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DREDGING ALTERNATIVES – THE CURRENT DEFLECTION WALL MINIMIZING DREDGING ACTIVITIES IN HARBOURS

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Abstract: Siltation is a permanent problem in harbours at tidal rivers and in estuaries causing continuous maintenance dredging to guarantee safe navigation. Costs can increase exponentially if cohesive sediments and contaminations are involved and deposition of dredged materials on landfills is necessary.

To prevent sedimentation of fine materials entering a harbour, a Current Deflecting Wall (CDW) can be used, which passively alters the water exchange. A sill at the bottom deflects the near-bed density currents away from the entrance. As a result the exchange during rising tide originates from the upper layer, which has a reduced density and containing less sediment.

A density difference along the harbour entrance is the driving force for density driven currents into the harbour. Velocities show strong directional changes over time and depth with a three-dimensional flow pattern into the harbour. Density currents cause opposite flows in the upper and lower layers. The resulting in/out boundary layer is varying over the tidal cycle. Currents are directed into the harbour over the bed from the first half of rising tide to the first half of falling tide.

A numerical Case Study "Bremerhaven" was designed to investigate the effects of a CDW on water exchange and turbulent mixing in the transition area between river and harbour basin for a real world scenario, where sufficient field records were available.

Boundary conditions were calculated by a 3-dimensional numerical model for the River Weser, where hydrodynamics and sediment transport were evaluated in a regional context.

The numerical experiment showed, that

- Hydrodynamics can be modelled.
- Water exchange, and thus sediment input, between river and harbour entrance in an inhomogeneous (brackish) environment can be reduced to a certain extent.
- CDW captures water needed for tidal filling from the top layer of the river.
- CDW cannot reduce density driven currents to zero and has to be optimized for specific locations and tidal conditions.

However, a reduction of siltation is possible.

Keywords: harbour sedimentation, 3D numerical modelling, Current Deflection Wall

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1 INTRODUCTION

To prevent sedimentation of fine sediment entering a harbour, the so called Current Deflecting Wall (CDW, shown in Fig. 1, can be used, which passively alters the water exchange between river and harbour.

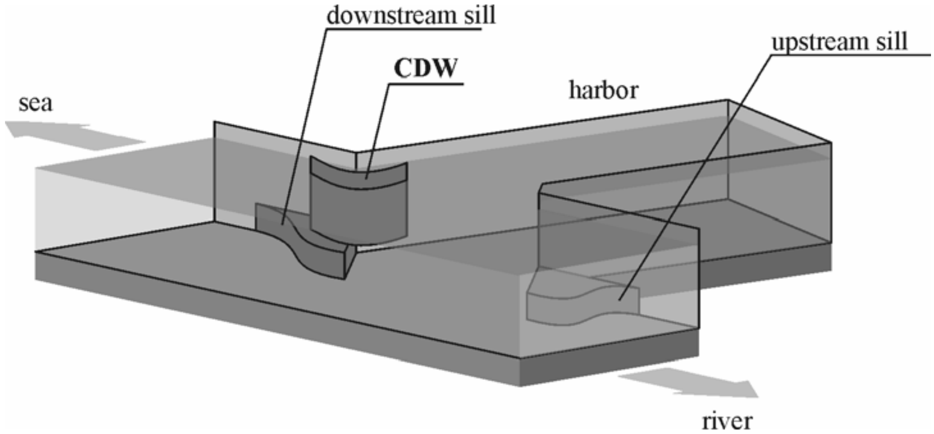


Fig. 1: Current Deflecting Wall downstream a tidal harbour entrance

Beneficiary effects of CDW under tidal conditions with inhomogeneous density could be are:

(a) Reduction of density driven currents into the harbor,
(b) Capture of water needed for tidal filling from the top layer of the river and
(c) Reducing and diverting the mixing layer from the harbour basin.

To investigate the effects of a CDW on water exchange and turbulent mixing in the transition area between river and harbour basin, a numerical 3D model for the Case Study “Bremerhaven Nordschleuse” was setup to simulate hydrodynamics and sediment transport processes near the structure and in the harbour.

Boundary conditions of the model were derived from a regional 3D model of the Weser Estuary. This model was calibrated by gauge data, flow measurements at fixed locations in different water depths and ADCP measurements.

Basic idea of the Case Study was to evaluate the complex flow pattern near the structure and the impact of boundary condition specification (salinity variations over time and depth, sediment transport across boundaries etc.) on sediment transport results as the final step. The Case Study uses well known hydrodynamic conditions to get a view to applicability and accuracy of existing modelling techniques for a real world case.

The coupled 3D models were driven by gauge data for a real scenario (spring tide) observed in the past. Input for sediment transport modelling was calculated from stationary gauges and additional field observations.

Model results describe the complex hydrodynamic situation and give a first view on applicability and accuracy of numerical models.

They show also applicability and restrictions using sediment transport modelling, providing a qualitative result on sedimentation and its reduction by CDW.

2 FLOW AND SEDIMENT TRANSPORT PHENOMENA IN HARBOURS WITH TIDAL BRAKISH ENVIRONMENT

A detailed description of flow characteristics in harbour entrances and changes in flow patterns in a tidal brakish environment using a CDW can be found in van Leeuwen and Hofland (1999) and Langendoen (1992). Thus, only a short introduction is given here to understand basic mechanism to be influenced using a CDW. This introduction focuses on critical flow pattern under discussion in research and practice. It keeps discussion on sediment transport modelling down to a minimum, well knowing, that sediment transport modelling can give only a first view to the problem, indicating which mechanism must be influenced to minimize sedimentation.

2.1 Basic characteristics without CDW

A density difference along the harbour entrance is the driving force for the density current into the harbour. Velocities show strong directional changes over time and depth with a three-dimensional flow pattern into the harbour. Strength of this 3D flow structure and intensity of vertical components are under discussion. Density currents cause opposite flows in the upper and lower layers (Fig. 2).

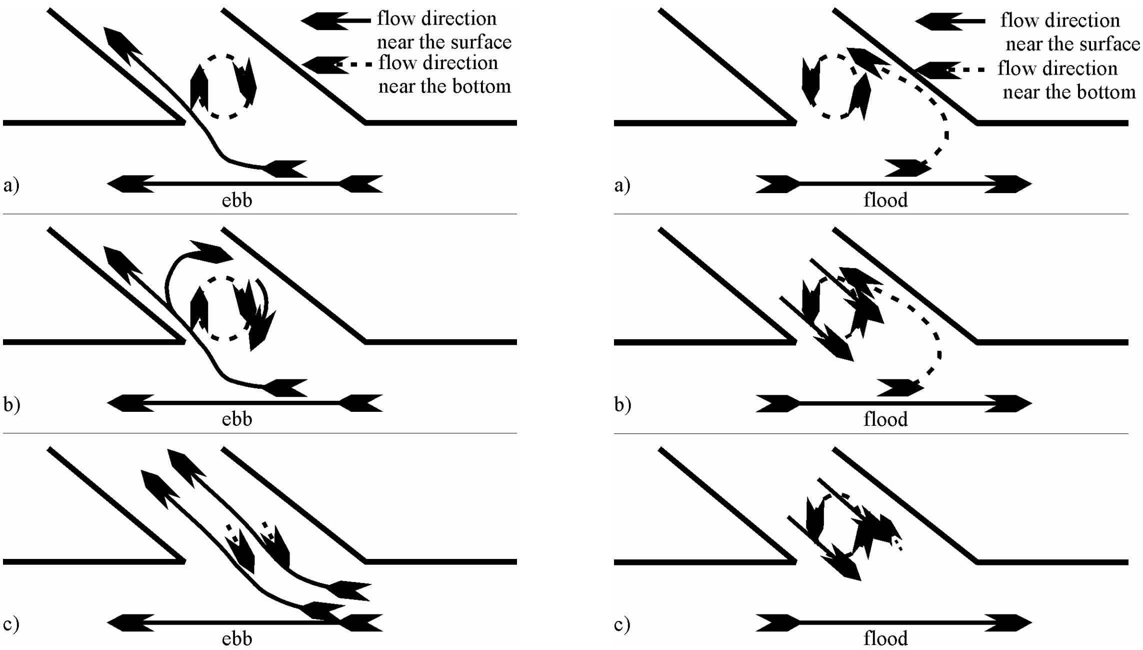


Fig. 2: Flow pattern in the harbour entrances (based on Drifter and ADCP measurements, IWA 2003)
(a) Beginning of ebb/flood, (b) During ebb/flood and (c) End of ebb/flood

The combined flow mechanisms can be distinguished in the order of influence (1) and 2) changing specific to the site) as follows (Langendoen 1992):

1) Density driven currents (salinity) between river and harbour,
2) Turbulent mixing in a transition zone between river and harbour and
3) Tidal filling of the harbour.

The resulting horizontal in/out boundary layer is varying in depth over the tidal cycle. Currents are directed into the harbour over the bed from the first half of rising tide to the first half of falling tide, where the largest sediment flux into the harbour takes place.

2.2 Flow and sediment transport characteristics using a CDW

At present, complete consensus of researchers on the precise function and effects of the CDW does not exist. Leeuwen and Hofland (1999) stated latest for inhomogeneous environments, that CDW function is best during the critical tidal phase of rising tide, where

- CDW directs water from the river top layer into the harbour (tidal filling),
- A downstream sill deflects the near-bed density currents away from the harbour entrance and
- CDW creates a vertical vortex near the bed with its axis across the harbour entrance.

Leeuwen and Hofland (1999) explain that the vortex is the main cause for the reduction of near bed water influx during rising tide. It is created by a pressure gradient over the vertical behind the CDW. It is stated that this secondary current blocks density currents from the sediment loaded currents from river into the harbour.

It can be suspected, that it also fixes the turbulent mixing zone and hinders a turbulent flux into the harbour. For other possible effects, such as “extra pressure against the pressure of the density current”, “decrease the effective width of the harbour entrance” and “increase of friction between the layers” they concluded, that these effects cannot have a significant influence.

3 MODELING TECHNOLOGY

The Test Case "Bremerhaven Nordschleuse" was setup using the three-dimensional hydrodynamic and mud transport modules of the MIKE 3 modelling system.

The theoretical background of MIKE 3 HD can be found in Vested et al. (1992) and Ekebjaerg and Justensen (1991). Basic equations and theory is well known and skipped here.

The hydrodynamic module solves the primitive equations in three dimensions by a FD scheme on an Arakawa C-grid using the Alternating Direction Implicit technique with double sweep algorithm. The implementation provides common turbulence models (only results of the mixed Smagorinsky/k-ε model are considered to be satisfactory), grid nesting by a fixed scheme and a fixed layer concept (equidistant) for vertical resolution. Horizontal grid nesting ($\Delta x_{min}=\Delta y_{min}$=const.=$1/3\Delta x_{max}=1/3\Delta y_{max}$ with a limit of nine sub grids) limits applicability near structures, where grid resolutions down to 0.25m are necessary to provide an adequate mesh resolution for turbulence modelling. The vertical top layer must cover all tidal variations. This should be changed, so more than one layer can fall dry/become wet during a simulation. Resolution near the bottom should be more flexible with variable layer thickness to get flow velocities for shear stress calculation more accurate. At the moment a logarithmic velocity profile is estimated between the last vertical nodes and the bottom.

The FD implementation requires limitation of fluxes during one time step satisfying the Courant-Criterion. This leeds to time steps with Δt=1s or less (satisfying C_R in nested areas with lowest grid resolution), which results in job times of several days (coupled hydrodynamic and sediment transport calculations) for one tide and will limit the future application of the programs, due to the fact that code is not written for parallel execution.

Theory of sediment transport implementation can be found in DHI (2000). The of mass conservation equation for cohesive sediment is given by:

$$\frac{\partial c}{\partial t}+\frac{\partial}{\partial x_j}\left(c(u_j - w_{s,j})\right)=\frac{\partial}{\partial x_j}\left(D_c\frac{\partial c}{\partial x_j}\right)+S_c \tag{1}$$

where $w_{s,j}$ = (0, 0, w_s [ms^{-1}]) is the settling velocity vector, D_c is the dispersion of sediment, and S_c is a local source. Suspended sediment influences the hydrodynamics by changing the density and the kinematic viscosity of the mixture. By altering the density due to suspended material, damping of turbulence at lutoclines is automatically taken into account in the hydrodynamic modelling by a Richardson damping technique. The kinematic viscosity is approximated:

$$\frac{\nu_m}{\nu}\approx 100^{\frac{c}{600\,g/l}} \tag{2}$$

where ν_m [m^2s^{-1}] is the kinematic viscosity of the mixture, and ν [~1-2 10^{-6} m^2s^{-1}] is the kinematic viscosity of the water. The settling velocity (w_s) used is related to dissipation and concentration (hindered settling is taken into account):

$$w_s = w_{s0}\cdot\left(\frac{c}{c_{s0}}\right)^n\cdot\left(1-\frac{c}{\frac{5+2n}{n}c_{s0\mathrm{MAX}}}\right)^{5+n}\cdot\max\left(0\,;1-\sqrt{\frac{\varepsilon}{\varepsilon_0}}\right)\cdot\left(1-e^{-S/3.0}\right) \tag{3}$$

where w_{s0} (~1 mms^{-1}) is a reference settling velocity, c_{s0} (~1 gl^{-1}) is a reference sediment concentration, n (~1) is a dimensionless suspended material parameter, c_{s0MAX} (~7 gl^{-1}) is the concentration at a higher settling velocity, ε [m^2s^{-3}] is the dissipation, ε_0 [m^2s^{-3}] is the 'floc destruction dissipation', i.e. the dissipation at which the flocs are destroyed, and S is the salinity. The present formulation relating the settling velocity to dissipation is used in the entire water column. It has been implemented to ensure that the dependence on the dissipation should simplify to the successful Krone's probability function in the viscous sub-layer close to the bed. The dissipation is taken into account in all-computational points in the water column, but only in the point nearest to the bed is the dissipation from short period surface waves also taken into account. The floc destruction dissipation is unknown, but can be related to the critical shear stress for deposition (τ_d [Nm^{-2}]):

$$\varepsilon_0=\frac{\tau_d^2}{\rho_m^2\nu_m} \tag{4}$$

whereby the dependence of the dissipation becomes exactly Krone's probability function in the viscous sub-layer. The critical shear stress for deposition is related to the concentration as follows:

$$\tau_d = \tau_{d,full} + (\tau_{d,part} - \tau_{d,full}) \cdot \left(\frac{c}{c_{d0}} \right)^4 \quad (5)$$

where $\tau_{d,full}$ (~0.1 Nm^{-2}) is the critical shear stress for full deposition (weak and strong aggregates of all orders deposit), $\tau_{d,part}$ (~1.5 Nm^{-2}) is the critical shear stress for partial deposition (only the strong first-order aggregates in the distribution deposit), and c_{d0} [~3 gl^{-1}] is a typical maximum sediment concentration. If erosion takes place it is determined by:

$$E = e_0 \cdot \sqrt{\rho_b \tau_{ce}} \cdot \frac{\tau_b - \tau_{ce}}{\tau_{ce}}, \; when \; \tau_{bmax} > \tau_{ce} \quad (6)$$

where e_0 ($\sim 4 \cdot 10^{-5}$) is a dimensionless bed material parameter, τ_b [kgm^{-3}] is the dry density of the bed surface, τ_{ce} [Nm^{-2}] is the critical shear stress for erosion, and τ_b [Nm^{-2}] is the bed shear stress (index max gives that it is the maximum during a short period surface wave cycle). The critical shear stress is:

$$\frac{\tau_{ce}}{\tau_{ce0}} = \left(\frac{\tau_y}{\tau_{y0}} \right)^m \quad (7)$$

where τ_{ce0} (~1.0 Nm^{-2}) is a reference critical shear stress, τ_{y0} (~1.5 Nm^{-2}) is a reference yield stress and m (~0.5) is a constant. If the biological activity taking place in the bed is low, or if resuspension takes place one or more times a day, then generally the yield stress is related to the dry density:

$$\frac{\tau_y}{\tau_{y0}} = e^{\left(4 \frac{\rho_b - \rho_{b0}}{\rho_{b0}} \right)} \quad (8)$$

where ρ_{b0} (~325 gl^{-1}) is a reference concentration. Combining (7) and (8) gives the following relation:

$$\frac{\tau_{ce}}{\tau_{ce0}} = e^{\left(m 4 \frac{\rho_b - \rho_{b0}}{\rho_{b0}} \right)} \quad (9)$$

which can be used to estimate the critical shear stress for erosion on the basis of the bed dry density profile. If deposition (D [$gm^{-2}s^{-1}$]) of weak and/or strong flocs takes place it is determined by (ignoring possible dispersion of flocs):

$$D = c_b \cdot w_s, \; when \; \tau_{bmax} < \tau_{ce} \quad (10)$$

where c_b [gl^{-1}] is the concentration close to the bed (point nearest to the bed). Full deposition occurs at low velocities having a smooth-wall. Close to the wall in the viscous sub-layer the dissipation can be determined:

$$\varepsilon = \frac{\tau_b^2}{\rho_m^2 \nu_m} \quad (11)$$

Inserting Eq. (11) and Eq. (4) into Eq. (3) we get:

$$w_s = w_{s0} \cdot \left(\frac{c}{c_{s0}} \right)^n \cdot \left(1 - \frac{c}{\frac{5+2n}{n} c_{s0\mathrm{MAX}}} \right)^{5+n} \cdot \left(1 - \frac{\tau_b}{\tau_d} \right) \cdot \left(1 - e^{-S/3.0} \right) \quad (12)$$

When calculating the settling velocity, (11) is only used in the lower half of the grid cell nearest to the bed. In the water column away from the bed the local dissipation is used. It is possible (to specify) that the critical shear stress for deposition is larger than the bed shear stress and the critical shear stress for erosion, and in this case erosion and deposition will both take place.

Multiple layers describe the bed, where depositing material always enters the first layer. The consolidation of the bed as well as the settling/consolidation of high concentration suspension layers (fluid mud layers) is not considered in detail. Consolidation is simplified by a transfer function (T_f [$gm^{-2}s^{-1}$]) between the layers. When the concentration in the water column just above the bed gets very high, the settling velocity becomes zero. In this case the deposition should be calculated on the basis of the used consolidation theory, e.g. transfer function.

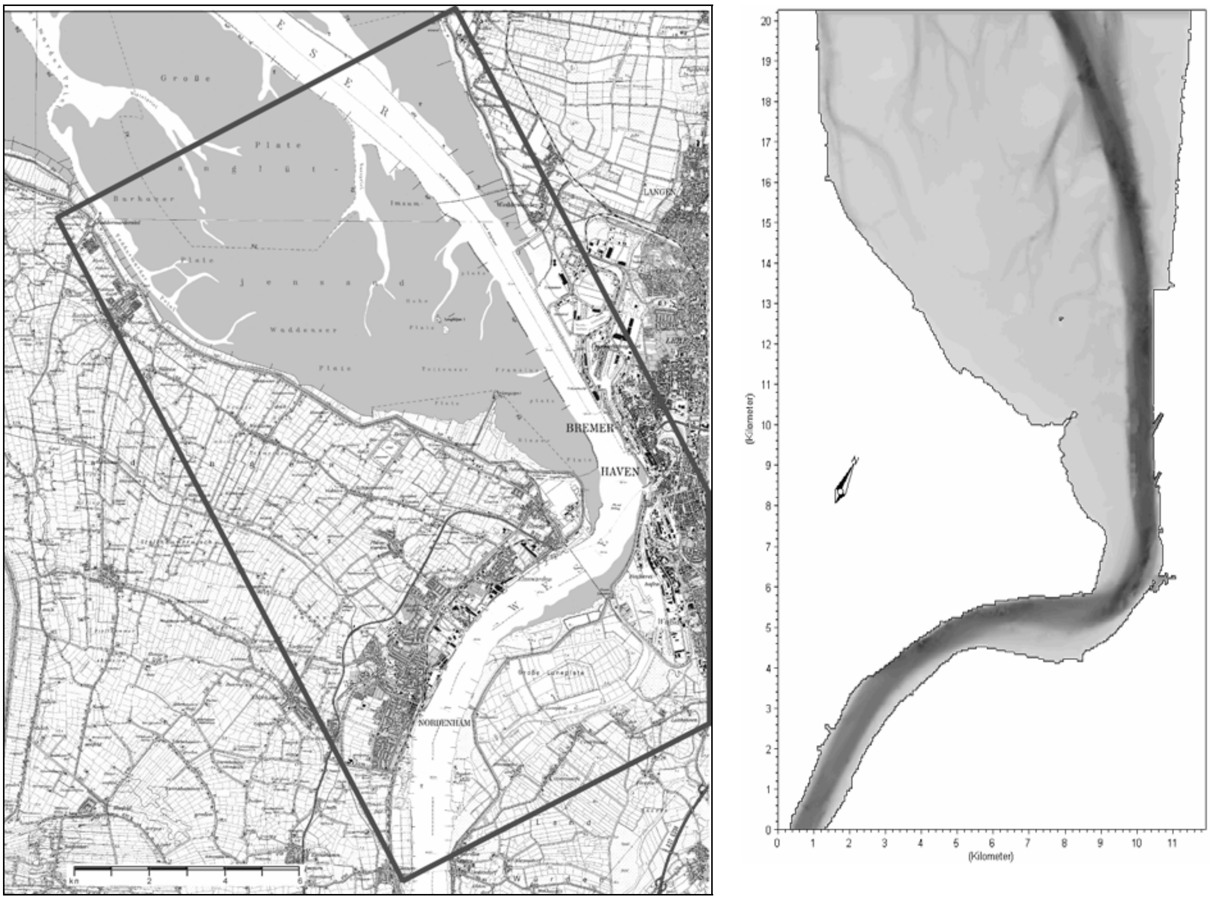
The influence of short surface waves is neglected in this Case Study. Details can be found in DHI (2000).

A complex Case Study was performed during the COSINUS-Project for the Tamar Estuary by Petersen et a. (2002). Intensive model tests were also done byFranzius-Institute (2003) for the Ems and Weser Estuaries.

4 CASE STUDY "BREMERHAVEN NORDSCHLEUSE"

4.1 Model Setup

For the Test Case "Bremerhaven Nordschleuse" a regional 3D model for the mouth of the Weser Estuary between Kleinensiel-Dedesdorf (Unterweser-km 53, h=f(t) boundary condition, depth integrated salinity S=f(t)) and Wremer Tief/Fedderwarder Siel (Unterweser-km 79.5, h=f(t) boundary condition, depth integrated salinity S=f(t)) was set up (Fig. 3).



(a) Model Area (b) Bathymetry (WSA Bremerhaven, 2000)

Fig. 3: Regional 3D model for the Test Case "Bremerhaven Nordschleuse"

Grid resolution of the model is $\Delta x=\Delta y=18m$ (regional model), $\Delta x=\Delta y=6m$ (nested harbour and entrance) and $\Delta x=\Delta y=2m$ (near CDW) with $\Delta z=1m$ respectively.

Water level calibration (Fig. 4) was done for two periods (Period A – spring tide: 13.09.2000 2^{00} to 14.09.2000 22^{00} and Period B – neap tide: 14.5.2001 6^{00} to 16.05.2001 1^{00}). Maximum water level differences are 10cm at one distinct point. Mean differences are 5cm. During neap tide results are equivalent Accuracy at high/low tide is better than 5cm. A time shift between measured and calculated tidal water levels is not visible.

Salinity varies between 4 and 21‰ at "Bremerhaven Alter Leuchtturm". Maximum difference between measured and calculated values at this gauge is 4.5‰ (Fig. 5). Additional measurements over the depth in the River Weser and the harbour (Fig. 6) showed same maximum differences over time and depth. Differences are concentrated to the bottom. At the surface, differences are at any time and any location below 1‰. Differences in Period B are below 5‰.

Small eddy structures can not be modelled due to grid resolution. Thus, turbulence modelling becomes an important topic. Only the mixed Smagorinsky/k-ε model was stable and produced satisfactory results. This surprising phenomena (all other models unstable and bad results) was tested in a parameter study for a well known harbour without density effects (modelled formerly by a physical model) and found to be true.

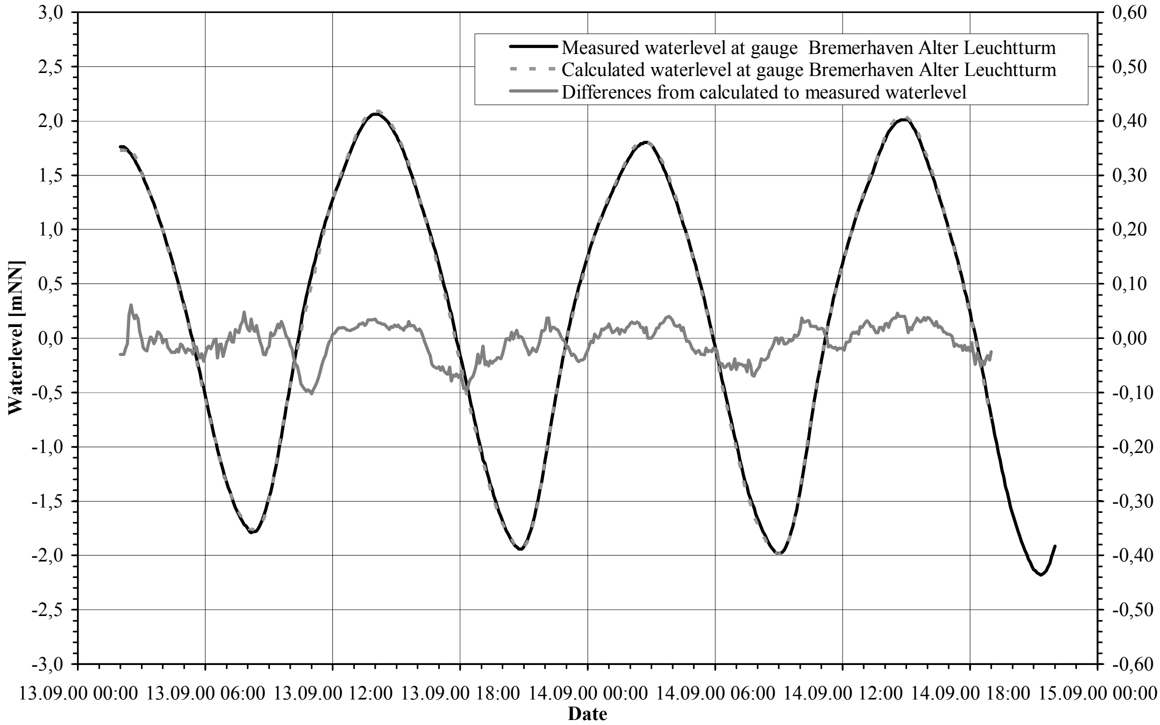


Fig. 4: Water level differences at gauge “Bremerhaven Alter Leuchtturm” for Period A

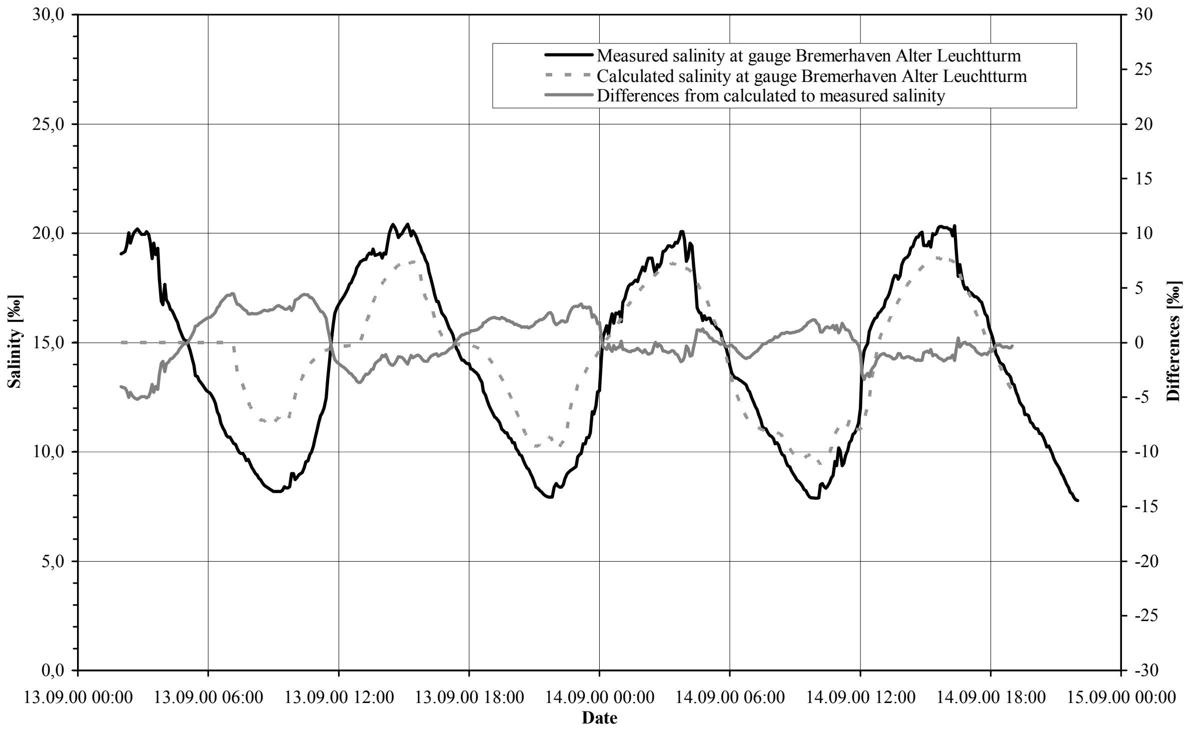


Fig. 5: Salinity differences at gauge “Bremerhaven Alter Leuchtturm” for Period A at -3.2 mNN

Flow velocities were tested against intensive ADCP measurements (IWA, 2001). This comparison must be extremely careful, due to the necessary time to produce an area map of flow velocities by ADCP profiles. Thus, flow velocities were selected also point by point and compared with model results at precise time and location of the measurement (Franzius-Institut., 2003). Flow velocities were averaged over MIKE 3 cells and transferred to calculation nodes for each vertical ADCP measurement (spot).

Flow velocities in the harbour entrance differ with a maximum of 10cm/s. The in/out boundary layer in the harbour entrance is varying in time, oscillating around a water depth of 6m with ±1m.

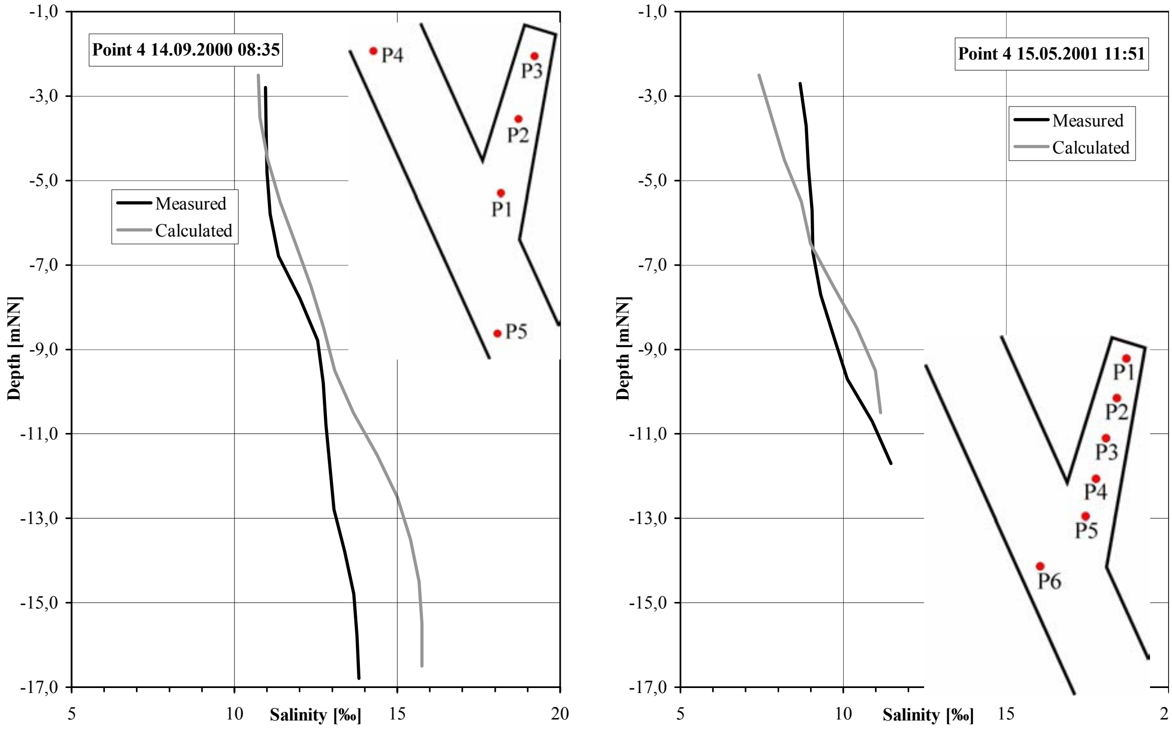


Fig. 6: Measured and calculated salinity during Period A (left, Point 4: 14.09.2000 8^{35}, around low water) and Period B (right, Point 4: 15.05.2001 11^{51}, 1h before low water)

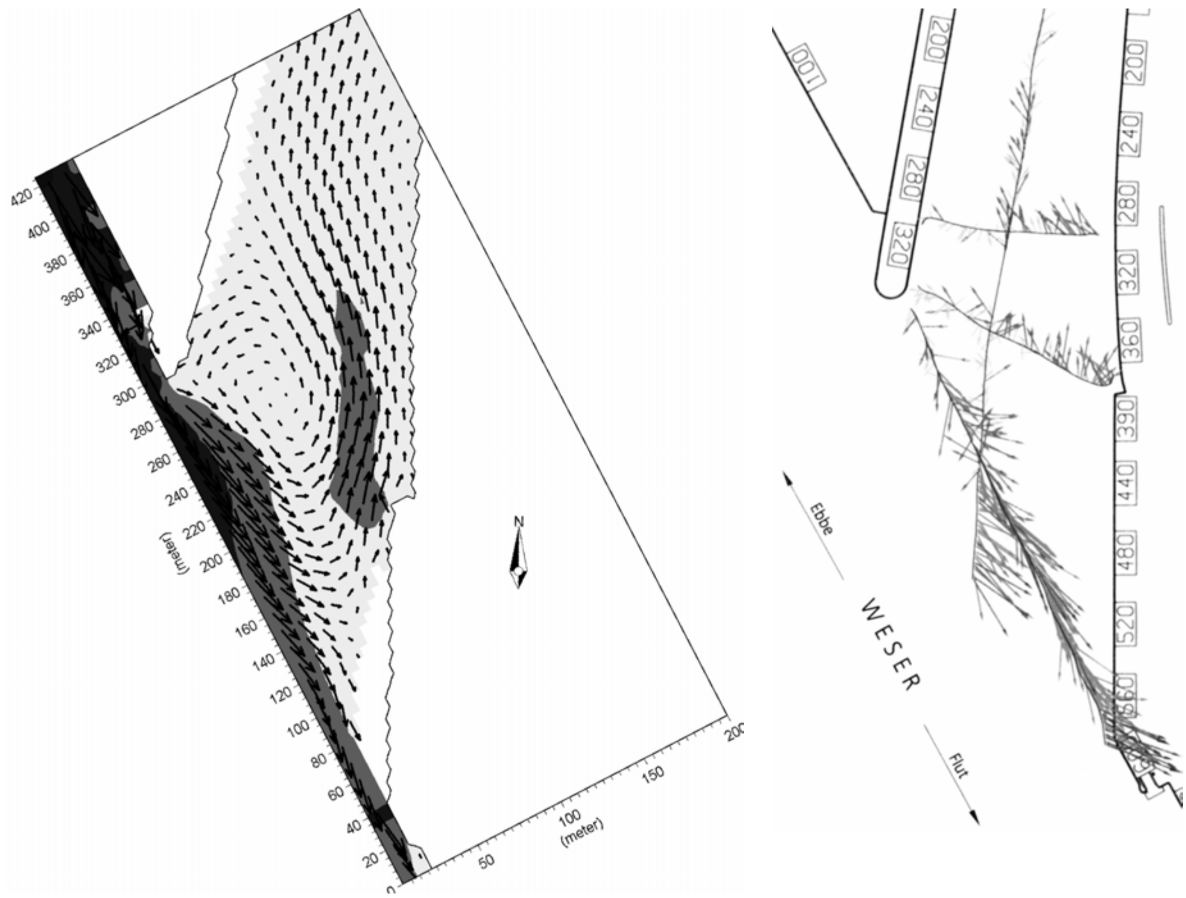


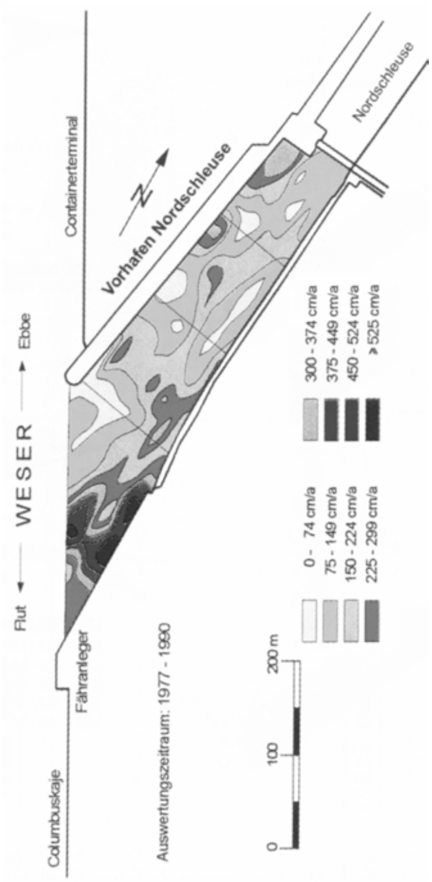
Fig. 7: Comparison of calculated flow velocities with ADCP measurements (IWA, 2001)

Comparison shows, that major eddy structures are reproduced (dimension, time of development, rotation, shape and movement through the harbour during tidal cycle). Unless the in/out layer is moved slightly to a different level, the vertical shape of large eddy structures is reproduced (Fig. 7). The resulting parameter set after the hydrodynamic calibration is shown in Tab. 1.

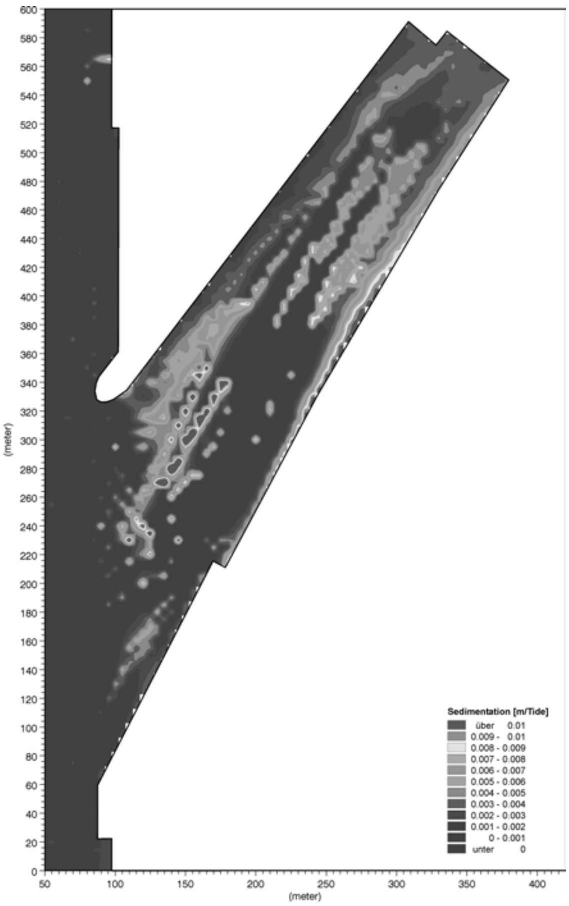
Tab. 1: Parameter set after hydrodynamic model calibration

Bottom roughness for the whole model area: k=0.05m
Turbulence Model: c_{sm}=0.5, c_μ=0.09, c_1=1.44, c_2=1.92, c_3=0, σ_k=1, σ_ε=1.3, k=1e-7, ε=5e-10

Calibration of the sediment transport model had to focus on main sediment fractions (silt<0.01mm to fine sand ~0.1mm), known from measurements (Nasner, 1997). Due to maintenance activities in the harbour (water injections) and traffic to/from the lock, development of sedimentation in the harbour is difficult to compare with model results. Thus, a sediment fraction with d_{50}=10µm was used to reproduce measured long term sedimentation (Nasner, 1997) and to check concentrations over the depth at location shown in Fig. 6. Fig. 8 shows, that areas of main sedimentation are reproduced. Their shape and dimension is slightly changed, because sediment movement caused by accelerating ships can not be modelled (e.g. upstream corner of the harbour entrance).



(a) Measured sedimentation (Nasner, 1997)



(b) Calculated sedimentation using Period (A)

Fig. 8: Comparison of sedimentation measured by Nasner (1997) for the period 1977-1990 and calculated sedimentation (silt, Period A) at „Bremerhaven Nordschleuse"

The parameter set shown in Tab. 2 was achieved after checking the model to reproduce general sediment transport processes. It is in accordance with "Hollands Diep2", "Breskens Harbour" and "Delfzijl Harbour" (van Rijn, 1993).

Tab. 2: Parameter Set for Sediment Transport Modelling

dispersion coefficient [-]: 0.01	critical bed shear stress for erosion (layer b) [N/ m^2]:
settling velocity [mm/s]: 0.2	0.3 (Harbour), 0.7 Wadden Area), 2.0 (River Weser)
max. concentration for deposition [kg/m^3]: 3.0	Density of bed material (layer b) [kg/ m^3]:
partial bed shear stress for deposition [N/m^2]: 1.5	1000 (Harbour)
critical bed shear stress for deposition [N/ m^2]: 0.06	1100 (Wadden Area)
empirical constant for erosion [-]: 1.00E-05	2000 (River Weser)

Model results show, that sedimentation applies mainly at the ending flood period (last 30%) and starting ebb period of spring tide (Period A). Consequently, the following investigations concentrate on Period A.

4.2 Hydrodynamic Simulation of CDW

The implemented CDW (Fig. 9) is situated on a sill with a height of 4.5m. There is an opening of 13m between CDW and shore.

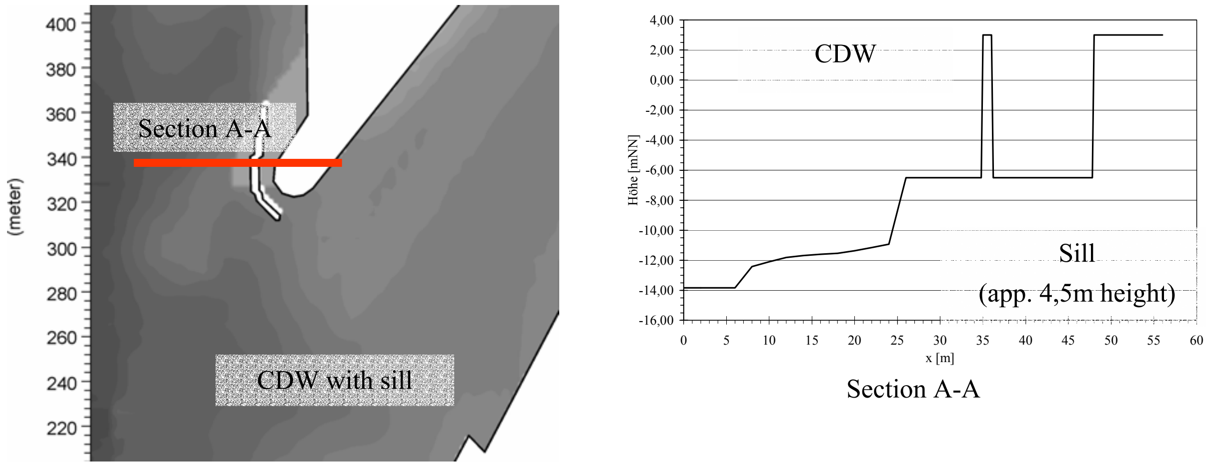


Fig. 9: CDW design for the Test Case “Bremerhaven Nordschleuse”

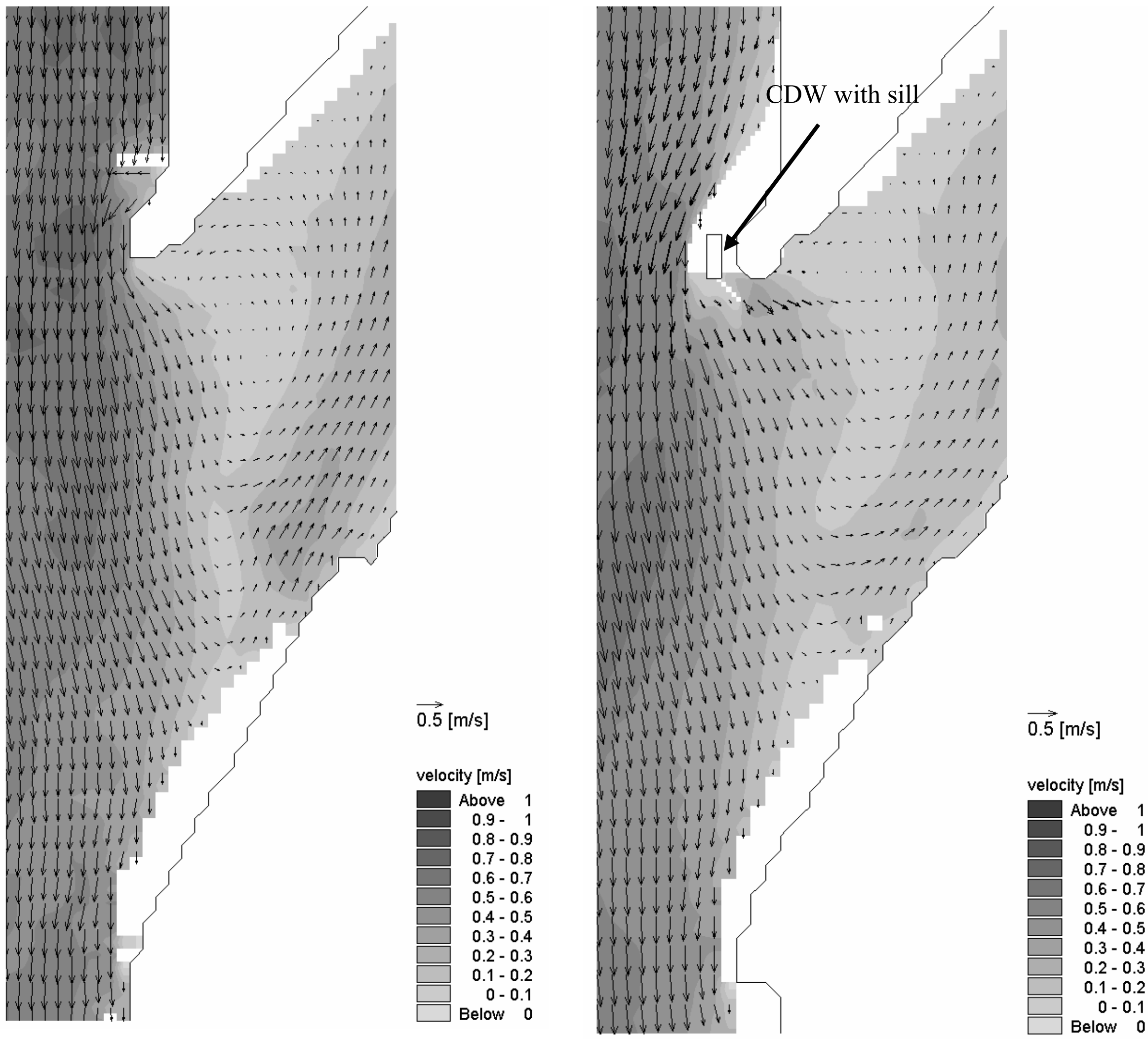


Fig. 10: Flow velocities near the harbour bottom (Period A, 14.09.200 10^{15}, 1.75h after low water): left - Reference model without CDW, right – Model with integrated downstream CDW and sill

One hour before low water with maximum flood velocities, flow directions into the harbour are comparable, but magnitudes of flow velocities are lower (Fig. 10). Water exchange over one tidal cycle is reduced from 3.75 Mio. m³ to 3.38 Mio. m³ (10%). Dynamic 3D visualization of flow pattern during flood shows an efficient capturing of water from the higher water column, which is used for tidal filling. A movement of the stagnation zone is not visible.

The vertical vortex described by Leeuwen and Hofland (1999) could not be found. In some areas unstable eddies behind CDW developed, but rapidly disappeared. CDW was not able to build up a stable secondary flow across the harbour entrance.

Simulation showed that the CDW has to be placed at such a distance from shore that the discharge needed for tidal filling plus an extra of 10% is captured. In the Test Case the CDW captures 155.000m³ for tidal filling instead of 209.000m³.

4.3 Effects of CDW on Sediment Transport Processes

The Case Study shows for the selected sediment fraction (silt) and for investigated tidal conditions (spring tide), that the main reduction is located in an area upstream the CDW (Fig. 11). This area has an extension of approximately 75m into the harbour, indicating the capturing of water by CDW.

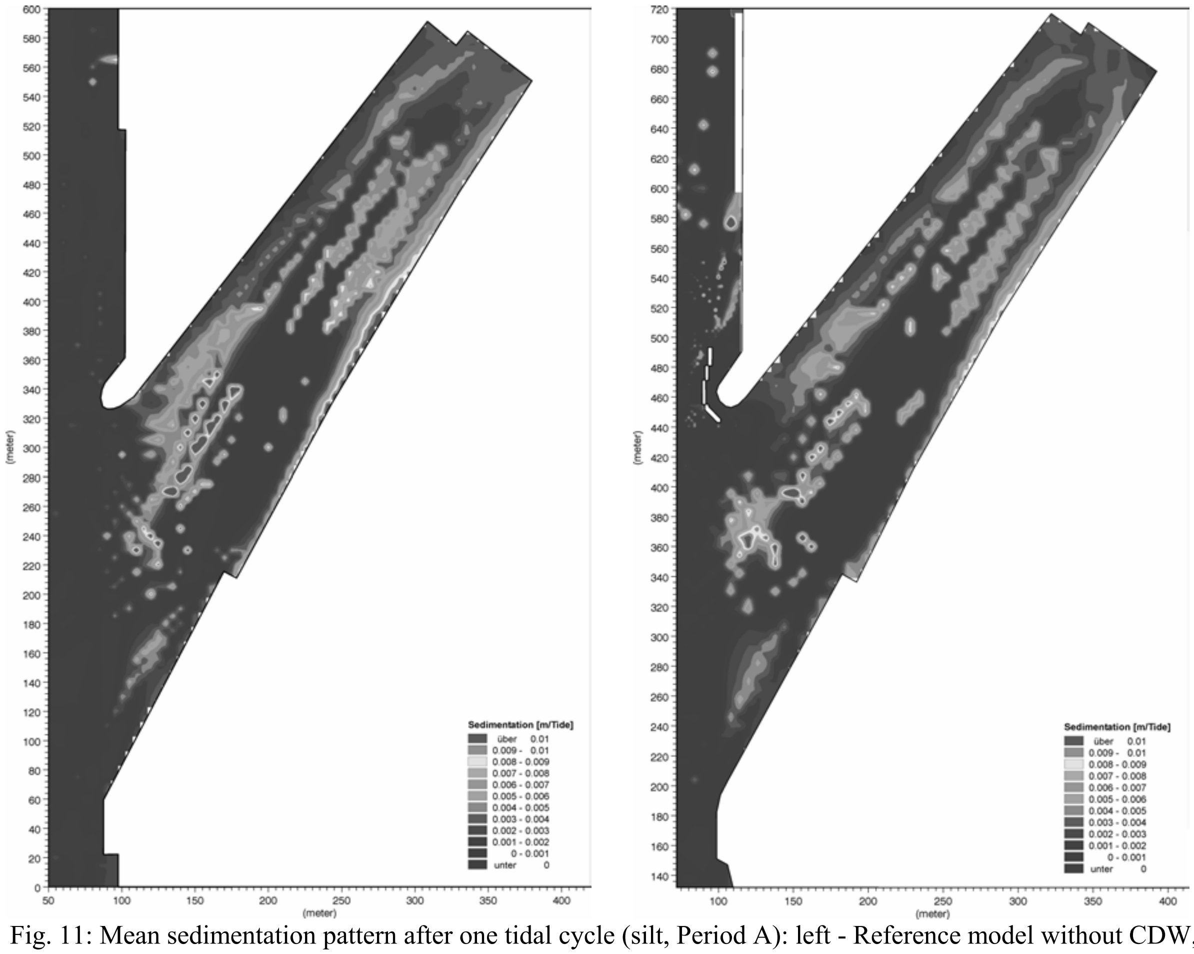


Fig. 11: Mean sedimentation pattern after one tidal cycle (silt, Period A): left - Reference model without CDW, right – Model with integrated downstream CDW and sill

The total volume of deposited sediment is reduced, which does not mean that sediment flux through the harbour entrance is reduced.

In the centre of the harbour entrance sedimentation with CDW is higher, indicating that flow velocities behind the structure are (a) reduced by the structure itself and (b) energy losses of water captured by CDW are so high after leaving the CDW, that flow velocities drop down under critical values for deposition. This energy loss could be also an argument that the vertical vortex can not be developed by CDW in our case.

Comparing the total water exchange of 3.38 Mio. m³ during one tide with the captured volume of 155.000m³ (~5%), the effect on sediment transport is surprising.

Thus, capturing of water from the top layer has to be seen as the main source of reduction by CDW, unless the vertical vortex is not found.

5 DISCUSSION AND REMARKS

It must be mentioned, that a final conclusion about CDWs functionality cannot be given. CDW layout and arrangement in tidal environments with existing density driven currents requires special, site specific adjustments, because small changes in length, curvature and position have substantial influence to flow pattern and sediment transport.

The vortex described by Leeuwen and Hofland (1999) was not reproduced in numerical model results, although reduction of sedimentation was visible. We suppose that grid resolution was not fine enough to reproduce this “secondary” flow. It could also be the case that CDW in its actual configuration for “Bremerhaven Nordschleuse” is not able to produce the necessary pressure gradient over the vertical and/or the vortex is unstable over time and space and not visible in numerical results. Both could indicate that CDW construction is sensitive due to vortex stability and/or other effects are responsible for the reduction of sedimentation, which was simulated without having the vortex. This should be clarified by future experiments with higher grid resolution.

The Case Study “Bremerhaven Nordschleuse” showed that CDW

- does not reduce the mixing area between river and harbour,
- does not move the stagnation point out of the harbour,
- redirects some water from the harbour entrance to the river with the implemented sill,
- reduction of water exchange between river and harbour is rather limited,
- reduces large eddies in the harbour and
- captures water from layers with lower sediment concentration to some extent.

The capturing of water from layers with a lower sediment concentration is the main mechanism for the reduction of sedimentation in this Case Study.

The Case Study showed that hydrodynamic modelling is able to predict the complex flow pattern in a brackish tidal environment across harbour entrances measured in nature by ADCP for the whole tidal cycle. Quality of model output depends on the specification of boundary conditions and grid resolution. Salinity should be introduced as a function of time, location and depth, which would improve model results.

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