Prepared for: Rijkswaterstaat/Dienst Getijdewateren

Morphological modelling in estuaries and tidal inlets

Part I

A literature survey

December 1991

delft hydraulics

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delft hydraulics

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1. Introduction

The Directoraat-Generaal Rijkswaterstaat/Dienst Getijdewateren of the Ministry of Public Works and Transport (from now : Rijkswaterstaat) is interested in morphological models predicting the long-term consequences of (human) interference in the geometry of estuaries, with emphasis on the estuary of the Western Scheldt and the Wadden Sea.

Two options are available for long-term models, viz. physical and numerical models. Physical models are very expensive in both maintenance and operation. Numerical models are an alternative second option. They can roughly be divided into three types :

- the dynamic models, using the hydrodynamic and transport equations;
- the empirical models, using empirical relationships only, and
- mixed models using hydrodynamic models to predict the water movement and empirical relationships to predict the morphological changes (from now : dynamic/empirical models).

Long term numerical models are not (yet) available at Rijkswaterstaat.

With letter GWAO-91.9569 dated August 22, 1991, Rijkswaterstaat commissioned DELFT HYDRAULICS to perform a preliminary study and a study of literature on the subject of long term morphological modelling of estuaries (ESTMORF). The study should lead to recommendations with respect to the development of a 1-dimensional model, using a dynamic model to calculate the water movement and empirical relationships to determine the morphological development. The possibility and necessity of extension of the model to 2 dimensions should also be studied.

The first phase of the project, the preliminary study, has been completed with a note (Karssen and Wang, 1991).

This report is the result of the study of literature. In Chapter 2, the physical processes in estuaries and the way nature responds to (human) interference in the estuaries are presented, while Chapter 3 gives an overview of already existing morphological models. Chapter 4 contains an overview of empirical relationships that have been or can be used in numerical models describing the morphological changes in estuaries. In Chapter 5, characteristics of the model to be developed are discussed. Chapter 6 contains the conclusions. Finally, Chapter 7 lists the references.

This study was performed and reported by B. Karssen and Dr. Z.B. Wang.

2. Physics of the morphology of estuaries

2.1. Introduction

Several definitions of an estuary can be found in literature. Ray B. Krone (Krone, 1976) gives the following definition :

"Estuaries are the transition regions from the fresh water unidirectional flows of streams to the tidal, saline ocean. The flows in an estuary are affected by the conditions at both ends of this transition zone and are modified by the configuration of the estuary, by winds, and waste discharges".

Morphological change is a result of the interaction between the water movement and the bed topography. Any influence on the water movement in the estuary will also induce morphological development and vice versa.

If an estuary is considered as a water body, then all influences can be detected at the boundaries of this water body. At the upstream boundary the river inflow, at the downstream boundary the saline water intrusion and the tidal motion, at the water surface the wind which generates waves and exerts forces on the flow, and at the bed the bed forms which induce resistance.

The morphological development in an estuary is in addition influenced by the sediment influx from the river and the sediment exchange with the sea, and by the properties of the sediment. (Wang, 1989).

After environmental changes, the geometry of an estuary tends to a stable situation. Human interference (e.g. waste discharge, dredging, construction works, etc.) and sea level rise are examples of such changes in the environment. These changes can be considered as a perturbation on the existing geometry. Due to this perturbation, the flow characteristics are changed. This leads to a response of the bed configuration. This, again, leads to new flow characteristics, etc. In this way, the morphological system adjusts itself from an unstable situation to a (new) stable situation. Although from this description one might think that morphological changes behave like an iterative process (the way it is modelled in some numerical models), in reality the reaction of bed level change on flow change (and vice versa) is instantaneous.

In the next sections of this chapter, the physical processes and parameters affecting the morphological processes in an estuary are discussed in more detail. Empirical relationships between the parameters describing the physical processes and the parameters describing the geometry of an estuary can be found in Chapter 4. An extensive overview of estuarine hydrodynamics can be found in Ippen (1966).

2.2. The tide

2.2.1. The tidal range

In his article, Hayes (1975) concludes that the tidal range has the principal control over the distribution and form of sand deposits affiliated with estuaries.

A classification of tides, according to the tidal range was proposed by Davies (1964) (original unit: ft) :

- Microtidal tidal range 0 2 m
- Mesotidal tidal range 2 4 m
- Macrotidal tidal range > 4 m.

In view of this, estuaries can be classified as microtidal, mesotidal or macrotidal.

The tidal range in the Western Scheldt estuary is not constant over its length. At the mouth of the Western Scheldt estuary (tidal station : Vlissingen) the tidal range for an average spring tide situation is approximately 4.4. m. Towards the head of the estuary (tidal station : Bath), the tidal range for an average spring tide situation is approximately 5.2 m.

The tidal range in the Wadden Sea area is smaller than in the Western Scheldt estuary. The range for average spring tide situations varies from approximately 1.7 m at Den Helder, 2.0 m at West-Terschelling, 2.1 m at Harlingen and up to 2.6 m at Huibertgat. As was already noted by Steijn (1991), the boundaries between the classes in the classification made by Davies are somewhat artificial. In view of this, both the Western Scheldt estuary and the Wadden Sea are classified as mesotidal.

In mesotidal estuaries, the relative importance of the tide on sediment deposition begins to predominate when compared with wind and wave effects. This is also reflected in the empirical relations presented in Chapter 4 : the equations almost all relate the channel geometry to tidal parameters.

The tidal wave in an estuary is affected by several factors. Firstly, the convergence of the estuary banks concentrates the tidal wave energy into a smaller crest length. Secondly, the energy is reflected by the banks and at the head of the estuary, while the length of the estuary is also important in view of resonance of the tidal wave. Finally, the shallow water causes frictional dissipation of energy. The former two mechanisms cause an increase of tidal range towards the estuary head, and the last a decay. The balance of the three is complex, depending on the estuarine depth and top-ography, and consequently is difficult to predict without fairly detailed numerical models. (Dyer, 1986)

2.2.2. The tidal currents

Tidal asymmetry is generated by non-linear interactions and, represented as higher harmonics and compound tides, can have profound effects on sediment transport in an estuary (tidal pumping).

Due to distortion of the tide by friction and other non-linear interactions (see previous section), the flood period may become longer than the ebb period or vice versa. This effect is called tidal asymmetry. Due to this tidal asymmetry, the maximum tidal velocities during the shortest period will be larger than the maximum tidal velocities during the longest period (continuity).

The tidal currents are very important for the sediment transport in an estuary, since the sediment transport is proportional to the flow velocity above a certain threshold raised to some power. As a consequence, higher

flood velocities than ebb velocities result in a stronger up-estuary transport of material. Vice versa, higher ebb velocities result in a stronger down-estuary transport of material.

In terms of astronomical tidal constituents, tidal distortion causes an increase of importance of higher harmonics and compound tides in the tide. For semi-diurnal tides, strongly determined by the tidal constituent M2, the most important higher harmonic resulting from non-linear interactions is the M4 constituent (Aubrey and Speer, 1985; Speer and Aubrey, 1985). Therefore, the amplitude ratio of M4 and M2 can be used as a measure of the tidal distortion. DiLorenzo (DiLorenzo, 1988) worked out the impact of the phase relation between M2 and M4 on the type of distortion.

Conclusion : In a dynamic model, the net transport of sediment is determined by the higher harmonics. Therefore, the higher harmonics have to be modelled correctly in the hydrodynamic module of such a model. In dynamic/empirical models, the necessity depends on the empirical relations used. If relations are used between tidal velocities and the channel geometry, higher harmonics have to be modelled correctly in the hydrodynamic module. If relations are used between other parameters like for instance the tidal prism and the channel geometry, the higher harmonics are not very important.

2.3. Wind and wind waves

Wind can have an important influence on estuarine circulation and mixing. The wind stress exerted on the surface can produce a net transport of water, while the waves generated will increase the intensity of vertical mixing. In the surface layer the water transport will be mainly in the direction of the wind. As a consequence, the normal seaward flow at the surface will be increased and the landward tidal flow will be decreased if the wind is blowing down-estuary. If the wind is up-estuary, the seaward flow intensity at the surface will be decreased, or even reversed in direction (Bowden, 1967).

The effect of the wind is important in the surface layer, but may extend to considerable depths in the water column. For example, in a narrow closed

basin, the flow in the lower part of the water column will be directed in the opposite direction of the surface flow (mass balance). The effect on the flow in the bottom layer will than be stronger in shallow channels than in deep channels.

In a long, narrow estuary the flow may be predominantly tidal, and the wind has little chance to generate much current. If the estuary is wide, or consists of a series of bays, wind stresses can generate currents of considerable importance (fetch) while the effect of the waves can vary much over the estuary.

Waves are foremost important in the mouth of the estuary but also internally generated in estuaries. Waves normally break at the outer margin of tidal flats so that strong swells cannot actually develop on the flats. At the mouth of the tidal creeks, however, refraction leads to the reorientation of the wave direction into the creek. Therefore, characterized by a permanent loss of energy, waves can extend deep into the tidal creeks and contribute to sediment movement (Ehlers, 1988)

Shoals in estuaries exist in a state of dynamic equilibrium with tidal currents and wave action. In their initial formation phase such shoals are mainly formed by tidal currents. However, when shoals grow above the subtidal level, waves become more important for the morphological development of shoals.

It is postulated that the net spatial morphological effect of fair weather conditions and storms over a year strongly depends on the specific morphological circumstances of the investigated year, e.g. no storms, only NW storms, many kinds of storms etc. To estimate the spatial morphological development it is necessary to study the difference between the wind characteristics over the investigated year and the wind climate of the last 10 or 20 years (Kohsiek et al., 1988).

Conclusions : Wind and wave effects are important as driving force for sediment transport in the mouth of the estuary and in the tidal flat area. There is not much experience with the incorporation of waves in a morphol-

ogical model. It is therefore advised to model waves using empirical relations.

2.4. The (horizontal) density gradient

Where tidal currents, wind waves and river discharge are responsible for mixing of the river water and sea water, residual currents are dominated by density differences between the river runoff and the sea. Density gradients can occur in the horizontal plane and in the vertical plane.

Density gradients in an estuary are caused by the intrusion of salt water from the sea (or ocean) into the estuary, and the outflow of fresh water from a river into the same estuary. The density of the salt water is higher than the density of the fresh water. Due to this density difference, salt water tends to move land inward over the lower part of the water column, while fresh water tends to move seaward over the upper part of the water column (gravitational circulation). In this way, both a horizontal and a vertical density gradient occurs.

The vertical density difference is called stratification. Energy is needed to overcome the stratification. In estuaries the vertical mixing is effected by turbulence, and the required energy is supplied by the (tidal) flow. Therefore, stratification is most pronounced in estuaries, where a river issues into a non-tidal sea.

On this basis, Cameron & Pritchard (1963) and Dyer (1973) have classified estuaries by their stratification and their salinity distributions. They define the following types of estuaries :

- highly stratified salt wedge type estuaries;

- partly mixed estuaries, and

- well mixed estuaries.

In well-mixed estuaries the density varies primarily in the horizontal direction and hardly over the depth of the estuary (van Os and Abraham, 1988).

A more quantitative method to classify estuaries by their stratification is based on the (dimensionless) estuary number E_D (Thatcher and Harleman, 1981) :

$$E_{p} = \frac{1}{\pi} \frac{\rho u_{t}^{3}}{\Delta \rho g h_{0} u_{r}}$$
(1)

where :

ut	- amplitude of profile averaged tidal velocity at the	
	mouth of the estuary	(m/s)
Δρ	- difference in density between the river and sea water	(g/m^3)
ρ	- density of sea water	(g/m ³)
g	- acceleration of gravity	(m/s ²)
h ₀	- depth at the mouth of the estuary	(m)
u,	- river velocity, i.e. river flow rate divided by	
	cross-sectional area at the mouth of the estuary	(m/s)

The estuary number is a measure for the ratio of the energy input by the tidal current and the energy needed for mixing. A similar relationship was defined by Fischer (1976), using an "Estuary Richardson Number", which is inversely proportional to the estuary number. For highly stratified estuaries $E_D < 0.2$, for well mixed estuaries $E_D > 8$, while for $0.2 \le E_D \le 8$ the estuary is considered partly mixed.

The horizontal density gradient results at the bottom in a landward residual current in the maritime part of the estuary and a seaward residual current in the fluvial part of the estuary. As a consequence, there is a point in the estuary where both residual currents meet at the bottom. At this location fine sediments, like silt and fine sand, accumulate and sedimentation occurs by residual circulation. This feature is called turbidity maximum.

In the Wadden Sea, no large density differences occur, due to the fact that there is not much inflow of fresh water. Also in the Western Scheldt estuary, the inflow of fresh water is small. In other words, the denominator in formula (1) is for both estuaries very small, resulting in an estuary number > 8. Conclusion: Both estuaries can be considered as well-mixed and are thus tide- and wave-driven.

2.5. Geometry of the estuary

2.5.1. Introduction

Stability of tidal channels is often related to the cross-sectional stability of these channels. Tidal channels/inlets, however, can also migrate horizontally. This is related to location stability. In this report emphasis is placed on cross-sectional stability. In the next sections, both types of stability are described.

2.5.2. Cross-sectional stability

Cross-sectional stability is related to the shape and size of the crosssectional area of a channel/inlet. In an originally stable situation, sedimentation (erosion) can occur due to changes in tidal characteristics and/or geometry e.g. by human interference. As a result, the area of a cross-section will decrease and/or increase and the shape of the crosssection will change. In Chapter 4, several empirical relations involving the cross-sectional area are presented.

Conclusion : Cross-sectional stability should be one of the phenomena to be modelled if human interference is to be modelled.

2.5.3. Location stability

Location stability is related to the location of a channel/inlet in the horizontal plane. Migration of tidal inlets may lead to different (tidal) characteristics and will therefore also affect the stability of individual channels in the system. Gerritsen (1990) lists some factors that play a role in location stability. From these factors, the tide and wave influence are already discussed in the previous sections. Other factors are : human interference, littoral drift, flood and ebb channels, location of the storage basin with respect to the inlet and other driving forces such as the Coriolis force and centrifugal forces. Three of above mentioned factors are very important for the modelling of the tidal channels inside the basin : human interference, flood and ebb channels and the 'other driving forces', e.g. Coriolis and centrifugal forces. In this section, ebb and flood channels and the 'other driving forces' are treated. Human interference is discussed in Section 2.6.

Different flood and ebb channels are a common feature. In flood (ebb) channels, the total amount of water flowing through the channel during the flood (ebb) period, is larger than the amount of water flowing through the channel during the ebb (flood) period. Channels for which flood and ebb volumes are equal, are called neutral. Between flood and ebb channels one finds shallow areas.

The 'other driving forces' cause the so-called meandering, and thus the generation of the ebb and flood channels. Meandering maintains the regime as the same amount of material must be eroded as is deposited, and the energy of flow required to erode deposited sediment by curved flow is less than required to scour the deposits by straight flow (Inglish and Kestner, 1958). This difference in energy is particularly large where the material eroded is fine. A relation between the 'other forces' and the size, location and shape of ebb- and flood-channels is hard to define.

Conclusion : In the model a distinction should be made between flood and ebb channels. Meandering is important, but is difficult to model with a 1dimensional model. Furthermore (see Chapter 4), no consistent empirical relations for meandering were found. The generation of new channels and the migration of channels should not be modelled. Therefore the 'other forces' will not be taken into account in the model.

2.6. Human interference

2.6.1. Barrages

Construction of barrages in a river prevents river sediment to reach the estuary, and can lead to accelerated coastal erosion downstream of the estuary mouth due to loss of sediment input.

If the barrage is constructed in an estuary, they obstruct the free passage of water and sediment during the flood tide. The tidal volume is therefore reduced resulting in greater deposition on the flood tide. On the ebb tide, velocities are lower than previously and less energy is available to move sediment. The net result is progressive siltation until a new equilibrium is reached. The reduced ebb flow may also affect the depth and lateral position of the sea channels at the mouth of the estuary (O'Connor, 1982).

Conclusion : The construction of barrages can have important effects on the sediment balance in an estuary. If barrage construction is foreseen in the area of interest, the model to be developed should be able to simulate the effects of the barrage. It is noted that a barrage can easily be modelled in a 2-dimensional model. In a 1-dimensional model, the 2- and 3-dimensional effects of a barrage (circulation, etc.) can not be modelled. A noncomplete closure can be modelled in a 1-dimensional model by adjustment of the cross-sectional area. The effects of barrages will be simulated 'automatically' through the tidal model.

2.6.2. Dock entrances/jetties

The effect of jetties is increasing local siltation rates and local reduction of depths. In areas with low concentrations of clay material, siltation may be enhanced by increased flocculation arising from turbulence generated by the jetty structure. In areas with large silt load, the reduction in flow produced by extra drag from the structure can be a principal cause of siltation.

The effect of dock entrances is to produce areas of reduced or eddying flow. As a consequence, enhanced siltation occurs in the place it is least required. Siltation rates are usually greatest on spring tides when sediment concentrations are greatest in the main channel. Eddy zones are particularly bad since sediment is drawn to them by secondary flows and their unstable nature makes navigation uncertain (O'Connor, 1982).

Conclusion : The effects of dock entrances and jetties are typically two dimensional. It is therefore concluded that these effects cannot be simulated in a 1-dimensional model.

2.6.3. Dredging and dumping activities

In offshore areas, increased depths can change wave refraction patterns and flow patterns so that wave and tidal energy is focused on particular areas, leading to accelerated coastal erosion problems. In inshore areas dredging effects are less complex. Dredged holes may attract extra flow, if on a large enough scale and may alter the pattern of flood and ebb channels, some of which may be navigation routes (O'Connor, 1982).

To increase the efficiency of the dredging process it is important that dredged material is dumped on such site where it cannot return. Still, the material carried up/down within the estuary will deposit on dredging locations (Inglish and Kestner, 1958).

Conclusion : Dredging and dumping can have profound effects on the flow pattern and the pattern of the residual sediment transport in the estuary. It is therefore important that the model should be able to simulate these effects.

2.6.4. Land reclamation

Land reclamation may totally alter local geometry. If smooth embankment walls are used, flow attraction on the embankment may result, leading to changes in channel configuration. Large scale reclamation may decrease inter-tidal volume (tidal prism), which will affect sedimentation and erosion patterns and thus (navigation) channel depths in the estuary (0'Connor, 1982).

Conclusion : The effect of land reclamation is important. It is however noted that the global effects will be simulated by the tidal model. Therefore, no extra requirements are needed for the morphological module. The effect of flow attraction is a two- or even three-dimensional one and can thus not be modelled in a one-dimensional model.

2.7. Sediment transport

Hayes (1975) notes that mesotidal estuaries differ from the microtidal estuaries in that sediments deposited by tidal currents begin to predominate over wind and wave effects. In macrotidal estuaries the most important feature is the total dominance of tidal currents.

The principal sand deposits in mesotidal inlets are the tidal deltas (Hayes, 1975) :

- the ebb-tidal delta or outer delta. Sediment accumulation seaward of a tidal inlet, deposited by ebb-tidal currents;
- the flood-tidal delta or inner delta. Sediment accumulation landward of an inlet, deposited by flood-tidal currents.

Channels in the ebb tidal delta of an inlet are much less stable with respect to location than channels in the flood tidal delta. For example in the Wadden Sea, sand transport from one tidal flat area to the next is accomplished via the ebb tidal bars at the mouth of tidal streams, which migrate at rates of about 20 m/year (Ehlers, 1988). In view of this high instability, it is very difficult to model these channels.

The flood tidal delta only contributes to the redeposition of sediments in a very minor way. For the Wadden Sea, 90% of the sediment movement occurs in the ebb delta (Ehlers, 1988). The transport into and out of the tidal inlet plays a subordinate role. Sediment transport across the tidal flats is accordingly low, but it cannot be ignored.

The availability of sediment limits, of course, sediment transport. Assuming a net sediment transport into the estuary on the seaward boundary of a model, sediment should be available on this boundary location in nature to meet this boundary condition. In models, the availability of sediment is often assumed to be unlimited (see Section 5.2).

Littoral drift is one of the most important sources of the sand that is transported landwards through the channel system of an estuary or inlet. Other sources are the onshore transport by waves, the sand loss from adjacent beaches and losses from the ebb tidal deltas. Conclusion : In case migration of channels is omitted from the model (see conclusion in Section 2.5.3) and in view of the complex process determining availability of sand, it is advised not to model the outer delta. The effect of the outer delta on the availability of sediment should be incorporated by an empirical relation in the boundary conditions of the model, if possible.

Other important factors determining the sediment transport rate are the properties of the sediment involved, like sediment size, shape, density, fall velocity, on the basis of which a distinction can be made between sand and silt, and sediment availability.

In the area of interest the bottom material mainly consists of sand $(D_{50} - 200 \text{ to } 300 \text{ }\mu)$. Silt has no important influence on the morphology.

Sand transport generally occurs in two forms : bed load transport and suspended load transport.

Bed load transport takes place in a thin sheet of sediment moving above the bottom. An important property of the bed load transport is that it is directly related to the local flow conditions. Several (bed load) sediment transport formulas are known, each containing variables describing the sediment characteristics like density and grain size. Examples of these formulas are those by Meyer-Peter and Muller (1948), Einstein (1950), Bag-nold (1956), Engelund-Hansen (1967), Ackers and White (1973) and Van Rijn (1984a).

Suspended load transport is often described by an advection/diffusion equation.

In both the Wadden Sea and the Western Scheldt, suspended load transport is much larger than bed load transport.

Conclusion : The morphology in the area of interest is mainly determined by the sandy bottom material. Sediment transport in both the Wadden Sea and the Western Scheldt is dominated by suspended load transport.

3. Morphological models

3.1. Introduction

In the previous chapter, the factors influencing the morphological processes are described. Considerable knowledge appears to exist about the individual elements, but appears to be lacking about the integration of the elements into a conceptual framework for morphological development. Mathematical modelling of the entire process is still in an early stage.

Roughly, three types of morphological models can be distinguished :

- Models, using hydrodynamic and transport equations to predict changes in morphological behaviour, the 'Dynamic models';
- Models, using hydrodynamic models to simulate the water movement and empirical relationships between parameters calculated from the water movement and parameters representing the geometry of the estuary to predict the morphological changes, the 'Dynamic/Empirical models';
- Models using empirical relationships only, the 'Empirical models'.

As was already mentioned in Chapter 1, this study has to lead to conclusions and recommendations with respect to the development of a 1-dimensional model, using a dynamic/empirical model. In this chapter an overview of existing morphological models of all three types is given. This overview provides extra information on the characteristics of the model to be developed and also serves as a state-of-the-art.

3.2. Dynamic models

3.2.1. Introduction

Dynamic models use the complete hydrodynamic equations, both for fluid transport and sediment transport. For low Froude numbers (as occur in the field) the hydraulic equations can be solved independently from both the transport equation and the equation describing bed level change, under the assumption of quasi-steadiness.

The dynamic model can thus be split into 3 separate modules (De Vries, 1959):

- the hydraulic module;
- the sediment transport module;
- the bed level change module.

Dynamic modelling of morphological processes in estuaries is still in an early stage of development. Morphological computations for estuaries have hardly been carried out so far.

3.2.1.1. The hydraulic module

For the hydraulic module a choice exists between a 1D-, 2DH- or 3D- model. For the study of morphological changes in the Western Scheldt a 1D-model (IMPLIC) can be very effective and less effective in the Wadden Sea. It is noted that such a hydraulic model is not yet available in the Wadden Sea and available for the Western Scheldt. Use of a 1D-model is particularly of interest if the estuary is long and the channels are relatively narrow. (Gerritsen, 1990).

Gerritsen (1990) suggests to use a 2DH model for the Wadden Sea, which has already been done by Ridderinkhof (1990) for the calculation of residual currents and mixing in the Wadden Sea, making use of WAQUA. The results agree well with observations.

For the Wadden Sea a grid size of 500 m was used by Ridderinkhof (1990). For morphologic simulations, a grid size of 200 or 100 m is suggested by Gerritsen (1990). Gerritsen notes that Langerak has successfully used a 100 m grid for the Western Scheldt.

3D-models are used in river morphology only once, by Wang and Adeff (1986).

3.2.1.2. The transport module

The sediment transport module is the most important part of the morphodynamic model. Many formulations exist for the sediment transport. These formulations can roughly be divided into two groups, viz. the total transport formulae and those making distinction between bed load transport and suspended load transport. The use of total transport formulae in tidal areas usually assumes that the sediment transport rate is related to the local instantaneous flow condition in the same way as in the case of steady uniform flow in rivers. Many sediment transport formulae are available for calculating the transport rate. Well known ones applied for river problems are those of Einstein (1950), Bagnold (1956), Meyer-Peter and Müller (1948), and Englund and Hansen (1967). A recently developed sediment transport formula is the one of Van Rijn (1984a, 1984b).

For coastal regions the influence of short waves is often important. Therefore specific transport formulae have been developed for coastal problems. An overview of these group formulas is given by Van de Graaf (1978). Well known formulae are the Frijlink-Kalinske formulae (Frijlink, 1952), the formulae of Bijker (1968) and the formulae of Van Rijn (1984a, 1984b). It should be noted that the sediment transport formula of Van Rijn (1984a, 1984b) uses bed shear stress as basic flow parameter. This means that it can also be used for coastal problems if the influence of the short waves on the bed shear stress are properly taken into account.

Here no detailed description is given of the particular formulae. For more information reference is made to the original publications and state of the art overview books like that of Vanoni (1977) and Van Rijn (1989).

In the morphological model of Holly and Rahuel (1990) the influence of nonuniform flow on the sediment transport rate is taken into account by using the so called bed load loading law. The transport rate is then not only dependent on the local flow conditions but also on the flow conditions upstream.

When a distinction is made between bed load transport and suspended load transport, the bed load transport is calculated in the same way as the total sediment transport rate. For the suspended load transport different approaches can be applied varying from a simple transport formula to a complete three-dimensional model. Usually the sediment concentration near the bed is related to the local flow parameters in a similar way as for the bed-load transport rate. The sediment concentration distribution in the water column is described by a convection-diffusion model. In a fully

three-dimensional model the complete three-dimensional convection-diffusion equation is solved (see e.g. Van Rijn, 1987). A simpler approach is the depth-integrated model with sediment loading (see e.g. Lin et al, 1981; Galappatti, 1983) with limited validity (Wang and Ribberink, 1986; Wang, 1989).

It is further important to mention the down-slope effect of the bed. The down-slope effect is the effect on the bed load sediment transport of the slope of the bed due to gravity (it is easier for sediment to move 'downhill' than 'uphill'). This effect on the bed-load sediment transport is significant in the morphological model. In one-dimensional models the downslope effect influences the magnitude of the transport rate and it also has a stabilizing effect on the morphological computation. In two-dimensional (horizontal) models the down-slope effect on the transport direction is also important. In river bends the down-slope effect is a dominating factor for the morphological development (Struiksma et al, 1985, Olesen, 1987).

In most sediment transport models, the bed material is dealt with as a single fraction. In the model of Ribberink (1988) the sediment is divided in different fractions and the transport rate of each fraction is calculated separately. A similar approach is applied in the model of Holly and Rahuel (1990).

In dynamic models, advection is often dominant over dispersion. It is however noted that generally, the incorporation of dispersion in dynamic models is very easy. Moreover, the numerical solution of the equations is easier if dispersion is included.

3.2.1.3. The bed level module

The bed level change is obtained from the sediment continuity equation :

$$\frac{\partial z_b}{\partial t} = -\frac{1}{(1-p_b)} \left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial (h\overline{c})}{\partial t} \right)$$

where :

- $z_{\rm b}$ bed level above reference plane
- $p_{\rm b}$ porosity of the bed
- q, sediment transport in x direction (bed load + suspended load)
- q sediment transport in y direction (bed load + suspended load)
- h = channel depth
- c depth averaged concentration

The time step for the bed level computation can be much larger than the time step in the flow computation. In case of a quasi-steady flow this is easily realized by assuming that the flow field does not change during the time step of the bed level computation. In the case of tidal flow it is often assumed that the flow field and the sediment transport field remain periodic during several tidal periods. This means that the time step for the bed level to a number of tidal periods (de Vriend, 1985), (Wang, 1989).

3.2.2. Overview of existing dynamic models

3.2.2.1. Holly and Rahuel

Description of the model :

F.M. Holly Jr. and J.-L. Rahuel have developed a computational framework for one-dimensional mobile river-bed simulation. The approach is based on the full St. Venant flow equations, treatment of sediment mixtures, recognition of the distinctly different physics of suspended load and bed load movement, use of a spatial-delay loading law for bed load, and a fully coupled, implicit solution of the resulting set of partial-differential equations, i.e. the water movement and the sediment transport are solved simultaneously.

In (Holly and Rahuel, 1990) the exact equations used in their model are described.

Results :

Tests are described without longitudinal dispersion of suspended load. All tests are performed on a schematic river reach, with constant width of 30.5 m. The water discharge is imposed constant at the upstream boundary. A constant initial slope is assumed, together with a constant initial water depth of 3.05 m.

The method remains stable for a wide range of time steps (from 6 minutes to 10 hours), as the computational method is robust.

Application :

The method as it is, is not applicable for estuaries. It is a single direction model for rivers. The discharge is assumed to be constant, and imposed upstream.

3.2.2.2. Puls

Description of the model :

W. Puls (1984) describes a 2-dimensional model of the Elbe estuary. The purpose of the model was to forecast bathymetric changes due to dredging and other hydraulic works. The model contains a flow module, calculating the (steady state) depth averaged currents for four "representative" phases of a mean tidal cycle. Empirical formulas are used to compute the wave height and the wave frequency. Combined action of waves and currents are taken into account making use of a combined shear stress formulation. The sand transport is also computed using a steady, vertically averaged approach. Distinction is made between suspended load and bed load. Spatial divergencies of transport rates result in a pattern of bathymetric changes for each of the four phases. The bed level change is calculated by taking the sum of these changes, multiplied by the corresponding durations of each tidal phase.

Results :

The morphological time step was taken approximately 425 days. The bathymetric changes are considerable during this time step. Therefore, the results can only be regarded to show the trend and not the realistic values of the changes in bathymetry. The comparison of the computed bathymetric changes with field data yields unsatisfactory results. According to Puls, this is due to the lack of knowledge about the actual physics of sand transport in nature.

Application :

The method shows unsatisfactory results. No recent information was found about improvement of the method in literature. The conclusion can therefore be that the method cannot yet be used for estuaries.

3.2.2.3. Wang

Description of the model :

Wang (1989) describes a model (ESMOR) for the morphological development in estuaries. Density flow, influence of wind and short waves are not taken into account. The model consists of three submodels, i.e. the flow model, the sediment transport model and the bed level model. The flow model is simple two-dimensional tidal flow model and a simplified secondary flow model. The sediment transport is divided in suspended load and bed load transport. The bed load transport is calculated with a transport formula while the suspended load is calculated from a depth-integrated model. This model is derived from an asymptotic solution of the convection-diffusion equation following the theory of Galappatti (1983). In the model of Galappatti, the concentration is expressed in terms of a series of previously determined profile functions multiplied by the mean concentration and its derivatives in time and space. The profile functions are based on known families of equilibrium profiles and velocity profiles. The bed level change is calculated from the total sediment transport field in a tidal period based on the mass balance.

Results :

The ESMOR model has been applied to the Yantze estuary in China. The available measurement data was not sufficient to calibrate, verify and test

the model extensively. Nevertheless, reasonable agreement has been achieved between the measured and calculated concentration as well as sediment transport. Concerning the morphological development, the comparison between the measured and calculated bed level change after one year shows a reasonable agreement in regions with sufficient change while in regions with minor changes the agreement is worse.

Applications:

A similar model has been implemented in the DELFT HYDRAULICS software environment (TRISULA). Recent applications of this model show that it is not able to make accurate long-term predictions but is a useful research tool for studying the morphodynamic processes in tidal inlets (Wang et al 1991).

3.3. Dynamic/empirical models

3.3.1. Introduction

From Section 3.1, the following definition of dynamic/empirical models can be derived :

'Dynamic/empirical models use hydrodynamic models to simulate the water movement and empirical relations to predict morphological changes. Empirical relationships relate parameters calculated from the water movement and parameters representing the geometry of the estuary.'

The hydraulic module of dynamic/empirical models is of the same type as for dynamic models (see Section 3.2.1.1). Several empirical relationships are possible between parameters describing the geometry of an estuary (channel) and hydraulic parameters. In Chapter 4, an overview of empirical relations is given. 24

3.3.2. Overview of existing dynamic/empirical models

3.3.2.1. O'Connor, Nicholson and Rayner

Description of the model :

O'Connor at al. (1991) describe a model for the prediction of the new geometry of a disturbed estuary following the so-called regime approach. In this approach, a simple one-dimensional tidal model is combined with four empirical expressions which relate the flow field to the estuary geometry. The advantage of this type of modelling is the simplicity, the relatively low running costs and ease of operation.

The first empirical expression of this approach relates the estuary crosssectional area to the tidal prism, following Jarrett (1976) :

$$A_c = a (P_a)^b$$

where :

a - constant (m⁻¹)
 b - constant
 P_s - spring tidal prism (m³)
 A_c - cross-sectional area below mean sea level (m²)

The second empirical formula relates the centre line depth at mean water level of a cross-section to the spring tidal prism, following Vincent and Corson (1981). In fact, Vincent and Corson relate the maximum depth in the cross section of the inlet throat at minimum width to the area of the cross-section with minimum width (see Section 4.3.4).

The third empirical formula relates the mean sea level width of a crosssection and the centre line depth at mean sea level of that cross-section.

The fourth and final expression concerns the shape of a cross-section (see Figure 1) :

 $b/b_c = (h/h_c)^s$

where :

Ъ	- cross-sectional width (m)
Ъ _с	- mean sea level width of a cross-section (m)
h	- water level above centre line bottom (m)
h _c	- centre line depth at mean sea level of a cross-section (m)
s	- side slope at the mean water level

From the above, it is clear that A_c , h_c , b_c and s can be expressed as a relation with the tidal prism.

Assuming an estuary in initial equilibrium, O'Connor et al changed their boundary conditions of the tidal model, resulting in a new set of tidal prisms for various locations along the estuary. With above mentioned formulae, the new estuary geometry was found. This new geometry resulted in a change of the hydrodynamic conditions. In turn, a new tidal computation was performed resulting again in a new estuary geometry. This process was repeated until a new equilibrium was reached.

The time needed to attain a new equilibrium was integrated in the model by O'Connor et al by adding two elements :

- Introduction of a relationship between the mean sediment concentration and the tidal range;
- The assumption that the rate of change of the estuary capacity varies exponentially with time until nominal equilibrium has been attained.

Results :

The method was applied to the River Usk, a river discharging into the Severn Estuary. In the River Usk, a tidal power barrage was planned to be build. After checking with a tidal model that the existing geometry was in a state of equilibrium, a tidal model was run for new boundary conditions valid for post-barrage hydraulic conditions. The above described iterative process was used to reach an equilibrium estuary geometry (checked with the Jarrett relationship). The results show that the method can be applied without difficulties and yields meaningful results. It is, however, not yet checked with both pre-disturbance and post-disturbance information available.

3.3.2.2. Di Silvio

Description of the model :

Di Silvio (Di Silvio, 1989; Di Silvio and Gambolati, 1990) describe a model which departs from the advection-diffusion equation. The model, however, neglects the transport of sediment by advection with respect to the (dominant) transport of sediment by dispersion. The advective part of the equation can, however, easily be included in the model. An empirical relationship was used between a vaguely defined annually averaged sediment concentration, the channel depth and a local parameter which depends on channel width, the average tidal volume through the section and the grain size.

The following equations are used :

where :

$$\frac{\partial}{\partial x} (hD \frac{\partial c}{\partial x}) = w_g(c - c_i)$$
(3)

$$\frac{\partial h}{\partial r} = w_g(c_i - c) \tag{4}$$

$$c_{i} = \frac{f_{c}}{h^{5}}$$
(5)

In Karssen and Wang (1991), an extensive description of the dispersion model of Di Silvio is given.

Results :

Preliminary results are given by Di Silvio and Gambolati (1990). Although the results show some interesting features of the model, experience with the model lacks. It is noted that the definition of the equilibrium concentration is vague and it is not clear how Di Silvio has determined the values for this concentration. As Karssen and Wang have furthermore found, it is not clear how human interference should be modelled making use of the equilibrium definitions of Di Silvio.

3.3.2.3. Allersma

Description of the model :

The model described by Allersma (Allersma, 1988) assumes that a change in the transport of sediment is determined by the change in cross-section profile only. This yields :

$$\frac{\partial T}{\partial x} = \frac{\partial A}{\partial t} \tag{6}$$

(m⁵)

(m/year)

where :

Т	= sediment transport rate	(m ³ /year)
x	- horizontal coordinate	(m)
A	= area of cross-section	(m ²)
t	- time parameter	(years)

The model uses different expressions for deposition and erosion :

Deposition is modelled by :

$$\frac{\partial A}{\partial t} = \sigma T \frac{A_{o} - A}{A_{o}}, \qquad (A > A_{o})$$
(7)

where :

 $A_e = equilibrium cross-section area (m²)$ $\sigma = constant (m⁻¹)$

Erosion is modelled by :

$$\frac{\partial A}{\partial t} = \epsilon \frac{A_{\phi} - A}{A_{\phi}}, \quad (A < A_{\phi})$$
(8)

(m²/year)

where :

 ϵ - constant

It is clear that deposition is determined by the availability of sediment, while erosion only depends on the local conditions with respect to the water movement, the geometry and the bottom erodability (modelled using the parameter ϵ).

The equilibrium cross-section area is defined by Allersma as :

$$A_{-} = \alpha M V + (1 - \alpha) N V^{5/6}$$
(9)

 (m^{-1})

where :

 α - relative importance of the tidal discharge and the river discharge (α - 0 river discharge only, α - 1 tide only)

M = constant

V - total quantity of water flowing through the cross-section per tidal period (m³)
N - constant (m^{*}^h)

Results :

The model described by Allersma has been used in applications by both Rijkswaterstaat and DELFT HYDRAULICS, but mainly for rivers. Karssen and Wang (1991), however, showed that care should be taken with the boundary

conditions of the model. They show that the time scale of morphological change is fully determined by the two parameters σ and ϵ , which are to be determined by the user.

3.4. Empirical models

Empirical models are models based completely on empirical relationships. This type of models is very useful to make a quick evaluation of an anticipated development. It is assumed that the relationships remain valid after natural or man made changes to the system. In general this assumes changes which have to be relatively small compared to the existing conditions (Gerritsen, 1990).

The basic principle of all empirical models is the same : empirical relations describe the equilibrium situation of the estuary/channels. The difference between the existing models lays mainly in the use of different parameters involved. Gerritsen (1990) lists three different existing empirical models, described by Stive and Eysink (1989), Rakhorst (1987) and Misdorp et al (1990). The differences between the models are small.

Most empirical relations only describe the equilibrium situation of the estuary, but not the way and time in which the new equilibrium is reached. Stive and Eysink (1989) assume a logarithmic time scale between the initial disturbance and the equilibrium situation.

In this study, a further and more extensive overview of the existing empirical models is omitted, because the principle of empirical models is the same; the empirical relations themselves and/or the parameters involved differ. A more extensive overview of empirical relations with respect to the geometry of estuaries as found in literature is given in Section 4.3.

4. Empirical relations

4.1. Introduction

In this chapter, an overview of possible empirical relations between the geometry of estuaries and physical parameters (like the tide) are presented.

The overview is given for estuaries in general. However, for each relation and the parameters involved, the specific values for the Western Scheldt and the Wadden Sea are also presented, if available.

In the study of tidal inlets emphasis has originally been placed on the size of the gorge of the inlet. Empirical relationships were established for the narrowest section of the inlet. Later investigations (de Jong and Gerritsen, 1984; Gerritsen and de Jong, 1985) have shown that those empirical relationships also apply to the tidal channels in a much larger part of the estuary. Therefore, relationships are given in the next sections which can be applied both for tidal inlets and for tidal channels.

4.2. Definitions

The ebb volume (EV) through a cross-section, is defined as the total amount of water flowing through that cross-section during the ebb period. In a similar way, the flood volume is defined, leading to the following equations :

$$EV = \int_{T_0}^{T_1} Q_{\theta} dt$$

1.0

(10a)

$$FV = \int_{T_1}^T Q_f dt$$

(10b)

where	:		
To	-	start of ebb period	(s)
T ₁	-	end time of ebb period	(s)
Т	-	end time of tidal period	(s)
Q.	-	discharge during ebb period	(m^3/s)

 Q_f - discharge during flood period (m^3/s)

The tidal volume (TV) is defined as the sum of the ebb volume (EV) and the flood volume (FV) :

$$TV = FV + EV \tag{11}$$

The tidal prism (P) is defined as the storage volume of the estuary between the low tide and the high tide levels. It is usually approximated by multiplying the mean surface area of the estuary by the mean tide range of the estuary. The tidal prism can be approximated by half of the tidal volume (TV) :

$$P \approx \frac{TV}{2} \tag{12}$$

The sinusoidal discharge is defined as :

$$Q_{\min} = \frac{\pi \left(FV + EV\right)}{2T} \tag{13}$$

We further have to define A_c . It is noted that A_c normally is defined as the cross-sectional area below mean sea level. In most empirical relationships with respect to Dutch tidal channels/inlets, A_c is defined as the cross-sectional area below the Dutch Ordnance Level, called NAP (Nieuw Amsterdams Peil). NAP is approximately equal to the mean sea level. A_c' is defined as the cross-sectional area at the time of maximum discharge.

4.3. Overview of relationships

4.3.1. Introduction

In the next sections, the possible empirical relationships are described. For almost all relations, mean tidal conditions are assumed, except for the relations incorporating the stability shear stress; these are valid for spring tide conditions.

4.3.2. Relationships involving A

4.3.2.1. A <> Tidal prism (P)

Several relationships between the cross-sectional area at mean sea level (A_c) and the tidal prism (P) have been published. M.P. O'Brien published relationships for inlets with and without jetties at both the Pacific and Atlantic coast of the United States (O'Brien, 1931, 1969). The first relationship published (O'Brien, 1931) assumed an exponential relation between the cross-sectional area at mean sea level and the tidal prism :

$$A_{a} = CP^{0.85}$$
 (14)

with c a constant.

In a later article (O'Brien, 1969) a linear relationship was used.

If a large range of depths exists among the channels under investigation the following equation generally is valid (Jarrett, 1976; Gerritsen, 1990) :

$$A_c = c_1 P^{c_1} \tag{15}$$

with c1 and c2 constants, depending on the area of interest.

If such large depth range does not exist, a linear relationship between the cross-sectional area and the tidal prism can be expected (Gerritsen, 1990). A linear relation between A_c and a characteristic tidal volume was indeed found by Eysink (1990) for the Wadden Sea.

Gerritsen et al. (1990) prove for the Western Scheldt and the Wadden Sea inlets that the hydraulic radius of the inlet/channel influences the dependency of A_c and P:

$$A_c = 1.269 \times 10^{-4} \frac{P}{R^{0.25}}$$
(16)

where :

R - hydraulic radius

For relatively wide and shallow channels the hydraulic radius is almost equal to the mean depth h, so equation (16) develops into :

$$A_c = 1.269 \times 10^{-4} \frac{P}{h^{0.25}}$$
(17)

where :

h - mean depth

(m)

(m)

Using $R = A_c/p$, with p the wetted perimeter, and p = b, with b the width of the channel, it follows for shallow channels from equation (16) that :

$$A_{-} = 7.64 \times 10^{-4} b^{0.2} P^{0.8} \tag{18}$$

4.3.2.2. A <> Characteristic volume (EV. FV or TV)

Wadden Sea

In Biegel (1991), separate relations for ebb and flood volumes are described for the Frisian inlets :

$$EV = 17106 A_c - 79.2 \times 10^6$$
(19)

$$FV = 16092 A_c - 48.4 \times 10^6$$

In Biegel (1991), also two relations between the tidal volume and the cross-section area are mentioned for the Frisian inlets. The first relation was found by Gerritsen and de Jong (1985) and is based on tidal-discharge measurements, the second was found by Misdorp et al (1990) and is based on bathymetric surveys :

$$TV = 33198 A_c - 127.6 \times 10^6$$

$$TV = 35959 A_c - 152.0 \times 10^6$$
(20)

Biegel (1991) also relates the maximum of the ebb volume and the flood volume to A_c :

$$V = 17519 A_{c} - 75400000 \tag{21}$$

where :

V - max (EV, FV)

Western Scheldt

Gerritsen and de Jong (1983) relate the ebb volume, the flood volume and the tidal volume to A_c for the Western Scheldt by :

$$EV = 12770.6 A_{c} + 12621823$$

$$FV = 12943.3 A_{c} + 1602151$$

$$TV = 25712.8 A_{c} + 14310117$$
(22)

Gerritsen and de Jong (1983) describe similar relations for ebb channels and flood channels as well :

$$EV^{BC} = 12994 A_c + 21.1 \times 10^6$$

$$FV^{FC} = 13000 A_c + 23.2 \times 10^6$$
(23)

EVEC	-	ebb volume in ebb channel	(m ³)
FVFV	-	flood volume in flood channel	(m^3)

4,3.2,3, A ◇ Discharge (Q)

Wadden Sea

In Biegel (1991) an equation for the sinusoidal discharge and the crosssectional area can be found :

$$A_c = 0.857 \ Q_{sin} + 3841$$
 (24)

For the Wadden Sea, no other relationships between A_c and a characteristic discharge were found in literature.

Western Scheldt

For the Western Scheldt, several relations were found. The first relationships between A_c and the maximum tidal discharge during ebb conditions and flood conditions are described in Gerritsen and de Jong (1983) :

$$Q_{\max,e} = 0.9142 A_c - 2735$$

$$Q_{\max,f} = 1.174 A_c + 2782$$
(25)

Similar relations are presented in Gerritsen and de Jong (1983) for ebb channels and flood channels :

$$Q_{\text{max}}^{BC} = 0.949 \ A_c - 743$$
 (26)
 $Q_{\text{max}}^{PC} = 1.2 \ A_c + 2434$

where :

 Q^{EC}_{max} - maximum discharge in ebb channel (m³/s) Q^{FC}_{max} - maximum discharge in flood channel (m³/s)

Gerritsen and de Jong (1983) also present the following equation for the sinusoidal discharge and the cross-sectional area in the Western Scheldt :

while Kreeke and Haring (1979) find in the Rhine-Meuse delta for the tidal inlets :

$$A_{c} = 1.17 \ \underline{\mathcal{Q}}_{sin} \tag{28}$$

4.3.3. Relationships involving A.

In Gerritsen and de Jong (1983) a relationship between the maximum discharge (Q_{max}) and the cross-sectional area at the time of maximum discharge (A_c') is suggested. In that case, the ratio Q_{max}/A_c' defines an actual velocity.

Wadden Sea

As was already noted, it is possible to define a maximum discharge during flood conditions and a maximum discharge during ebb conditions. Also an 'overall' maximum discharge can be defined. For the Frisian inlets in the Wadden Sea the following relations were found (Biegel, 1991) :

$Q_{\max,f} = -7503 + 1.24A_{c}^{\prime}$	
$Q_{\max,e} = -8026 + 1.28 A_c'$	(29)
$Q_{\rm max} = -69337 + 1.27 A_{\rm c}^{\prime}$	

Western Scheldt

Similar relations were found by Gerritsen and de Jong (1983) for the Western Scheldt :

$$Q_{\max,f} = -1330 + 1.081A'_{c}$$
(30)
$$Q_{\max,e} = -4118 + 0.957A'_{c}$$

4.3.4. Relationships involving the shape of the cross-section

Gerritsen (1990) describes two relations between the average velocity and the hydraulic radius :

$$\overline{v} = \frac{2P}{A_c T} = \alpha R^{\beta} \tag{31}$$

where :

Ť	-	tidal period	(s)
ī	-	average velocity	(m/s)

For the Wadden Sea, $\alpha = 0.353$ and $\beta = 0.25$ (Gerritsen, 1990). It is clear that this relation is similar to equation (16).

Another equation relating the average velocity to the hydraulic radius is :

$$\overline{\mathbf{v}} = \overline{\mathbf{v}}_0 + \gamma R \tag{32}$$

For the Wadden Sea, $\overline{v}_0 = 0.42 \text{ m/s}$ and $\gamma = 0.017 \text{ sec}^{-1}$. For Wadden Sea inlets and Western Scheldt, $\gamma = 0.016 \text{ sec}^{-1}$, while \overline{v}_0 remains unchanged. Equation (32) is only valid for channels and inlets with a hydraulic radius of 2 m or higher.

It is also possible to relate the maximum velocity (V_{max}) to the hydraulic radius (Gerritsen, 1990) :

 $V_{\rm max} = 0.5 \pm 0.032R$ (33)

This relation is valid for the Wadden Sea channels. A similar equation was not found for the Western Scheldt.

A relationship between the mean depth of a tidal channel and the tidal volume is suggested by Eysink, although it is noted that this relation shows deviations for the channels investigated (Eysink, 1990). He finds : where :

h - mean depth below NAP or msl

Bruun and Gerritsen (1960) describe the tendency for sediment to move towards the side of the channel because of higher sand concentrations in the middle due to higher velocities. Therefore, deposits of material on the banks will occur, causing an increased channel slope, which will cause bed load movement towards the middle of the channel by the combined action of gravity, current shear and uplift forces. At a certain slope an equilibrium condition will be established. Van Bendegom (1949) has shown that for the Dutch Wadden Sea the width-depth ratio of the channels is a function of channel width.

The width b is a non-linear function of the depth h :

$$b = c_{\rm h}h^2$$

where :

Ъ	-	channel widt	:h					(m)
с _ь	-	coefficient	(2.5	m ⁻¹	for	Wadden	Sea)	(m ⁻¹)
h	-	depth						(m)

Gerritsen (1990) states that this relationship can be used in empirical models.

O'Connor et al (1991) describe a relation between the cross-sectional width and the water elevation (see Figure 1) :

$$\frac{b}{b_{\sigma}} = \left(\frac{h}{h_{\sigma}}\right)^{\sigma}$$
(36)

where	3	
Ъ	- cross-sectional width at water level h	(m)
bc	- mean sea level width of a cross-section	(m)
h	- water level above centre line bottom	(m)

38

(35)

(m)

- h, centre line depth at mean sea level of a cross-section (m)
- s side slope at the mean sea level

Vincent and Corson (1981) relate for 67 inlets on the Atlantic and Pacific coasts of the USA and on the Gulf of Mexico the area of the cross-section with minimum width (A_c) to (see Figure 2) :

- the length of a channel from the minimum width line to the shallowest depth in the channel as it passes across the edge of the ebb shoal (L), and
- the shallowest depth in the channel (d_c) .

Furthermore they relate the average and maximum depth $(d_a \text{ and } d_m)$ in the cross-section of the inlet throat at minimum inlet width.

$$L = 23.92 A_c^{0.53}$$

$$d_m = 0.5479 A_c^{0.38}$$

$$d_c = 0.2367 A_c^{0.34}$$

$$d_a = 1.42 - 0.347 d_m$$
(37)

Eysink (1990) presents the following relation between the channel volume below mean sea level and the tidal volume :

$$V_c = C_c (TV)^{1.5}$$
(38)

where :

V_c - channel volume below MSL

(m³)

c_c - empirical coefficient

In Rijkswaterstaat (1985), some relations between meander characteristics and tidal characteristics are given. In the conclusions of that report, however, it is noted that these relations do not show high correlations. Therefore, these relations are omitted in this report.

4.3.5. Relationships involving adaptation time

It is generally assumed that the adaptation processes show an exponential behaviour with an immediate response to changes in the system. Eysink (1990) describes this character in the following equation :

$$\frac{X}{X_0} = \exp\left(\frac{-t}{\tau}\right) \tag{39}$$

where :

- t time since disturbance of existing equilibrium
- X quantity (depth, area or volume) representing the present difference from the new equilibrium

X_n - initial difference from new equilibrium

= characteristic time for adaptation, equal to : $\tau = X_0 / \Delta(X_0)$ where : $\Delta(X_0)$ -initial rate of adaptation

A system with a immediate response to changes should be treated differently from a system with a time lag. The initial disturbance of the system can in case of an immediate response be derived via the change in the hydraulic conditions and the morphologic relations. The initial adaptation rate (τ in equation (39)) can be obtained through monitoring or computations with mathematical models. This approach can not be applied in case of for instance a change in sea level rise. In that case it is not realistic to assume that this will result in an immediate response of the sea bed in a tidal basin. It is more likely that the sea bed will follow with a certain time lag. If the sea level suddenly rises faster, more sand has to be trapped. Due to the extra sea level rise the over-depth in the basin initially increases. Consequently, gradually more sand is trapped until the sea bed again rises at the same rate as the sea level (Eysink, 1990).

Gerritsen (1990) introduces the time lag in a similar expression as (39) :

 $\frac{\Delta A(t)}{\Delta A_0} = e^{\frac{-bv_s(t)\frac{1}{(1-\epsilon)}}{\Delta A(t)}}$

(40)

where :

$$A = A(t) = A - A_0$$
 (m²)
 $A = A_0 = A_1 - A_0$ (m²)
 $A_1 = initial cross-section area after disturbance$ (m²)
 $A_0 = equilibrium value of cross-section area$ (m²)
 $b = width of the channel$ (m)
 $v_s(t) = sedimentation velocity = \partial z_b / \partial t$ (m/s)
 $z_b = bottom level$ (m)
 $\epsilon = pore content of the deposited sediment$

4.3.6. Stability shear stress

4.3.6.1. Without wind and wind waves

The relations involving A_c (see Section 4.3.2) can be given for more general applicability with the definition of the stability shear stress (Bruun and Gerritsen, 1960) :

$$\tau_g = \frac{\rho g \mathcal{Q}_{\text{max}}^2}{(A_c^2)^2 C^2} \tag{41}$$

where :

ρ	- fluid density	(kg/m^3)
g	- acceleration of gravity	(m/s ²)

leading to the following relation between A_c and Q_{max} :

$$A'_{\sigma} = \frac{Q_{\max}}{C \sqrt{\left(\frac{\tau_s}{\rho g}\right)}}$$
(42)

Gerritsen and de Jong (1983) show that this relation has the highest correlation of all relations studied.

The value for the stability shear stress in (41) and (42) depends on the littoral drift, and varies from 3.5 to 5.0 N/m² for small to large littoral drift and subsequent sediment load. For the Western Scheldt a value of 4.3 N/m^2 was found under spring tide conditions. Bruun and Gerritsen assumed that bed forming conditions would be closer to spring tide than to the average tide. Therefore, the mean spring tide was used as governing tidal condition.

Next to τ_c it is also possible to define a dimensionless shear stress τ_s^* depending on sediment properties :

$$\tau_{g}^{*} = \frac{\mu \tau_{g}}{(\rho_{g} - \rho) g D_{50}}$$
(43)

where :

ш

 $\rho_{\rm m}$ - density of sediment

 (kg/m^3)

- ripple coefficient of sediment : $\mu = (C_{pon}/C)^{1.5}$ with

C_{D90} - Chezy coefficient corresponding to grain size D₉₀

C - overall Chezy coefficient

In Gerritsen (1990), an average value for τ_{*g} of 0.331 can be found (in the report erroneously : 0.336) for the dimensionless shear stress for Wadden Sea inlets.

Corresponding with the method of the stability shear stress, the following relationship between the time averaged velocity (or the time averaged maximum velocity) and sediment properties can be found in Gerritsen (1990) and Gerritsen et al. (1990) :

$$\frac{V}{\left(\frac{\Delta D}{\mu}\right)^{0.5}C} = A_1 \tag{44}$$

where :

- Δ relative density of sediment
- D diameter bottom material (m)

 $A_1 = constant$

For the Western Scheldt an average value of 0.37 for A_1 was found. For the Wadden Sea values of 0.43 and 0.40 for A_1 were found for inlets and channels, respectively.

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It is noted that the determination of the Chezy-value (incorporated in the parameter μ) is the main difficulty in the calculation of τ_s , τ_s^* and ∇ . Gerritsen et al. therefore conclude that, although the approach of using the dimensionless shear stress as stability parameter is promising, further research is required to refine the method.

4.3.6.2, With wind and wind waves

The stability shear stress is affected by wave action. It is possible to adjust the stability shear stress for this effect by using Bijker's formula (Bijker, 1967). In Gerritsen (1990) the following relation between the stability shear stress with and without wave action is given :

$$\sqrt{\tau_{g}}e^{-0.27\frac{\Delta D_{pg}}{\mu\tau_{g}}} = \sqrt{\tau_{g}^{7}}e^{\frac{-0.27}{\mu\tau_{g}^{\prime}}\left(1+0.5\left(\frac{u_{0}}{\nu}\right)^{2}\right)}$$
(45)

where :

by :

τ'8	- stability shear stress with wave activity	(N/m ²)
u _o	- maximum orbital velocity near the bottom	(m/s)
v	= average velocity over cross-section	(m/s)
ξ	- factor for the increase in shear stress by	wave action, defined

$$= c \left(\frac{f_{v}}{2g}\right)^{0.5}$$
(46)

where :

 f_{μ} - friction factor for waves defined by Jonsson (1965) and C - Chezy coefficient.

4.3.7. Other empirical relationships

Eysink (1990) assumes the validity of a relation for the Dutch coast between the sand volume stored in the ebb tidal deltas in front of tidal inlets for outer deltas in the USA that was published by Bruun (Bruun, 1978) :

$$V_{o} = c_{o} (TV)^{1.23} \tag{47}$$

where :

 V_{c} - sand volume stored in outer delta (m^{3})

c = empirical coefficient

A similar relationship between the tidal prism and the sand volume of the ebb tidal deltas was published by Walton and Adams (1976).

Stive and Eysink (1989) describe a relation between the intertidal surface area and the total area covered by water at mean high water. This relation is only presented graphically in their report and can therefore form not directly be used in the model.

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ξ

4.4. Summary

In the following table, the empirical relations are summarized :

5	Р	EV	FV	тv	R	h	b	Q _{max,f}	Q _{max,e}	Q _{max}	a _{sin}
A _c	(14) (15) (16) (17)	(19) (22) (23)	(19) (22) (23)	(20) (22)	(16)	(17)	(18)	(25)	(25)	(26)	(24) (27) (28)
A _c '					1	2	1	(29) (30)	(29) (30)	(29)	
V		1	17		(31) (32)	27		5.16	 		
vmax		X			(33)	5		÷.,		- Ky	120
h		1	198	(34)			(35) (36)			- Ky	
S				1.5	1	(36)	(36)				
v _c			1.50	(47)	1. An			11	1 - 5 - F	1	4
vo		- 2		(47)	16	1			1	140	

In the row and column headers, the parameters are listed. In case of a relation between two or more parameters, the number of that relation can be found in the cell on the intersection of the row and column belonging to the parameters involved.

5. Characteristics of the model to be developed

5.1. Introduction

The model to be developed (from now : the model) should have general characteristics and specific characteristics with respect to the area of interest (Wadden Sea and Western Scheldt). In this chapter, both types of characteristics are described, using the information contained in the previous chapters concerning estuaries. Section 5.2 contains a description of the independent parameters of the model, while Section 5.3 describes the parameters the model should describe according to the requirements of client. Finally, Section 5.4 contains the main properties of the model.

5.2. Independent parameters

The most important driving force, the tide, should be used as (seaward) boundary condition in the tidal model. In both areas of interest, the dominant tide is semi-diurnal. At least M2 and S2 should thus be present in the boundary conditions. Furthermore, M4 should be included in the boundary conditions to represent the tidal deformation, although it will also be generated inside the model by non-linear interactions. A more detailed study of the most important tidal constituents will have to reveal the exact boundary conditions. It is noted that almost all empirical relations are defined under mean tide conditions. It is therefore advised to perform model runs with boundary conditions corresponding to a mean tide situation.

Wind and wind waves are mainly important in the mouth of the estuary and on shoals above the subtidal level (see Section 2.3). In literature hardly any information is available on the representation of wind and wind waves in a morphological model of an estuary. In only one empirical relationship described in the previous chapter, the effect of wind and wind waves is built in, i.e. in the stability shear stress expression of Bruun and Gerritsen (1960) by using Bijker's formula (1967). The stability shear stress, however, needs further research before it can be used in a model in order to take into account the effect of wind and waves (Gerritsen et al, 1991). If the effect of wind and waves is not taken into account the relation involving the stability shear stress is in fact the same as the stability relationship involving tidal prism. Therefore it is recommended to neglect the effect of wind and waves but to use the stability shear stress in the model such that it can easily be extended in the future to include the effect of wind and waves.

In the Wadden Sea no large density differences occur both in the horizontal and in the vertical plane. In the tidal model of the Wadden Sea, the influence of the density gradient can thus be neglected. Also the Western Scheldt is a well-mixed estuary. The horizontal density gradient should, although the river discharge of the river Scheldt is small (105 m^3/s ; Gerritsen et al, 1990), however not be neglected.

In the Sections 4.3.2.2 and 4.3.2.3, separate empirical relations are presented for ebb channels and flood channels in the Western Scheldt. For the Wadden Sea, no specific relations for ebb channels and flood channels were found in literature. There are, however, relations for the Wadden Sea between the ebb and flood volume and the cross-sectional area that can be used separately for the ebb and flood channels. Therefore, it is possible to model ebb and flood channels separately in both areas. The distinction between ebb and flood channels will be based on the direction of the residual flow.

Important factors determining the sediment transport rate are the properties of the sediment involved, like sediment size, shape, density, fall velocity and sediment availability. The ideal model should therefore use these sediment properties in the sediment transport equations. In the dynamic/empirical model of the Di Silvio, the fall velocity is used in the process equations.

In the expression of the dimensionless stability shear stress, parameters involving sediment properties are used (see equation (43)).

The incorporation of limitations in the sand availability will be very complicated. It is only possible via empirical relations.

5.3. Dependent parameters

This section describes the (physical) parameters the model should describe to meet client's requirements : the model should give global information about the change in cross-sectional area of channels and the size of the intertidal and subtidal area over the model area for a period of at least 50 years.

Cross-sectional and location stability are both important for channel/inlet morphology. Assuming a one-dimensional model, the location of a channel in the horizontal plane cannot be modelled. Furthermore, the location of tidal inlets can only be modelled with (at least) a 2-dimensional model covering a large area including the ebb tidal delta. Finally, no information was found in literature about a dynamic/empirical model combining morphological development of both the location of a channel and the cross-sectional shape of a channel.

In view of above mentioned requirements, the dependent parameters can easily be listed :

- the cross-sectional area for grid cells for each time step from the initial disturbance to the equilibrium situation;
- the time span from the initial disturbance to the time at which the equilibrium is attained.
- size of intertidal and subtidal area.

5.4. Main properties

The model should consist of two or more modules. The 'first' module will always be a the hydrodynamic module, describing the water movement. The other module(s) will describe the morphology. These modules have to be linked in a proper way. De Vriend (1985) lists three properties so-called 'compound models' should have :

- each module has to be able to perform its specific tasks;
- the combination of modules should be well-balanced;
- the combination of modules must not give rise to spurious interactions, neither physical nor numerical.

In Karssen and Wang (1991), the advantages and the disadvantages of the model of Allersma (see Section 3.3.2.3) and the model of Di Silvio (see Section 3.3.2.2) are presented. A summary of the results can be found in the following table :

	Di Silvia	Allersma
Equilibrium		+
Proces description	+	0
Soundary conditions		
Behaviour of the solution (linear)	0	0
Modelling of human interference	-	+
Experience		+
Possibility of extension	+	

It is clear that a combination of both models would give a model with good properties. In the same note, it was concluded that such a straight forward combination is not possible. It is, however, possible to use a different empirical relation in the Di Silvio model, with respect to the equilibrium condition.

In view of its properties, the model to be developed will be a dynamic/empirical model. The exact formulation of the model will be determined in the next phase of the study.

6. Conclusions

Mathematical modelling of morphological changes is still in an early stage of development. Three types of morphological models can be distinguished : dynamic models, dynamic/empirical models and empirical models. Dynamic models are very time-consuming and can (at the moment) only be used properly for short-term prediction (0-10 years). The empirical models are very useful to make a quick evaluation of an anticipated development. The changes that can be modelled, however, have to remain relatively small compared to the existing conditions. Dynamic/empirical models use hydrodynamic models to predict the water movement and empirical relationships between parameters calculated from the water movement and parameters representing the geometry of the estuary to predict the morphological changes. The advantage of these models is that they are not very time-consuming, and almost always tend to stable situations. For long-term predictions, the development of a dynamic/empirical model is advised.

There is some experience with the dynamic/empirical model of Allersma, but in this model the boundary conditions are not well-defined. The process description of the model of Di Silvio is better, but there is not much experience with this model. Furthermore, the definition of the equilibrium in his model is unclear.

Most empirical relations found in literature are relationships between the cross-sectional area and the characteristic volumes. These relationships show high correlation. Also relationships between the maximum discharge and the cross-sectional area are found. A (theoretical) relation between the maximum discharge and the cross-sectional area during this maximum discharge involving the so-called stability shear stress is most promising (highest correlation !).

There are also empirical relations with respect to the shape of the channel, intertidal and subtidal area, and empirical relations relating the depth and the width of the channel.

Given the different relations between the cross-sectional area and the ebb volume and between the cross-sectional area and the flood volume, it should be possible to model the areas of interest with a distinction between ebb and flood channels. The neutral channels can then be modelled using empirical relations between the tidal volume and the cross-sectional area.

In this stage of the study it is difficult to determine if separate modelling of channels and intertidal flat areas is possible, because this depends in large extent on properties of the tidal model to be used. The decision if these areas should be modelled can be taken in a later stage of the study.

Only one empirical relation was found incorporating wind and wind waves (i.e. the stability shear stress expression). It was already stated that this expression cannot yet be used in the model. It is therefore advised not to model wave influences.

The empirical relations are valid for the Western Scheldt and/or the Wadden Sea. This means that the sediment properties are incorporated in the empirical relations. In view of the fact that the bottom material in both estuaries is mainly sand, the relations are valid for sandy bottoms. There were no empirical relations found for silt bottoms. However, in case specific empirical relations for silt will be found, these empirical relationships can be used in the model without difficulties, provided that these relationships use the same parameters.

It is necessary to develop a model combining a sediment transport model and a (already existing) hydrodynamic model. The influence of morphology on the water movement is very important. This effect cannot be modelled without using the interaction between morphological change and hydrodynamic conditions. Therefore, a transport model only cannot be used to perform long-term morphological computations.

In the transport model, both advection and dispersion should be incorporated. If the sediment transport model is based on the instantaneous flow field (dynamic model) the advection term is often dominant, but taking the dispersion term into account only makes the numerical computation easier. For the long-term models where the transport model is based on the residual

flow field, dispersion is often dominant. Advection, however, can be incorporated easily.

The sediment transport model will need a boundary condition at the open sea. The exact form of the conditions depends on the physical relations to be used in the model. Most empirical relations found are for the gorge and the inner areas of the estuary or tidal inlet. Empirical relations for the ebb tidal delta area are scarce and they are difficult to be implemented in a one-dimensional model. Therefore it is recommended to place the open sea boundary not outside the gorge. However, the system to be modelled is thus not a closed one because of the boundary condition and because there will always be an exchange of sediment through the boundary.

The exact formulation of the model to be developed will be worked out in the next phase of the study : formulation of physical relations.

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Figure 1. Definitions of parameters of relation (36) (from : O' Connor et al., 1991)



Figure 2. Definitions of parameters of relation (37) (from : Vincent and Corson, 1981)

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