Delft Cluster

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Abstract

The report gives a description of the combined application of a physical scale model and various numerical models to assess the effect of a Current Deflecting Wall (CDW) on the rate of exchange of water between river and harbour (or access channel) and consequently on siltation. The physical model is a slightly distorted model (with vertical and horizontal scales of 150 and 300), which includes a section of the Beneden Zeeschelde in Belgium. The model takes into account the effects due to tidal filling, horizontal entrainment and density currents resulting from salinity differences. Calibration of the model was done on the basis of variations of water levels, velocities and salinities on the river near the access channel. The boundary conditions of the physical scale model were derived from an overall two-dimensional (depth-averaged) numerical model as well as from a three-dimensional model, which was nested in the overall model. The benefits of a CDW for the Beneden Zeeschelde, as determined from the model tests, will be described elsewhere. To describe adequately the three-dimensional flow, as induced by a CDW, numerical models needed to be extended with proper process formulations and newly developed numerical schematisation techniques. These were implemented in DELFT3D and relate to: (i) a fixed layer schematisation of the numerical grid in vertical direction, (ii) inclusion of a non-hydrostatic pressure distribution and (iii) application of a smooth boundary approximation. Comparison with results from the physical scale model will have to indicate the potentials and limitations of numerical modelling in these matters. Vice versa, the numerical scale model allows the assessment of possible scale effects of the physical model due to the distortion and relatively low Reynolds numbers (viscosity). The approach thus combines the strong points of both modelling approaches and offers the opportunity to develop and validate numerical models for complicated three-dimensional flow phenomena.

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Executive Summary

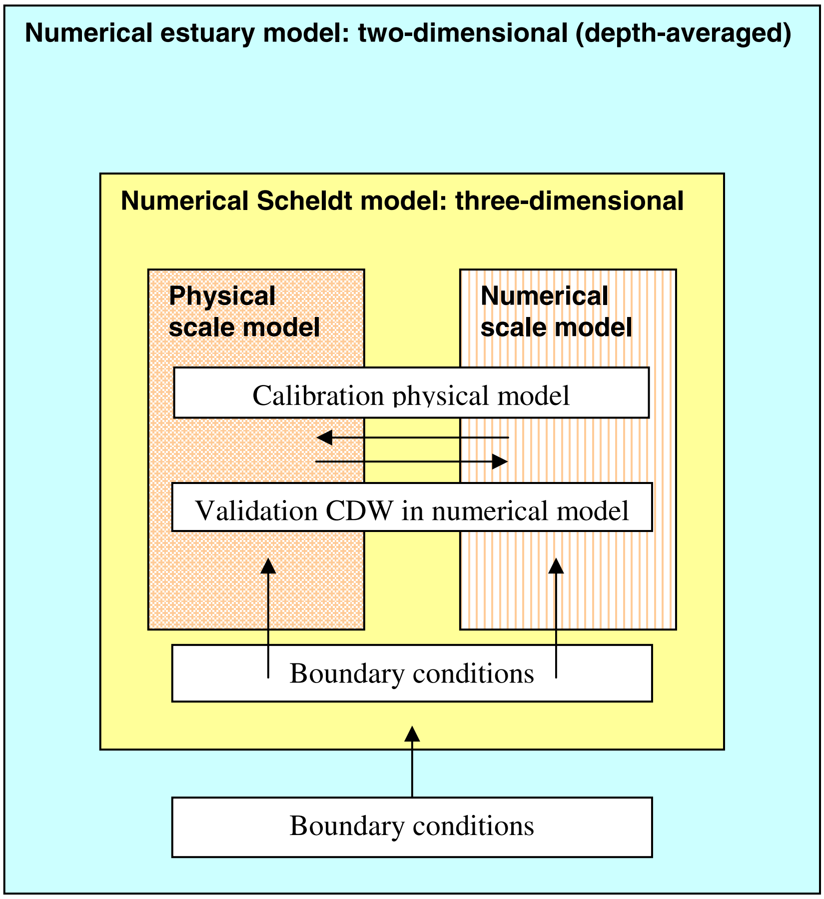
Accumulation of sediment in harbour basins and access channels forms a practical problem for managing authorities of ports in the world. Dumping in the environment poses negative side-effects and the storage of polluted sediments in depots is not always an option. Therefore there is a universal need to seek for solutions that combat siltation of harbours and channels at a more fundamental level. The application of passive devices, such as the construction of a Current Deflecting Wall (CDW), offers promising prospects to achieve a permanent reduction of net siltation. This report describes a hybrid approach on the design of a CDW for the maritime access of the Port of Antwerp, located along the Beneden Zeeschelde in Belgium. The approach consists of the set-up of *tools*, which are capable to arrive at such a design. Distinction is made between physical scale models and numerical models and the purpose of this report is to discuss the capabilities and limitations of these models in these matters and to demonstrate how, through their combined effort, an additional value is obtained.

The sedimentation rate in a semi-enclosed water body, which is in open connection with coastal water, estuary or river, is governed by the mutual exchange of water due to tidal action, horizontal entrainment and density currents (following from spatial differences of temperature, salinity or suspended sediment concentration). The resulting three-dimensional flow pattern may be influenced by a CDW in such a way that the rate of water exchange is decreased. A reduction of the resulting sediment import into the harbour or channel is accomplished through (i) a diversion of high concentration, near-bed suspensions from the harbour entrance, (ii) a decrease in the strength of the mixing layer, (iii) a diversion of the mixing layer away from the harbour mouth and (iv) the generation of a helical flow pattern in the harbour mouth. Experiences with an actually built prototype for the Köhlfleet harbour in Hamburg (tidal, homogeneous conditions) and fundamental research for a schematised estuarine harbour show that reductions in the exchange flow rate of 30-50% can be achieved.

Research on the optimum design of a CDW requires an accurate reproduction of the relevant, three-dimensional processes. Based on previous experiences, the application of a physical scale model is still considered to be the most suitable research tool to arrive at reliable answers. This has led to the construction of a physical scale model in the Tidal Flume of WL | Delft Hydraulics encompassing a section of the Beneden Zeeschelde and a planned access channel for which siltation-reducing measures need to be studied. The large-scale hydrodynamic processes are characterised by tidal flow with a tidal range of 4 m, an annual-mean river discharge of 110 m^3/s and gravity-driven flow. Salinities vary strongly with the tide and the river run-off. Suspended sediment concentrations are generally limited to several hundreds mg/l, however near-bed concentrations can be considerably larger. Local sedimentation rates, as observed in access channels, amount several meters per year.

The physical scale model takes into account the large-scale hydrodynamic processes, including gravity-driven flow due to salinity differences between sea and river water. Due to practical constraints the model employs different vertical and horizontal length scales, 150 and 300 respectively. This implies a distortion factor of 2, which is still acceptable to reproduce the local three-dimensional flow. The model was calibrated against field data, aiming at a correct reproduction of the vertical and horizontal tide and salinity variation at the river near the location of the future access channel.

Simultaneously, a numerical scale model, based on the DELFT3D software package, was set-up as a pilot model for the Delft Tidal Flume to supply the physical scale model with boundary conditions. The numerical scale model derived its boundary conditions from an overall two-dimensional (depth-averaged) model for the Scheldt estuary as well as from a more detailed three-dimensional model, which was nested in the overall model. The general approach adopted in the study is given schematically in the figure below.



Until recently, the application of numerical models to study local three-dimensional flow, as induced by a CDW, was not feasible, because these models were lacking the proper representation of the physical processes. Furthermore, simulations required a high resolution grid, which resulted in excessive computing time. New process formulations and numerical schematisation techniques were implemented in the DELFT3D code to account for the relevant phenomena and to employ an efficient computational approach:

- A fixed layer schematisation of the numerical grid in vertical direction;
- Inclusion of a non-hydrostatic pressure distribution;
- Application of a smooth-boundary approximation.

Comparison with results from the physical scale model will indicate the potentials and limitations of numerical modelling in these matters. Vice versa, the numerical scale model allows the assessment of possible scale effects of the physical model due to the distortion and relatively low Reynolds numbers (viscosity). The approach thus combines the strong points of both modelling approaches and offers the unique opportunity to develop and validate numerical models for complicated three-dimensional flow phenomena.

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The Current Deflecting Wall: mitigating harbour siltation

Set-up and integration of physical and numerical modelling techniques

C. Kuijper
J.C. Winterwerp

Report

June 2003

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1 Introduction

1.1 Background

Harbours along coasts, rivers and in lakes contain water bodies with relatively low energetic hydrodynamic conditions. These conditions favour the accumulation of sediment, often necessitating continuous maintenance dredging works (Table 1.1).

Location	Annual dredging volume and costs
Port of Rotterdam	10 Mm^3
Port of IJmuiden	3.2 Mm^3
Port of Hamburg	2.6 Mm^3
Dutch marinas	200 M€ (backlog: 700 M€)

Table 1.1: Annual dredging volumes in Ports and marinas.

The costs involved in these dredging activities are substantial, if available dumping locations of the dredged material are scarce. This especially holds for contaminated sediments, when legislation forbids the disposal in open water so that storage locations need to be constructed, e.g. the Slufter on the Maasvlakte in the Netherlands and depot IJsseloog in Lake Ketel, see Figure 1.1. Consequently, Port Authorities seek for various measures to reduce the net import of sediment from the environment into harbours.

Figure 1.1: Slufter depot (left panel) and IJsseloog depot (right panel).

A successful strategy to minimise harbour siltation requires a thorough understanding of the underlying mechanisms. Numerous studies on processes, methods and models have therefore been carried out (e.g. Dursthoff, 1970; Booij, 1986; Eysink, 1988, 1989; Langedoen, 1992; and Kolahdoozan et al., 1998). An innovative approach was followed by the Port of Hamburg for the Köhlfleet harbour. Near the harbour entrance a training wall (denoted as Current Deflecting Wall or CDW) was constructed. With this device the local flow field could be affected in such a way that a significant reduction of the annual siltation was achieved (Christiansen and Kirby, 1991, Winterwerp et al, 1994, Kirby, 2001). Further investigations were carried out in The Netherlands through basic research in the Delft Tidal Flume within the framework of the European LIP programme. Through the establishment of the Dutch foundation ANTI-SLIB studies were sponsored and co-ordinated to arrive at feasible solutions for the reduction of siltation in small-scale enterprises, like marinas.

1.2 Objective of the study

The potential of siltation-reducing measures with passive devices, such as a Current Deflecting Wall, is nowadays stimulating research programmes for specific harbour geometries. Currently, an extensive project is being carried out by Delft Hydraulics for the Beneden Zeeschelde in Belgium. The aim of the project is to arrive at an optimum design for a CDW to minimise siltation of entrance channels of harbour docks in the Port of Antwerp. The underlying processes for sedimentation are complicated, and the assessment of the efficiency of a CDW requires the application of advanced, state-of-the-art, research tools. Until recently, only physical scale models were capable to address these types of managerial questions. Recent, and on-going, developments allow the application of numerical models in these matters. This offers the opportunity to combine the strong points of both modelling approaches.

It is therefore the objective of this report:

- to describe the capabilities and limitations of both modelling approaches in addressing the aforementioned research issues and
- to demonstrate how, through their combined effort, additional value to the design of a CDW is obtained.

1.3 Contents of the report

In Chapter 2 the various mechanisms are described responsible for the net import of sediment into harbours. Distinction is made between stagnant water systems (shallow lakes), rivers, tidal rivers and coastal areas and estuarine systems (Winterwerp, 2003). Additionally, mitigating measures are discussed in relation to their effect on the siltation processes and the experiences gained so far with these (passive) devices. Chapter 3 focuses on the specific conditions for the Beneden Zeeschelde and the on-going research to guarantee the accessibility of the Port of Antwerp. The modelling approach is described in general terms in Chapter 4, addressing (i) the use of a physical scale model, (ii) the necessary extension and utilisation of numerical models and (iii) their combined application. This is elaborated in more detail in Chapter 5 where, for the physical model, the scaling laws and resulting spatial and time scales are derived taking into account practical constraints for the construction of the model. Numerical models should describe adequately the dominant water exchange processes and avoid that results are biased by numerical artefacts. This is also addressed in Chapter 5. Conclusions of the study are presented in Chapter 6.

1.4 Relation with Delft Cluster

The project follows closely the objectives of Delft Cluster, aiming at fundamental research in the field of hydraulic engineering. The added-value of Delft Cluster to the project is (i) that processes are studied in a more fundamental way, (ii) that existing tools and techniques are made operational and (iii) that they are further improved and validated on the basis of laboratory and field measurements. In this way the product of the project is generic, thus strengthening the competitive position of The Netherlands in consultancy projects on siltation-reducing methods for harbours and access channels along *coasts and rivers*.

2 Siltation-reducing measures

2.1 Water-exchange processes

The attention is focused on the transport of fine suspended sediment. By a variety of mechanisms water will enter the harbour basin and the sediment, carried by the water, is allowed to settle because of the relatively low velocities in the harbour. Thus, although the net flux of water through the harbour entrance is nil[1], there will be a net import of sediment into the harbour. The siltation rate F_s depends on the total volume of exchanged water over a certain period of time (rate of exchange Q), the sediment properties (especially the settling velocity) and the characteristics of the harbour geometry (volume and surface area). The combined effect of the sediment properties and harbour geometry for a specific harbour basin can be described by the trapping efficiency p, so that the siltation rate is given by:

$$F_s = p\, c_a\, Q \tag{2.1}$$

where c_a is the suspended sediment concentration of the ambient water (Winterwerp, 2003).

The rate of exchange of water Q between a harbour basin and its environment may be governed by a number of processes:

I. exchange flow by horizontal entrainment (mixing layer) Q_e,
II. exchange flow by tidal filling Q_t,
III. exchange flow by fresh-salt driven density currents Q_d,
IV. exchange flow by warm-cold driven density currents Q_T, and
V. exchange flow by sediment-induced density currents Q_s,

In some cases also navigation may play a role. Ships, sailing past a harbour basin, induce outflow of that basin through the local acceleration of the flow in the channel, followed by an inflow after passage. The water exchange cannot be reduced through constructional measures, although regulations with respect to sailing speed and sailing distance from the harbour mouth may be effective.

Exchange flow by horizontal entrainment

The horizontal exchange of water results from velocity differences between the river and the harbour. Along the harbour entrance, starting from the upstream corner where flow instabilities and separation occurs, a plane mixing layer originates, consisting of river water and water from the harbour. The rate of growth of the mixing layer is given by:

$$Q_e = f_e\, A\, U_0 \tag{2.2}$$

[1] *After averaging over a long enough period and assuming that no discharge in, or withdrawal of water from, the harbour takes place.*

where f_e is the exchange coefficient which depends on the harbour mouth configuration and the local flow conditions, A is the cross-sectional area of the harbour entrance and U_0 is the velocity of the river flow. Typical values for f_e range between 0.005 and 0.05 (Winterwerp, 2003).

The flow interferes with the downstream geometry of the harbour basin, giving rise to the existence of eddies in the harbour. Depending on the topography of the harbour one or more eddies may develop. It is remarked here that the shape of the harbour at the downstream side of the entrance, where the flow encounters the rigid wall at the so-called stagnation point, is of great importance and also affects the flow inside the harbour. The line connecting the upstream separation point and the downstream stagnation point is the dividing 'streamline', where the flow velocity is the average of the harbour and river water. In the mixing zone the flow velocity gradually varies from a low value, representing the velocity inside the harbour, towards the velocity of the river water outside the mixing zone. The sediment concentration in the mixing zone will be the result of the mixing between the river and harbour water.

Exchange flow by tidal filling

The exchange flow rate by tidal filling Q_t can be computed in a straightforward manner if tidal elevations and basin surface area are known. It is noted that the effects of tidal filling and entrainment cannot simply be added, as the mixing layer will be advected into the harbour mouth during tidal filling (e.g. Eysink, 1989, Van Rijn, 2003):

$$\overline{Q_e} = \left(f_e/\pi\right)\overline{A}\,\hat{U}_0 - f_t\overline{Q_t} = f_e'\overline{A}\,\hat{U}_0 - f_t\overline{Q_t} \qquad (2.3)$$

where an overbar denotes averaging over the tidal period and $\hat{U}_0$ is the amplitude of the tidal velocity (Winterwerp, 2003). The coefficient f_t amounts to 0.1 – 0.25.

Exchange flow by density currents

The effect of density currents, induced by gradients in salinity or water temperature, on the exchange flow rate can be significant. Such density currents are not only generated by fresh or warm water releases in the harbour basin itself, but also in the case of density gradients in the ambient water (e.g. Eysink, 1989). In particular, the discharge of cooling water within a harbour basin has a dramatic effect on the exchange flow rate. As a rule of the thumb, this exchange flow amounts to three to five times the cooling water discharge.

These exchange flow processes have been described extensively in the literature. Only recently it has been appreciated that already at moderate suspended sediment concentrations, as low as 100 mg/l, density currents can be induced by the sediment suspension itself. Winterwerp and Van Kessel (2002) showed that sediment-induced density currents increase the sediment fluxes into the harbour area of the Port of Rotterdam by at least a factor of three. A second effect of these sediment-induced density currents is that they augment the trapping efficiency within the basin.

It is further noted that the effects of Q_t, Q_d, Q_T and Q_s can be computed with a reasonable accuracy with the current generation of three-dimensional numerical hydrodynamic models.

However, accurate computation of Q_e is still beyond the present state-of-the-art of numerical modelling.

2.2 Mitigating measures

The successful implementation of adequate measures to reduce harbour siltation depends on the characteristics of the water system, where the harbour is located. Winterwerp (2003) proposes a classification scheme where distinction is made between the following water systems:

A. Stagnant systems, such as shallow lakes;
B. Upland rivers;
C. Tidal rivers and coastal areas, i.e. in either homogeneous fresh or homogeneous salt conditions;
D. Estuarine systems within the fresh-salt water region.

The classification scheme is given in Table 2.1.

water systems		water exchange mechanisms				
		I.	II.	III.	IV.	V.
A.	stagnant water / shallow lakes	yes	no	no	sometimes	no
B.	rivers	yes	no	no	sometimes	no
C.	tidal rivers and coastal areas	yes	yes	no	sometimes	sometimes
D.	estuarine systems	yes	yes	yes	sometimes	sometimes

Table 2.1: Classification of ambient water systems with the water exchange mechanisms I - V, as described in Section 2.1 (Winterwerp, 2003). Shaded combinations have relevance to this report.

A number of devices and/or measures are available, or have been developed in the last decade, to decrease one of the five processes I – V by reducing p, Q and/or c_a in (2.1). This is discussed in detail by Winterwerp (2003). Here the attention will be directed to siltation-reducing measures for harbours along tidal rivers, coasts and estuarine systems (shaded in Table 2.1). Emphasize will be on the application of a Current Deflecting Wall and connected sill. Figure 2.1 shows an example of a CDW-configuration for homogeneous flow conditions.

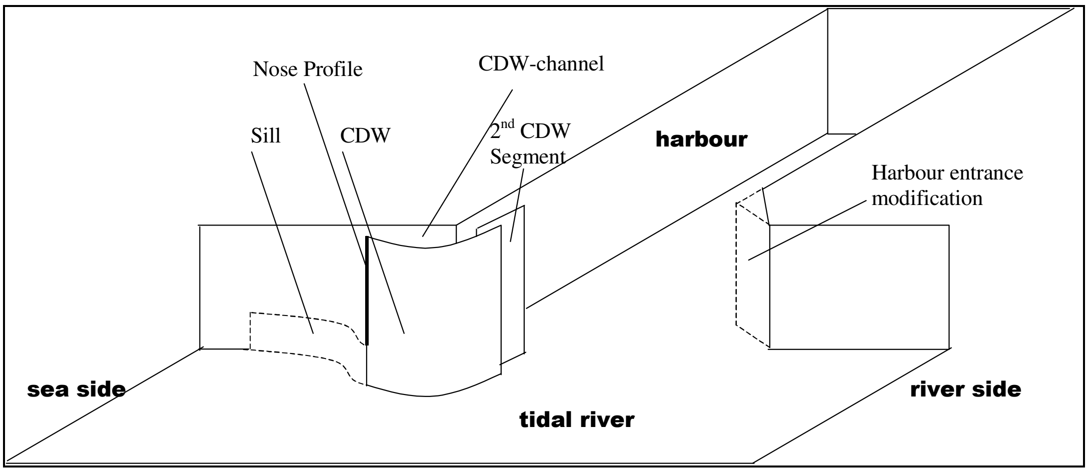


Figure 2.1: Sketch of Current Deflecting Wall for homogeneous flow conditions.

A Current Deflecting Wall[2] has the following advantageous effects:

- Diversion of high concentration, near-bed suspensions from the harbour entrance by means of a sill and/or other submarine constructions. In this way the effective value of c_a can be reduced.
- Decrease in the strength of the mixing layer and hence a reduction in horizontal entrainment rate through the adjustment of the local flow field in front of the harbour.
- The exchange flow rate can also be decreased by a diversion of the mixing layer away from the harbour mouth.
- Modification of the harbour geometry near the stagnation zone as this also affects the exchange flow rate.
- The exchange rate by salt-fresh water induced density currents can be decreased by the generation of a helical flow pattern with a longitudinal axis in the harbour mouth and a near-bed velocity component out of the harbour, see Figure 2.2.

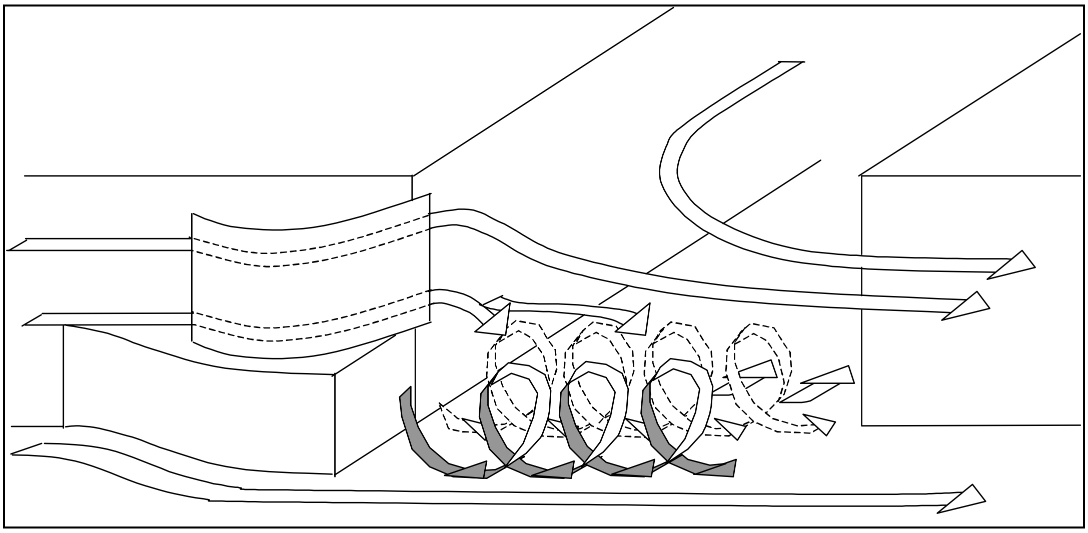


Figure 2.2. Schematic helical flow pattern in entrance during flood, with CDW present (non-homogeneous flow conditions).

2.3 Gained experiences

This section presents results of model investigations with physical scale models on the feasibility of CDW-systems for tidal rivers without and with the effect of density driven flow. All research was carried out between 1990 and 2000. One case study is not discussed as it relates to non-tidal conditions:

- A marina located on the right bank of the Meuse River near Roermond in The Netherlands. Research was carried out in a non-distorted scale model at the Delft University of Technology; see van Schijndel and Kranenburg (1998) and Winterwerp (2003).

[2] *Developed and patented by dr. H. Christiansen*

Köhlfleet harbour (Hamburg)

In 1991 a Current Deflecting Wall was constructed for the Köhlfleet harbour in Hamburg, see Figure 2.3. The harbour is located along the Elbe river, beyond the limit of salt intrusion but still with a considerable tidal influence. The design of the device was derived from model tests in a hydraulic model (Franzius-Institut, 1989).

Figure 2.3: Current Deflecting Wall in entrance of the Köhlfleet harbour.

Experiences during the last decade show that a considerable reduction in siltation volume in the harbour has been achieved (Christiansen, 1997). In Figure 2.4 the net siltation in the Köhlfleet harbour is compared with that in other harbours of Hamburg. In 1991, 1992 and 1995 the CDW was operational, whereas in 1993 and 1994 the CDW was damaged and did not function properly. The figure clearly illustrates that the CDW reduces net siltation considerably: 23%, 36% and 60% for the years 1991, 1992 and 1995, with an average of 40% for the three years. No reduction was observed in 1993 and 1994.

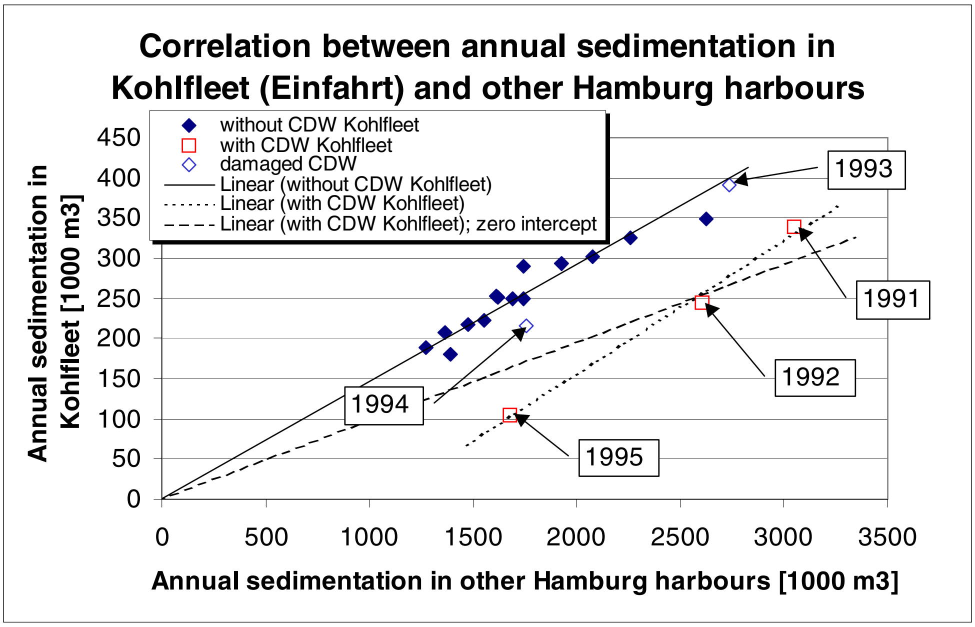


Figure 2.4: Annual siltation in the Köhlfleet harbour without and with CDW.

Parkhafen (Hamburg)

Due to the success of the CDW for the *Köhlfleet harbour* the Port Authorities of Hamburg decided to investigate the possibilities in reducing siltation for the adjoining Parkhafen, see Figure 2.5.

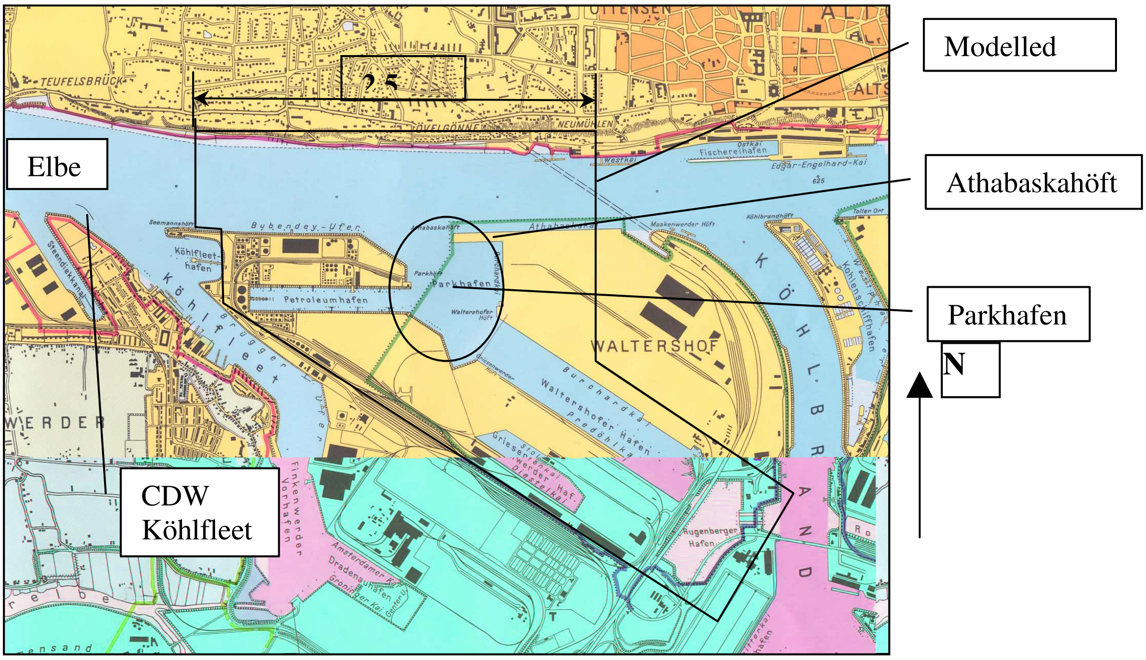


Figure 2.5: Plan view of Parkhafen, Köhlfleet and river Elbe and modelled area.

Tests with a physical scale model were carried out in 1992 and again in 1999 because of changed geometrical conditions for the Parkhafen (i.e. deepening of the Elbe and Parkhafen and closure of the Griesenwerder Hafen), see van Kruiningen et al (1992) and Kuijper and Winterwerp (2001). Optimisation of numerous CDW-configurations was done with floats and dye. The final result was assessed through the injection with dye (see Figure 2.6), from which a potential reduction of siltation in the Parkhafen of 30-50% was derived. Preparations for the construction of the real-scale prototype are presently being carried out.

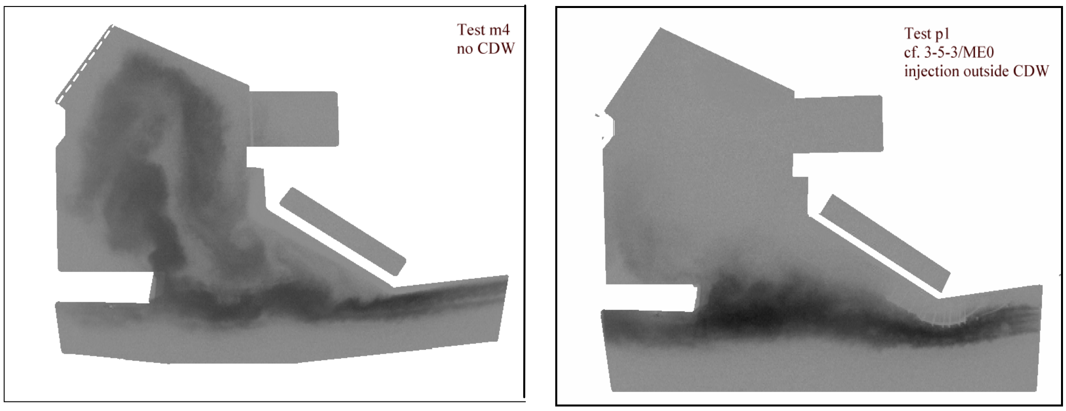


Figure 2.6: Dispersion of dye in the Parkhafen 700 s (model time) after start of dye injection upstream of the harbour entrance during flood. Left panel: without CDW; right panel: with CDW and dye injection at the outside of the CDW.

LIP-3 investigations

The two case studies, as described above, related to two existing harbours situated along a tidal river without the effects of density currents on harbour siltation. Fundamental research on exchange flows between a schematised estuarine harbour and river was carried out in the Delft Tidal Flume, as part of the EU-financed Large Scale Facilities and Installations Programme (LIP). Model tests were carried to investigate the potentials of a CDW, taking into account effects due to salt water-fresh water induced density currents. For the considered experimental conditions, resembling those prevailing in the Scheldt River near Antwerp, this resulted in a new CDW-sill system, which included an "upstream sill' at the opposing corner of the harbour entrance, see Figure 2.7.

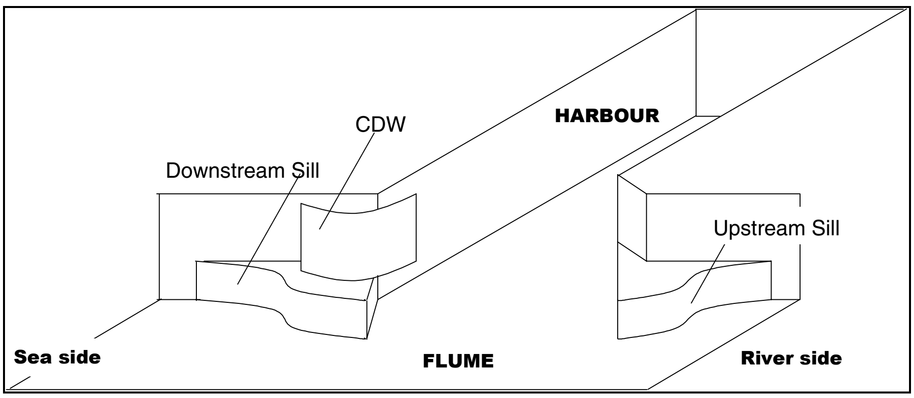


Figure 2.7: CDW-SILL system in the Delft Tidal Flume for non-homogeneous conditions (Hofland et al, 2001).

The results show that during subsequent phases during the tidal cycle the exchange flow of water between river and harbour can be affected by the CDW-sill system:

- During flood the helical flow pattern (Figure 2.2) results in a time-varying reduction of the near-bed flow into the harbour with an average of 70% during the whole flood period, see Figure 2.8;

- Around high water slack there is a small increase of 5% (compared with the situation without CDW), which is due to a decreased salinity in the harbour;
- During ebb the density current near the bed and the net flow are directed outward from the harbour, which is expected to have a minor impact on the siltation rate

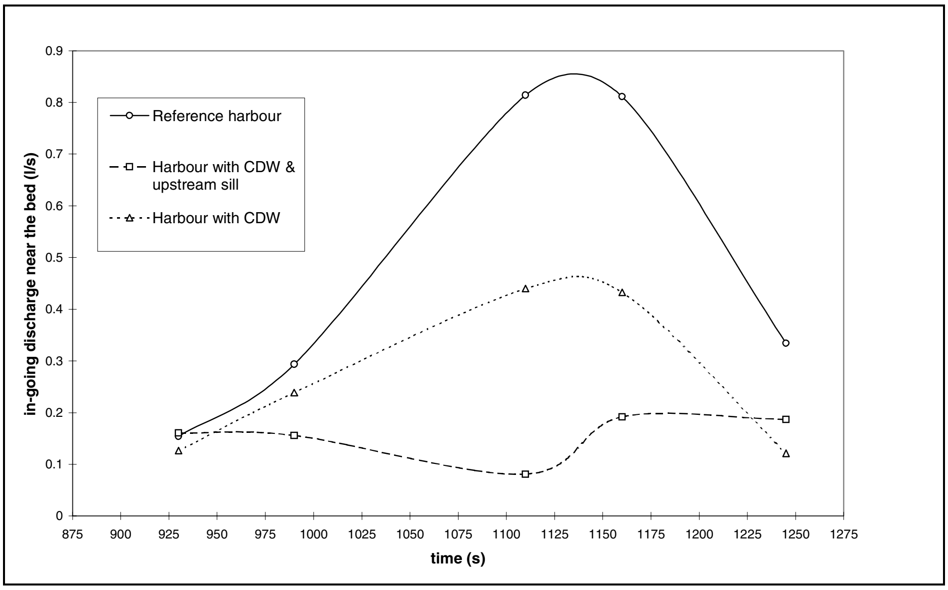


Figure 2.8: In-going discharge near the bed during periods from 925s to 250s in the next tidal cycle (Hofland et al, 2001).

3 Physical description of the Zeeschelde

3.1 General

The Zeeschelde is the section of the Scheldt between Ghent at the head of the Scheldt estuary and the Belgian-Dutch border near Doel. A sub-section of the river between Rupelmonde (the limit of salt intrusion) and Doel is known as the Beneden Zeeschelde and part of it forms the connection between the Port of Antwerp and the Western Scheldt estuary, see Figure 3.1.

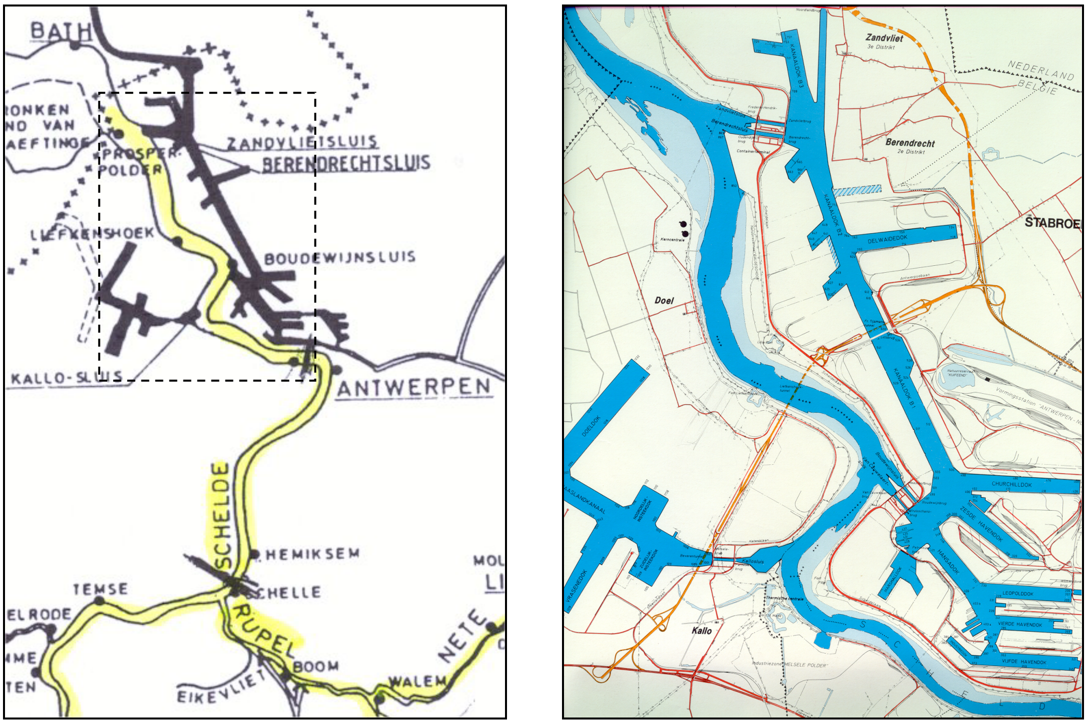


Figure 3.1: Beneden Zeeschelde (left panel) and detail (right panel).

The managing authorities, responsible for a proper functioning of the system, need to evaluate various measures that are anticipated in the nearby future to guarantee standards on safety, transport and water quality issues in the area. The fate of sediments in general and of fine-grained cohesive sediments in particular, forms a crucial role in many of these managerial questions. One of the important tasks is to guarantee safe navigational conditions, which requires that access channels and harbour docks are kept at specific design depths by means of continuous maintenance dredging. How to keep the dredging works at a minimal level is one of the goals of the CDW-project.

3.2 Hydrodynamics

The Scheldt estuary can be classified as macro-tidal, which implies that the average tidal range is more than 4 m. The tidal dynamics result in a periodic variation of water levels, flow velocities, salinities and suspended sediment concentrations. During propagation in upstream direction the tidal wave is deformed due to bed friction and convergence of the geometry of the estuary. The latter effect causes an increase of the tidal range between Vlissingen and Schelle and also an increase of the propagation velocity. The duration of the flood period becomes shorter (and thus the ebb period becomes longer), resulting in an asymmetric velocity variation in time with a flood dominant character.

Average flood and ebb velocities amount to 0.6 to 0.8 m/s. Change in flow direction from ebb to flood (i.e. during low water slack) does not occur simultaneously over the width of a cross-section. The turn of the tide in the shallow parts precedes that in the deeper ebb channels. During high water slack the change in flow direction occurs more synchronic. Due to the density gradient the near-surface flow velocities during flood decrease, whereas an increase occurs during ebb. Tidal discharges between Prosperpolder and Antwerp are one to two orders of magnitude larger than the mean river discharge (110 m^3/s at Schelle for the period 1997-1998, with extreme daily values of 25 and 567 m^3/s).

Long-term measurements (between October 1997 and December 1998) show salinities between 20 ppt in Properpolder and ppt g/l in Oosterweel. The river discharge has a large influence on the salinity distribution of the Beneden Zeeschelde. Typical longitudinal distributions are given in Figure 3.2, showing the effects of river discharge and tide on chlorinities and salinities.

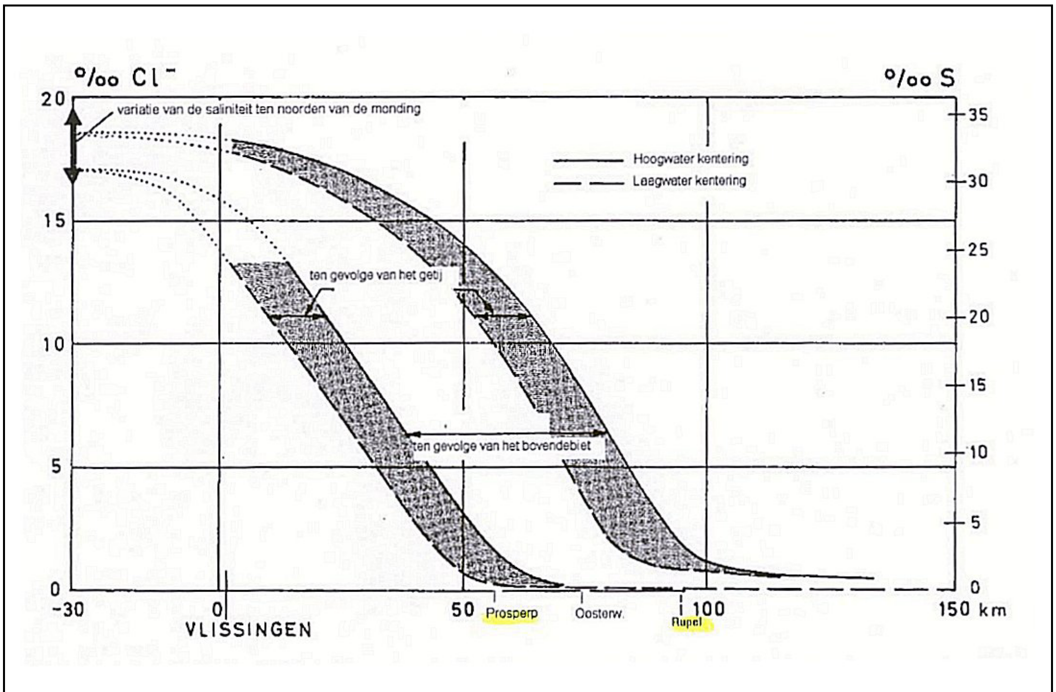


Figure 3.2: Salinity distribution in the Scheldt estuary (Peters and Sterling, 1976).

The salinity variations on the river determine the exchange flows due to density currents between the river and access channels to sluices and harbour docks. The magnitude of these variations near Prosperpolder during average flow conditions is 3-4.5 ppt. In the upstream regions (Oosterweel) the river run-off determines the salinity levels with values up to 15 ppt in the dry period and almost nil during the wet period.

Only for high river discharges the flood number is larger than 0.1, which implies stratified salinity conditions. However, also for lower river discharges the river can be stratified because of the meandering of the river.

3.3 Sediment dynamics

The suspended sediment concentrations vary in time and space due to the non-steady hydrodynamics (on time-scales of the semi-diurnal tidal cycle, the neap-spring tidal cycle and the seasons), the horizontal circulations (as induced by the geometry, intertidal areas etc.) and due to the vertical circulations (resulting from the gravitational flow but also from river bends). Generally, concentrations in the Beneden Zeeschelde are limited to several hundreds mg/l. However, near the bed layers with much higher concentrations, ranging between 1 and 100 g/l, can be present. They are formed during slack water and resuspended during the subsequent phase of the tide when flow velocities increase. Parts of the high-concentration layer may survive, especially during neap tide conditions. In that case the layer will consolidate resulting in a net accumulation of sediment.

Measurements indicate that suspended sediment concentrations are larger during spring tide conditions than during neap tide conditions (Oosterweel and Prosperpolder). Measurements near Zandvliet and Oosterweel show that resulting sediment fluxes during spring tide are three times higher than during neap tide. Concentrations are also higher during winter than during summer. This is attributed to variations in river discharge (effect on turbidity maximum, sediment input), temperature (biological activity, flocculation etc.), storm effects (also on sea resulting in an increase of ambient suspended sediment concentrations) and land erosion. It is estimated that, on a yearly basis, 10-30 10^6 ton of sediment mass (mainly mud) is transported in upstream direction and a comparable amount in downstream direction.

The turbidity maximum is located upstream of the zone with salinities of several ppt. The location varies with the river discharge and from measurements it follows that during dry periods the turbidity maximum can penetrate as far as St. Amands, i.e. 110 km from Vlissingen. High river discharges cause the turbidity maximum to be shifted in downstream direction between Prosperpolder and Bath, i.e. 50 km from Vlissingen.

Sedimentation rates have been measured in the access channel to the Kallo sluice, indicating a net accumulation of 0.014 m/day (or 5 m/year). Similar measurements in- and outside a sediment trap near the entrance of the Berendrecht sluice show values of 0.04-0.07 m/day inside the trap and 0.01-0.02 m/day outside. Net sedimentation rates in the navigation channel are much lower, amounting to 0.002 m/year as based on bathymetric surveys. Accumulation of wetlands and intertidal areas vary between 0.002 to 0.02 m/year.

4 Modelling approach

Spatial domain

A study on the effectiveness of a Current Deflecting Wall on the reduction of siltation in a harbour basin or access channel requires the proper modelling of small-scale processes in a natural environment. The time-varying local conditions with respect to hydro-dynamics and sediment dynamics are determined by the large-scale processes in the water system, such as tidal propagation, salinity intrusion and advection of suspended sediment, as discussed in the previous chapter. The length scale of these processes is at least the tidal excursion (the order of ten kilometres) in both directions from the area of interest and even more if dispersion due to the three-dimensional character of the processes is taken into account. The locations of the model boundaries of this *local* model should be at least at distances from the area of interest as given by this length scale, to avoid that the prescribed boundary conditions are influenced by modifications in the interior of the model. Conditions at these boundaries (Boundaries 1 and 2) with respect to hydrodynamics (and sediment dynamics for a sediment transport model) should be prescribed as a function of time for the whole simulation period. They can be based on field measurements or derived from a *global* model. The *local* model may be either a physical or a numerical model. A schematic representation of the *local* model is given in Figure 4.1.

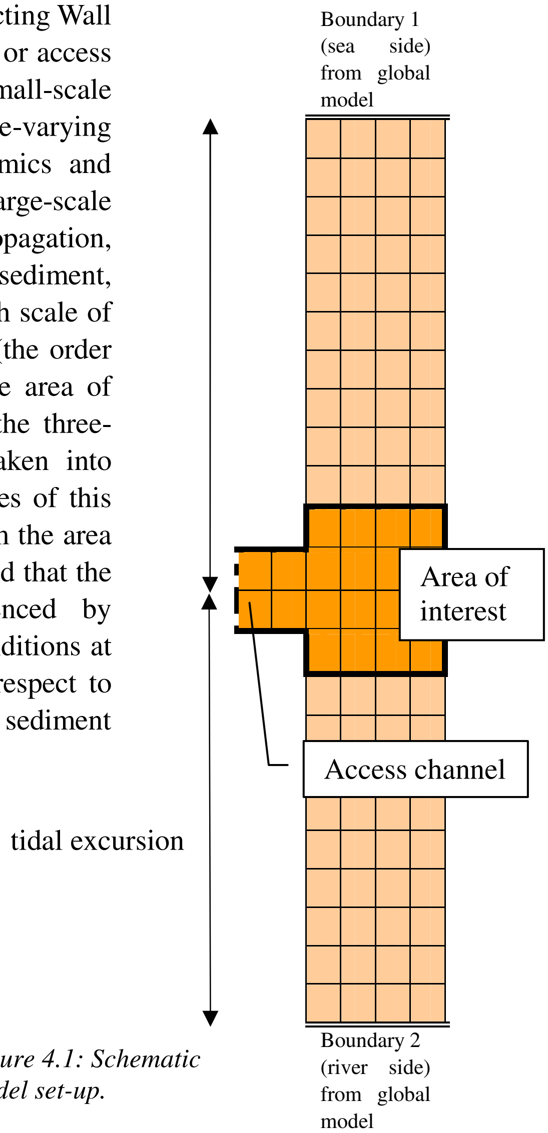


Figure 4.1: Schematic model set-up.

Model requirements

The anticipated CDW in the Beneden Zeeschelde will be located in the area with salt intrusion, where the flow is highly three-dimensional, see Chapter 3. Thus, even in the case without CDW the model should be three-dimensional for a proper assessment of the exchange flows between river and harbour. To study the effect of a CDW, model requirements are even more severe due to flow separation along the CDW, the induced three-dimensional helical flow in the harbour entrance and the flow across the sill. Three-dimensional numerical models based on the shallow water approximation are not suitable, because vertical accelerations of the flow are neglected. Furthermore, flow separation depends on the curvature and roughness of the CDW and poses major requirements on turbulence modelling.

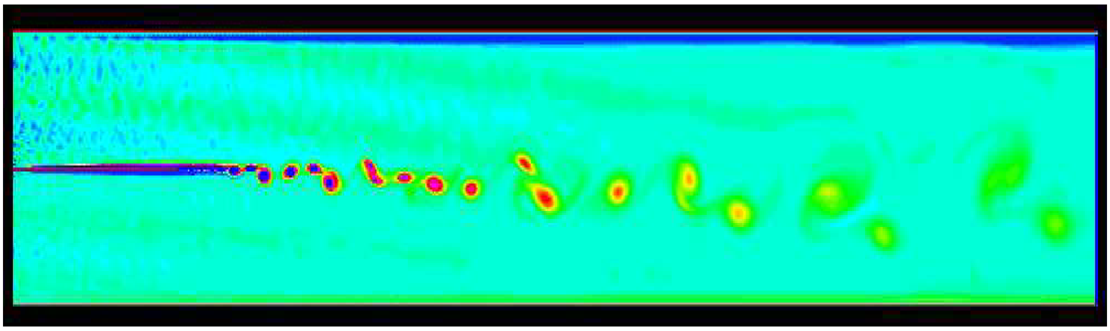


Figure 4.2: DELFT3D simulations: mixing layer due to velocity differences between upper and lower layer (figure is slightly distorted).

Although small-scale simulations on flow separation already existed (see Figure 4.2), computing resources were insufficient to perform simulations for practical situations, because of the required high resolution of the grid near the CDW. Increased computational speed of computers and recent developments such as horizontal large eddy simulations, domain decomposition and the inclusion of the momentum equations for all three spatial directions in the numerical model presently allow for a much better representation of the relevant processes. However, no experiences have been gained yet with numerical models to obtain an optimum design of a CDW for a specific case.

The aforementioned processes are implicitly accounted for by a physical model, provided that the model is properly scaled and scale effects can be neglected. Because of the three-dimensional character of the flow the model should be non-distorted (i.e. the horizontal and vertical length scales are the same) or, at most, weakly distorted. Model tests for homogeneous tidal conditions have shown that with a distortion of 2 the rate of horizontal exchange between river and harbour is not affected (van de Graaff and Reinalda, 1977). It is noted that for a proper modelling of the longitudinal salinity intrusion larger values of the distortion can be applied (up to 10, see van der Heijden et al, 1984), but in that case local three-dimensional flow patterns are not represented correctly.

From the comparison between physical and numerical models, as given above, it follows that a study regarding the implications of a CDW on exchange flows and its design is still most appropriate employing a physical model. However, numerical models become more and more suitable for this task as relevant physical processes are being implemented in the software code and increased computing speed and numerical techniques allow the necessary detailed simulations. A comparison with results from a physical model will have to show the capabilities of numerical models. If validated satisfactorily, a sediment transport model may be coupled to the hydrodynamic model for an assessment of the net siltation in the harbour basin. The latter is not feasible in a physical model, as the many physical process associated with the transport of (cohesive) sediments can not be scaled correctly.

Modelling suite

The tools for the design of a CDW in the Beneden Zeeschelde that have been developed as part of the current project are as follows:

1. *Numerical Scheldt model (global model).* A three-dimensional hydrodynamic model for the Scheldt between Bath/Waarde and Schelle/Rupelmonde. The model is used to

supply the boundary conditions for the local models. In future the model will be coupled with a sediment transport model.

2. *Numerical scale mode (local model). (i)* a two-dimensional (depth-averaged) application of DELFT3D for the calibration of the physical scale model and (ii) a three-dimensional application to simulate the effect of a CDW. Calibration will be done on the basis of field measurements. With the model the effects of the distortion of the physical scale model will be investigated.
3. *Physical scale model (local model).* Following calibration against field data, the model will be used (i) to arrive at an optimum design of the CDW and (ii) to validate the *numerical* scale model regarding the effect of a CDW.

In addition, an existing two-dimensional (depth-averaged) model for the whole Scheldt estuary was used, to supply boundary conditions for the numerical Scheldt model (and also both local models).

The development of tools, as carried out within the framework of Delft Cluster, thus consists of the integration of physical and numerical models, which are calibrated against field data. This is schematically shown in Figure 4.3.

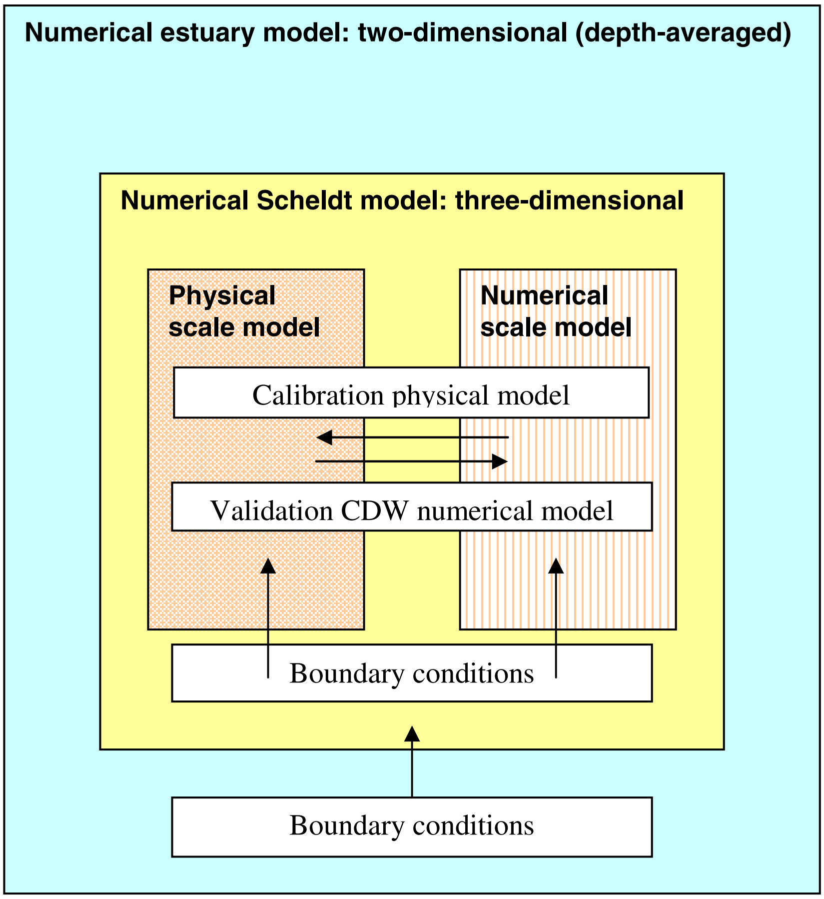


Figure 4.3: Schematic representation of modelling tools and their interactions.

5 Modelling elements

5.1 Project area

The project area is shown in Figure 5.1, where the (approximate) location of the future access channel is already indicated.

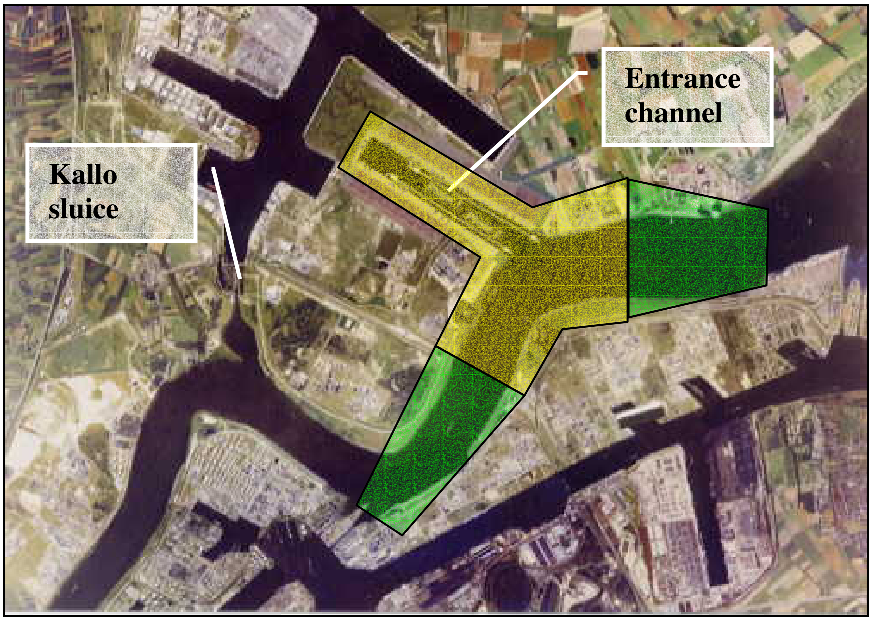


Figure 5.1: Location of future access *channel.*

In the yellow area the bathymetry of the prototype is accurately represented in the model. This is over a distance of 1-2 km along the river to both sides, starting from the entrance channel. The green areas form transition zones that gradually connect to the Delft Tidal Flume.

5.2 Physical scale model

Configuration

The Delft Tidal flume in its original state consists of a straight flume with a length of 130 m and a cross-section of 1x1 m^2, which is connected to a sea basin, see van Leussen and Winterwerp (1990). The water level is controlled by a cylinder with a horizontal axis. By means of rotation of the cylinder an overflow weir is moved vertically. At the upstream side of the flume tidal discharges as well as a constant river flow can be imposed. With the injection of brine the sea water can have a density up to 1030 kg/m^3. Tidal periods may vary between 30 and approximately 1800 s. A schematic sketch of the Tidal Flume is given in Figure 5.2.

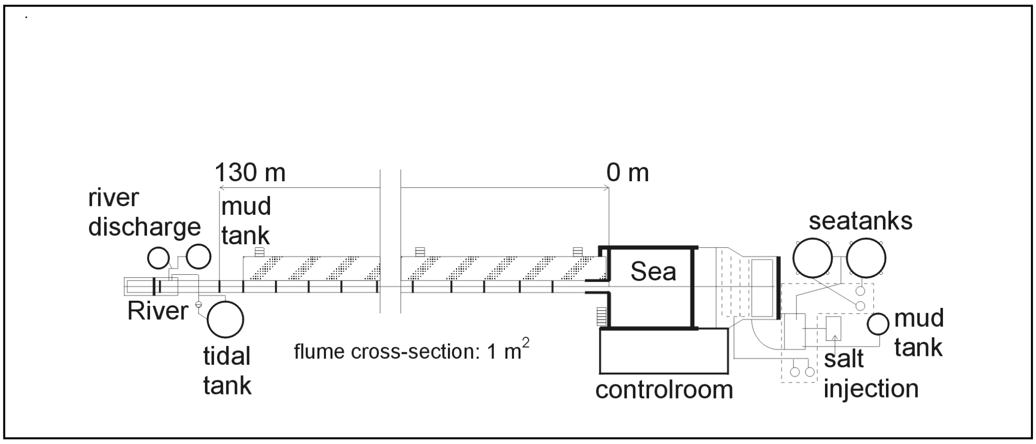


Figure 5.2: Delft Tidal Flume

The model was constructed on an elevated platform alongside the flume. Glass panels were removed over a certain stretch of the flume to connect the flume with the modelled site area. Furthermore, topography and bathymetry were 'mirrored', because space to construct the model was only available at the right side of the river (looking in seaward direction). In the area of interest the bathymetry was modelled with concrete, whereas for the transition areas coarse sand was used.

Model scales

In a model with free surface flow the ratio of the inertia and the gravity forces should be preserved as these are the dominant forces. This ratio is given by the Froude number (Fr):

$$Fr = \frac{u}{\sqrt{gh}} \tag{5.1}$$

with,

u - flow velocity [m/s]
g - gravitational acceleration [m/s^2]
h - water depth [m].

If density-driven flow plays a role, the internal Froude number instead of the Froude number should be used:

$$Fr_{\text{internal}} = \frac{u}{\sqrt{\frac{\Delta\rho}{\rho} gh}} \tag{5.2}$$

The scale factor (n) for quantity Γ is defined as:

$$n_\Gamma = \frac{\text{value of } \Gamma \text{ in prototype}}{\text{value of } \Gamma \text{ in model}} \tag{5.3}$$

The internal Froude number in the prototype and the model should be equal and thus $n_{Fr,\ internal}$ = 1. From this it follows that the length scales for the water depth and the longitudinal velocity are related according to:

$$n_u = \sqrt{n_h} \tag{5.4}$$

if the relative density difference $n_{\Delta\rho/\rho}$ is scaled as 1.

The choice for the horizontal length scale n_L is based on practical considerations (required space and costs); for the model n_L = 300 is chosen. In an undistorted model this would result in a water depth of maximal 20/300 = 0.07 m, which is too small for accurate measurements. The model is therefore slightly distorted and a vertical scale of 150 is selected, implying a distortion of 2. With Eq. (5.4) the velocity scale becomes 12.25, resulting in a time scale $n_t = n_L/n_u$=300/12.25=24.5. In that case the duration of the semi-diurnal tide in the model becomes 1830 s. Characteristic values of various parameters for the prototype and the model are given in Table 5.1.

Parameter	**Scale**	**Prototype**	**Model**
Water depth	150	14-20 m	0.09-0.13 m
Tidal range	150	5.2 m	0.035 m
River width	300	400 m	1.3 m
Tidal excursion	300	15000 m	50 m
Salinity	1	0-10 ppt	0-10 ppt
Horizontal velocity	12.25	0.9 m/s	0.07 m/s
River discharge	$0.55\ 10^6$	120 m^3/s	0.2 l/s
Tidal period	24.5	12.5 hrs	1830 s

Table 5.1: Values of parameters in the prototype and the physical scale model.

With the selected scales it is verified, if turbulence is sufficiently developed and scale effects due to viscosity can be neglected. The Reynolds number is given by:

$$\mathrm{Re} = \frac{u\,h}{\nu} \tag{5.5}$$

where u is the horizontal velocity [m/s], h is the water depth [m] and ν is the cinematic viscosity [m^2/s]. With the characteristic values of Table 5.1 and $\nu = 10^{-6}$ m^2/s, the Reynolds number amounts $6\ 10^3$–$9\ 10^3$, which is sufficiently large for well-developed turbulent flow.

In Figure 5.3a the physical scale model for the Beneden Zeeschelde, close to the future entrance channel, is shown. A detail, near the project area, is given in Figure 5.3b.

Figure 5.3a: General view of physical scale model.

Figure 5.3b: Detail of physical scale model: river (from left to right) and access channel. (top).

The bathymetry, as reproduced in the physical scale model, is shown in Figure 5.4.

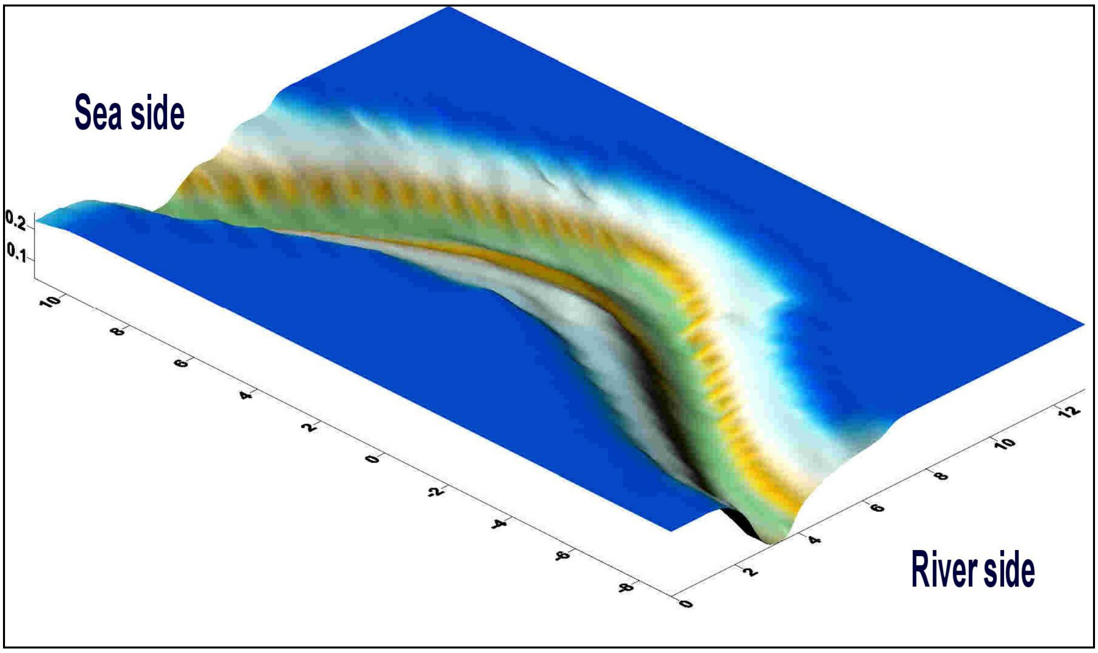


Figure 5.4: Bathymetry of the Beneden Zeeschelde between Kallo and Zandvliet in the physical scale model (figure is distorted).

Target conditions

The target conditions are defined as those conditions for which the hydrodynamics, including the salinity distribution, near the entrance channel should be reproduced by the model. These are considered to be characteristic for the mean, long-term siltation processes. Starting point is the average tide at Liefkenshoek, a few hundred m ‘up-estuary’ from the entrance channel. The amplitude is increased with a factor 1.14, which is representative for the spring tide on June 12, 2002, during which field measurements were done. The tide also resembles the long-term average spring tide at Liefkenshoek. The salinity difference between sea and river water is chosen somewhat larger than the mean difference to account for distortion effects of the model, implying that a conservative approach is followed.

Calibration

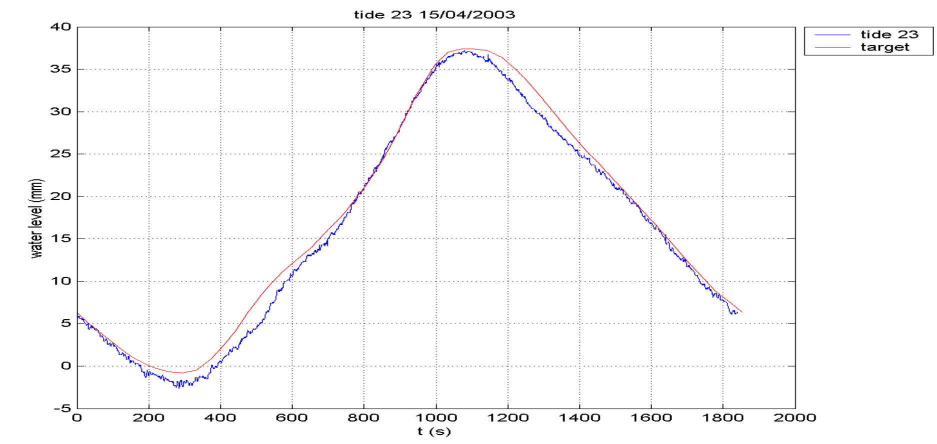
Calibration aimed at the reproduction of water levels, flow velocities and salinities on the Beneden Zeeschelde near the entrance channel for the target conditions as described above. During calibration, the entrance channel was not connected to the river, thus representing the present conditions in the Beneden Zeeschelde. The adopted procedure was as follows:

- Set-up of a numerical coarse grid and depth-averaged model (100x10 grid cells in longitudinal and lateral direction) for the Tidal Flume, see Chapter 4. With the model the boundary conditions were set in such a way that the water level and the flow velocity near the entrance channel were reproduced for the target conditions.
- With a few model tests in the Tidal Flume the boundary conditions were further optimised.
- Additional mixing elements were fixed to the flume bed and attached to the side walls in the transition between the sea basin and the river bend to influence vertical mixing.

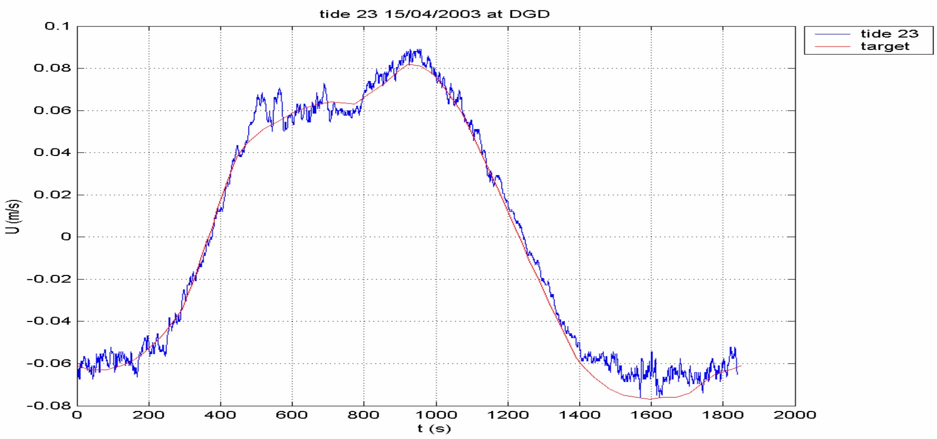
- Reduction of the density difference between the sea and the river and 10% modifications of river discharge and tidal range.
- Fine tuning of the boundary conditions by slightly changing the phases and amplitudes.

Final results of the calibration are given in Figure 5.5a (water level), Figure 5.5b (velocity) and Figure 5.5c (salinity). Water level and flow velocity in the Beneden Zeeschelde near the entrance channel are fairly good reproduced. Figure 5.5c shows the realised salinity variation (coloured dots), the measured salinities in 1995 and 2002 as well as the target conditions. The coloured dots represent measured values at different positions in vertical direction (varying between 0 and 7.2 cm below the reference level), indicating the vertical stratification. For further details reference is made to (van Kessel and Cornelisse, 2003).

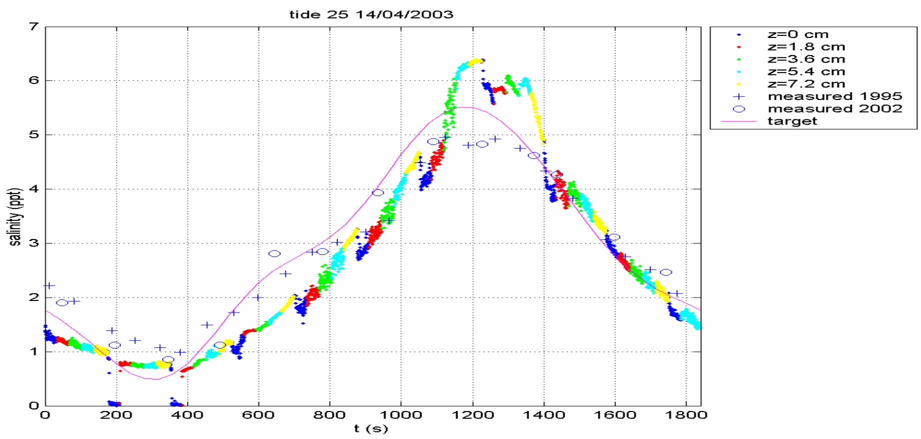
Figure 5.6 presents phase-amplitude diagrams for the vertical tide (Figure 5.6a) and the horizontal tide (Figure 5.6b). The target and realised values for each of the four Fourier-components are compared (the first component relates to the semi-diurnal tide, the second to the quarter-diurnal tide etc.). The results show a good correspondence between target and realised conditions.



a: Water level

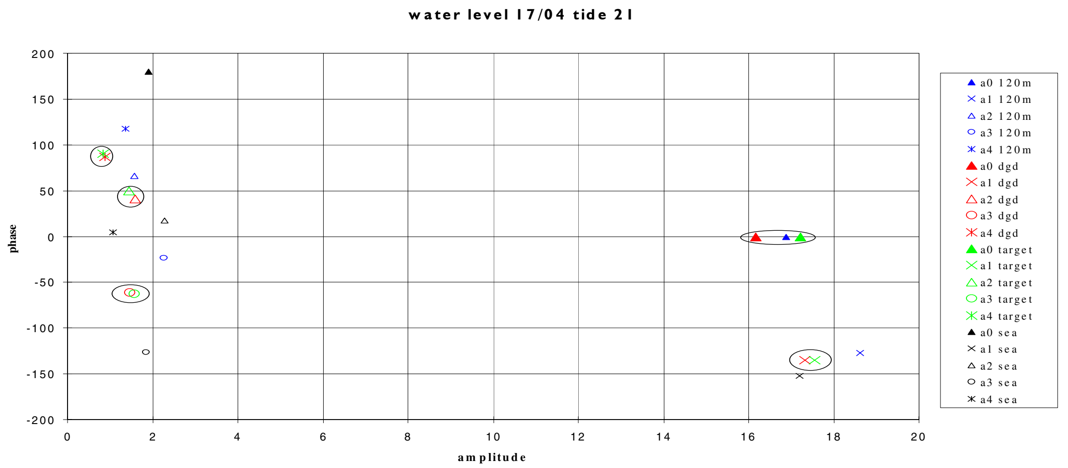


b: Velocity

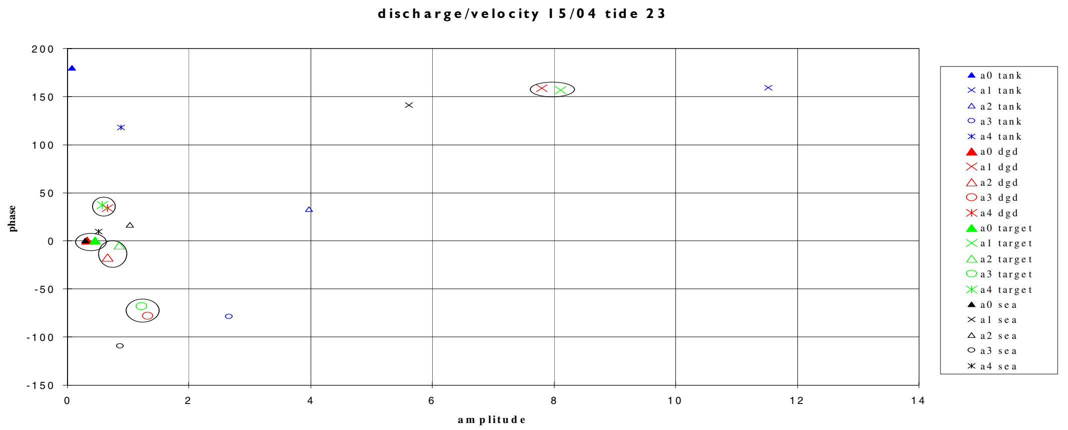


c: Salinity

Figure 5.5: Calibration results for the physical scale model (van Kessel and Cornelisse, 2003).



a: Water level near entrance channel.



b: Velocity near entrance channel.

Figure 5.6: Amplitude-phase diagrams.

Finally, Figure 5.7 compares the phase differences between the water level, velocity and salinity for the target conditions and as realised in the model. A good reproduction of these phase differences is essential, because of the 4 different time-intervals that can be distinguished during the tidal period:

- Phase I: tidal filling, density current out
- Phase II: tidal filling, density current in
- Phase III: tidal emptying, density current in
- Phase IV: tidal emptying, density current out

The transition between Phase I and II occurs near LW slack (zero velocity). The transition between Phase II and III occurs near HW. The transition between Phase III and IV occurs near HW slack (zero velocity). The transition between Phase IV and I occurs near LW. The transitions I–II and III–IV are not very sharp, as the peaks of minimum and maximum salinity are quite wide. These transitions have therefore been indicated with thick grey lines.

The figure shows that the four periods are adequately reproduced.

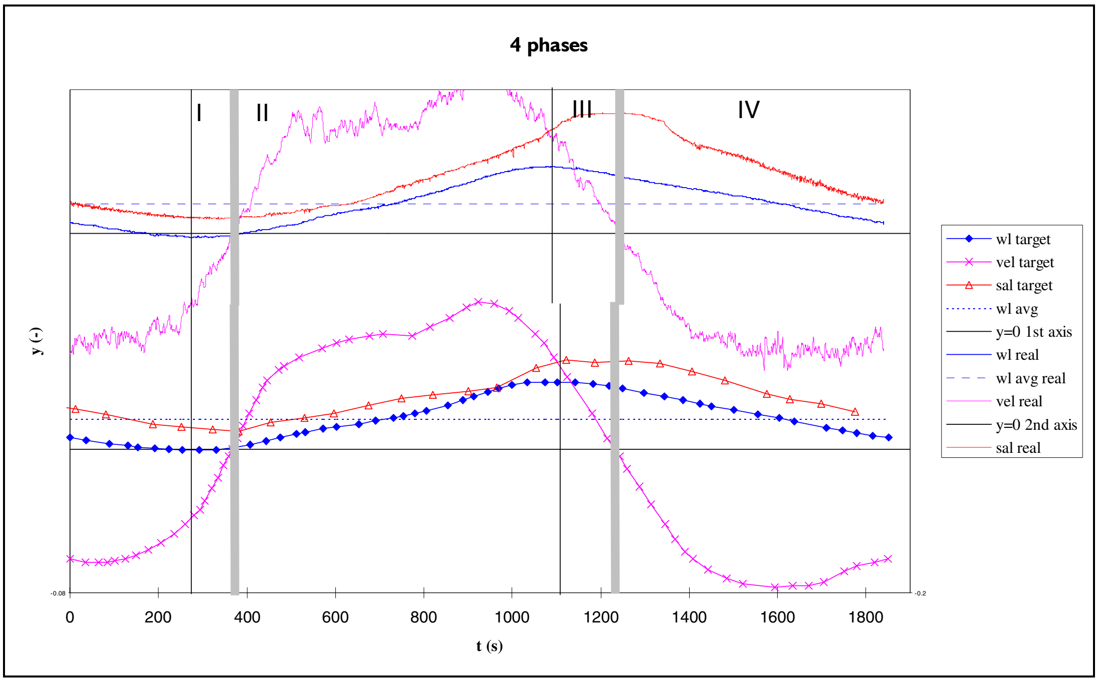


Figure 5.7: Comparison of phase differences of target water level, velocity and salinity. Top panel: realised curves (tide 23 15/05/2003); bottom panel: target curves.

CDW tests

With the construction and calibration of the physical scale model in the Delft Tidal Flume and the parallel set-up of the numerical models it can be concluded that the necessary tools for an optimum design of the CDW have been made available. Following calibration, the entrance channel is connected to the Beneden Zeeschelde and model tests with various CDW-configurations are carried out. Recalibration is required after opening of the tidal dock. The target conditions do not significantly change, as results from the 2DH numerical Scheldt model demonstrate (not shown herein). Because of the storage volume added by opening the dock, the boundary conditions have to be adapted, however. This is done, again, with the numerical model suite.

Assessment of the various CDW-configurations is obtained by means of visual observation using dye injection and Particle Image Velocimetry (PIV). In the latter case floats are monitored with a digital video camera from which the spatial distribution of the velocities is computed. In a number of locations along the river and in the access channel vertical salinity distributions and horizontal flow velocities are measured. Exchange flow rates between river and access channel are derived from the PIV-measurements as well as from dye tests. The latter is illustrated in Figure 5.8, showing the spreading of dye in the entrance of the access channel. The red-coloured dye represents the near-bed water with a high density entering the harbour, while the blue-coloured dye represents the out flowing near-surface water with a relatively low density.

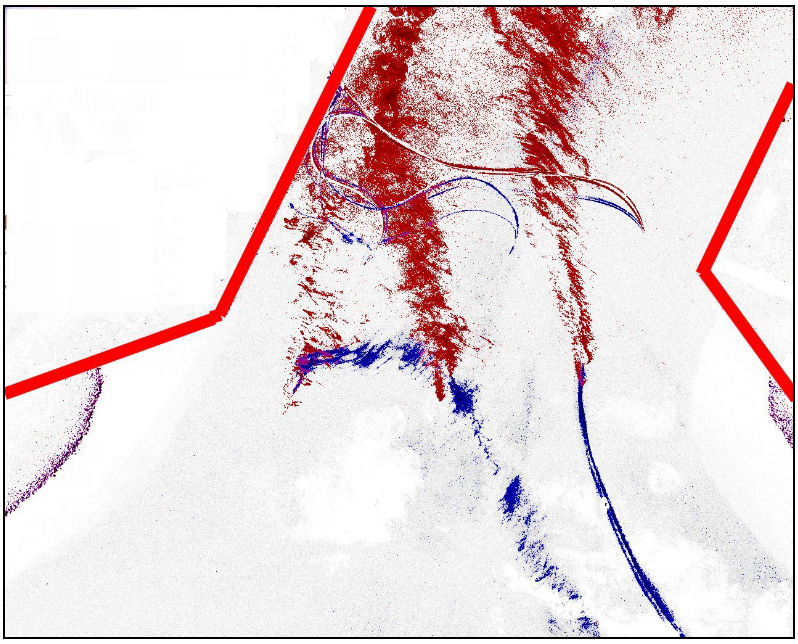


Figure 5.8: Density currents in entrance of channel as visualised with dye (red: inflowing near-bed water; blue: out flowing near-surface water).

Detailed results of the model tests on the design of the CDW will be reported elsewhere.

5.3 Numerical modelling

The numerical modelling consists of a DELFT3D schematisation for:

1. the Delft Tidal Flume, denoted as *numerical scale model,* as a depth-averaged (2Dh) and as a three-dimensional model;
2. the Beneden Zeeschelde between Waarde in the Western Scheldt and Schelle on the river Scheldt, referred to as *numerical Scheldt model*, as a fully three-dimensional application.

Numerical scale model

Firstly, as described in Section 5.2, the *numerical* scale model is used to optimise the boundary conditions for the *physical* scale model. For this purpose a relatively coarse grid was used of 100 grid cells in longitudinal direction and 10 grid cells in lateral direction, see Figure 5.9

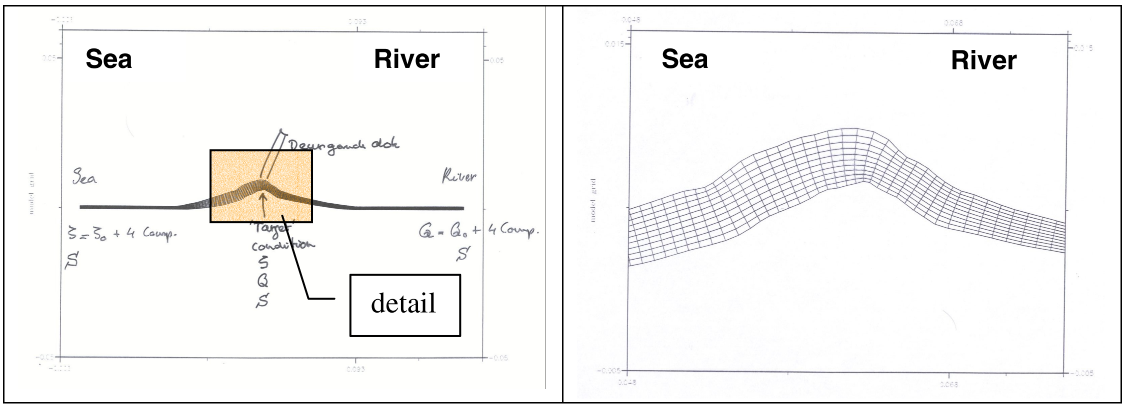


Figure 5.9: Grid schematisation of numerical scale model (coarse version).

Part of the modelled area thus consists of two straight flume sections and, in between, the Beneden Zeeschelde, with an accurate representation of the bathymetry, and two transition zones. All dimensions are on model scale. The model is run in 2D (depth-averaged) mode and first estimates of the boundary conditions are derived from a third overall model for the whole Scheldt estuary. This estuary model includes the coastal region, the Western Scheldt and the upper tributaries of the Scheldt, see Figure 5.11. The roughness of the numerical scale model was determined on the basis of a steady state test, i.e. a constant river discharge without tidal forcing. Results with respect to the obtained Nikuradse roughness are given in Figure 5.10.

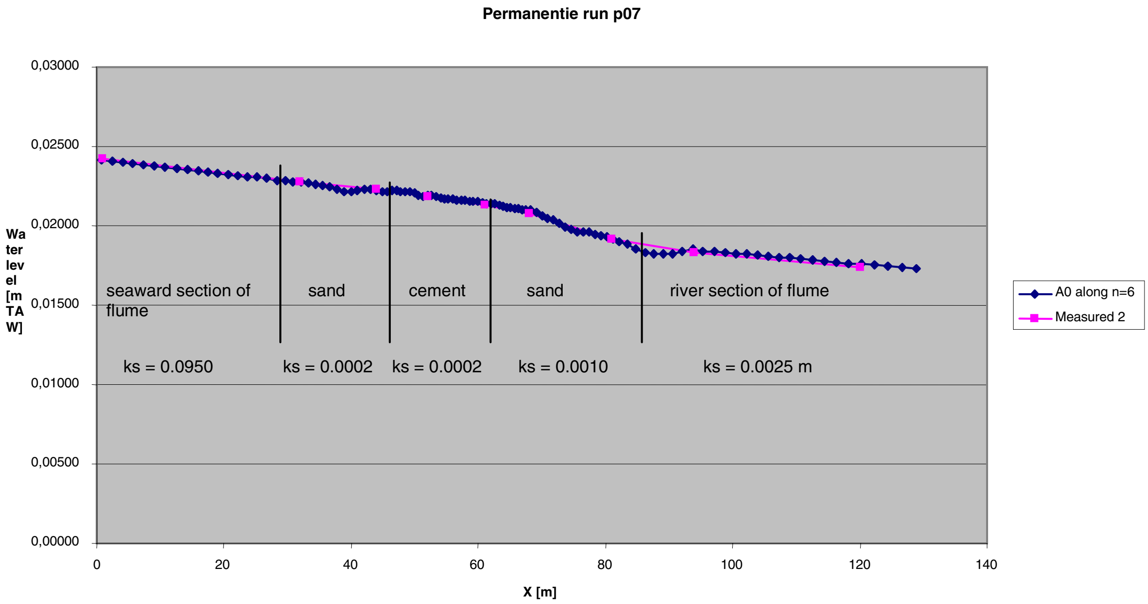


Figure 5.10: Calibration of numerical scale model and derived Nikuradse roughness.

Secondly, the numerical scale model is used to simulate in 3D the rate of exchange between the river and the entrance channel without and with CDW. Results of the numerical scale model are compared with the results of the physical scale model. For this purpose the grid is refined and the software code is extended with process formulations and new numerical schematisation techniques:

- Fixed-layer schematisation in vertical direction instead of the σ-transformation;
- Inclusion of a non-hydrostatic pressure distribution;
- Smooth-boundary approximation;

in combination with existing, recently developed, features:

- Horizontal Large Eddy Simulation (HLES);
- Domain decomposition.

The new features of the DELFT3D software package will be discussed hereafter for the numerical Scheldt model.

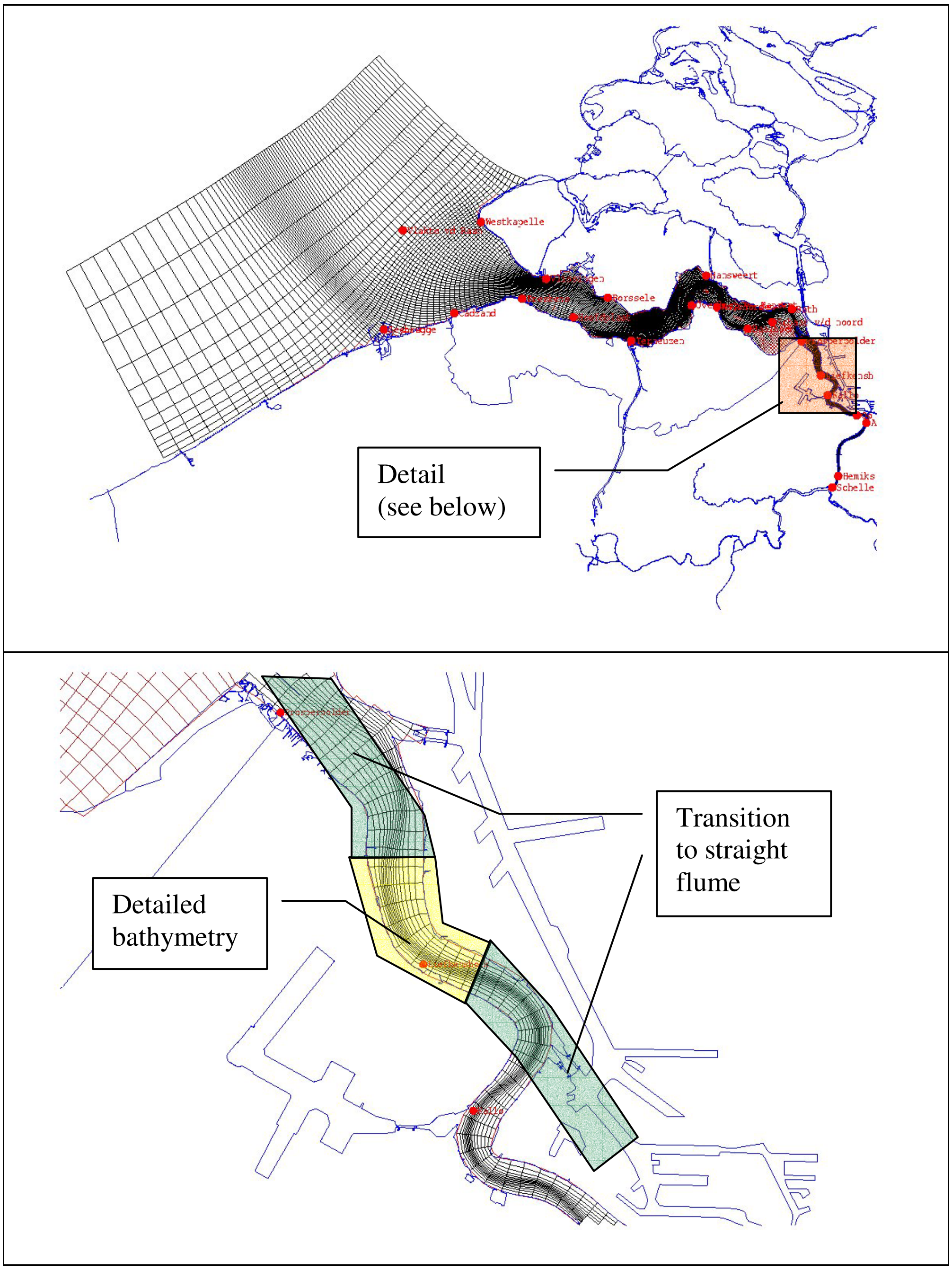


Figure 5.11: Overall Scheldt estuary model (upper) and detail for Beneden Zeeschelde with Tidal Flume.

Numerical Scheldt model

The numerical Scheldt model will be used by the responsible authorities of the Beneden Zeeschelde to address adequately the multiple managerial questions related to sediment transport. This requires the availability of a predictive model on hydro- and sediment dynamics with the following functionality:

- Accurate modelling of the complex, meandering geometry and bathymetry of the Beneden Zeeschelde, including the river banks, channels and intertidal areas;
- Modelling of the geometry and flow near guiding walls, over sills and around CDW's and other constructions;
- Simulation of the horizontal and vertical tide, including the effects of drying intertidal areas;
- Simulation of the horizontal and vertical salinity distributions and the induced density currents;
- Adequate representation of the isotropic turbulence field, including the effects of vertical salinity gradients;
- Modelling of the horizontal circulation patterns;
- Modelling of the secondary flow in river bends;
- Options for the future extension of the hydrodynamic model with a sediment transport model.

To fulfil these requirements a number of features, already available in research versions of DELFT3D, have been made operational as part of the current project:

Fixed-layer schematisation

For the 3D-modelling of hydrodynamics in coastal areas, estuaries and lakes two commonly used vertical grid systems can be distinguished: the z-coordinate system (ZCM) and the σ-coordinate system (SCM), see Figure 5.12.

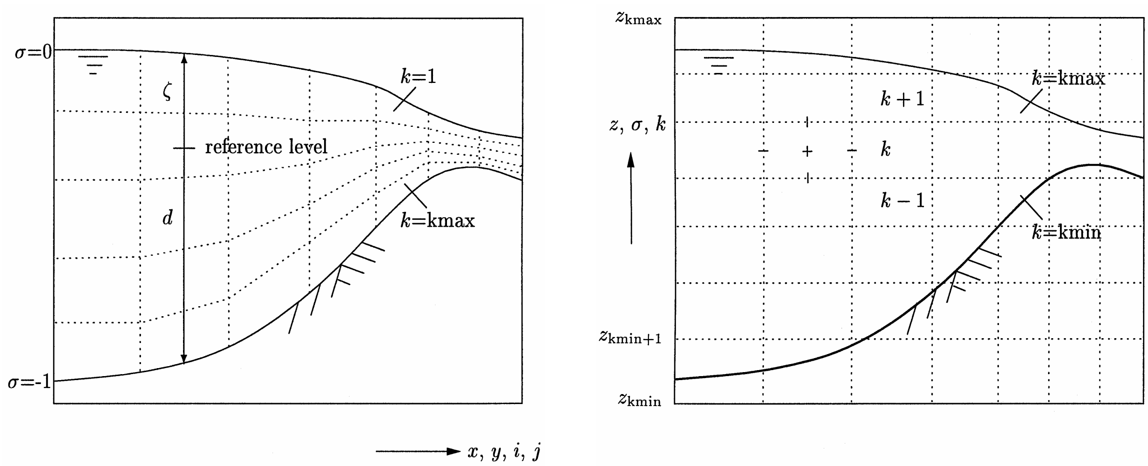


Figure 5.12: Vertical computational grid z-coordinate model (left) and the σ–model (right) (Van Kester et al, 2002).

The vertical grid should (i) resolve the boundary layer near the bottom, (ii) be fine around pycnoclines and (iii) avoid large truncation errors in the approximation of strict horizontal gradients (van Kester et al, 2002). Neither of the coordinate systems meets all the requirements. The ZCM model has horizontal coordinate lines which are (almost) parallel with isopycnals. In that case an irregular bottom will be represented by a staircase (zig-zag boundary), resulting in inaccuracies in the approximation of the bed shear stress and horizontal advection near the bed. The SCM model is boundary fitted, but will not have enough resolution around the pycnocline. This leads to significant errors in the approximation of strictly horizontal gradients (Van Kester, 2002) and also in the accurate modelling of structures. For the CDW-project a special version of the ZCM was implemented, allowing the free surface to move through the vertical grid and taking into account variable grid sizes near the bed. This permits a more accurate modelling of training walls (with their top at mean water level), sills etc. In addition, the representation of a non-hydrostatic pressure distribution (see hereafter) is presently implemented for the ZCM.

Non-hydrostatic pressure distribution
Three-dimensional models for boundary layer type of flows usually assume a hydrostatic pressure distribution in vertical direction. This assumption is valid if vertical accelerations are relatively small and the length scale of the flow phenomena in horizontal direction is much larger than in vertical direction. DELFT3D makes use of this shallow water approximation, which makes it more efficient in solving the equations. However, in cases with orbital movements under short waves, vertical circulations, with abruptly changing flows over a varying bottom-topography and around constructions the non-hydrostatic pressure component cannot be neglected. The latter case is relevant for the flow as influenced by a Current Deflecting Wall such as the helical flow downstream of the CDW and the flow overtopping a sill. In order to capture non-hydrostatic flow phenomena, the hydrostatic version of the z-model is extended with a non-hydrostatic module. For a detailed description of the numerical backgrounds reference is made to Bijvelds (2003).

For local three-dimensional flow the grid spacing in horizontal direction should be of the same order of magnitude as the vertical grid spacing. It will increase considerably the computing time, however by computing the non-hydrostatic pressure only in the region of interest (i.e. near the CDW) the computational effort can be reduced significantly. This option has been made available in DELFT3D.

The implementation of the non-hydrostatic module has been tested for a number of cases, see Bijvelds (2003). Figure 5.13 illustrates the effect of the non-hydrostatic pressure on the vertical mixing near the front of a gravity current, which is much larger in the hydrostatic model.

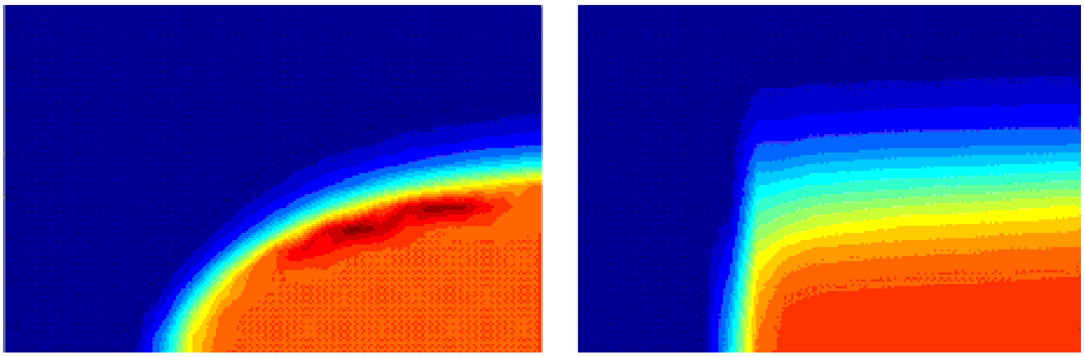
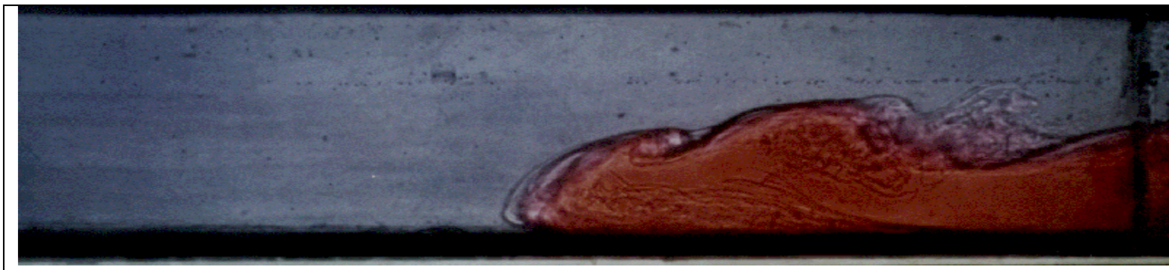
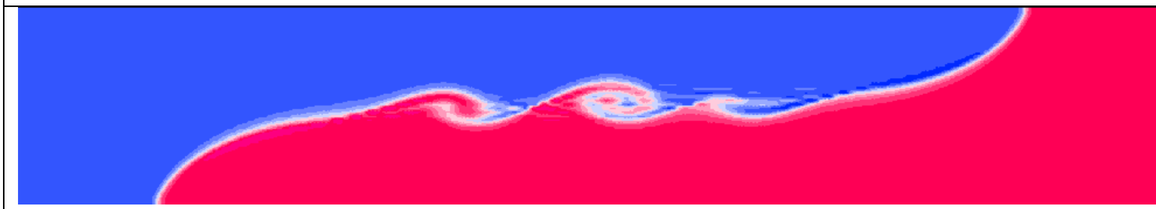


Figure 5.13: Characteristic salinity distribution (side view) of the front of a gravity current; non-hydrostatic model (left) and hydrostatic model (right) (Bijvelds, 2003).

Figure 5.14 shows the growing of Kelvin-Helmholz instabilities at the density interface of gravity current. The growing of these instabilities can be simulated by a non-hydrostatic model only, provided that the grid resolution is sufficiently large.



The development of Kelvin Helmholtz instabilities near the front of gravity current.



Characteristic salinity distribution (computed) during a lock-exchange experiment using a high resolution grid (300 x 50 grid cells).

Figure 5.14: Exchange flow, observed in a flume (top panel) and computed (lower panel).

Smooth-boundary approach

The use of an orthogonal grid, such as in DELFT3D, can result in so-called ‘stair-case’ schematisations along river banks, training walls, etc. These stair-cases can influence the local flow by introducing additional, artificial roughness, which leads to spurious effects if these local flow phenomena are of special importance (such as the inflow in a harbour basin along the harbour boundaries). It has necessitated the implementation of a smooth-boundary approach to circumvent these effects. Figure 5.15 gives the effect of a smooth-boundary approach on the velocity distribution in a river bend. It shows that the longitudinal velocity, as simulated with a smooth-boundary approach (Figure 5.15c), follows more closely the expected distribution (Figure 5.15a), than in the case without such an approximation (Figure 5.15b).

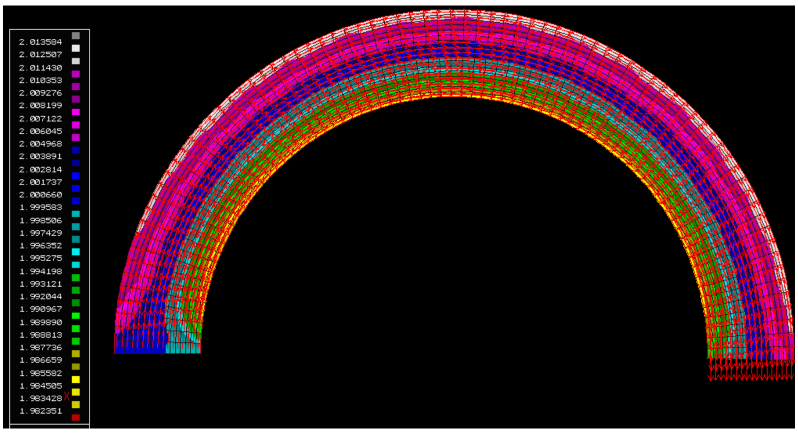


Figure 5.15a: Velocity distribution in river bend with ‘ideal’ circular grid

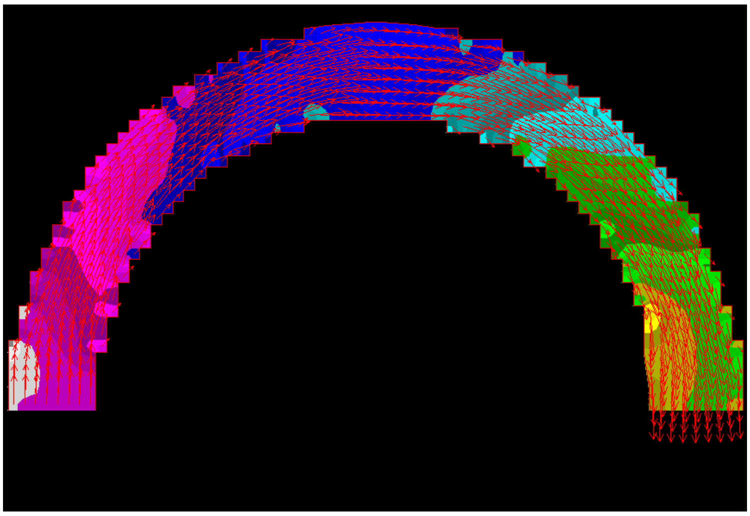


Figure 5.15b: Velocity distribution in river bend with rectangular grid without smooth boundary approximation

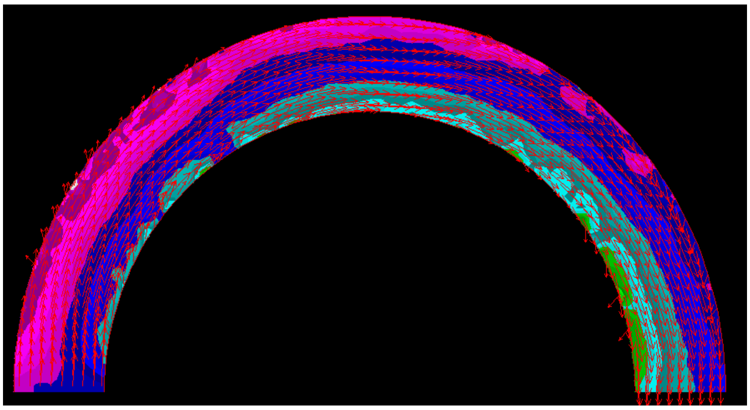
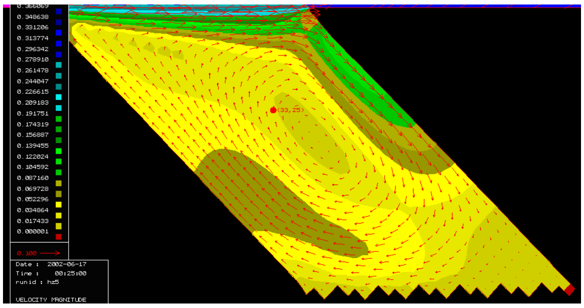
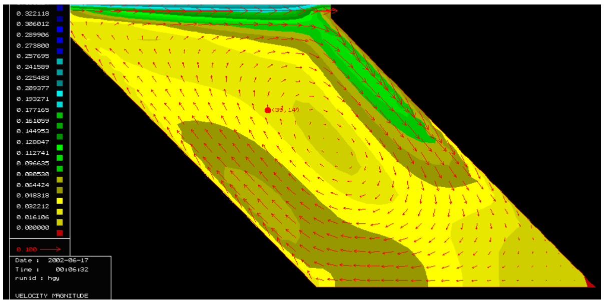


Figure 5.15c: Velocity distribution in river bend with rectangular grid with smooth boundary approximation

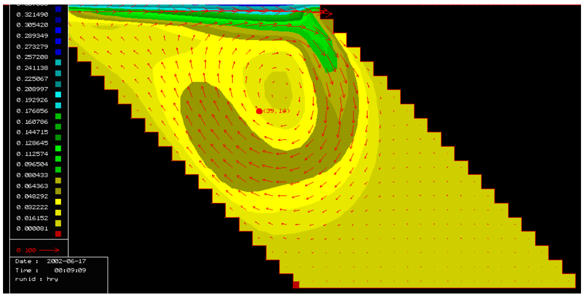
For the current application, the effect of a stair-case boundary becomes apparent in Figure 5.16, showing the effects of stair-case-induced artificial roughness on the eddy pattern in a harbour basin. Figure 5.16a gives the computational results of a simulation on a curvi-linear grid. In that case the domain is boundary-fitted, meaning that grid lines and harbour boundaries coincide. This schematisation results in a too large water exchange between river and harbour, because of the orientation of the grid lines. The application of a rectangular grid affects the eddy pattern in the harbour, due to the artificial roughness of the boundaries (Figure 5.16b). A more realistic pattern is obtained with the smooth-boundary approximation as given by Figure 5.16c.



a. Curvi-linear grid



c. Rectangular grid with smooth-boundaries



b. Rectangular grid

Figure 5.16: Effect of a stair-case boundary on the velocity distribution in a harbour basin.

6 Conclusions

The study has resulted in the following conclusions:

1. The potential effect of a Current Deflecting Wall (CDW) to reduce siltation in harbours and access channels has been demonstrated with a prototype in the entrance of the Köhlfleet harbour in Hamburg and fundamental research in the Delft Tidal Flume for an estuarine harbour. Reductions may be as large as 30-50%.
2. The functioning of a CDW follows from its effect on the local three-dimensional flow field, in particular the reduction of the rate of horizontal exchange of water between river and harbour (or access channel). For an optimal design, a proper modelling of the processes is essential, which requires advanced modelling tools.
3. Physical scale models have proven their value over the years in addressing the aforementioned research questions. Interpretation of the results should take into account possible scale effects due to the distortion of the model and viscosity effects resulting from relatively low Reynolds numbers.
4. Until recently, numerical models lacked a reliable representation of the governing processes, while computer resources were insufficient to perform the necessary detailed simulations. Recent developments now offer the opportunity to investigate the capabilities and limitations of numerical models in these matters.
5. With the modelling suite, as set-up for the Beneden Zeeschelde in Belgium, the strong points of physical and numerical models are combined. The physical scale model was calibrated on the basis of field measurements while the software code of DELFT3D was extended to account for the non-hydrostatic pressure distribution. Furthermore, the adopted fixed-layer schematisation of the vertical grid and the smooth-boundary approximation circumvent that spurious numerical effects arise.
6. A successful validation of the numerical model (still to be carried out) on the basis of measurements in the physical scale model and in the field, will increase the confidence in applying numerical models to design and assess the effectiveness of CDW-type constructions. The present project has made operational the necessary tools to carry out such a validation.

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General Appendix: Delft Cluster Research Programme Information

This publication is a result of the Delft Cluster research-program 1999-2002 (ICES-KIS-II), that consists of 7 research themes:
►Soil and structures, ►Risks due to flooding, ►Coast and river , ►Urban infrastructure, ►Subsurface management, ►Integrated water resources management, ►Knowledge management.

This publication is part of:

Research Theme	:	Coast and River		
Baseproject name	:	Hydraulic engineering and Geotechnology		
Project name	:	The Current Deflecting Wall: mitigating harbour siltation.		
Project leader/Institute		Ir. J.W. de Feijter		GeoDelft
Project number	:	DC1-323-1		
Project duration	:	01-03-2000	-	30-06-2003
Financial sponsor(s)	:	Delft Cluster		
		WL l Delft Hydraulics		
		Vlaams Gewest – Ministerie van de Vlaamse Gemeenschap		
Project participants	:	GeoDelft		
		WL l Delft Hydraulics		
		Vlaams Gewest – Ministerie van de Vlaamse Gemeenschap		
Total Project-budget	:	€ 368470,-		
Number of involved PhD-students	:	0		
Number of involved PostDocs	:	0		

Delft Cluster is an open knowledge network of five Delft-based institutes for long-term fundamental strategic research focussed on the sustainable development of densely populated delta areas.

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