\left( \frac{\partial \tau_{bn}}{\partial z} \right)_{x=h} = -2 \frac{m^2}{m^2 + k} \left( \frac{\rho k^2 u}{d} (\nu_b - \nu_n) \right)
\]  \hspace{2cm} (A 49)

Using \( \tau_{bn} = \nu_{bn} \), \( \tau_{bn} = \nu_{bn} \) and \( u_b = \frac{U}{k} \), this is written as:

\[
\left( \frac{\partial \tau_{bn}}{\partial z} \right)_{x=h} = -2 \frac{m^2}{m^2 + k} \frac{\rho k^2 u}{d} \left( \frac{\tau_{bn}}{k} - \frac{\tau_{vn}}{k} \right)
\]  \hspace{2cm} (A 50)

Now Equations (A 45) and (A 50) are inserted into Equation (A 44), resulting in:

\[
\tau_{bn} = -\frac{\rho k(2m + 1)(m + 1)}{m^2(2m^2 + k(m + 1))} \frac{h u^2}{r} - \tau_{vn}
\]  \hspace{2cm} (A 51)

**Bed Level Description**

Based on a lateral balance of forces on a bed particle, Odgaard [1981] relates the lateral bed shear stress to the bed slope in the same direction using the following equation:

\[
\frac{dh}{dn} = -\frac{m}{\rho c u_w \sqrt{g \rho \Delta g D}} \tau_{bn}
\]  \hspace{2cm} (A 52)

**Continuity Equation**

The averaged vane drag forces are taken into account in the Chézy equation as a reduction of the effective longitudinal water surface slope. Fulfillment of flow continuity is then described by:

\[
Q = \int_B hU \, dn = \int_B hC \sqrt{\frac{h_i}{h_{in}}} \frac{\tau_{bn}}{\rho g} \, dn
\]  \hspace{2cm} (A 53)
APPENDIX 11B  Wang Vane Description

The water motion induced by the vanes is described in a theoretical way for the vane tip vortex, which is calculated based on a finite wing theory. In order to take into account the influence of the water surface and river bed an infinite number of mirror images of this vortex are used. The resulting near bed transverse velocities are given by:

\[
v_{vn} = \frac{F_l}{\pi \rho u_v H} \sum_{j=1}^{\infty} \frac{(-1)^{j+1}}{\eta_j} \left[ 1 - \exp \left( \frac{-U}{4 \pi \eta_j} \right) \right] \frac{z_j}{\eta_j}
\]  

(A 54)

In this equation the distance from the vortex core can be written as \( \eta_j = \sqrt{\eta_j^2 + \gamma^2} \). The vertical distance from the bed level to the main vortex and the most important first 5 mirror images is given with:

\[
\begin{align*}
1 & : H \\
2 & : 2h-H \\
3 & : 2h+H \\
4 & : 2h+2H \\
5 & : 2h+3H \\
\end{align*}
\]

The lateral bed shear stress induced by these transverse near-bed velocities is calculated, using \( \tau_{vn} = \frac{v_{vn}}{u_b} \) and \( u_b = \frac{U}{k} \) and \( \tau_{bh} = \rho U^2 \kappa \left( \frac{m+1}{m^2} \right)^2 \). This results in:

\[
\tau_{vn} = \frac{F_l k \kappa \left( \frac{h}{H} \right)^{1/m} \left( \frac{m+1}{m^2} \right)^2 \sum_{j=1}^{\infty} \frac{(-1)^{j+1}}{\eta_j} \left[ 1 - \exp \left( \frac{-U}{4 \pi \eta_j} \right) \right] \frac{z_j}{\eta_j}}{u_b}
\]  

(A 55)

Furthermore the streamwise component of the vortex induced bed shear stresses is described with:

\[
\tau_{sv} = \frac{F_d}{F_l} \tau_{vn} = \frac{F_d k \kappa \left( \frac{h}{H} \right)^{1/m} \left( \frac{m+1}{m^2} \right)^2 \sum_{j=1}^{\infty} \frac{(-1)^{j+1}}{\eta_j} \left[ 1 - \exp \left( \frac{-U}{4 \pi \eta_j} \right) \right] \frac{z_j}{\eta_j}}{u_b}
\]  

(A 56)

The lift and drag forces exerted by the vanes are calculated with:

\[
F_L = \frac{1}{2} c_l \rho H \int_0^H u^2 dz = \frac{1}{2} c_l \rho H L \left( \frac{m+1}{m(m+2)} \right)^{2/n} \left( \frac{H}{h} \right)^{2/n}
\]  

(A 57)

\[
F_D = \frac{1}{2} c_D \rho H \int_0^H u^2 dz = c_D \frac{F_l}{c_l}
\]  

(A 58)

An idealized vertical distribution of the vertical circulation around the vane is assumed, which is maximum at the bed level and zero at the vane tip. This results in:

\[
c_l = \frac{2 \pi \alpha}{1 + \frac{H}{L}}
\]  

(A 59)

\[
c_D = \frac{1 - \frac{L}{2 \pi H}}{\alpha}
\]  

(A 60)

In the model a field is assumed consisting of equal-sized, equal-spaced vanes. The contributions to the horizontal circulation from all vanes are added, taking into account a reduction factor \( \lambda \), because of interaction of the individual tip vortices. According to wing theory a vertical circulation develops around each vane, reducing the effective angle of incidence of the vane. If vanes are close to each other the effective angle of incidence is further reduced by the circulation from neighboring vanes. This phenomenon is described by Wang resulting in the following expression for the reduction factor \( \lambda \):
\[ \lambda = \frac{1 + 2\mu}{1 + (1 + 2\sigma_1)^2 + 2\sigma_2} \quad (A \, 61) \]

While
\[ \sigma_1 = \frac{1}{\pi} \int_0^1 \frac{\eta^2}{\left((\delta_n/L)^2 + \eta^2\right)^{1/2}} d\eta \quad (A \, 62) \]
\[ \sigma_2 = \frac{1}{1 + 4(\delta_n/L)^2} \quad (A \, 63) \]

This results in the graph plotted in Figure A 172.

![Graph](image)

**Figure A 172** Interaction factor \( \lambda \) as a function of transverse vane spacing (Wang [1991]).
APPENDIX 11C  Vane Description Vs Measurements

<table>
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<th>Test</th>
<th>Relative transverse near-bed discharge</th>
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<td>Ti,2</td>
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<tr>
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<td>ZG2</td>
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<td>ZG3</td>
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<tr>
<td>Array of Flat Plates</td>
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<tr>
<td>x (m)</td>
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<td>0.81</td>
</tr>
<tr>
<td>T12</td>
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<tr>
<td>T13</td>
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<tr>
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<td>1.06</td>
</tr>
<tr>
<td>T18</td>
<td>0.76</td>
<td>1.15</td>
</tr>
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Table A 21  Transverse near-bed flow per unit length John predicted with Wang theory relative to those from measurements in scale model tests, together with total vane-induced transverse near-bed flow John.
APPENDIX 12A  Basic Equations In Rivcom

General Description
The 2DH Rivcom model was developed in the early eighties as a result of a cooperation of the Dutch Delft Hydraulics Laboratory and the French Laboratoire National d’Hydraulique in Chatou. The model has the following general properties:

- Time dependent Navier Stokes equations are numerically solved in a 2 dimensional curvilinear grid. In this case 2 dimensional vertically averaged equations are used, in which a hydrostatic pressure distribution is assumed.
- Equations are solved using a rigid lid approximation, which is justified by the low Froude number.
- Spiral flow intensity is calculated based on the velocity field and the stream line curvature.
- With the velocity and spiral flow fields the sediment transport and subsequent changes in bed topography are calculated, using a local sediment balance.
- Constant boundary conditions.

Modelling of Vanes
The influence of the vanes is taken into account both in the momentum and in the spiral flow intensity equation. The lift and drag forces are calculated using the velocity vector averaged over the vane height, calculated from the average velocity field and the spiral flow intensity field. These are added to the bed friction in the grid cell within which the vane is located.
Secondly a vortex generated by the tip of the vane is added to the spiral flow intensity in the particular grid cell. This extra intensity is damped relatively fast downstream of the vane compared to the natural spiral flow intensity generated by the curved stream lines.

Basic Equations
The theory behind Rivcom is based on the Navier Stokes momentum equations, which describe the velocity field for this 2DH case in orthogonal coordinates averaged over the depth:

\[
\frac{\partial u}{\partial t} + (u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y}) = - \frac{\partial p}{\partial x} + \frac{1}{\rho} \frac{\partial}{\partial x} \left( \frac{1}{2} \rho (u^2 + v^2) \right) + f v - \frac{g}{C^2 h} \sqrt{u^2 + v^2} + \gamma_x \tag{A 64}
\]

\[
\frac{\partial v}{\partial t} + (u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y}) = - \frac{\partial p}{\partial y} + \frac{1}{\rho} \frac{\partial}{\partial y} \left( \frac{1}{2} \rho (u^2 + v^2) \right) - f u - \frac{g}{C^2 h} \sqrt{u^2 + v^2} + \gamma_y \tag{A 65}
\]

Furthermore the continuity equation:

\[
\frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = 0 \tag{A 66}
\]

Engelund Hansen formula is used for the calculation of the sediment transport:

\[
S_b = \alpha \frac{0.084}{\sqrt{gC} A D} u^3 \tag{A 67}
\]

Direction of this transport follows from the following equation:

\[
\tan \delta = \frac{v}{u} = E_1 A \frac{I}{U} - 1.18 \frac{E_2 \partial z_b}{\sqrt{g} \partial n} \tag{A 68}
\]

Sediment continuity equation:

\[
\frac{\partial z_b}{\partial t} + \frac{\partial S_b}{\partial x} + \frac{\partial S_y}{\partial y} = 0 \tag{A 69}
\]

Finally the spiral flow intensity in calculated based on the following formula:

\[
\beta \frac{C}{\sqrt{g}} \frac{h}{s} \frac{\partial h}{\partial y} + 1 = \frac{h}{R} \sqrt{u^2 + v^2} \tag{A 70}
\]

The influence of the vanes is added to this intensity.

In which

- $C$  Chézy friction coefficient \[m^{1/2}/s\]
- $E_1, E_2$  Coefficients in sediment direction equation
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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</tr>
<tr>
<td>g</td>
<td>Acceleration of gravity</td>
<td>m/s²</td>
</tr>
<tr>
<td>h</td>
<td>Water depth</td>
<td>m</td>
</tr>
<tr>
<td>I</td>
<td>Spiral flow intensity</td>
<td>m/s</td>
</tr>
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<td>p</td>
<td>Pressure on rigid lid</td>
<td>N/m²</td>
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<td>R</td>
<td>Local radius of curvature of stream line</td>
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<td>s, n</td>
<td>Spatial coordinates relative to stream line</td>
<td>m</td>
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<td>Bulk sediment transport per unit width in x and y</td>
<td>m³/s/m</td>
</tr>
<tr>
<td>t</td>
<td>Time</td>
<td>s</td>
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<tr>
<td>x, y</td>
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<td>u</td>
<td>Velocity in x direction</td>
<td>m/s</td>
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<tr>
<td>v</td>
<td>Velocity in y direction</td>
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<tr>
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</tr>
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<td>Sediment transport correction factor</td>
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</tr>
<tr>
<td>β</td>
<td>Coefficient in equation for spiral flow intensity</td>
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</tr>
<tr>
<td>δ</td>
<td>Direction of sediment transport with x axis</td>
<td>rad</td>
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<tr>
<td>ν</td>
<td>Turbulence viscosity</td>
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<tr>
<td>(\chi_x)</td>
<td>Influence of the vanes and spiral flow intensity</td>
<td>m/s²</td>
</tr>
<tr>
<td>(\chi_y)</td>
<td>on the velocity field in x direction</td>
<td>m/s²</td>
</tr>
</tbody>
</table>
APPENDIX 12B  Lateral Bed Slope In Rivcom

In Rivcom the 2D vertically averaged equations of motion are solved, which describe the average velocity vector \((u, v)\) for each grid cell. On top of that a spiral flow intensity \(I\) is calculated describing the secondary flow, which is defined as \(I = \frac{1}{2} \sqrt{\psi}\). In case submerged vanes are included in the model, the spiral stream intensity is composed of two components:

\[
I = I_s + I_v
\]

(A 71)

\(I_s\) models the natural spiral flow intensity in the river bend, its description is based on is described by:

\[
I_s = \frac{h}{R \sqrt{u^2 + v^2}} \beta \frac{C}{g} \frac{h}{\sqrt{g}} \frac{\partial \xi}{\partial n}
\]

(A 72)

The two main terms in this formula describe the following influences:

- Source term, generation of spiral motion as a result of local streamline curvature.
- Exponential damping of spiral motion along river axis.

\(I_v\) describes the spiral flow corresponding to the lateral near bed velocities generated by the vanes. This is explained in Appendix 12 B.

Engelund-Hansen formula is used in this case to describe the magnitude of the sediment transport, based on the magnitude of the vertically averaged velocity. The heading of this transport is determined by three factors, which are included in the following formula:

- Heading of the velocity vector.
- Spiral flow intensity, which induces lateral near-bed velocities.
- Lateral bed slope.

\[
\tan \phi = \frac{v}{u} - E_1 A \frac{I}{U} - 1.18 \frac{E_2}{\sqrt{g}} \frac{\partial \xi}{\partial n}
\]

(A 73)

In which

\[
A = \frac{2}{\kappa^2} \left(1 - \frac{\sqrt{g}}{\kappa C}\right)
\]

\[
E_1 = 1.33 \text{ Calibration coefficient}
\]

\[
E_2 = 1.00 \text{ Calibration coefficient}
\]

\[
\theta = -\frac{U^2}{C^2 \Delta D}
\]

\[
\phi \quad \text{Heading of sediment transport rad}
\]

The two calibration coefficients \(E_1\) and \(E_2\) were calibrated using the existing situation in the River Waal in the bend at Hulhuizen, thus the non steady situation. As the overshoot in transverse bed slope is reproduced more or less correctly, it is reasonable to assume that, in case the bend would be semi-infinite, the equilibrium cross section in an axi-symmetric situation is described correctly as well.

Axi-Symmetric Approximation

An axi symmetric approximation of the bed level can be obtained with these same equations. In this case all derivatives with respect to \(s\) are 0, depth averaged transverse velocity \(v = 0\), while the sediment transport will be just in streamwise direction, therefore \(\tan \phi = 0\). A combination of equations (A 73) and (A 72) now leads to:

\[
\frac{\partial \xi}{\partial n} = 0.85 \frac{E_1}{E_2} \frac{2}{\kappa^2} \left(1 - \frac{\sqrt{g}}{\kappa C}\right) \sqrt{\theta} \frac{h}{R}
\]

(A 74)
APPENDIX 12C  Description Of Vanes In Rivcom

The influence of submerged vanes are included in two parts of Rivcom. In the first place the generated lateral near bed velocities are included by an additional term in the spiral flow intensity. Secondly the lift and drag forces are averaged over the grid cell, within which the vane is located, and taken into account as extra contribution to the bed shear stress. Wang’s theory, as described in Appendix 13B, has been the basis for the model description.

Spiral Flow Contribution of Vanes
The contribution of the submerged vanes to the spiral flow intensity is based on the near bed velocities induced by the tip vortex. This vortex is assumed to be contained by the chain of curvilinear grid cells downstream of the one in which the vane is located.

For a cell in this chain (each approximately 5 m wide and 25 m long in the River Waal model) the peak value of the transverse near bed velocity is calculated with equation (A 81) in a cross section at a downstream distance from the vane of the center of the particular cell. Based on this peak transverse near bed velocity an average transverse near bed velocity over the Rivcom cell is calculated with:

\[
\overline{v_{vb}} = v_{vb,p} \frac{\Lambda_c}{\Lambda_e} = v_{vb,p} \frac{\psi h \Delta l}{\Lambda_e}
\]  

(A 75)

In which
- \( \Lambda_e \): Plan surface of a Rivcom grid cell in m²
- \( \Lambda_v \): Bed surface assumed to be influenced by a vane in m²
- \( h \): Water depth in m
- \( \Delta l \): Streamwise length of a Rivcom grid cell in m
- \( v_{vb,p} \): Peak value of lateral near bed velocity induced by vane in cross section.
- \( \psi \): Calibration coefficient for integration

Calibration of the model on the mobile bed scale model test Q98 T7, mentioned in Paragraph 3.5, led to a value of \( \psi = 1.6/\pi \).

The spiral flow intensity corresponding to this the cell average transverse velocity is given by:

\[
I_c = \frac{2m k^2}{3m - 6} \overline{v_{vb}} = \frac{2m k^2}{3m - 6} \frac{\Lambda_v}{\Lambda_e} v_{vb,p}
\]  

(A 76)

Lateral Near Bed Velocities
Descriptions for the lift and drag forces exerted by a vane are adopted from Wang:

\[
F_L = \frac{1}{2} \rho C_L H_v L (\overline{v_e}^2 + \overline{v_v}^2) = \frac{1}{2} \rho \frac{2 \pi a_x}{1 + \frac{L}{H_v}} H_v L (\overline{v_e}^2 + \overline{v_v}^2)
\]  

(A 77)

\[
F_D = \frac{1}{2} \rho C_D H_v L (\overline{v_e}^2 + \overline{v_v}^2) = \frac{1}{2} \rho \left( \frac{2 \pi a_y}{1 + \frac{L}{H_v}} ight) L^2 (\overline{v_e}^2 + \overline{v_v}^2)
\]  

(A 78)

In the effective angle of attack of the vanes the lateral velocities just upstream of the vanes are taken into account in \( \alpha_v = \alpha_v - \gamma \). The angle \( \gamma \) describes the deviation of the velocity vector from the longitudinal axis just upstream from the vane:

\[
\gamma = \arctan \left( \frac{u_v \sin \beta + v_v \cos \beta}{u_v \cos \beta - v_v \sin \beta} \right)
\]  

(A 79)

In which
- \( \beta \): Angle between velocity vector averaged over vane height and streamwise coordinate.
These lift forces are linked to the near bed lateral velocities generated behind the vane by:

\[ v = \frac{F_b}{\rho U H} \left( \frac{1}{1 + \left( \frac{y - \mu}{\delta H} \right)^2} \left( 1 - \exp \left( -\frac{\lambda_v}{x} \right) \right) \right) \]  \hspace{1cm} (A 80)

The first part of this formula gives a value for the maximum lateral velocity occurring just behind the vane, it is based on the theory developed by Wang [1991], elucidated in Appendix 2B. The second factor describes a gauss-like transverse distribution of the velocities which was introduced by Struikisma and De Groot [1996]. Finally the third factor describes the damping of the velocities in longitudinal direction. This distribution of the velocities behind the vanes was based on measurements.

Finally the factor \( \alpha_v \) is replaced by \( \tan \alpha_v \), the resulting formula reads:

\[ v = \frac{t \left( \frac{H}{h} \right) \tan \alpha_v}{\left( 1 + \frac{H}{L} \right) \delta} \left( \frac{1}{1 + \left( \frac{y - \mu}{\delta H} \right)^2} \left( 1 - \exp \left( -\frac{\lambda_v}{x} \right) \right) \right) \]  \hspace{1cm} (A 81)

The length scale used for the damping is:

\[ \lambda_v = \frac{3}{2k} \left( \frac{H}{h} \right)^2 \]  \hspace{1cm} (A 82)

**Contribution In Bed Friction**

Finally the vane forces are averaged over the area of the Rivcom grid cell within which the vane is located. The resulting shear stresses are added in the 2DH equations that describe the depth averaged water motion.

\[ \tau_{vn} = -\frac{F_{nv} \cos \alpha_v}{\Lambda_R} - \frac{F_{nv} \sin \alpha_v}{\Lambda_R} \]  \hspace{1cm} (A 83)

\[ \tau_{vn} = -\frac{F_{n} \sin \alpha_v}{\Lambda_R} + \frac{F_{v} \cos \alpha_v}{\Lambda_R} \]  \hspace{1cm} (A 84)
APPENDIX 12D  Single Vane Test $T_{1,14}$ Vs Rivcom Equation

Figure A 173 Transverse near-bed velocities measured in test $T_{1,14}$ in cross sections at 0.05 m, 0.81 m and 1.81 m downstream from single vane ($H=0.1$ m, $L=0.4$ m, $\alpha=18^\circ$) together with prediction by Rivcom equation (1/2)
Figure A.174 Transverse near-bed velocities measured in test T1,4 in cross sections at 2.81 m and 3.81 m downstream from single vane (H=0.1 m, L=0.4 m, $\alpha=18^\circ$) together with prediction by Rivcom equation (2/2)
Figure A 175 Transverse near-bed velocities measured in test $T_{12}$ in cross sections at 0.05 m, 0.81 m and 1.81 m downstream from array of vanes ($H=0.1$ m, $L=0.4$ m, $\alpha=18^\circ$) with 0.20m spacing together with prediction by Rivcom equation (1/2)
Figure A 176 Transverse near-bed velocities measured in test $T_{12}$ in cross sections at 2.81 m, 3.81 m and 5.81 m downstream from array of vanes ($H=0.1$ m, $L=0.4$ m, $\alpha=18^\circ$) with 0.20m spacing together with prediction by Rivcom equation (2/2)
Figure A.177 Transverse near-bed velocities measured in test T27 in cross sections at 0.05 m, 0.81 m and 1.81 m downstream from array of vanes (H=0.1 m, L=0.4 m, α=18°) with 0.30 m spacing together with prediction by Rivcom equation (1/2)
Figure A.178 Transverse near-bed velocities measured in test T_{c2} in cross sections at 2.81 m, 3.81 m and 5.81 m downstream from array of vanes (H=0.1 m, L=0.4 m, α=18°) with 0.30 m spacing together with prediction by Rivcom equation (2/2)
Figure A179 Transverse near-bed velocities measured in test \( T_{0.7} \) in cross sections at 0.05 m, 0.81 m and 1.81 m downstream from array of vanes (\( H=0.1 \) m, \( L=0.4 \) m, \( \alpha=18^\circ \)) with 0.40m spacing together with prediction by Rivcom equation (1/2)
Figure A180 Transverse near-bed velocities measured in test T17 in cross sections at 2.81 m, 3.81 m and 5.81 m downstream from array of vanes (H=0.1 m, L=0.4 m, α=18°) with 0.40m spacing together with prediction by Rivcom equation (2/2)
APPENDIX 12H  Single Vane Test T14 Vs Rivcom Equation

Figure A 181 Transverse near-bed velocities measured in test T14 in cross sections at 0.05 m, 0.81 m and 1.81 m downstream from single vane (H=0.05 m, L=0.4 m, α=18°) together with prediction by Rivcom equation (1/2)
Figure A 182  Transverse near-bed velocities measured in test T1,4 in cross sections at 2.81 m and 3.81 m downstream from single vane (H=0.05 m, L=0.4 m, α=18°) together with prediction by Rivcom equation (2/2)
APPENDIX 12I Array Vanes 0.30 m Spacing Vs Rivcom Equation

Figure A.183 Transverse near-bed velocities measured in test T_{gb} in cross sections at 0.05 m, 0.81 m and 1.81 m downstream from array of vanes (H=0.05 m, L=0.4 m, α=18°) with 0.30 m spacing together with prediction by Rivcom equation (1/2)
Figure A.184 Transverse near-bed velocities measured in test $T_n$ in cross sections at 2.81 m, 3.81 m and 5.81 m downstream from array of vanes ($H=0.05$ m, $L=0.4$ m, $\alpha=18^\circ$) with 0.30 m spacing together with prediction by Rivecom equation (2/2)
APPENDIX 12J  Rivcom Vane Equation Vs Measurements

<table>
<thead>
<tr>
<th>Test</th>
<th>Transverse near-bed flow per unit length in cross section</th>
<th>Total transverse near-bed flow</th>
</tr>
</thead>
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**Flat Plate**

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<th>1.81</th>
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<tbody>
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<td>1.11</td>
</tr>
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<td>1.17</td>
<td>0.75</td>
<td>0.87</td>
</tr>
<tr>
<td>T1,13</td>
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<td>1.58</td>
</tr>
<tr>
<td>T1,14</td>
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<td>0.95</td>
<td>1.03</td>
<td>0.94</td>
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<tr>
<td>T1,15</td>
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<td>0.83</td>
<td>0.72</td>
<td>0.77</td>
<td>1.03</td>
</tr>
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</table>

| Q1   | 1.23 | 1.56 | 2.10 | 1.64 | 3.60 |
| Q2   | 1.20 | 1.59 | 2.61 | 2.73 | 3.04 |
| Q3   | 1.08 | 1.42 | 2.19 | 2.45 | 2.80 |

**Sheet Pile**

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**Array of Flat Plates**

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<th>3.81</th>
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<td>1.16</td>
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</tr>
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</table>

Table A 22: Transverse near-bed flow per unit length $J_{bd, Rv}$ in each cross section and total transverse near-bed flow $J_{bd, Rv}$ induced by the vane. Value predicted by Rivcom equation (5.1) relative to those from scale model measurements.
### APPENDIX 12K  Rivcom Vane Integration Vs Measurements

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<th>Transverse near-bed flow per unit length in cross section</th>
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<td>$\frac{\text{John Revit}}{\text{John measured}}$</td>
<td>$\frac{\text{John Revit}}{\text{John measured}}$</td>
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<tr>
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<tr>
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<tr>
<td>$T_{133}$</td>
<td>0.60 0.60 0.78 0.79 2.31</td>
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<td>$T_{134}$</td>
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<td><strong>Array of Flat Plates</strong></td>
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<td></td>
</tr>
<tr>
<td>$x (m)$</td>
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<tr>
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<tr>
<td>$T_{18}$</td>
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Table A 23  Transverse near-bed flow per unit length $\frac{\text{John Revit}}{\text{John measured}}$ in each cross section and total transverse near-bed flow $\frac{\text{John Revit}}{\text{John measured}}$ induced by the vane. Value predicted by Rivcom integration equation (5.2) relative to those from scale model measurements.
## APPENDIX 13A  Enhanced Equation Vs Measurements

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### Array of Flat Plates

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<tr>
<td>T1,7</td>
<td>1.48 1.76 2.15 2.61 2.11 1.83</td>
<td>1.97</td>
</tr>
<tr>
<td>T1,8</td>
<td>0.89 1.89 4.62 4.18 4.89 5.46</td>
<td>2.68</td>
</tr>
</tbody>
</table>

**Table A 24** Transverse near-bed flow per unit length $j_{en}$ in each cross section and total transverse near-bed flow $j_{en}$ induced by the vane. Value predicted by the enhanced equations (4.10) and (4.11) relative to those from scale model measurements.
## APPENDIX 13B  Array Reduction Factors In Enhanced Equation

<table>
<thead>
<tr>
<th>Vane Array Tests</th>
<th>Reduction factors for $v_{exp}(x=0)$</th>
<th>B</th>
<th>$\lambda_v$</th>
<th>Transv. near-bed flow per unit length in cr. section $J_{meas}/J_{measured}$</th>
<th>Total transv. flow $J_{meas}/J_{measured}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$x$ [m]</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>spacing</td>
<td>0.05</td>
<td>0.81</td>
<td>1.81</td>
<td>2.81</td>
<td>3.81</td>
</tr>
<tr>
<td>$T_{0.2}$ 0.7 h</td>
<td>0.6</td>
<td>1.0</td>
<td>0.6</td>
<td>0.97</td>
<td>1.15</td>
</tr>
<tr>
<td>$T_{0.7}$ 1.0 h</td>
<td>0.7</td>
<td>1.0</td>
<td>0.6</td>
<td>1.00</td>
<td>1.17</td>
</tr>
<tr>
<td>$T_{0.3}$ 1.3 h</td>
<td>0.7</td>
<td>0.9</td>
<td>0.3</td>
<td>1.00</td>
<td>1.04</td>
</tr>
<tr>
<td>$T_{0.8}$ 1.0 h</td>
<td>1.2</td>
<td>0.9</td>
<td>0.2</td>
<td>0.96</td>
<td>1.17</td>
</tr>
</tbody>
</table>

**Table A 25** Reduction factors added to the enhanced equation (13.8) in the initial transverse near-bed velocity peak, the width of the velocity profile and the damping length. Tests in $h = 0.30$ m water depth, $T_{0.2}$, $T_{0.3}$ and $T_{0.7}$ with vane dimensions $H=0.10$ m, $L=0.40$ m and $\alpha=18^\circ$ (similar to $T_{14}$). Test $T_{0.8}$ with dimensions $H=0.05$ m, $L=0.40$ m and $\alpha=18^\circ$ (similar to $T_{14}$).