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A Review on Coastal Sediment Transport Modelling

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Introduction

Coastal and estuarine sediment transport is a complex, multidimensional, multiscale, dynamic process. Modelling efforts need to describe both sediment and ambient fluid (water) motions and their interactions. Many issues arise from the multiscale nature of the problems studied: coastal models are usually developed at a scale (order of at least tens of meters) that is much larger than that on which physical processes such as turbulence, sediment-sediment interactions and fluid sediment interactions occur. Theses processes thus have to be implemented as sub-grid scale modelling, which is not specific to sediment transport, but particularly crucial to coastal morphological change and sediment transport given the importance of near-bed processes of different nature (sediment-sediment interactions and near-bed hydrodynamics).

We will first review how the sediment motions and the different physical processes are commonly described in coastal area and river models. We will show how the suspended sediment concentration can be obtained and how interactions between the suspended sediment and the ambient fluid flow are parameterized. We will also describe how near-bed processes are implemented as a sub-grid scale modelling both in terms of sediment transport but also in terms of hydrodynamics. We will then review in more details several existing coastal sediment transport models, and finally we will discuss the implementation of such sediment parameterizations in POLCOMS.

Suspended sediment transport and morphological modelling

2.1 Suspended sediment transport

2.1.1 Suspended sediment concentration

The suspended sediment concentration ($c(x, y, z, t)$ in Cartesian coordinates) is usually either prescribed by a Rouse profile or calculated by solving a balance equation for the concentration, which results from sediment mass conservation.

The Rouse profile was developed assuming a uniform equilibrium suspension and balance between upward sediment transfer due to turbulent mixing and downward settling. It specifies the concentration in the water column following

$$\frac{c}{C_{ref}} = \left(\frac{z}{z_{ref}}\right)^{(0.74W_s/\kappa u_\star)} \tag{2.1.1}$$

where c is the concentration at level z, C_{ref} is the concentration at a reference level z_{ref}, W_s is the sediment settling velocity, κ the von Karman constant, and $u_\star$ the friction velocity. In this equation, C_{ref}, z_{ref}, W_s, κ, and $u_\star$ still need to be determined. C_{ref} and z_{ref} can be seen as a concentration boundary condition at the bottom of the numerical domain. Specification of W_s will be discussed in a further section. Even though reduction of the von Karman constant value has been repeatedly observed (see *Vanoni* (1975) for example), κ is usually taken to be the clear fluid value of 0.4. $u_\star$ is given by the bottom boundary layer model associated.

The sediment mass conservation reduces to an advection-diffusion equation, and can be expressed as

$$\frac{\partial c}{\partial t} + \frac{\partial u_i c}{\partial x_i} = \frac{\partial W_s c}{\partial x_3} + \frac{\partial}{\partial x_i}\left(K_{s,i}\frac{\partial c}{\partial x_i}\right) + S_c \tag{2.1.2}$$

with u_i the i^{th} component of the flow velocity, x_3 the vertical direction, $K_{s,i}$ the i^{th} component of the sediment diffusivity and S_c is a source/sink term (e.g., *Lesser et al.*, 2004; *Warner et al.*, 2008). The fall velocity W_s, the sediment diffusivities $K_{s,i}$ (usually split in two, a vertical diffusivity and a horizontal diffusivity) and the source/sink term all need to be explicitly prescribed. Boundary conditions are also required to solve this balance equation.

2.1.2 Boundary conditions

Using a Rouse profile requires the appropriate specification of a bottom concentration boundary condition (both C_{ref} and z_{ref}). When solving the advection-diffusion equation, boundary conditions at lateral boundaries and at the top and bottom boundaries are all necessary, but we will focus here on the top and bottom ones.

At the top boundary, flux conditions are commonly used and are implemented in mainly two ways. The first approach is to set that the total sediment flux must vanish at the top boundary (*Zhang et al.*, 1999; *Harris and Wiberg*, 2001), which can be written as follows:

$$-W_s c + K_{s,z}\frac{\partial c}{\partial z} = 0. \tag{2.1.3}$$

The second approach is to only set the vertical diffusive flux to be zero (e.g., *Lesser et al.*, 2000).

At the bottom boundary, the condition can either be of Dirichlet type (concentration condition), which will specify the the concentration at a reference level in the near-bed region, or of Neumann type (flux condition), which will specify the vertical sediment flux.

Concentration bottom boundary condition

This condition, also called reference concentration, usually provides a formula for the concentration at some reference level (C_{ref} at z_{ref}) where both C_{ref} and z_{ref} are functions of the flow and sediment parameters (e.g., Shields parameter, specific gravity, ...). An issue with using such a boundary condition is that the bottom grid location may not coincide with the reference level, and the concentration at the bottom grid location then needs to be extrapolated from the reference concentration, usually using a Rouse profile, (*Lesser et al.*, 2000).

There exist quite a few reference concentration relationships, eight of which were assessed in *Garcia and Parker* (1991), and the most commonly used formulas in large scale models are that of *Smith and McLean* (1977) and that of *van Rijn* (1984c).

The expression proposed by *Smith and McLean* (1977) is such that the reference concentration C_{ref} is

$$C_{ref} = C_0\frac{\gamma_0 S_0}{1+\gamma_0 S_0}, \tag{2.1.4}$$

where $\gamma_0 = 2.4\times10^{-3}$ is a constant, $C_0 = 0.65$ is the maximum permissible concentration and S_0 is the normalized excess bed shear stress,

$$S_0 = \frac{\tau_b - \tau_{cr}}{\tau_{cr}}, \tag{2.1.5}$$

with τ_b the bed shear stress and τ_{cr} the critical bed shear stress for incipient motion of sediment. In the *Smith and McLean* (1977) formulation, C_{ref} is calculated at the level

$$z_{ref} = \alpha_0(\tau_b - \tau_{cr})/[(\rho_s - \rho_w)g] + k_s \tag{2.1.6}$$

with ($\alpha_0 = 26.3$), ρ_s and ρ_w respectively the sediment and water densities, and k_s the bed roughness.

The expression proposed by *van Rijn* (1984c) was developed for open channel flows and is such that

$$C_{ref} = 0.015\frac{D_{50}}{z_{ref}}\frac{{S_0}^{1.5}}{{D_\star}^{0.3}} \tag{2.1.7}$$

where

$$D_\star = D_{50}\left[\frac{(s-1)g}{\nu^2}\right] \tag{2.1.8}$$

with D_{50} the median sediment grain diameter, $s = \rho_s/\rho_f$ and ν the kinematic viscosity of water. The reference level in this expression is taken to be half the bed-form height or the equivalent bed roughness with a minimum value of 0.01 times the flow depth.

The main difference between the two formulas (equations 2.1.4 and 2.1.7) presented is the absence of maximum concentration in the *van Rijn* (1984c) formula. Although the experimental data used to derive such formulas was usually collected for small to moderate Shields parameters, a limit on the reference concentration should exist for large Shields parameter. In that sense, the *Smith and McLean* (1977) presents a distinct theoretical advantage. Formulas similar in form to that of *Smith and McLean* (1977) have since been introduced (e.g., *Garcia and Parker*, 1991; *Zyserman and Fredsoe*, 1994) and usually follow the form

$$c_{ref} = \frac{AX^n}{1 + \frac{AX^n}{c_m}} \tag{2.1.9}$$

where A, c_m and n are constants, and X is a combination of the appropriate dimensionless variables (such as the Shields parameter θ). Formulas following equation 2.1.9 give zero reference concentration for $X = 0$ and tend to c_m for large values of X, which is the expected behavior. The *Smith and McLean* (1977) formula almost follows equation 2.1.9 with $X = S_0$, $A = \gamma_0$ and $n = 1$. Other uses include *Garcia and Parker* (1991) with $X = u_\star R_p^{0.6}/W_s$ and $R_p = D_{50}\sqrt{(s-1)gD_{50}}/\nu$, $A = 1.3 \times 10^{-7}$, $c_m = 0.3$, $n = 5$ and *Zyserman and Fredsoe* (1994) with $X = \theta - \theta_{cr}$ where θ_{cr} is the critical Shields parameter for initiation of motion, $A = 0.331$, $c_m = 0.46$, $n = 1.75$.

Flux boundary conditions

Flux boundary conditions aim to provide some kind of information on the vertical sediment flux at the bottom boundary. One then can either directly specify the net sediment flux at the bottom boundary (e.g., *Harris and Wiberg*, 2001; *Wai et al.*, 2004), or specify the erosional and depositional fluxes as sources and sinks in the advection-diffusion equation and then set the diffusive (and advective) flux of sediment through the bottom boundary to zero (e.g., *Lesser et al.*, 2000; *Warner et al.*, 2008).

The net bottom boundary sediment flux is commonly divided in an upward part (erosion, pick-up, E) and a downward part (deposition, D). While deposition is usually calculated as the flux due to particle settling, there exist several ways to calculate the erosion. Even though the objective is always to calculate an erosion value from flow and sediment parameters, two main methods that have historically been closely linked to the cohesiveness of the sediment can be discerned:

- Assuming that the disequilibrium introduced by the unsteadiness remains mild, the erosion flux can be considered to be equal to the entrainment rate under equilibrium condition and related to the reference concentration value through the settling velocity (*Garcia and Parker*, 1991). This approach has been widely used in area 2D and 3D model but almost exclusively for non-cohesive sediments (e.g., *Harris and Wiberg*, 2001; *Wai et al.*, 2004).

- The other main method is to provide a formula relating directly the erosion flux to flow and sediment parameters (e.g. *Lumborg and Windelin*, 2003). Although this has typically been used mainly for cohesive sediments, some empirical formulas also exist for non-cohesive sediments (e.g., *van Rijn*, 1984b). In large scale models, this method is however almost exclusively used to describe erosion of cohesive sediments, the notable exception being the recent parameterization employed in the NOPP Community Sediment Transport Model, which introduces a "universal" direct formula ($E = f(\theta)$) to be used for both cohesive and non-cohesive sediments.

Erosion and deposition fluxes

The erosion rate has been one of the most studied issue in fine sediment transport and theoretical, laboratory studies and field observations have been used to investigate the rate of erosion. The general consensus is that bottom shear stresses are the dominant forces causing erosion while the sediment bed characteristics control the resistance to erosion. Mathematically, two formulations, a power law and exponential erosion, have been introduced, defended and are still employed. Power laws relate the erosion E to a power of the available excess shear stress:

$$E = M\left[\tau_b - \tau_{ce}\right]^n \tag{2.1.10}$$

where M and n are empirical constants and τ_{ce} is the critical stress for erosion. In particular, a linear formula ($n = 1$) has been widely used (e.g., *Ariathurai and Krone*, 1976; *Mehta et al.*, 1989; *Sanford and Halka*, 1993; *Mei et al.*, 1997). Exponential forms are usually written in the following form (e.g., *Parchure and Mehta*, 1985)

$$E = E_0 \exp\left[\alpha\left(\tau_b - \tau_{ce}\right)^{\beta}\right] \tag{2.1.11}$$

where E_0 is the empirical floc erosion rate and both α and β are empirical constants. The exponential form has mostly been used for depth-limited erosion with $\tau_{ce} = \tau_{ce}(z)$, while a linear erosion formula (equation 2.1.10 with $n = 1$) has almost always been used for unlimited erosion. However, *Sanford and Maa* (2001) recently showed that a linear erosion formula may be used to represent both depth-limited and unlimited erosion, provided that the critical bottom shear stress increases with depth.

Deposition of fine particles is usually parameterized following *Krone* (1962):

$$D = W_s c\left(1 - \frac{\tau_b}{\tau_{cd}}\right), \text{ for } \tau_b < \tau_{cd} \tag{2.1.12}$$

where τ_{cd} is the critical shear stress for deposition. For larger and non-cohesive particles, this critical stress is not used and the deposition is then simply

$$D = W_s c. \tag{2.1.13}$$

This difference in deposition flux formulation also corresponds to the difference in method discussed in the previous section. To the exception of the NOPP Community Sediment Transport Model, equation 2.1.12 is used in conjunction with $E = f(\theta)$, and equation 2.1.13 in conjunction with the reference concentration.

Since τ_{ce} is taken to be greater than τ_{cd} in equations 2.1.10, 2.1.11 and 2.1.12, erosion and deposition are mutually exclusive. This defines three states (e.g., *Li and Amos*, 2001) depending on the value of the bottom shear stress:

- When $\tau_b < \tau_{cd}$, there is no erosion and only deposition, this is the depositional state.
- When $\tau_{cd} < \tau_b < \tau_{ce}$, there is neither erosion nor deposition, and this is a stable state.
- When $\tau_{ce} < \tau_b$, there is no deposition and only erosion, this is the erosional state.

It is worthwhile noting that such a concept of mutually exclusive deposition and erosion is foreign to non-cohesive sediments, and somewhat counterintuitive to non-buoyant particles under the action of gravity. In particular, the deposition representation has been challenged. *Sanford and Halka* (1993) observed a decrease of the suspended sediment concentration in phase with the decceleration of the flow, which can not be modelled with equation 2.1.12 since this formula predicts a continuous increase of the suspended sediment concentration until the bed shear stress decreases below its critical value for deposition. Instead, they were able to reproduce the observed concentrations by taking a continuous deposition (equation 2.1.13). *Winterwerp and van Kesteren* (2004) argue that mutually exclusive deposition and erosion is not supported by a sound explanation of the physical processes involved, and also assume that the deposition is continuous (equation 2.1.13), thus allowing simultaneous erosion and deposition.

2.1.3 Sediment diffusivities

Two different specifications are typically used for the sediment diffusivities: one for diffusion in the horizontal plane with K_h being the horizontal diffusivity used both for x and y, one for vertical diffusion with K_v the vertical diffusivity. It is common to relate the sediment diffusivity to the turbulent eddy viscosity through a Schmidt-Prandtl number that can be taken to be unity (sediment turbulent diffusivity and eddy viscosity are then equal), a constant different than one, or that can be specified as a function of flow and sediment parameters. The turbulent diffusivity is usually larger than the eddy viscosity because of centrifugal forces in turbulent eddies ejecting particles to the outside of the eddies, thus leading to Schmidt numbers less than one.

van Rijn (1984c) related the sediment turbulent diffusivity to the turbulent eddy viscosity through two parameters, one of which is a function of the settling velocity and of the friction velocity, and as such can be seen as expressing the relative importance of the particles' gravitational inertia respect to the flow turbulence

$$\beta = 1 + 2\left(\frac{W_s}{u_\star}\right)^2 \text{ for } 0.1 < \frac{W_s}{u_\star} < 1. \tag{2.1.14}$$

The other is a function of the concentration and represents the effect the presence of particles has on the sediment diffusivity, which can then be written as:

$$K_v = \beta\left[1 + \left(\frac{c}{C_0}\right)^{0.8} - 2\left(\frac{c}{C_0}\right)^{0.4}\right]\nu_v \tag{2.1.15}$$

where $C_0 = 0.65$ is again the maximum permissible concentration and ν_v is the vertical eddy diffusivity.

More recently, *Rose and Thorne* (2001) only considered the β parameter and introduced another formula relating it to the velocity ratio. *Amoudry et al.* (2005) only considered a concentration dependence and provided an expression by an empirical fit of model's results to experimental data. Still, the vertical diffusivity in large scale model remains described in relatively simple ways (see chapter 3 for more details) that often neglect the concentration dependence.

2.1.4 Sediment settling velocity

The settling velocity of sediment is an important parameter in the determination of the suspended concentration profile as well as in the near bed conditions. In a rather obvious and expected way, it depends on the sediment and flow parameters (sediment diameter, specific gravity, fluid viscosity). The settling velocity of a single sphere of diameter d and specific gravity s in a semi-infinite fluid domain gives

$$W_{s0} = \sqrt{\frac{4(s-1)gd}{3C_D}} \tag{2.1.16}$$

where C_D is the drag coefficient and is a function of the settling particle Reynolds number $Re_s = W_s d/\nu$. Common approaches are then to set the settling velocity either as a user-specified, sediment specific parameter, or to employ formulas relating the drag coefficient or the settling velocity to the sediment and flow parameters.

Furthermore, it has been observed that the settling of sediment also depends on the local concentration. For non cohesive sediments, such dependence is commonly taken to follow the experimental results of *Richardson and Zaki* (1954), for which

$$\frac{W_s}{W_{s0}} = (1-c)^n \tag{2.1.17}$$

where W_{s0} is the settling velocity of a single sediment particle, and n depends on Re_s:

$$\begin{aligned} n &= 4.35 Re_s^{-0.03} && 0.2 < Re_s < 1 \\ n &= 4.45 Re_s^{-0.10} && 1 < Re_s < 500 \\ n &= 2.39 && 500 < Re_s \end{aligned} \tag{2.1.18}$$

For cohesive sediments, *Mehta* (1986) argued that the sediment settling velocity does not depend on the concentration at very low concentrations. At moderate concentrations, the settling velocity is found to follow (e.g., *Burt*, 1986; *Mehta*, 1986)

$$W_s = k_1 c^n \tag{2.1.19}$$

where k_1 depends on the sediment composition and n varies from 1 to 2 with a mean of about 1.3. At higher concentrations, settling is hindered and the settling velocity is found to decrease with concentration (e.g., *Mehta*, 1986)

$$\frac{W_s}{W_{s0}} = (1 - k_2 c)^n \tag{2.1.20}$$

which is similar to the experimental relationship of *Richardson and Zaki* (1954) and where W_{s0} is the settling velocity of a single sediment particle, k_2 is a coefficient that depends on the sediment composition and $n \approx 5$ (again similar to the value found by *Richardson and Zaki* (1954) for small Reynolds numbers). The limit between the two regimes is approximately for a mass concentration of 3 g/l, or a volumetric concentration of $c \approx 10^{-3}$. More recently, *Winterwerp* (2002) and *Dankers and Winterwerp* (2007) introduced another formula for hindered settling of suspended cohesive sediments:

$$\frac{W_s}{W_{s0}} = \frac{(1 - c_f)^m (1 - c)}{1 + 2.5 c_f} \tag{2.1.21}$$

where c_f is the volumetric concentration of flocs and the exponent m accounts for possible non-linear effects.

2.1.5 Effect of sediment on density

All models following one of the approaches previously described to calculate the suspended concentration profile actually implicitly assume a dilute mixture of fluid (water) and sediment. As such the density in the momentum equations should be that of the mixture and thus should account for the sediment concentration:

$$\rho = \rho_w + \sum_k c_k (\rho_{s,k} - \rho_w) \tag{2.1.22}$$

where the sum is done on the different sediment classes and allows the the representation of different sediments in the same model.

2.2 Near-bed and bed treatment

Capturing the details of near-bed sediment processes in coastal models would require prohibitively expensive high vertical resolution and small time stepping. Instead, these processes are modelled by specifying appropriate sub-grid scale relationships in order to relax the vertical length scale constraint. In addition, wave-averaged parameterizations help relax the time scale constraints.

2.2.1 Bed load

Bed load is the part of sediment transport that is due to interparticle interactions and which occurs in a near-bed region of high sediment concentration. It can not be resolved by typical multidimensional models for which sediment is implicitly assumed to be dilute, and is instead described by relating bed load transport rate to bottom shear stress. Such relationships have now been investigated both empirically and theoretically for several decades. The bed load transport rate has been measured directly in many experimental studies using bed load traps that lead to empirical formulas such as that of *Meyer-Peter and Mueller* (1948); *Wilson* (1966); *Ribberink* (1998). Originally, such formulas focused on open channel flows (steady, uniform flow), but have since evolved to account for wave and current combined flows.

Several studies also proceeded to provide theoretical and semi-empirical relationships for the bed load transport rate. *Einstein* (1950) used a statistical description of the near-bed sediment motions and related the bed load transport rate to the probability of a particle being eroded from the bed, itself relate to the flow intensity. *Bagnold* (1966) introduced equations giving the bed load, suspended load and total load transport rates as functions of the stream power for steady flows using considerations of energy balance and mechanical equilibrium. An extension of this approach was pursued later by *Bailard* (1981) for unsteady flows and gave the transport rates as functions of powers of the time-dependent free stream velocity. *Engelund and Fredsøe* (1976) assumed that bed load corresponds to the "transport of a certain fraction of the particles in a single layer", and obtained a semi-empirical law by considering the motion of individual particles and the most important forces of relevance. *van Rijn* (1984a) computationally solved equations of motion of individual saltating particles and calculated saltation characteristics, then used these results to deduce a semi-empirical bed load transport rate formula.

Bed load formula for currents

For currents, bed load formulas generally follow the following form:

$$\Phi_B = m(\theta - \theta_{cr})^n \tag{2.2.1}$$

where m and n are constants and θ_{cr} is the critical Shields parameter for initiation of motion. Φ_B is the dimensionless bed load transport

$$\Phi_B = \frac{Q_B}{\sqrt{(s-1)gD_{50}^3}} \tag{2.2.2}$$

Meyer-Peter and Mueller (1948)'s formula specifies $m = 8$, $n = 1.5$, *Wilson* (1966)'s takes $m = 12$, $n = 1.5$ for sheet flow, *Ribberink* (1998) found $m = 10.4$ and $n = 1.67$. More recently, *Soulsby and Damgaard* (2005) introduced a slightly different formula

$$\Phi_B = 12\theta^{1/2}(\theta - \theta_{cr}) \tag{2.2.3}$$

Bed load formula for waves and currents

Although numerous studies report bed load transport rates for currents, oscillatory flows or even co-linear combinations of the two, relatively few studies deal with the more realistic case of waves and currents superimposed at an angle. Some recent studies that deal with such situations include the work of *Ribberink* (1998), *Camenen and Larson* (2005), and *Soulsby and Damgaard* (2005).

The formula given by *Ribberink* (1998) provides the bed load transport for an oscillatory flow combined with a superimposed current under an arbitrary angle and under sheet flow conditions. It is similar in form to the formulas used for currents only:

$$\vec{\Phi}_B(t) = m\left[|\theta(t)| - \theta_c\right]^n \frac{\vec{\theta}(t)}{|\theta(t)|} \tag{2.2.4}$$

with $m = 11$, $n = 1.65$ and

$$|\theta(t)| = \sqrt{\theta_x(t)^2 + \theta_y(t)^2}. \tag{2.2.5}$$

The net sediment transport over a wave period is then the time average of equation 2.2.4. The components of the bed shear stress are calculated using a quadratic friction formula and the intra-wave near-bed velocity of the combined wave-current motion $u_b(t)$

$$\vec{\tau}_b(t) = \frac{1}{2} f_{cw} |u_b(t)| \vec{u}_b(t) \tag{2.2.6}$$

with

$$\begin{aligned} \vec{u}_b(t) &= \vec{U}_c + \vec{u}_w(t) && (2.2.7)\\ |u_b(t)| &= \sqrt{u_{b,x}^2(t) + u_{b,y}^2(t)} && (2.2.8) \end{aligned}$$

where $\vec{U}_c$ is the time-averaged current velocity vector, and $\vec{u}_w(t)$ the oscillatory flow vector of amplitude $\tilde{U}$. The friction coefficient f_{cw} is calculated as a combination of the wave friction factor f_w and of a current friction factor f_c

$$f_{cw} = \frac{U_c}{U_c + \tilde{U}} f_c + \frac{\tilde{U}}{U_c + \tilde{U}} f_w \qquad (2.2.9)$$

The formula given by *Camenen and Larson* (2005) is for the case of an asymmetric wave combined with a current under an arbitrary angle ϕ. The instantaneous bed shear stress is defined by

$$\tau_b(t) = \frac{1}{2} f_{cw} |U_c \cos\phi + u_w(t)| \left(U_c \cos\phi + u_w(t)\right) \qquad (2.2.10)$$

where the wave-current friction coefficient is calculated the same way as for *Ribberink* (1998)'s formula. The mean and maximum Shields parameters are respectively calculated following

$$\begin{aligned} \theta_{cw,m} &= \left(\theta_c^2 + \theta_{w,m}^2 + 2\theta_c\theta_{w,m}\cos\phi\right)^{1/2} && (2.2.11)\\ \theta_{cw} &= \left(\theta_c^2 + \theta_w^2 + 2\theta_c\theta_w\cos\phi\right)^{1/2} && (2.2.12) \end{aligned}$$

The net sediment transport under waves and current is:

$$\begin{aligned} \Phi_{B,w} &= a_w \sqrt{\theta_{cw,onshore} + \theta_{cw,offshore}}\,\theta_{cw,m} \exp\left(-b\frac{\theta_{cr}}{\theta_{cw}}\right) && (2.2.13)\\ \Phi_{B,n} &= a_n \sqrt{\theta_{cn}}\,\theta_{cw,m} \exp\left(-b\frac{\theta_{cr}}{\theta_{cw}}\right) && (2.2.14) \end{aligned}$$

where w and n correspond to the wave direction and its normal, $\theta_{cn} = 0.5 f_c (U_c \sin\phi)^2/[(s-1)gD_{50}]$. a_w, a_n, and b are coefficients such that $b = 4.5$, $a_n = 12$, and

$$a_w = 6 + 6\frac{\theta_c}{\theta_c + \theta_w} \qquad (2.2.15)$$

The formulas given by *Soulsby and Damgaard* (2005) can be applied to cases of asymmetric waves combined with a current under an arbitrary angle ϕ. The asymmetry is added through the presence of a second harmonic (Stokes 2nd order wave). The resulting oscillatory bed shear stress is then

$$\theta_w(\omega t) = \theta_w \left[\sin(\omega t) + \Delta \sin\left(2\omega t - \pi/2\right)\right] \qquad (2.2.16)$$

in which Δ is the ratio between the amplitudes of the first and second harmonics. Taking the x axis to be the current direction, the net bed load transport rate is given by:

$$\begin{aligned} \Phi_{B,x1} &= A_2 \theta_c^{1/2}(\theta_c - \theta_{cr}) && (2.2.17)\\ \Phi_{B,x2} &= A_2(0.9534 + 0.1907\cos 2\phi)\theta_w^{1/2}\theta_c + A_2(0.229\Delta\theta_w^{3/2}\cos\phi) && (2.2.18)\\ \Phi_{B,x} &= \max\left(\Phi_{x1}, \Phi_{x2}\right) && (2.2.19)\\ \Phi_{B,y} &= A_2 \frac{0.1907\theta_w^2}{\theta_w^{3/2} + (3/2)\theta_c^{3/2}}\left(\theta_c \sin 2\phi + 1.2\Delta\theta_w \sin\phi\right) && (2.2.20) \end{aligned}$$

with $A_2 = 12$. Such a formula corresponds to an approximation of the integral over a wave period of the time-dependent transport rate:

$$\vec{\Phi}_B(t) = A_2|\vec{\theta}|^{1/2}\left(|\vec{\theta}| - \theta_{cr}\right)\frac{\vec{\theta}}{|\vec{\theta}|} \tag{2.2.21}$$

and where the vector Shields parameter can be written:

$$\vec{\theta} = \vec{\theta}_c + \vec{\theta}_w\left[\sin(\omega t) + \Delta\sin\left(2\omega t - \pi/2\right)\right]. \tag{2.2.22}$$

It also does reduce to the current only formula for $\theta_w = 0$, and the symmetric wave with no current ($\theta_c = 0$ and $\Delta = 0$) gives no net bed load transport rate.

The *Ribberink* (1998) and *Soulsby and Damgaard* (2005) bed load approaches are relatively similar, since both are based on the integration of the time-dependent transport rate over a wave period. Still, the later is more practical in that it provides directly the wave-averaged net bed load transport rate. In both cases, the time-dependent transport rate is assumed to follow a relationship similar to that for uniform steady flows and thus both methods follow a quasi-steady approach. *Ribberink* (1998) specified that quasi-steadiness only remains valid for fine sediments ($D < 0.2$ mm), light material or small wave periods ($T < 3$ s). *Sleath* (1994) and *Zala Flores and Sleath* (1998) limited quasi-steadiness to values of the parameter $U_0\omega/(s-1)g$ less than 0.3.

The formula introduced by *Camenen and Larson* (2005) is significantly different in its form. In particular, the exponential form used implies that sediment transport is theoretically possible for bottom shear stresses below the critical bed shear stress. Even though this formula does show good agreement with data, and even though some data sets do give small sediment transport under the critical bed shear stress, the exponential form contradicts the physical intuition and general consensus that the driving forces due to shear first need to exceed the resisting forces such as friction in order to induce motion.

2.2.2 Bed morphology evolution

The bed morphology evolution is treated by deriving an equation for the sediment bed location from the sediment mass conservation. Such an equation is often called the Exner equation, which states in its original form:

$$\frac{\partial \eta}{\partial t} + A\frac{\partial Q}{\partial x} = 0 \tag{2.2.23}$$

where η is the bed elevation respect to a fixed datum, A is a coefficient and Q is the sediment flux. Other terms are now commonly added, such as a term representing the temporal changes of the sediment concentration profile. A more general form to the original Exner equation can be derived by considering the sediment mass conservation for a layer contained between $\eta_b(x, y, t)$ and $\eta_t(x, y, t)$ (e.g., *Paola and Voller*, 2005). Assuming that there is no mass source or sink in this layer and that the bottom of the layer (η_b) is the sediment bed (no normal mass flux of sediment), the conservation of sediment mass states that:

$$\int_{\eta_b}^{\eta_t}\frac{\partial}{\partial t}\left(\rho_s c\right)dz + \nabla_H\cdot\vec{Q} + q_{vt} = 0 \tag{2.2.24}$$

where η_b and η_t are the bottom and top of the layer, ∇_H is the horizontal gradient, $\vec{Q}$ is the horizontal entire layer mass flux (sediment transport rate) vector given by:

$$\vec{Q} = \int_{\eta_b}^{\eta_t}\rho_s c\vec{u_s}dz \tag{2.2.25}$$

with $\vec{u_s}$ the sediment velocity vector. Finally, q_{vt} is the sediment mass flux per unit surface across the top of the layer (positive if out of the layer). Equation 2.2.24 can also then be rewritten as

$$\frac{\partial}{\partial t}\int_{\eta_b}^{\eta_t}\rho_s c dz - \rho_s c_t \frac{\partial \eta_t}{\partial t} + \rho_s c_b \frac{\partial \eta_b}{\partial t} + \nabla_H \cdot \vec{Q} + q_{vt} = 0 \tag{2.2.26}$$

which is the form most commonly used in morphological models, albeit simplified by making further assumptions.

When the layer considered is the entire water column, the top of the layer is typically taken to be the mean water level, which is assumed to be constant with time and across which there is no sediment flux. Equation 2.2.26 then simplifies to the Exner equation with an added storage term (time rate of change of the total mass of suspended sediment), which has commonly been used in morphological models (e.g., *Zhang et al.*, 1999; *Wu et al.*, 2000; *Harris and Wiberg*, 2001)

$$\frac{\partial}{\partial t}\int_{\eta_b}^{MWL}\rho_s c dz + \rho_s c_b \frac{\partial \eta_b}{\partial t} + \nabla_H \cdot \vec{Q} = 0 \tag{2.2.27}$$

with Q being the total sediment transport rate.

Equation 2.2.26 can also be applied to a near-bed layer. Many models that do so use the following equation to determine the bed location

$$\rho_s(1-p_c)\frac{\partial \eta_b}{\partial t} + \nabla_H \cdot \vec{Q}_B + E - D = 0 \tag{2.2.28}$$

where the vertical flux is split into an upwards portion (erosion, E) and a downwards portion (deposition, D), $\vec{Q}_B$ is the bed load transport rate vector and p_c the bed porosity. While the conservation of mass (equation 2.2.26) can indeed lead to equation 2.2.28, and q_{VT} can be split into upward and downward fluxes, other additional assumptions are required, and appropriate discussion on such assumptions is rarely done. Applying equation 2.2.26 to the bed load layer, the top of the layer, $\eta_T = \eta_B + \delta_B$ where δ_B is the bed load layer thickness, is not necessarily constant with time and the mass conservation then reads

$$\frac{\partial}{\partial t}\int_{\eta_b}^{\eta_t}\rho_s c dz - \rho_s c_t \frac{\partial \eta_t}{\partial t} + \rho_s c_b \frac{\partial \eta_b}{\partial t} + \nabla_H \cdot \vec{Q_B} + E - D = 0 \tag{2.2.29}$$

where E and D are estimated at the top of the bed load layer. Using equation 2.2.28 thus implicitly assumes that the storage term in the bed load layer and the time dependence of the top of the bed load layer are both negligible. While this may not be true if equation 2.2.29 is used in intrawave modeling, the wave averaged version for a periodic forcing will indeed reduce to equation 2.2.28.

In addition to using Exner-like equations, layered structures are becoming more popular to describe the sediment bed, especially when different sediments are considered in the model (e.g., *Gessler et al.*, 1999; *Hydroqual*, 2002; *Warner et al.*, 2008). In general, such layered structure uses the concept of an active layer from which sediment is eroded and deposited, and of underlying layer(s) of sediment. In *Gessler et al.* (1999), the mass conservation is applied to a sediment class in the active layer but the equation for the bed elevation still follows the form of equation 2.2.28 (the last three terms being summed over all sediment classes). In *Warner et al.* (2008) the bed tracking process is a little more complicated and can be summarized as follows. First, an active layer thickness is calculated. If the top layer of sediment is less thick than the active layer, sediment is entrained from deeper layers until the top layer has the same thickness as the active layer. Sediment is then transported, eroded and deposited, limited by sediment availability (the mass of available sediment is the mass contained in the active layer). If deposition results in a top layer thicker than a user defined value, a new layer is created. Finally, the active layer thickness is recalculated and the bed layers adjusted accordingly. The main concept of such layered structure consists of setting a limit to the amount of sediment that can be eroded, which introduces the notion of availability of sediment and expresses that not all of the sediment can be eroded at once.

2.3 Bottom boundary layer model

In a way similar to the near-bed sediment processes, the vertical resolution of regional scale models will not be sufficient to resolve the fluid flow gradients and algorithms that parameterize the bottom boundary layer processes are thus required. The appropriate determination of the bottom boundary layer is particularly important for sediment transport, which strongly depends on the bottom shear stress. For our purposes, the bottom boundary layer model will seek to provide bottom shear stress specification, which, again similar to the near-bed sediment processes, are aimed at wave-averaged predictions, instead of intrawave predictions.

2.3.1 Pure current boundary layer

The bottom shear stress in the case of a pure current is commonly calculated using simple drag coefficient expressions that, in turn rely on linear bottom drag, quadratic bottom drag, or a logarithmic velocity profile.

The linear and quadratic drag-coefficient approaches relate the bottom shear stress to the near-bed velocity (usually the velocity in the bottom grid, u) and can be written as:

$$\tau_b = \rho_f c_1 u \tag{2.3.1}$$

for the linear approach, where c_1 is a drag coefficient and has the dimensions of a velocity. For the quadratic approach, the bottom shear stress is given by

$$\tau_b = \frac{1}{2}\rho_f f_c u^2 \tag{2.3.2}$$

where f_c is a non-dimensional coefficient.

The logarithmic approach assumes that the flow velocity follows the classic rough wall log-law vertical profile close to the bed, for which the velocity at a given elevation is given by

$$u(z) = \frac{u_\star}{\kappa} \ln\left(\frac{z}{K_s}\right) \tag{2.3.3}$$

where $u_\star = \sqrt{\tau_b/\rho_f}$ is the friction velocity, K_s is the bed roughness and κ the von Karman constant. This approach can be rewritten in the form of equation 2.3.2 and the bed shear stress in then given by

$$\tau_b = \rho_f \left[\frac{\kappa}{\ln\left(z/K_s\right)}\right]^2 u(z)^2 \tag{2.3.4}$$

An important advantage of the log-law approach lies in the elevation dependence in equation 2.3.4. Because of morphological changes, the elevation of the bottom numerical grid will also change. In turn, such elevation changes can relate to different bed shear stresses, which is not accounted for by the linear and quadratic approaches. An appropriate value for κ also has to be specified, and the clear fluid value of 0.41 is usually implemented. We already mentioned that it has been observed that the value of the von Karman constant should be lowered in presence of sediments (see for example *Vanoni* (1975)). While this effect is neglected for the suspended sediment far from the bed, mainly based on the diluteness of sediment, higher sediment concentration in the boundary layer could have a more significant effect. Values as low as 0.3 have been reported in the literature from experiments (e.g., *Vanoni*, 1975; *Bennett et al.*, 1998) and from numerical results (e.g., *Longo*, 2005; *Amoudry et al.*, 2008). The specification of the roughness will be discussed in a further section.

2.3.2 Wave and wave-current boundary layers

Most of the wave bottom boundary layer models use the concept of a wave friction factor f_w to describe the bottom shear stress through a quadratic friction law:

$$\tau_b = \frac{1}{2}\rho_f f_w u_b^2 \tag{2.3.5}$$

where u_b is the wave orbital velocity. Similarly, the bed shear stress in a wave-current boundary layer is commonly defined using a wave-current friction factor

$$\tau_{b,x} = \frac{1}{2}\rho f_{cw}\left(u^2+v^2\right)^{1/2} u \tag{2.3.6}$$

$$\tau_{b,y} = \frac{1}{2}\rho f_{cw}\left(u^2+v^2\right)^{1/2} v \tag{2.3.7}$$

where u and v are the components of the horizontal velocity $\left(\left(u^2+v^2\right)^{1/2} = |u_{cw}|\right)$. The bottom boundary layer models then mainly differ through the determination of the friction factor (f_w for pure waves or f_{cw} for wave-current flows). Many different expressions for the friction factor are available in the literature both explicit (e.g., *Swart*, 1974; *Kamphuis*, 1975; *Fredsøe*, 1984; *Justesen*, 1988; *Nielsen*, 1992; *Madsen*, 1994) and implicit (*Jonsson*, 1966; *Grant and Madsen*, 1979, 1986).

The wave friction factor usually depends on the wave Reynolds number $A^2\omega/\nu$ with A the wave orbital amplitude and on the relative bed roughness (*Jonsson*, 1966). One of the most used explicit wave-friction formula is the *Swart* (1974) formula, which is an approximation of the implicit semi-empirical formula of *Jonsson* (1966) and gives the wave friction factor as a function of the relative bed roughness only:

$$f_w = \exp\left[5.213\left(\frac{A}{K_s}\right)^{-0.194} - 5.977\right] \tag{2.3.8}$$

Madsen (1994) presented an application of a spectral wave-current boundary layer model and derived friction factor formulas of the same type as *Swart* (1974) for wave-current combinations

$$f_{cw} = C_\mu \exp\left[7.02 X_w^{-0.078} - 8.82\right] \text{ for } 0.2 < X_w < 10^2 \tag{2.3.9}$$

$$f_{cw} = C_\mu \exp\left[5.61 X_w^{-0.109} - 7.30\right] \text{ for } 10^2 < X_w < 10^4 \tag{2.3.10}$$

where

$$X_w = \frac{C_\mu u_b}{K_s \omega} \tag{2.3.11}$$

and

$$C_\mu = (1 + 2\mu|\cos\phi| + \mu_2)^{1/2} \tag{2.3.12}$$

with u_b the representative wave orbital velocity amplitude, μ the ratio of current and wave bed shear stress (usually much smaller than one), and ϕ the angle between the current and the wave propagation direction.

Another approach to the parameterization of the wave-current interactions has been provided by *Mellor* (2002) and is based on the approximating the results of an intra-wave model. A two-equation turbulence model (*Mellor and Yamada*, 1982) is used in combination with the law of the wall as an intrawave boundary layer model and is tested and validated using the laboratory results of *Jensen et al.* (1989). The wave effects on the mean flow are then approximated through an increase of the turbulent kinetic energy production:

$$\mathcal{P} = \mathcal{P}_s + \mathcal{P}_A \tag{2.3.13}$$

where $\mathcal{P}_s$ is the "traditional" (mean) shear production and $\mathcal{P}_A$ is an apparent production due to waves. This extra production is obtained from the numerical results of the intrawave model with a modified law of the wall. It is found not to depend on $u_\star/u_b$ and is represented by the following form:

$$\left(\frac{\mathcal{P}_A}{\omega u_b^2}\right)^{1/3} = F_\phi(\phi) F_z \left(\frac{z\omega}{u_b}, \frac{K_s\omega}{u_b}\right) \tag{2.3.14}$$

where ϕ is the angle between the oscillation amplitude vector and the mean current vector. F_z is obtained from the shear production from pure oscillatory cases, and the functional relationship is found by a curve fit to numerical results for different roughness and mean shear stress values

$$\begin{aligned} F_z &= -0.0488 + 0.02917Lz + 0.01703Lz^2 \\ &\quad + \left[1.125\,(LK_s + 5) + 0.125\,(LK_s + 5)^4\right] \\ &\quad \times \left(-0.0102 - 0.00253Lz + 0.00273Lz^2\right) \end{aligned} \tag{2.3.15}$$

where

$$\begin{aligned} Lz &= \ln\left(\frac{z\omega}{u_b}\right) \\ LK_s &= \log_{10}\left(\frac{K_s\omega}{u_b}\right) \end{aligned}$$

A best fit between calculations resolving the oscillations and parameterized calculations for a given angle ϕ provide values for F_ϕ and a curve fit leads to

$$F_\phi = 1.22 + 0.22\cos 2\phi \tag{2.3.16}$$

2.3.3 Bed roughness

Bed roughnesses are commonly associated with the grain roughness, bed load sediment transport and with the presence of ripples. Roughness lengths are generally considered to be additive and the total bed roughness has traditionally been the sum of the three roughnesses just introduced (e.g., *Grant and Madsen*, 1982; *Xu and Wright*, 1995; *Li and Amos*, 2001). Still *Harris and Wiberg* (2001) argued that the total roughness should only be the larger of the bed load and bed form roughnesses. The grain roughness is taken to be proportional to the sediment grain diameter, and $K_{sg} = 2.5D_{50}$ is commonly used. The bed load roughness is related to the value of the excess Shields parameter and several expressions have been introduced (e.g., *Grant and Madsen*, 1982; *Wiberg and Rubin*, 1989; *Xu and Wright*, 1995; *Li and Amos*, 2001). Interestingly, such a dependence on the Shields parameter also means that methods for the determination of the bed shear stress will be implicit, which in turn justifies the use of iterative techniques. The bed form roughness is estimated as

$$K_{sr} = a_r \frac{\eta_r^2}{\lambda_r} \tag{2.3.17}$$

where a_r is a constant. *Grant and Madsen* (1982) proposed $a_r = 27.7$, *Nielsen* (1992) $a_r = 8$. The geometric characteristics of the bed forms, η_r and λ_r, are calculated using "ripple predictors" that aim to prescribe ripple length and height empirically.

Nielsen (1981) argued that the ripple length depends on the mobility number $\Psi = (a\omega)^2/(s-1)gd$, while the ripple steepness depends on the Shields parameter. He also introduced different formulas for laboratory and field data. Under irregular field waves, the ripple length and height are given by:

$$\frac{\lambda_r}{a} = \exp\left(\frac{693 - 0.37\ln^7\Psi}{1000 + 0.75\ln^8\Psi}\right) \quad (2.3.18)$$

$$\frac{\eta_r}{\lambda_r} = 0.342 - 0.34\theta^{1/4} \quad (2.3.19)$$

$$\frac{\eta_r}{a} = 21\Psi^{-1.85} \quad (2.3.20)$$

Wiberg and Harris (1994) differentiated orbital, suborbital and anorbital ripples based on increasing values of the wave orbital diameter. For anorbital ripples, the ripple length is approximated by a constant $\lambda_r = 535D$ and the ripple steepness is given by

$$\frac{\eta_r}{\lambda_r} = \exp\left[-0.095\left(\ln\frac{d_o}{\eta_r}\right)^2 + 0.442\ln\frac{d_o}{\eta_r} - 2.28\right]. \quad (2.3.21)$$

where d_o is the diameter of the orbital motions.

However, both roughness predictors presented were developed for wave dominated cases and might not be appropriate in all wave-current situations. Based on observations on the Scotian shelf *Li and Amos* (1998) distinguish between five categories: no transport, ripples in weak transport, ripple in equilibrium range, ripples in break-off range and plane bed (sheet flow). These five regimes are defined based on the values of the skin-friction shear velocity ($u_{\star s}$ found by only considering the grain roughness) and of the bed load shear velocity ($u_{\star b}$ found by considering the grain roughness and the bed load roughness). In the no transport regime, preexisting ripples will increase the bed shear stress at the crests, which determines sediment transport, and a ripple-enhanced shear velocity is calculated following:

$$u_{\star e} = \frac{u_{\star s}}{1 - \pi\eta_r/\lambda_r} \quad (2.3.22)$$

where the ripple dimensions are that of the preexisting ripples and $u_{\star s}$ is the skin-friction velocity. If the enhanced shear stress is still less than the critical shear stress for motion, the ripple geometry remains unchanged. In the case for which $u_{\star s} < u_{\star cr} < u_{\star e}$, weak localized transport occurs and the ripple geometry is then predicted by

$$\frac{\eta_r}{D} = 19.6\frac{u_{\star s}}{u_{\star cr}} + 20.9 \quad (2.3.23)$$

$$\frac{\eta_r}{\lambda_r} = 0.12 \quad (2.3.24)$$

The equilibrium regime happens for $u_{\star s} > u_{\star cr}$ and $u_{\star b} < u_{\star bf}$ where $u_{\star b}$ is the bed load shear velocity and $u_{\star bf}$ the break-off criterion such that $u_{\star bf} = 1.34 S_\star^{0.3} u_{\star cr}$ with $S_\star^{0.3} = (D/4\nu)[(s-1)dD]^{1/2}$ (*Grant and Madsen*, 1982). The ripples are then predicted using

$$\frac{\eta_r}{D} = 22.15\frac{u_{\star b}}{u_{\star cr}} + 6.38 \quad (2.3.25)$$

$$\frac{\eta_r}{\lambda_r} = 0.12 \quad (2.3.26)$$

for current dominated ripples and using

$$\frac{\eta_r}{D} = 27.14\frac{u_{\star b}}{u_{\star cr}} + 16.36 \quad (2.3.27)$$

$$\frac{\eta_r}{\lambda_r} = 0.15 \quad (2.3.28)$$

for wave dominated ripples. Break off ripples are described by

$$\lambda_r = 535D \tag{2.3.29}$$

$$\frac{\eta_r}{\lambda_r} = 0.15\frac{u_{\star p} - u_{\star b}}{u_{\star p} - u_{\star bf}} \tag{2.3.30}$$

where $u_{\star p}$ is the threshold for plane bed.

2.4 Cohesive sediments

For cohesive sediments, the suspended sediment concentration profile is determined by a combination of processes more complicated than those accounted for in the typical advection-diffusion equation (equation 2.1.2):

- flocculation, which is the formation and break-up of flocs of cohesive sediment and is a key process in differentiating cohesive and non-cohesive sediments.
- consolidation and liquefaction, processes by which the bed is either strengthened or weakened.

Settling, deposition, the interaction between particles and flow turbulence, erosion and entrainment are all processes that are not specific to cohesive sediments, even though they will usually be modelled differently for cohesive and non-cohesive sediments. So far, most models that account for the cohesive nature of sediment by including processes such as flocculation and consolidation are implemented in one-dimension (e.g., *Winterwerp*, 2002; *Neumeier et al.*, 2008; *Sanford*, 2008). In most multidimensional models, cohesive sediments are modelled in simpler ways that do not fully account for all cohesive processes: in particular, only settling, deposition and erosion are considered while both flocculation and consolidation are neglected.

2.4.1 Flocculation

In flocculation models, mud flocs are commonly treated as self-similar fractal entities (*Kranenburg*, 1994; *Winterwerp and van Kesteren*, 2004) and fractal theory is employed to derive equations for the floc's properties (size, settling velocity, density). The density of the flocs is given by (*Kranenburg*, 1994)

$$\Delta\rho_a = \rho_a - \rho_w = (\rho_s - \rho_w)\left(\frac{D_p}{D_a}\right)^{3-n_a} \tag{2.4.1}$$

where ρ_a, ρ_s, and ρ_w are the floc, sediment particles, water densities, and D_a and D_p are the floc and sediment particle diameters. n_a is the fractal dimension and typically in the range $1.7 < n_a < 2.2$ with a mean value of $n_a = 2$ (*Kranenburg*, 1994). The volumetric concentration of the flocs c_a is then such that

$$c_a = \left(\frac{\rho_s - \rho_w}{\rho_a - \rho_w}\right)c = c\left(\frac{D_f}{D}\right)^{3-n_a} = f_s N D_f^3 \tag{2.4.2}$$

where f_s is a shape factor (equal to $pi/6$ for spherical particles) and N the number concentration of flocs (number of flocs per unit volume). More recently, *Winterwerp* (2002) presented a model developed in three dimensions but only implemented in one by striping all horizontal gradients. This turbulence-induced flocculation model is based on a mass balance and on considering the flocs as fractal entities. *Winterwerp* (2002) then derives balance equations for both the floc size D_a and for the number of mud flocs in the turbulent fluid, both of which can be viewed as advection diffusion equations with an extra non-linear term due to the aggregation and floc break-up processes (*Winterwerp and van Kesteren*, 2004).

However, the main issue for multidimensional models is really how to parameterize the effect of flocculation on the particle size, floc density, floc settling velocity without resolving the flocculation processes per se. For example, *Neumeier et al.* (2008) use a set of equations directly relating the floc length-scale, effective diameter and median settling velocity to the suspended sediment concentration following *Whitehouse et al.* (2000). This issue is similar to that encountered in bedload modelling and the importance of empirical studies should be relatively evident. These usually seek to relate the floc's properties to some parameterization of the turbulent cohesive suspension and common quantities used are the suspended sediment concentration c and the Kolmogorov time scale τ_η (e.g., *Lick et al.*, 1993; *Dyere and Manning*, 1999; *Manning and Dyer*, 1999). The derived empirical expressions usually relate the floc diameter D_a to both c and τ_η, while the floc settling velocity W_s is a function of both its size and density. *Lick et al.* (1993) derived

$$D_a = k\left(\frac{c}{\tau_\eta}\right)^{-q} \tag{2.4.3}$$

where k and q are dimensional empirical constants and then related the floc settling velocity to the floc diameter via a power function. *van Leussen* (1994) introduced a formula modifying the settling velocity in still water depending on the turbulence:

$$W_s = W_{s0}\frac{1 + a\left(\tau_\eta\right)^{-1}}{1 + b\left(\tau_\eta\right)^{-2}} \tag{2.4.4}$$

where W_{s0} is a concentration dependent settling velocity, a and b are empirical dimensional constants. *Manning and Dyer* (1999) and *Dyere and Manning* (1999) found an expression of the type

$$D_a = k_w W_s + k_\eta\left(\tau_\eta\right)^{-1} + k_c c + k_0 \tag{2.4.5}$$

for the floc diameter where k_w, k_η, k_c and k_0 are dimensional constants, which can also be rewritten for the settling velocity. A noticeable drawback from these empirical studies is that the relationships derived are not expressed in non-dimensional form.

2.4.2 Consolidation

Self-weight consolidation is the consolidation of cohesive sediment deposits under the influence of their own weight. When flocs settle and accumulate on the bed, they are squeezed by the flocs settling on top of them. Pore water is then driven out of the intra-floc and inter-floc spaces. This process can result in large vertical deformations of the bed.

Consolidation is commonly described by the Gibson equation (*Gibson et al.*, 1967), which is a one-dimensional equation for the void ratio e and can be expressed in Eulerian coordinates or, in the classic form, in a material coordinate system:

$$\frac{\partial e}{\partial t} + (s-1)\frac{\mathrm{d}}{\mathrm{d}e}\left(\frac{k(e)}{1+e}\right) + \frac{\partial}{\partial \zeta}\left(\frac{k(e)}{g\rho_w(1+e)}\frac{\mathrm{d}\sigma_{zz}(e)}{\mathrm{d}e}\frac{\partial e}{\partial \zeta}\right) \tag{2.4.6}$$

where $k(e)$ is the permeability of the soil and $\sigma_{zz}(e)$ is the vertical effective stress. ζ is a vertical material coordinate that represents the volume of solids. In Eulerian coordinates the Gibson equation can be rewritten in terms of the volumetric concentration

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial z}\left[(s-1)kc^2 + \frac{kc}{g\rho_w}\frac{\mathrm{d}\sigma_{zz}}{\mathrm{d}c}\frac{\partial c}{\partial z}\right] \tag{2.4.7}$$

which can be seen as an advection-diffusion equation of some form. While empirical function for the effective stress and the permeability are commonly used, *Kranenburg* (1994) proposed to treat the bed as a self-similar structure and obtained relationships for both of them.

Sanford (2008) assumes that the effects of consolidation can be approximated as a first-order relaxation to an equilibrium state. Both the critical stress for erosion τ_c and the solids volume concentration ϕ_s are then given by:

$$\frac{\partial \tau_c}{\partial t} = r_c \left(\tau_{ceq} - \tau_c\right) H \left(\tau_{ceq} - \tau_c\right) - r_s \left(\tau_{ceq} - \tau_c\right) H \left(\tau_c - \tau_{ceq}\right) \tag{2.4.8}$$

$$\frac{\partial \phi_s}{\partial t} = r_c \left(\phi_{seq} - \phi_s\right) H \left(\phi_{seq} - \phi_s\right) - r_s \left(\phi_{seq} - \phi_s\right) H \left(\phi_s - \phi_{seq}\right) \tag{2.4.9}$$

where H is the Heavyside step function, r_c is the first order consolidation rate and r_s is the first order swelling rate, which is much smaller than r_c. Both r_c and r_s are determined empirically.

2.4.3 Viscosity

In addition to specific erosion, deposition, and settling velocity relationships, models for cohesive particle can include a dependence of the mixture viscosity on the sediment concentration (e.g., *Winterwerp*, 2002)

$$\mu_{eff} = \mu(1 + 2.5c_f) \tag{2.4.10}$$

Intercomparison of existing models

A dozen existing multidimensional models, both two dimensional and three-dimensional, are hereafter briefly described, then summarized and compared in tables 3.1, 3.2, 3.3 and 3.4. The eight three dimensional models reviewed are:

- the NOPP community sediment transport model incorporated in ROMS (*Warner et al.*, 2005; *Blaas et al.*, 2007; *Warner et al.*, 2008)
- the sediment transport module included in DELFT3D (*Lesser et al.*, 2000; *van Rijn and Walstra*, 2003; *Lesser et al.*, 2004)
- ECOM-SED, which is a model commercialized by Hydroqual (*Hydroqual*, 2002)
- the sediment transport module included in the MIKE models commercialized by DHI (*Zyserman and Ronberg*, 2001; *Lumborg and Windelin*, 2003; *Lumborg*, 2005)
- a quasi three-dimensional sediment transport model introduced by *Rakha* (1998)
- the coastal sediment transport model developed by *Wai et al.* (2004)
- the 3D open channel flow hydrodynamic and sediment transport model CH3D, used by the US Army Corps of Engineers and described in *Gessler et al.* (1999)
- the 3D model for sediment transport in open channels developed at the University of Karlsruhe, Germany (*Wu et al.*, 2000; *Fang and Rodi*, 2003)

Three two-dimensional models are also included in the comparison

- STORMSED (*Cookman and Flemings*, 2001)
- the continental shelves model of *Harris and Wiberg* (2001)
- the model described in *Zhang et al.* (1999)

The SEDTRANS model (*Li and Amos*, 1995, 2001; *Neumeier et al.*, 2008), which is only a one dimensional (vertical) model but could be seen as a 1DV sub-module in a three-dimensional coastal model and has recently been linked to a 3D hydrodynamic model (*Neumeier et al.*, 2008), is also included.

3.1 NOPP Community Sediment Transport Model (ROMS)

This three-dimensional model implements algorithms for an unlimited number of user-defined sediment classes and for the evolution of the bed morphology. It is incorporated in a coastal-circulation model with a two-way coupling between a wave model and the sediment transport module. Each sediment class is described by a grain diameter, grain density, settling velocity, critical stress for erosion and erodibility constant.

3.1.1 Suspended sediment transport

The model solves an advection and diffusion equation (equation 2.1.2) in which horizontal diffusion is neglected. A source or sink term is added to the computations for the bottom computational cells and represents deposition and erosion (see table 3.1 for the exact equations). The boundary conditions specify a zero vertical diffusive flux both at the top and bottom boundary. As mentioned previously, the settling velocity is a user-defined constant. The effect of the suspended sediment on the mixture density is considered. The sediment diffusivity is calculated in the same way as the diffusivity of other tracers and following one of the five turbulence closure models implemented in ROMS (see *Warner et al.* (2008) for more details).

3.1.2 Morphology and near bed treatment

The sediment bed is represented by a user-defined constant number of layers, each of which has a thickness, sediment-class distribution, porosity and age. The method used to track the location of the sediment bed has already been briefly described in section 2.2.2:

1. First, an active layer thickness is calculated.
2. If the top layer of sediment is less thick than the active layer, sediment is entrained from deeper layers until the top layer has the same thickness as the active layer.
3. Sediment is then transported, eroded and deposited. Transport and erosion are limited by the available sediment mass, which is the mass contained in the active layer.
4. The sea floor elevation is updated accordingly to the convergence or divergence of the near-bed sediment fluxes, to which a morphological factor is applied.
5. If deposition results in a top layer thicker than a user defined value, a new layer is created.
6. Finally, the active layer thickness in recalculated and the bed layers adjusted accordingly.

The bed load transport rate can be calculated following the *Meyer-Peter and Mueller* (1948) formula for unidirectional flow or following the *Soulsby and Damgaard* (2005) formulation for combined waves and currents, both of which are modified to account for bed slope effects.

3.1.3 Bottom boundary layer modelling

Simple drag coefficient expressions (linear, quadratic, logarithmic profile) or formulations representing waves and currents effects can be used. Three different methods are implemented to calculate the bed shear stress in wave-current boundary layers:

- the wave-current boundary layer algorithm and the bed roughness of *Styles and Glenn* (2000, 2002).
- the wave-current boundary layer model of *Soulsby* (1995) and the bed roughness predictors of *Grant and Madsen* (1982); *Nielsen* (1986) and *Li and Amos* (2001).
- the wave-current bottom boundary layer model of *Madsen* (1994) or *Styles and Glenn* (2000) and the bed roughness predictor of *Wiberg and Harris* (1994).

3.1.4 Cohesive sediment modelling

Cohesive sediment transport is not modeled in ROMS, even though the erosion is specified using a flux equation commonly used to describe erosion of cohesive sediment.

3.2 DELFT3D

The sediment module in DELFT3D implements algorithms for up to five different classes, which have to be specified as either "mud" or "sand".

3.2.1 Suspended sediment transport

The suspended sediment concentration is obtained by solving equation 2.1.2. Again a source/sink term that represents sediment exchange between the water column and the bed is added. This term is specified differently depending on the type of sediment ("mud" or "sand"). For sands, a reference height is calculated, and the sediment source/sink term is located in the first cell entirely above the reference elevation (reference cell). The sediment concentration at the reference height is given by a formula adapted from that of *van Rijn* (1984c). The source/sink term is then calculated assuming a linear gradient between the reference concentration and the concentration in the reference cell (see equations in table 3.1). For muds, the source/sink term is always added to the bottom grids and is computed using a linear equation for erosion (equation 2.1.10 with $n = 1$) and the Krone deposition formula (*Krone*, 1962). The boundary conditions state that the diffusive sediment flux is zero both at the top and bottom boundaries and that the advective flux is zero at the top boundary. The settling velocity is prescribed as a function of the fluid and the sediment grain properties, of the sediment concentration and of the salinity. The sediment diffusivity is related to the eddy viscosity using the β factor introduced by *van Rijn* (1984c) modified to account for wave and currents:

$$\beta_{eff} = 1 + (\beta - 1)\,\frac{\tau_c}{\tau_w + \tau_c} \tag{3.2.1}$$

The density also accounts for the presence of sediment in the flow.

3.2.2 Morphology and near bed treatment

Morphological changes are obtained by

- Calculating the change of bottom sediment mass from the near-bed fluxes and a correction for the suspended load transport under the reference level
- Translate such change of mass into a thickness change using the dry bed density. A morphological factor allows to accelerate morphological changes.
- Update the bed elevation

The bed load transport rate is calculated following expressions that are based on the *van Rijn* (1984a) formula and the effects of the bed slope are included. A particularity of the approach used in DELFT3D is that, contrary to many other bed load models such as *Ribberink* (1998) and *Soulsby and Damgaard* (2005) for example, the bed load transport rate is related directly to the flow velocities instead of the bed shear stress.

3.2.3 Bottom boundary layer modelling

The bed shear stress is given by:

$$\tau_b = \mu_c \tau_{b,c} + \mu_w \tau_{b,w} \tag{3.2.2}$$

where the bed shear stress due to waves is calculated using the *Swart* (1974) formula for the wave friction factor and μ_c and μ_w are efficiency factors for the current and the waves and are calculated from the water depth, the roughness and the wave parameters. The current efficiency factor is the ratio of the grain related friction factor and the total current-relate friction factor

$$\mu_c = \frac{0.24\left[\ln_{10}\left(\frac{12h}{3D_{90}}\right)\right]^{-2}}{0.24\left[\ln_{10}\left(\frac{12h}{k_c}\right)\right]^{-2}} \tag{3.2.3}$$

and the wave efficiency factor is

$$\mu_w = \max\left(0.063, \frac{1}{8}\left(1.5 - \frac{H_s}{h}\right)^2\right) \tag{3.2.4}$$

3.2.4 Cohesive sediment modelling

Even though different formulae are used for sands and muds at several instances, cohesive processes such as flocculation, consolidation and fluidization are not modelled.

3.3 ECOM-SED

Although ECOM-SED aims to model sediment transport for both cohesive and non-cohesive sediments, only two size classes (one of each) are allowed.

3.3.1 Suspended sediment transport

The suspended sediment concentration is calculated by solving the advection diffusion equation (equation 2.1.2). The top boundary condition specifies that the diffusive sediment flux is zero while at the bottom the diffusive sediment is taken to consist of erosion and deposition. Different formulae are used for the erosion and the deposition depending whether the sediment considered is cohesive or non-cohesive. For cohesive sediment, the erosion is modelled as a power of the excess bed shear stress (similar to equation 2.1.10) and the deposition is modelled following the formula of *Krone* (1962). For cohesive sediments, the erosion is modelled following a reference concentration approach with the *van Rijn* (1984c) formula to which a coefficient representing bed armoring is applied, while deposition is due to the self weight of the grains. The settling velocity is taken to be a function of the concentration for cohesive sediments and only a function of the particle parameters for non-cohesive sediments. The sediment diffusivities are calculated in the same way as for other tracers, that is: 1) the horizontal diffusivities are held constants and 2) the vertical diffusivities are calculated following a Mellor-Yamada turbulence closure.

3.3.2 Morphology and near bed treatment

The bed is segmented into seven layers. The layers' thickness is calculated by considering sediment mass conservation, while erosion and deposition only occur for the topmost layer. Bed load is not considered.

3.3.3 Bottom boundary layer modelling

The bottom shear stress is calculated using a logarithmic profile approach for currents and using the *Grant and Madsen* (1979) wave-current model otherwise.

3.3.4 Cohesive sediment modelling

Different formulae for cohesive and non-cohesive sediment are implemented for erosion, deposition and settling velocity. However, processes specific to cohesive sediments such as flocculation and consolidation are not considered.

3.4 SEDTRANS

SEDTRANS has been developed more than 20 years ago and the latest versions are SEDTRANS92 (*Li and Amos*, 1995), SEDTRANS96 (*Li and Amos*, 2001), and SEDTRANS05 (*Neumeier et al.*, 2008). SEDTRANS05 differs from SEDTRANS96 by

- including a new cohesive sediment algorithm that provides variations of sediment properties with depth, represents the suspended sediment as a spectrum of different settling velocities, includes the flocculation process and provides simulations of multiple erosion-deposition cycles
- implementing the *van Rijn* (1993) method for non-cohesive sediments
- calculating the still water settling velocity following *Soulsby* (1997)
- finding the density and viscosity of water from temperature and salinity data

One of the particularities of SEDTRANS is the presence of a bed form predictor that will provide the type of bed form present as a function of the bed shear stress and uses the same thresholds as the ripple geometry predictor. For current ripples, if $u_{\star cs} < u_{\star cr}$ there is no transport and the input ripple geometry will be used, if $u_{\star cs} > u_{\star p}$ current induced upper-plane bed occurs, in between active current ripples are present. For wave ripples, if $u_{\star ws} < u_{\star cr}$ there is no transport and the input ripple geometry will be used, if $u_{\star ws} > u_{\star p}$ wave induced upper-plane bed occurs, in between active wave ripples are present. For combined flows,

- if $u_{\star e} < u_{\star cr}$ there is no transport and the input ripple geometry will be used
- if $u_{\star e} > u_{\star cr}$ and $u_{\star s} < u_{\star cr}$ weak transport ripples occur
- if $u_{\star s} > u_{\star cr}$ and $u_{\star b} < u_{\star bf}$ equilibrium ripples are present and can be further divided in current-dominated rippled for $u_{\star ws}/u_{\star cs} < 0.75$, wave-dominated ripples for $u_{\star ws}/u_{\star cs} < 1.25$ and combined wave current ripples in between
- if $u_{\star b} > u_{\star bf}$, break-off ripples (wave dominated) are present
- if $u_{\star s} > u_{\star p}$ upper-plane bed occurs under combined flows.

3.4.1 Suspended sediment transport

Contrary to most other models, the suspended sediment concentration is prescribed by a Rouse profile and the boundary condition at the bottom sets the concentration to be the reference concentration as given by the *Smith and McLean* (1977) formula. The settling velocity was considered to be a function of only the sediment and fluid properties in SEDTRANS96, but more complex description has been incorporated in SEDTRANS05 (*Neumeier et al.*, 2008). The effect of the sediment on the mixture density is always neglected.

3.4.2 Morphology and near bed treatment

SEDTRANS does not calculate the morphology evolutions and bed load transport is not specifically computed either. The reason is that SEDTRANS is a 1DV model primarily aimed at cohesive sediment transport. Still, some formulae used to obtain the sediment transport do calculate the bed load, in particular the *van Rijn* (1993) method has been included in SEDTRANS05.

3.4.3 Bottom boundary layer modelling

A quadratic drag coefficient law is used to determine the bed shear stress. Different coefficients and roughnesses are used depending on the flow situation. For waves only, the wave friction coefficient of *Jonsson* (1966) as modified by *Nielsen* (1979) is used while, for waves and currents, the coefficient is found following *Grant and Madsen* (1986). The roughness predictor gives three components for the bed roughness:

$$K_s = 2.5D + 27.7\frac{\eta^2}{\lambda} + 180(2.9D(\theta - \theta_c)^{0.75}) \tag{3.4.1}$$

3.4.4 Cohesive sediment modelling

In SEDTRANS96 (*Li and Amos*, 2001), cohesive sediments are modeled by treating the transport in three states (depositional, erosional, stable). The depositional state occurs when the bed shear stress is less than the critical shear stress for deposition and the deposition rate is then given by the *Krone* (1962) formula. The stable state happens when the bed shear stress is more than the critical value for deposition but less than the critical value for erosion, and there is no change. The erosional state corresponds to bed shear stress higher than the critical value for erosion and the erosion rate is then calculated following an exponential relationship (equation 2.1.11 with $\beta = 0.5$)

$$E = E_0 \exp\left[\alpha\left(\tau_b - \tau_{ce}(z)\right)^{1/2}\right] \tag{3.4.2}$$

with

$$\tau_{ce}(z) = \tau_{ce}(0) + A\left(\rho_b - \rho\right) g z_s \tan\phi_i \tag{3.4.3}$$

where A is an empirical coefficient, ρ_b is the bulk sediment density, ϕ_i is the internal friction angle of cohesive sediment.

In SEDTRANS05 (*Neumeier et al.*, 2008), the cohesive sediment algorithm calculates, in order, for each time step:

1. the effective bed shear stress taking into account the drag reduction due to high sediment concentration and the drag enhancement due to sediment-transmitted stress

2. the mass of eroded sediment and erosion of the bed

3. the deposition rate for each sediment class including the flocculation process (flocculation hindered settling is modelled following *Whitehouse et al.* (2000)) and the corresponding mass removed from the suspended load and added to the bed

4. the mass of eroded sediment is added to the suspended load

5. the consolidation of the bed

3.5 Summary of different models.

Several characteristics of the different models reviewed are summarized in tables 3.1 for the method used to obtain the suspended sediment concentration and the required boundary conditions, in table 3.2 for some sediment and flow parameters, in table 3.3 for the morphodynamic and bed load modelling and in table 3.4 for the bottom boundary layer model.

Table 3.1: Suspended sediment concentration comparison table.

	Method	Top BC	Bottom BC
ROMS	Advection and diffusion equation without horizontal diffusion and with an added source/sink term which is the combination of downward settling and erosion and only for bottom computational cell: $$S_c = -\frac{cW_s}{s} + E_s$$ with $$E_s = E_0(1-\phi)\frac{\tau_b - \tau_{ce}}{\tau_{ce}}$$	Zero vertical diffusive flux.	Zero vertical diffusive flux.
DELFT3D	Advection and diffusion equation with added source/sink term. Source and sink calculated assuming a linear concentration gradient between reference concentration (*van Rijn*, 1984c) and concentration of reference cell. $$Source = c_{ref}(\frac{K_v}{\Delta z})$$ $$Sink = c_{cell}(\frac{K_v}{\Delta z} + W_s)$$	Zero diffusive flux.	No advective flux, no diffusive flux.
ECOM-SED	Advection and diffusion equation	Zero-diffusive flux	Diffusive flux specified as erosion and deposition.

Table 3.1 continued.

	Method	Top BC	Bottom BC
MIKE	Depth-averaged advection-diffusion equation with added source/sink terms. Source/Sink term accounts for deposition and erosion.		
SEDTRANS	Rouse profile.	Not needed.	Reference concentration specified after *Smith and McLean* (1977).
Rakha (1998)	Vertical diffusion equation with settling term. Lagrangian drift added to compensate neglected convective terms.	Zero sediment flux.	Reference concentration after *Zyserman and Fredsoe* (1994).
Wai et al. (2004)	Advection and diffusion equation.	No net flux of sediment.	Net flux q_s expressed in terms of deposition and erosion. $$q_s = \alpha_s W_s (c - c_{ref}),$$ and the reference concentration taken following *van Rijn* (1984c).
CH3D (*Gessler et al.*, 1999)	Advection and diffusion equation.		
Wu et al. (2000)	Advection and diffusion equation.	Zero net vertical flux.	Sediment flux specified with reference concentration approach, which follows *van Rijn* (1984c).
STORMSED	Rouse profile.	Not needed	Reference concentration specified after *Smith and McLean* (1977).
Harris and Wiberg (2001)	Advection and diffusion equation with vertical convective term neglected.	No net sediment flux.	Flux condition obtained from reference concentration formulation of *Smith and McLean* (1977).
Zhang et al. (1999)	Advection and diffusion equation without vertical advection neglected.	No net sediment flux.	Reference concentration from *Smith and McLean* (1977) for non-cohesive sediments, flux condition for cohesive sediments.

Table 3.2: Sediment transport parameters comparison table.

	W_s	K_h	K_v	Fluid density
ROMS	Constant	Not needed.	Same as other tracers, related to eddy viscosity depending on turbulence closure used.	$\rho = \rho_w + \sum_{m=1}^{N_{sed}} \frac{c_m}{\rho_{s,m}}(\rho_{s,m} - \rho_w)$
DELFT3D	Function of salinity, and of concentration, the latter following *Richardson and Zaki* (1954)		Related to vertical eddy viscosity through: $K_v = \frac{\nu}{\sigma_{c0}} + \frac{\beta}{\sigma_c(Ri)}\nu_v$	Same as ROMS
ECOM-SED	Function of concentration for cohesive sediment only.	Constant.	Mellor-Yamada turbulence closure	Function of temperature and salinity only.
MIKE	Function of concentration	User defined constant	Not needed.	Constant
SEDTRANS	Function of sediment and fluid properties only.	Not needed	Not needed	Constant

Table 3.2 continued.

	W_s	K_h	K_v	Fluid density
Rakha (1998)		Not needed	Same as eddy viscosity	Constant
Wai et al. (2004)	Calculated following *van Rijn* (1993)	Constant	Specified analytical expression: parabolic profile.	Constant
CH3D (*Gessler et al.*, 1999)			Following *van Rijn* (1984c): $K_v = \beta\Phi\nu_v$ with β function of $W_s/u_\star$ and Φ function of concentration.	Accounts for local concentration, temperature and salinity.
Wu et al. (2000)		Related to eddy viscosity through Schmidt number	Related to eddy viscosity through Schmidt number	
STORMSED		Not needed	Not needed	Constant
Harris and Wiberg (2001)	Constant value	Zero	Same as eddy viscosity	
Zhang et al. (1999)	Constant value	Same as eddy viscosity	Same as eddy viscosity	

Table 3.3: Morphology and bed load comparison table.

	Morphology	Bed Load
ROMS	Layered structure of bed: • Constant number of layers. • Thickness of "active layer" (different than top layer) following *Harris and Wiberg* (1997). • Top layer thickness modified by mass conservation accounting for erosion, deposition and bed load. • Erosion and bed load limited by sediment availability from active layer.	Choice between *Meyer-Peter and Mueller* (1948) and *Soulsby and Damgaard* (2005). Correction for bed slope.
DELFT3D	• Change of bottom sediment mass calculated from erosion, deposition, bed load, and correction for suspended load transported under reference level. • Change of mass then translated in bed elevation.	Bed load calculated using van Rijn formulae. Correction added for bed slope.
ECOM-SED	Layered structure of the bed, with seven layers. Conservation of mass with erosion and deposition only for the top layer.	No.
MIKE	Not in *Lumborg and Windelin* (2003)	Not in *Lumborg and Windelin* (2003)
SEDTRANS	NO	Not specifically, but some bed load formulae used to calculate sediment transport.

Table 3.3 continued.

	Morphology	Bed Load
Rakha (1998)	NO	NO
Wai et al. (2004)	Evolution of seabed level: $$\Delta z = \frac{\alpha_s W_s}{\rho_s}(c - \beta_c C_{ref})\Delta t$$	NO
CH3D (*Gessler et al.*, 1999)	Layered structure of bed. Thickness of active layer found by solving Exner equation with an active layer floor source term, erosion, deposition and bed load.	Following *van Rijn* (1984a).
Wu et al. (2000)	Bed elevation found by solving mass balance equation, without bed load, only erosion and deposition.	NO
STORMSED	Solution of Exner equation on total water column: suspended sediment transport, bedload transport and a diffusive transport term.	Following *Yalin* (1963).
Harris and Wiberg (2001)	Solution of: $$\frac{\partial \eta}{\partial t} = \frac{1}{C_{bed}}\left(\frac{\partial q_s}{\partial x} + \frac{\partial V_s}{\partial t}\right)$$	NO
Zhang et al. (1999)	Solution of similar equation than *Harris and Wiberg* (2001)	NO

Table 3.4: Bottom boundary layer model comparison table.

	τ_b for current only	τ_b for wave and wave-current BL	Roughness
ROMS	Linear drag coefficient, quadratic drag coefficient, logarithmic profile	Choice between: • *Styles and Glenn* (2000) • *Soulsby* (1995) • *Madsen* (1994)	Choice between: • *Grant and Madsen* (1982) • *Nielsen* (1986) • *Li and Amos* (2001) • *Wiberg and Harris* (1994)
DELFT3D		Wave-current bed shear stress given by: $$\tau_b = \mu_c \tau_{b,c} + \mu_w \tau_{b,w}$$ Wave bed shear stress using wave friction factor from *Swart* (1974) formula. Efficiency factors determined from water depth, roughnesses, and wave parameters.	Roughness predictor (*van Rijn and Walstra*, 2003)
ECOM-SED	Log law	*Grant and Madsen* (1979)	
MIKE	Logarithmic profile.	$$\tau_b = \frac{1}{2}\rho_f f_w \left(U_b^2 + U_\delta^2 + 2U_b U_\delta \cos\beta\right)$$ with *Swart* (1974) formula for the friction factor.	

Table 3.4 continued.

	τ_b for current only	τ_b for wave and wave-current BL	Roughness
SEDTRANS	Quadratic drag coefficient.	Wave friction coefficient. • *Jonsson* (1966) as modified by *Nielsen* (1979) for only waves • *Grant and Madsen* (1986) for wave-current cases	Own roughness predictor: $K_s = 2.5D + 27.7\frac{\eta^2}{\lambda} + 180(2.9D(\theta - \theta_c)^{0.75})$
Rakha (1998)		*Fredsøe* (1984)	$K_s = 2.5D$
Wai et al. (2004)		Choice between: • *Grant and Madsen* (1979) • *Fredsøe* (1984)	
CH3D (*Gessler et al.*, 1999)	Logarithmic profile, but $u_\star$ for bed load determined using Chezy coefficient.	No waves.	
Wu et al. (2000)	Logarithmic profile.	No waves.	
STORMSED	Rough wall log-law.	*Grant and Madsen* (1979)	
Harris and Wiberg (2001)	From velocity profile found by solving momentum equations for unstratified, steady uniform planetary boundary layer in geostrophic balance.	Wave friction factor after *Fredsoe* (1993)	
Zhang et al. (1999)	Rough wall log-law	Friction factor after *Swart* (1974) and then re-use of the log profile	

Sediment transport in POLCOMS

4.1 Existing sediment transport model in POLCOMS

Suspended sediment modelling is already included in POLCOMS and has been discussed in *Holt and James* (1999) and *Souza et al.* (2007) for example. The SPM concentration is calculated by solving an advection diffusion equation similar to equation 2.1.2 in which horizontal diffusion is not included. The advection terms are treated in the same manner as for other scalars. The bottom boundary condition transfers suspended matter between the lowest level of the ocean model and the bed by mutually exclusive deposition and erosion

$$\frac{\partial c}{\partial t} = \frac{E_0}{\Delta z}\left(\frac{\tau_b}{\tau_{ce}} - 1\right)\left(\frac{B}{\sum B}\right),\ \tau_b > \tau_{ce},\ B > 0 \tag{4.1.1}$$

$$\frac{\partial c}{\partial t} = -\frac{W_s c}{\Delta z}\left(1 - \frac{\tau_b}{\tau_{cd}}\right),\ \tau < \tau_{cd} \tag{4.1.2}$$

The bed mass B is such that both erosion and deposition are allowed for positive values, but only deposition is allowed for no bed mass. It is updated following the conservation of sediment mass:

$$\frac{\partial B}{\partial t} = -\frac{\partial c}{\partial t}\Delta z \tag{4.1.3}$$

The near-bed boundary layer is modeled following the approach described in *Souza and Friedrichs* (2005), in which the bed shear stress is given by the sum of the current stress $\tau_{b,c}$ and the wave stress $\tau_{b,w}$:

$$\tau_b = \tau_{b,c} + \tau_{b,w}. \tag{4.1.4}$$

The current stress is calculated using a rough-wall log law approach. The wave stress is given by

$$\tau_{b,w} = \frac{1}{2}\rho f_w U_0 \tag{4.1.5}$$

with U_0 the near-bed wave orbital velocity and f_w the wave friction factor, calculated using the empirical expression from *Grant and Madsen* (1982).

The settling velocity is taken to be a constant and the influence of sediment concentration on the flow density is neglected. The vertical sediment diffusivity is calculated along with the vertical eddy diffusivity based on a Mellor-Yamada 2.5 turbulence scheme. No near-bed and bed treatment (morphological changes and bed load transport) are included, neither is a wave-current boundary layer model to calculate the bed shear stress.

4.2 Sediment transport modelling to be implemented in POLCOMS

Several aspects of a sediment transport and bed morphology model, as discussed in previous chapters, are still lacking in POLCOMS and thus have to be implemented. Within the suspended sediment module, all of the following four issues need to be addressed or refined:

- the settling velocity specification and the influence of the sediment concentration on it.
- the influence of the sediment concentration on the density
- the horizontal diffusivity
- the boundary conditions (erosion and deposition at the bottom).

No near-bed and bed morphology is implemented so far. The layered approach to describe the bed offers several important advantages in that it is better suited to represent beds of mixed sediments, modelled with several sediment classes and also in that it implicitly includes the concept of sediment availability. We will thus implement a bed morphology model that follows that of the Community Sediment Transport Model in ROMS. To apply sediment conservation near the bed in a satisfactory manner, a bed load transport rate model is also required and will have to be able to simulate appropriately the cases of non-collinear asymmetric waves and currents (e.g., *Soulsby and Damgaard*, 2005). Both a wave-current bottom boundary layer model and a bed roughness predictor (e.g., *Wiberg and Harris*, 1994; *Li and Amos*, 2001) also have to be implemented. A bed form predictor such as that of SEDTRANS (*Li and Amos*, 2001) could also be implemented in combination with the bed roughness predictor.

Finally, while cohesive sediment will eventually need to be included in the model, it would be wise to first focus on non-cohesive sediments. Most current "state-of-the-art" models only deal with cohesive sediment by implementing modified relationships for erosion, deposition and settling velocity, which can all be easily added to a non-cohesive model.

4.3 Test cases

All models need to be tested and validated, and several sediment transport problems can be simulated to that end.

- Open channel flows can be simulated and the sediment transport model should then be able to reproduce appropriate suspended sediment concentration profiles (e.g., the Rouse profile).
- *Lesser et al.* (2004) tested their model with the work of *Hjelmfelt and Lenau* (1970) on the development of suspended sediment transport. However, such a situation equivalent to sediment transport downstream an apron with a fixed sediment bed is unrealistic: in natural and laboratory environments scour would occur and the sediment bed would not be fixed.
- The equilibrium slope of a straight flume can also be numerically simulated and compared to the theoretical values given the specified upstream discharge and bed roughness.
- The evolution of the bed can also be followed in a settling basin (domain where particles can only settle, or in other words no ambient flow). Again the numerical simulations can be compared to a theoretical solution.
- the trench migration of *van Rijn* (1987) can be reproduced numerically
- the wave and current flume experiment of *Dekker and Jacobs* (2000) can also be reproduced numerically

However, all these sediment transport problems represent simplified situations and the validation of the model for realistic natural application will require the simulation of cases for which field measurements are available.

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