

Chapter 1

The numerical modelling system

This dissertation has been carried out in the context of a research project whose main objectives were the development of a morphodynamic modelling system and its application to two tidal inlets in Portugal, the overall goal being a better understanding of the physical processes that govern the dynamics of wave-dominated tidal inlets. The modelling platform has been in continuous development over the last six years in an international and collaborative framework, involving researchers from the Oregon Health & Science University, the Virginia Institute of Marine Science, the Portuguese National Civil Engineering Laboratory, the Technical University of Darmstadt and the UMR7266 - LIENSs of La Rochelle. During the last three years, considerable efforts have been dedicated to the coupling between the wave and circulation models and the development of a morphodynamic module compatible with the 2DH version of the wave-current modelling system. These tasks rapidly appeared as important requirements for the success of the research project and the implementation of the spectral boundary forcing in the wave model and the development of the sediment transport and bottom update modules became integral parts of this dissertation. This chapter gives a general presentation of the numerical platform and a detailed description of the different models that compose it.

1.1 Overview

The modelling system that is being developed aims to simulate the hydrodynamics and non-cohesive sediment dynamics in estuaries and coastal zones under the combined effect of tides and waves. This modelling system couples a set of modules to simulate the hydrodynamic circulation, wave propagation, sediment transport and bottom evolution. An overview of the modelling system is represented in Fig. 1.1. The core model of the system is the Semi Eulerian-Lagrangian Finite Element hydrodynamic model [SELFIE, *Zhang and Baptista*, 2008a]. This unstructured-grid model is computationally very efficient, thanks to the Eulerian-Lagrangian Method (ELM) to treat the advection in the momentum equations and the parallelization through Message Passing Interface (MPI). The ELM method and the semi-implicit schemes relax the numerical stability constraints of the model (*e.g.*, CFL) and allow the use of larger time step than allowed by explicit methods. This key advantage motivated the choice for this model as the core of the morphodynamic platform instead of other existent morphodynamic modelling systems. Hence, SELFIE provides an ideal framework for modelling coastal processes along complex coastlines typical of estuaries and coastal inlets. A description of SELFIE is given in Section 1.2.

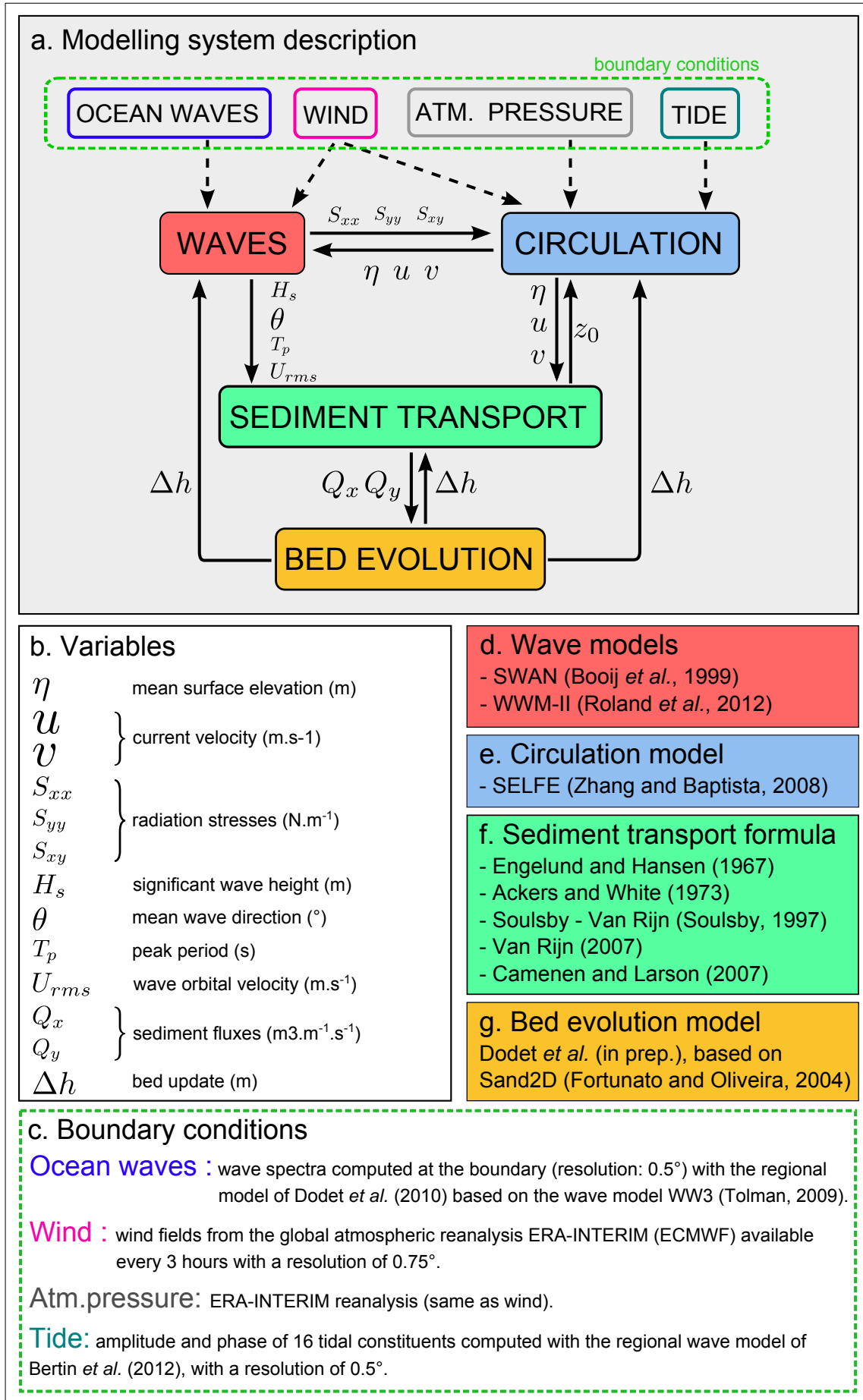


Figure 1.1: The numerical modelling system: schematic description (a), exchanged variables (b), boundary conditions (c), model details (d, e, f, g).

The wave module plays a key role in the modelling system. The impact of the waves on the dynamics is indeed preponderant in wave-dominated environments such as ephemeral tidal inlets and accurate wave model results are necessary to reproduce the hydrodynamic circulation, subsequent sediment transport and morphological changes of these systems. In addition, the small scales (in time, geographical space and spectral space) that need to be resolved to simulate the evolution of the wave field in shallow water amplify the numerical constraints. Therefore, without efficient numerical methods, most computational time is dedicated to wave modelling. The wave modelling strategy and the different existing wave models that are involved in the modelling system are described in Section 1.3.

Sediment transport rates are calculated with classical semi-empirical formulations based on the depth-averaged velocity, the water depth, the bottom roughness, the sediment properties and in some cases the wave field properties. The various sediment transport formulae implemented in the modelling system are described in Section 1.4.

Finally, the bottom evolution that results from the divergence of the sediment fluxes is computed within the bed update module by solving the Exner equation. The inherent instability of a morphodynamic model due to the non-linear coupling between the bed update module and the other modules requires adequate numerical methods and several filters to prevent the development of instabilities. The numerical method and the filters implemented are described in Section 1.5.

1.2 The hydrodynamic model

Since the original paper by *Zhang and Baptista* [2008a], SELFIE has evolved into a comprehensive modelling system that can be configured in many different ways; e.g., hydrostatic or non-hydrostatic modes; 3D bottom deformation as occurred during a seismically or landslide generated tsunamis; in 2D or 3D configuration; in Cartesian or spherical coordinates; with several tracer transport modules (sediment, oil, ecology etc.). In its default configuration, however, SELFIE solves the free surface elevation η and the 3D velocities (u, v, w) based on the shallow water equations. In a barotropic configuration, the governing equations in Cartesian coordinates are:

$$\nabla \cdot \mathbf{u} + \frac{\partial w}{\partial z} = 0 \quad (1.1)$$

$$\frac{\partial \eta}{\partial t} + \nabla \cdot \int_{-d}^{\eta} \mathbf{u} dz = 0 \quad (1.2)$$

$$\frac{D\mathbf{u}}{Dt} = -g\nabla\eta + \frac{\partial}{\partial z} \left(\nu \frac{\partial \mathbf{u}}{\partial z} \right) + \mathbf{f} \quad (1.3)$$

where D denotes the material derivative, d is the bathymetric depth, $\mathbf{u} = (u, v)$ is the horizontal velocity and w the vertical velocity, ν is the vertical eddy viscosity, $\nabla = (\partial/\partial x, \partial/\partial y)$ is the horizontal gradient operator, (x, y) are the Cartesian coordinates, z is the vertical coordinate

positive upward, t is the time, and $\mathbf{f}$ combines the explicit terms:

$$\mathbf{f} = \nabla \cdot (\mu \nabla \mathbf{u}) - f \mathbf{k} \times \mathbf{u} - \frac{1}{\rho_0} \nabla p_A + \mathbf{R}_s \quad (1.4)$$

where μ is the horizontal eddy viscosity, f is the Coriolis factor, $\mathbf{k}$ is a unit vector of the z -axis, ρ_0 is a reference water density, p_A is the atmospheric pressure, and $\mathbf{R}_s$ is the wave-induced radiation stress term.

SELFIE runs on unstructured horizontal grids with hybrid vertical coordinates: partly terrain-following S coordinates and partly Z coordinates (Fig. 1.2a). In 3D a basic computational unit is a triangular prism with uneven top and bottom surfaces, while in depth-averaged mode (2DH) only one vertical layer is considered and the computational unit is a triangular element. The elevation is computed at the vertices, the horizontal components of the velocity are computed at the sides, and the vertical velocity (in 3D) is computed at the element centre (Fig. 1.2b).

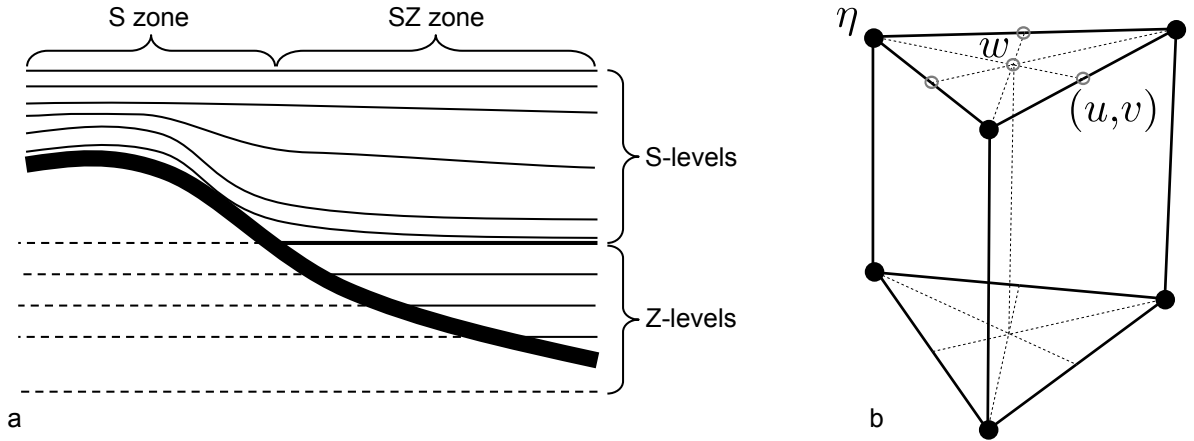


Figure 1.2: Schematic description of the vertical hybrid S - Z coordinates used in SELFIE (a) and location on computational unit of the computed elevations and velocities (b). Adapted from Zhang and Baptista [2008a].

SELFIE uses the bottom boundary condition to decouple the continuity equation 1.2 (in its depth-averaged form) from the momentum equation 1.3. At the bottom, SELFIE enforces the balance between the internal Reynolds stress and the applied bottom frictional stress as follows:

$$\nu \frac{\partial \mathbf{u}}{\partial z} = \boldsymbol{\tau}_b \text{ at } z = -d \quad (1.5)$$

The bottom stress is computed with a quadratic law:

$$\boldsymbol{\tau}_b = C_D |\mathbf{u}_b| \mathbf{u}_b \quad (1.6)$$

where $\mathbf{u}_b$ is the velocity at the top of the bottom computational cell and C_D is the drag coefficient computed as:

$$C_D = \left(\frac{\kappa}{\log(z_b/z_0)} \right)^2 \quad (1.7)$$

where $\kappa = 0.4$ is the von Karman's constant and z_b is the height from the bottom to the top of the bottom computational cell. The bottom roughness z_0 depends on the sediment grain size

and the dimensions of bedforms when the bed is not flat. Although z_0 can be estimated locally from current measurements, a map of the roughness distribution over the whole model domain is rarely available and z_0 is often used as a tuning parameter to calibrate the model. An alternative method is to use a bedform predictor that computes bedforms' dimensions from the simulated hydrodynamic conditions based on semi-empirical formulae. In shallow waters, the presence of waves significantly affects the bottom stress in the wave bottom boundary layer and an apparent bottom roughness z_{0app} can be used to represent the enhanced bottom stress by replacing z_0 with z_{0app} in Eq. 1.7. In the theory of *Grant and Madsen* [1979], implemented in SELFE by *Roland et al.* [2012], the maximum wave bottom stress is defined as:

$$\tau_w = 0.5\rho_0 f_w U_w^2 \quad (1.8)$$

where U_w is the wave orbital velocity amplitude:

$$U_w^2 = 2 \int_0^\infty \int_0^{2\pi} \frac{N\sigma^3}{\sinh^2(kh)} d\theta d\sigma \quad (1.9)$$

where N is the wave action, σ is the intrinsic frequency and θ is the wave direction. The combined wave-current friction factor f_w is a function of both the current and waves [*Zhang et al.*, 2004]:

$$\begin{cases} \gamma = \frac{|\tau_b|}{\tau_w} \\ C_\gamma = (1 + 2\gamma |\cos \theta_w| + \gamma^2)^{1/2} \\ f_w = C_\gamma \exp \left[5.61 \left(\frac{C_\gamma U_w}{30z_0\omega} \right)^{-0.109} - 7.3 \right] \end{cases} \quad (1.10)$$

where ω is the absolute wave frequency. Eqs. 1.8 and 1.10 are solved iteratively for $(\gamma, C_\gamma, \tau_w)$ by first assuming $\gamma = 0$, $C_\gamma = 1$. After these quantities are found, the apparent roughness is given by:

$$z_{0app} = \delta_{wc} \left(\frac{\delta_{wc}}{z_0} \right)^{-\sqrt{|\tau_b|/(C_\gamma \tau_w)}} \quad (1.11)$$

where the wave boundary layer thickness is given by:

$$\delta_{wc} = \frac{\sqrt{\frac{C_\gamma \tau_w}{\rho_0}}}{\omega} \exp \left[2.96 \left(\frac{C_\gamma U_w}{30z_0\omega} \right)^{-0.071} - 1.45 \right] \quad (1.12)$$

At the sea surface, SELFE enforces the balance between the internal Reynolds stress and the applied wind shear stress as follows:

$$\nu \frac{\partial \mathbf{u}}{\partial z} = \tau_b \text{ at } z = -d \quad (1.13)$$

where the wind stress τ_w is parametrized using the approach of *Pond and Pickard* [1983] or with a wave-dependent formula as described in *Bertin et al.* [2012b]. The vertical eddy viscosities are solved with the Generic Length Scale (GLS) turbulence closure model of *Umlauf and Burchard* [2003].

In 2DH (depth-averaged mode), the coupling is readily done using the assumption that the velocity profile is uniform and only one vertical layer is necessary. Eq. 1.3 becomes:

$$\frac{D\mathbf{u}}{Dt} = -g\nabla\eta + \frac{\partial}{\partial z} \left(\nu \frac{\partial \mathbf{u}}{\partial z} \right) + \nabla \cdot (\mu \nabla \mathbf{u}) - f\mathbf{k} \times \mathbf{u} - \frac{1}{\rho_0} \nabla p_A + \mathbf{R}_s + \frac{\boldsymbol{\tau}_w - \boldsymbol{\tau}_b}{\rho_0 h} \quad (1.14)$$

where h is the total water depth ($h = \eta + d$) and the quadratic law used to compute τ_b uses the depth-averaged velocity $\mathbf{u}$, while the drag coefficient C_D is computed from a Manning formulation. The wave-induced radiation stress term $\mathbf{R}_s(R_{sx}, R_{sy})$ is computed with the formulation of *Longuet-Higgins and Stewart* [1964]¹:

$$R_{sx} = -\frac{1}{\rho_0 h} \left(\frac{\partial S_{xx}}{\partial x} + \frac{\partial S_{xy}}{\partial y} \right) \quad (1.15)$$

$$R_{sy} = -\frac{1}{\rho_0 h} \left(\frac{\partial S_{xy}}{\partial x} + \frac{\partial S_{yy}}{\partial y} \right) \quad (1.16)$$

where S_{xx} , S_{xy} and S_{yy} are the components of the radiation stress tensor, defined for an irregular wave spectrum according to *Battjes* [1974] as:

$$S_{xx} = \int_0^\infty \int_0^{2\pi} N(\sigma, \theta) \sigma \left[\frac{c_g(\sigma)}{c_p(\sigma)} (\cos^2 \theta + 1) - \frac{1}{2} \right] d\theta d\sigma \quad (1.17)$$

$$S_{yy} = \int_0^\infty \int_0^{2\pi} N(\sigma, \theta) \sigma \left[\frac{c_g(\sigma)}{c_p(\sigma)} (\sin^2 \theta + 1) - \frac{1}{2} \right] d\theta d\sigma \quad (1.18)$$

$$S_{xy} = \int_0^\infty \int_0^{2\pi} N(\sigma, \theta) \sigma \frac{c_g(\sigma)}{c_p(\sigma)} \sin \theta \cos \theta d\theta d\sigma \quad (1.19)$$

where c_g is the wave group velocity and c_p is the wave phase velocity.

SELFIE solves Eqs. 1.2 and 1.3 simultaneously using a semi-implicit scheme, with an Eulerian-Lagrangian Method (ELM) for the advection in Eq. 1.3. A continuous Galerkin finite-element scheme is applied to Eqs. 1.2 and 1.3 resulting in a positive-definite, symmetric and sparse matrix that can be solved with an efficient iterative solver like the Jacobian Conjugate Gradient Method [*Zhang and Baptista*, 2008b]. Due to the hydrostatic assumption used in the shallow-water equations, w is solved from Eq. 1.1 with a finite-volume method after Eqs. 1.2 and 1.3 are solved. Other details can be found in *Zhang and Baptista* [2008a] and *Zhang et al.* [2011].

1.3 The wave models

With the purpose to investigate the dynamics of environments subjected to very energetic wave climate, a particular care was given to the choice and implementation of the wave module as well as the generation of the offshore wave boundary conditions. In this regard, a regional application to the North-East Atlantic Ocean of the wave model WAVEWATCH III [WW3,

¹Other formulations with depth-dependant radiations stresses are available in 3D mode [eg. *Bennis et al.*, 2011; *Mellor*, 2013].

[Tolman, 2009] was first implemented in order to generate spectral boundary conditions at desired spectral and spatial resolutions. This regional model is described by *Dodet et al.* [2010] and further validated by *Bertin and Dodet* [2010]. Then, a wave-current coupling was realized between SELFE and the Simulating WAVes Nearshore wave model [SWAN, *Booij et al.*, 1999] in its structured version (Version 40.81). This preliminary release of the modelling system [*Bruneau et al.*, 2011a] was used to investigate the complex wave-current interactions at the Albufeira inlet system, described in Chapter ?? and by *Dodet et al.* [2013]. Meanwhile, *Roland et al.* [2012] integrated the Wind Wave Model II (WWM-II) into SELFE and eventually WWM-II replaced the SWAN model in the modelling system. Indeed SWAN revealed some limitations when simulating wave breaking on very steep slopes (as the ones that occurred at the Albufeira lagoon) and a fully coupled wave-current model using the same unstructured grid appeared as an important step forward. The three spectral wave models WW3, SWAN and WWM-II solve the wave action equation, which is described in the following paragraph. A description of the source terms and the numerical approaches is then given, with a particular attention to the difference between the three models and the recent improvements that they benefited from.

1.3.1 The wave action equation

The wave action equation can be written as follows [eg. *Komen et al.*, 1994]:

$$\frac{\partial}{\partial t}N + \nabla(\dot{\mathbf{X}} N) + \frac{\partial}{\partial \sigma}(\dot{\theta} N) + \frac{\partial}{\partial \theta}(\dot{\sigma} N) = S_{tot} \quad (1.20)$$

where the wave action, which is invariant in slowly varying media [*Bretherton and Garrett*, 1967], is defined as:

$$N(t, \mathbf{X}, \sigma, \theta) = \frac{E(t, \mathbf{X}, \sigma, \theta)}{\sigma} \quad (1.21)$$

where E is the variance density of the sea level elevations. The advection velocities in the different spaces are obtained following the geometrical optics approximation [eg. *Keller*, 1958]:

$$\dot{\mathbf{X}} = \mathbf{c}_X = \frac{d\mathbf{X}}{dt} = \frac{d\omega}{dk} = \mathbf{c}_g + \mathbf{U}_{A(\mathbf{k})} \quad (1.22)$$

$$\dot{\theta} = c_\theta = \frac{1}{k} \frac{\partial \sigma}{\partial h} \frac{\partial h}{\partial m} + \mathbf{k} \cdot \frac{\partial \mathbf{U}_{A(\mathbf{k})}}{\partial s} \quad (1.23)$$

$$\dot{\sigma} = c_\sigma = \frac{\partial \sigma}{\partial h} \left(\frac{\partial h}{\partial t} + \mathbf{U}_A \cdot \nabla_X h \right) - c_g \mathbf{k} \frac{\partial \mathbf{U}_{A(\mathbf{k})}}{\partial s} \quad (1.24)$$

Here s represents the coordinate along the wave propagation direction and m is perpendicular to it. $\mathbf{X}$ is the Cartesian coordinate vector (x,y) in the geographical space, and $\mathbf{k}$ is the wave number vector (with $k = |\mathbf{k}|$). The effective advection velocity $\mathbf{U}_A$ depends in general on the wave number vector of each wave component [*Andrews and McIntyre*, 1978], but in the present formulation this is approximated by the depth-averaged current. Since scales of variation of depths and currents are assumed to be much larger than those of an individual wave, the quasi-uniform (linear) wave theory can be applied locally, giving the following dispersion relation and Doppler type equation to interrelate the phase parameters:

$$\sigma^2 = gk \tanh kh \quad (1.25)$$

$$\omega = \sigma + \mathbf{k} \mathbf{U}_A \quad (1.26)$$

S_{tot} is the sum of the source terms including the non-linear interactions in deep and shallow water (S_{nl4} and S_{nl3} , respectively), the energy input due to wind (S_{in}), the energy dissipation in deep and shallow water due to whitecapping and wave breaking (S_{ds} and S_{br} , respectively) and the energy dissipation due to bottom friction (S_{bf}). S_{tot} is defined as follows:

$$\frac{DN}{Dt} = S_{tot} = S_{nl4} + S_{nl3} + S_{in} + S_{ds} + S_{br} + S_{bf} \quad (1.27)$$

1.3.2 Non-linear interactions S_{nl4} and S_{nl3}

The treatment of the non-linear interactions in third generation wave models corresponds to the major improvement compared to the first and second generations of wave models. In deep and intermediate water depths resonant weakly non-linear interactions between sets of four waves play an important role in the evolution of the energy spectrum. *Hasselmann* [1962] found that a set of four waves, called a quadruplet, could exchange energy when the following resonance conditions were satisfied:

$$\begin{cases} \mathbf{k}_1 + \mathbf{k}_2 = \mathbf{k}_3 + \mathbf{k}_4 \\ \omega_1 + \omega_2 = \omega_3 + \omega_4 \end{cases} \quad (1.28)$$

The basic equation describing these interactions is known as the Boltzmann integral, which can be solved using exact methods such as the Webb-Resio-Tracy method [WRT, *Webb*, 1978; *Tracy and Resio*, 1982]. Although these methods solve the Boltzmann integral with great accuracy, they are computationally very demanding and cannot be used in operational applications. To overcome this hurdle, various approximations have been developed, of which the Discrete Interactions Approximations [DIA, *Hasselmann et al.*, 1985] is most commonly used. The DIA suffers from many shortcomings, yet it preserves a few but important characteristics of the full solution, such as the slow down-shifting of the peak frequency and shape stabilization during wave growth. This method is much faster than the WRT method and was selected for the three wave models.

The lower order triad interactions are non-resonant in deep and intermediate water depths, forcing second-order bound components that can be important locally but do not contribute to the wave evolution over large distances. However, as ocean surface waves propagate from deep to shallow water, triad interactions approach resonance and assume a significant role in the wave energy balance. This transition from quadruplet to triad interactions is the result of the change in the dispersion relation from a dispersive deep water regime that does not support resonant triad interactions [*Phillips*, 1960] to a non-dispersive shallow water regime where all wave components travel with the same speed. In general triad interactions transfer energy from the incident wave components to higher- (e.g., harmonic) and lower- (e.g., infra-gravity) frequency components. SWAN and WWM-II use the Lumped Triad Approximation (LTA) of *Eldeberky et al.* [1996] to compute S_{nl3} , while the actual release (V3.14) of WW3 does not include triad interactions.

1.3.3 Wave growth S_{in}

The source term S_{in} is composed of a linear growth term and an exponential growth term. The linear term, which allows for the consistent spin-up of the model from quiescent conditions

and improves initial wave growth behaviour, is included in the three wave models following the theory of *Cavaleri and Rizzoli* [1981], with a filter to eliminate wave growth at low-frequency as introduced by *Tolman* [1992]. Regarding the exponential growth, the widely used WAM-Cycle4 parametrization [*Günther et al.*, 1992] is available in WW3, SWAN and WWM-II. It is based on the wave growth theory of *Miles* [1957], modified by *Janssen* [1982]. The pressure slope correlations that give rise to part of the wave generation are parametrized following *Janssen* [1991].

In addition, several improvements are included in WW3 and WWM-II: first, the WAM-Cycle4 parametrization was further extended by *Bidlot* [2007] to take air density into account and increased wind gustiness in unstable atmospheric conditions. *Ardhuin et al.* [2010] also included a sink term to S_{in} to reproduce swell decay observations [*Ardhuin et al.*, 2009] due to shear stresses variations in phase with the orbital velocity. The resulting wave growth term S_{in}^* is expressed as $S_{in}^* = S_{in} + S_{out}$ and S_{out} is related to a laminar-to-turbulent transition of the oscillatory boundary layers over swell through the boundary Reynolds number defined as $Re = 4u_{orb}a_{orb}/\nu_a$, where u_{orb} and a_{orb} are the significant surface orbital velocity and displacement amplitudes, and ν_a the air viscosity. S_{out} is defined as:

$$S_{out}(k, \theta) = \begin{cases} -C_{dsv} \frac{\rho_a}{\rho_w} (2k\sqrt{2\nu\sigma}) F(k, \theta) & \text{for } Re < Re_c \\ -\frac{\rho_a}{\rho_w} (16f_e\sigma^2 u_{orb}/g) F(k, \theta) & \text{for } Re \geq Re_c \end{cases} \quad (1.29)$$

where F is the variance density spectrum defined as a function of k and θ and that can be inferred from $E(\sigma, \theta)$ using straightforward Jacobian transformation. Re_c is a threshold value set to $2 \cdot 10^5$ and $C_{dsv} = 1.2$.

1.3.4 Wave dissipation S_{ds} , S_{br} and S_{bf}

Spectral wave energy dissipation represents the least understood part of the physics relevant to wave modelling. There is a general consensus that the major part of this dissipation is due to the wave breaking, but physics of this breaking process, particularly for the spectral waves, is poorly understood. In deep water, the dissipation of spectral energy S_{ds} is mainly attributed to the whitecapping dissipation process. This process has been represented by the pulse-based model of *Hasselmann et al.* [1973] where S_{ds} depends on the mean spectral steepness. However, this model was shown to underestimate the dissipation of the spectral tail in the presence of swell, due to its dependence on the mean spectral wavenumber and steepness [*van Vledder and Hurdle*, 2002]. In addition *Ardhuin et al.* [2010] demonstrated that an increase in swell steepness would result in a decrease in swell dissipation, while the opposite effect is expected. To counteract these limitations, *van der Westhuysen et al.* [2007] implemented in SWAN an adapted form of the saturation-based model of *Alves and Banner* [2003]. The local treatment of whitecapping dissipation in the frequency space improved the model results in fetch and depth-limited conditions, yet the steepness-induced breaking dissipation was still underestimated in the presence of strong counter currents [*Ris and Holthuijsen*, 1996]. To overcome this limitation, *van der Westhuysen* [2012] used an enhanced-dissipation term with an additional term in S_{ds} proportional to the current gradient in the wave propagation direction, defined as:

$$S_{ds,curr}(\sigma, \theta) = -C_{ds} \max\left(\frac{c_\sigma(\sigma, \theta)}{\sigma}, 0\right) \left(\frac{B(k)}{B_r}\right)^{\frac{p}{2}} E(\sigma, \theta) \quad (1.30)$$

where $B(k)$ represents the spectral saturation, p is a function of the inverse wave age, based on scaling arguments involving a spectral balance between the wind input, whitecapping and non-linear interaction terms [see *van der Westhuysen et al.*, 2007, for details], and B_r is a threshold saturation level, as in *Alves and Banner* [2003], and C_{ds} is a calibration parameter. Although this parametrization improved model results in several cases [*van der Westhuysen*, 2012; *Dodet et al.*, 2013], the underlying physics is still questionable, particularly in blocking conditions. In WW3, the whitecapping dissipation term S_{ds} combines the parametrization of *Bidlot* [2007] with an additional saturation term. The effect of wave-turbulence interactions is also included in this source term following the expression of *Teixeira and Belcher* [2002]. The calibration settings used in the North-Atlantic regional model were provided by Dr. Fabrice Ardhuin and corresponded to TEST405 as described in *Ardhuin et al.* [2010].

To model the energy dissipation due to depth-induced breaking, the bore-based model of *Battjes and Janssen* [1978] is used in the three wave models. This model is based on the assumption that all waves in a random field exceeding a threshold height, defined as a function of bottom topography parameters, will break. For a random wave field, the fraction of waves satisfying this criterion is determined by a statistical description of surf-zone wave heights (a Rayleigh-type distribution, truncated at a depth-dependent wave-height maximum is generally used). The bulk rate δ of spectral energy density dissipation of the fraction of breaking waves is estimated using an analogy with dissipation in turbulent bores as:

$$\delta = 0.25Q_b f_m H_{max}^2 \quad (1.31)$$

where Q_b is the fraction of breaking waves in the random field, f_m is the mean frequency and H_{max} is the maximum individual height that a component in the random wave field can reach without breaking (conversely, above which all waves would break). In the model of *Battjes and Janssen* [1978] the maximum wave height H_{max} is defined using a Miche-type criterion [*Miche*, 1944],

$$\bar{k}H_{max} = \gamma_M \tanh(\bar{k}h) \quad (1.32)$$

where γ_M is a constant factor. This approach also removes energy in deepwater waves exceeding a limiting steepness and can potentially result in double counting of dissipation in deep-water waves. Alternatively, H_{max} can be defined using a McCowan-type criterion, which consists of a simple constant ratio:

$$H_{max} = \gamma h \quad (1.33)$$

where γ is a constant derived from field and laboratory observations of breaking waves (default value is 0.73). This approach represents exclusively depth-induced breaking and is the default option in the three models. The fraction of breaking waves Q_b is determined in terms of a Rayleigh-type distribution truncated at H_{max} (i.e., all broken waves have a height equal to H_{max}), which results in the following expression:

$$\frac{1 - Q_b}{-\ln Q_b} = \left(\frac{H_{rms}}{H_{max}} \right) \quad (1.34)$$

where H_{rms} is the root-mean-square wave height. With the assumption that the total spectral energy dissipation δ is distributed over the entire spectrum so that it does not change the

spectral shape [Eldeberky and Battjes, 1996] the following depth-induced breaking dissipation source function is obtained:

$$S_{br}(k, \theta) = -\alpha \frac{\delta}{E_{tot}} F(k, \theta) = -0.25\alpha Q_b f_m \frac{H_{max}^2}{E_{tot}} F(k, \theta) \quad (1.35)$$

where E_{tot} is the total energy (computed as the zero moment of the variance density spectrum, see Eq. ??) and $\alpha = 1.0$ is a tunable parameter.

Eventually, the interactions of the wave orbital motion with the sea floor in shallow water will dissipate spectral energy through bottom friction. In the three wave models, the default parametrization of the dissipation term S_{bf} is based on the empirical, linear JONSWAP parametrization [Hasselmann *et al.*, 1973], that can be expressed in the following form:

$$S_{bf} = -C_b \frac{\sigma^2}{g^2 \sinh^2 kh} E(\sigma, \theta) \quad (1.36)$$

where the empirical coefficient C_b is suggested to be $C_b = 0.038 \text{ m}^2\text{s}^{-3}$ for swell [Hasselmann *et al.*, 1973] and $C_b = 0.067 \text{ m}^2\text{s}^{-3}$ for wind sea [Bouws and Komen, 1983]. Alternatively, in SWAN and WWM-II, the coefficient C_b can be computed as a function of the bottom roughness height and the actual wave conditions, following the formulation of Madsen *et al.* [2011]:

$$C_b = f_w \frac{g}{\sqrt{2}} U_{rms} \quad (1.37)$$

where U_{rms} is the wave bottom orbital motion:

$$U_{rms}^2 = 2 \int_0^\infty \int_0^{2\pi} \frac{\sigma^2}{g^2 \sinh^2 kh} E d\theta d\sigma \quad (1.38)$$

and f_w is a non-dimensional friction factor estimated as:

$$\frac{1}{4\sqrt{f_w}} + \log -10 \left(\frac{1}{4\sqrt{f_w}} \right) = m_f + \log_{10} \left(\frac{U_w}{K_n} \right) \quad (1.39)$$

in which $m_f = -0.08$, U_w is the wave orbital velocity amplitude (Eq. 1.9) and K_n is the bottom roughness length scale.

1.3.5 Numerical approaches

The different numerical approaches implemented in the three wave models discussed herein emerged from the original applications of these models (basin-scale, nearshore or multi-scale). Although, WW3, SWAN and WWM-II share the same discretization of the spectral grid with equally-spaced directional bins and logarithmically-spaced frequency bins, they present important difference in their other numerical aspects.

WW3 is particularly adequate for hindcasting and forecasting wave conditions at ocean scales. It solves the wave action equation on structured grids based on the Fraction Step Method (FSM): an Operator Splitting Method developed by Yanenko [1971]. The propagation part of the wave

action equation is split in four one-dimensional parts which are solved with the third-order Ultimate Quickest (UQ) scheme of *Leonard* [1991]. The UQ-scheme solves very accurately the advection problem with very little numerical diffusion and is computationally effective in comparison to other schemes [Roland, 2009]. Afterwards the source terms are integrated with an adaptive time integration method using a semi-implicit integration scheme. Other important features such as the treatment of unresolved islands [Chawla and Tolman, 2008] or the multi-grid nesting strategy [Tolman, 2008] have been implemented in the last decade. Thanks to these improvements and the inclusion of the accurate source terms formulations of *Ardhuin et al.* [2010], WW3 represents one of the most effective approach for deep-water wave modelling.

SWAN runs on structured (regular or curvi-linear) grids and treats the propagation part fully implicitly with an upwind Euler method [Booij et al., 1999]. Two finite-difference schemes are implemented for solving the geographical advection part in the wave action equation: the Backward Time and Backward Space (BSBT), a first-order scheme and the higher-order SOR-DUP scheme [Rogers et al., 2002]. The discretization of the spectral space is based on an hybrid central-upwind scheme, in which the degree to which the scheme is upwind or central can be controlled by adjusting some coefficients. The fully implicit treatment of the advection part and the semi-implicit integration of the source terms within the advection part lead to a robust scheme that converges to a unique solution, independently of the chosen integration time step. Although this unconditional stability is a major advantage of SWAN, the inflexibility of the structured mesh makes the model applications cumbersome in complicated coastal environments [Roland, 2009].

WWM-II runs on unstructured (triangular) meshes and solves the wave action equation using the FSM method described by *Yanenko* [1971], similarly to WW3. The geographical part is solved using a Residual Distribution Scheme (e.g., *Abgrall* [2006]), which borrows ideas from the Finite Element (FE) and the Finite Volume framework [Roland et al., 2012]. Then the spectral advection is treated with the UQ scheme following the WW3 approach. Finally the source terms are integrated in three separate fractional steps according to their time scales or nonlinearity, using a TVD Runge-Kutta third-order scheme for the wave breaking and the bottom friction, a dynamic method for the triad interactions and a semi-implicit approach for the deep-water source terms. The flexibility of the unstructured mesh combined with the robust and accurate method of WWM-II makes this model particularly adequate for simulating wave-current interactions in complicated coastal zones such as tidal inlets.

1.4 The sediment transport model

Sediment transport rates are usually calculated with classical semi-empirical formulations based on the assumption that the bed load or the total load (both bed and suspended loads) are instantaneously in equilibrium with the local hydrodynamic conditions [with some exceptions, e.g., *Wu*, 2004]. These formulae often use the depth-averaged velocity to compute the total load directly [e.g., *Engelund and Hansen*, 1967] or the bed load and the suspended load separately [e.g., *van Rijn*, 2007b,a]. Such formulae have been implemented in several morphodynamics models, such as DELFT3D [Lesser et al., 2004] or MORSYS2D [Bertin et al., 2009a]. Alternatively,

the vertical profile of the suspended sediment concentration can be treated as a passive scalar and is determined by solving an advection/diffusion equation, as for instance in the morphodynamic modelling systems TELEMAC [Villaret *et al.*, 2013], ROMS [Warner *et al.*, 2008], DELFT3D [Lesser *et al.*, 2004] and MORSELE [Pinto *et al.*, 2012]. Although the latter method is more physical, it is also computationally much more expensive and only too few comparisons between both methods have been realized to this day to conclude about their relative accuracy. The choice is even more subtle when depth-averaged hydrodynamic models are used to compute the velocities, since additional parametrizations are needed to emulate the vertical distributions of currents and sediment concentrations. As the computational cost of a morphodynamic modelling system is usually a key factor for its applicability, a depth-averaged approach with semi-empirical formulae to compute the sediment transport rates was selected. Among the wide range of semi-empirical formulations available in the literature, each of them being developed for particular conditions (steady-state, with or without waves etc...) or validated against a specific test bed, five formulations have been implemented so far in the modelling system presented herein. These formulations are presented in the following paragraph.

1.4.1 Sediment transport formulae

Engelund and Hansen (1967)

Engelund and Hansen [1967] proposed a formula to compute the total sediment transport rate in rivers. This formula is less appropriate for use in the sea than in rivers, because of the assumption of quasy-steady flow and that the sandwaves/dunes are in equilibrium with the instantaneous flow velocity, which is not true in tidal flows. However this formula is still widely used in coastal environments in the absence of waves due to its simplicity and reasonable accuracy. The total sediment transport is computed as:

$$q_t = \frac{0.04C_d^{3/2}\mathbf{u}^5}{g(s-1)^2d_{50}} \quad (1.40)$$

where $\mathbf{u}$ is the mean flow velocity, $s = \rho_s/\rho_0$ is the specific sediment density, d_{50} is the mean particle diameter and C_d is the drag coefficient.

Ackers and White (1973)

Ackers and White [1973] derived a formula for total transport in rivers by considering the form of the transport relations for bedload (coarse sediments) and suspended sediment load (fine sediments) separately, and uniting them with a transition in the range $1 < D_* \leq 60$ through empirical coefficients that were fitted to a large data set. The total transport is computed as:

$$q_t = C_{AW}\mathbf{u}h\left(\frac{\mathbf{u}}{u_*}\right)^n\left(\frac{F_{AW}-A_{AW}}{A_{AW}}\right)^m \quad (1.41)$$

where

$$F_{AW} = \frac{u_*^n}{\sqrt{g(s-1)d_{35}}}\left[\frac{\mathbf{u}}{2.46\log(10h/d_{35})}\right]^{1-n} \quad (1.42)$$

The dimensionless grain size D_* is defined as $D_* = d_{35} [g(s-1)/\nu^2]^{1/3}$ with ν the kinematic viscosity of the fluid. For $1 < D_* \leq 60$

$$n = 1 - 0.243 \log D_* \quad (1.43)$$

$$A_{AW} = \frac{0.23}{\sqrt{D_*}} + 0.14 \quad (1.44)$$

$$m = \frac{6.83}{D_*} + 1.67 \quad (1.45)$$

$$C_{AW} = \exp [2.79 \log D_* - 0.426(\log D_*)^2 - 7.97] \quad (1.46)$$

and for $D_* > 60$: $n = 0$; $A_{AW} = 0.17$; $m = 1.78$; $C_{AW} = 0.025$ ². Given the threshold $D_* > 1$, this formula only applies for grain sizes larger than 0.1 mm.

Soulsby - Van Rijn (1997)

The Soulsby - Van Rijn formula [Soulsby, 1997] is used to compute total sediment transport in the presence of waves and tidal currents. The formula is:

$$q_t = (A_{sb} + A_{ss}) \mathbf{u} \left[\left(\mathbf{u}^2 + \frac{0.018}{C_D} U_{rms}^2 \right)^{1/2} - U_{cr} \right]^{2.4} (1 - 1.6 \tanh \beta) \quad (1.47)$$

where C_D is the drag coefficient due to current alone, U_{rms} is the root-mean-square wave orbital velocity, U_{cr} is the threshold depth-averaged current speed, β is the slope of the bed in the streamwise direction (positive if flow runs uphill), and A_{sb} and A_{ss} represent the contribution of the bedload and the suspended load respectively and are computed as follows:

$$A_{sb} = \frac{0.005h(d_{50}/h)^{1.2}}{[g(s-1)d_{50}]^{1.2}} \quad (1.48)$$

$$A_{ss} = \frac{0.012d_{50}D_*^{-0.6}}{[g(s-1)d_{50}]^{1.2}} \quad (1.49)$$

where D_* is the dimensionless particle size defined as $D_* = d_{50} [g(s-1)/\nu^2]^{1/3}$.

Van Rijn (2007)

van Rijn [2007a,b] proposed a unified transport formula for bedload and suspended load in the presence of waves and currents, that apply over a large range of particle sizes (0.05 mm to 2 mm). The bedload and suspended transport rates are computed as:

$$q_b = 0.015 \mathbf{u} h (d_{50}/h)^{1.2} M_e^{1.5} \quad (1.50)$$

$$q_s = 0.012 \mathbf{u} d_{50} M_e^{2.4} D_*^{-0.6} \quad (1.51)$$

where $M_e = (u_e - u_{cr}) / [(s-1)gd_{50}]^{0.5}$ is the mobility parameter (if $u_e \leq u_{cr}$ $M_e = 0$), $u_e = u + \gamma U_w$ is the effective velocity with $\gamma = 0.4$ for irregular waves (and 0.8 for regular waves), u is the depth-averaged flow velocity, $U_w = \pi H_s / [T_p \sinh(kh)]$ is the peak orbital velocity (based on linear wave theory), H_s is the significant wave height, T_p is the peak wave period, $u_{cr} =$

²The coefficients m and C_{AW} , given in Soulsby [1997], are taken from a revised set of values based on more recent data.

$\beta u_{cr,c} + (1 - \beta)u_{cr,w}$ with $\beta = u/(u + U_w)$, $u_{cr,c}$ is the critical velocity for currents based on Shields (initiation of motion), and $u_{cr,w}$ is the critical velocity for waves based on *Komar and Miller* [1975]:

$$u_{cr,c} = \begin{cases} 0.19(d_{50})^{0.1} \log(4h/d_{90}) & \text{for } 0.00005 < d_{50} < 0.0005m \\ 8.5(d_{50})^{0.6} \log(4h/d_{90}) & \text{for } 0.0005 < d_{50} < 0.002m \end{cases} \quad (1.52)$$

$$u_{cr,w} = \begin{cases} 0.24((s-1)g)^{0.66} d_{50}^{0.33} (T_p)^{0.33} & \text{for } 0.00005 < d_{50} < 0.0005m \\ 0.95[(s-1)g]^{0.57} d_{50}^{0.43} (T_p)^{0.14} & \text{for } 0.0005 < d_{50} < 0.002m \end{cases} \quad (1.53)$$

Camenen and Larson (2005, 2008)

Camenen and Larson developed a formula for bed load transport [Camenen and Larson, 2005] and for suspended transport [Camenen and Larson, 2008], taking into account the following mechanisms: presence of waves and currents, breaking and non-breaking waves, asymmetric wave velocity, angle between waves and current, phase-lag effects between water and sediment motions, and bed features in bed roughness estimate.

The bed load transport rate is computed as:

$$\frac{q_{bw}}{\sqrt{(s-1)gd_{50}^3}} = a_w \sqrt{\theta_{net}} \theta_{cw,m} \exp\left(-b \frac{\theta_{cr}}{\theta_{cw}}\right) \quad (1.54)$$

$$\frac{q_{bn}}{\sqrt{(s-1)gd_{50}^3}} = a_n \sqrt{\theta_{cn}} \theta_{cw,m} \exp\left(-b \frac{\theta_{cr}}{\theta_{cw}}\right) \quad (1.55)$$

where the subscripts w and n correspond respectively to the wave direction and the direction normal to the waves, θ_{cr} is the critical Shields number, θ_{net} the net sediment transporting Shields number (taking into account phase-lag effect), a_w , a_n , and b are empirical coefficients. θ_{cw} , $\theta_{cw,m}$, and θ_{cn} are defined as follows:

$$\begin{aligned} \theta_{cw} &= (\theta_c^2 + \theta_w^2 + 2\theta_w\theta_c \cos \phi)^{1/2} \\ \theta_{cw,m} &= (\theta_c^2 + \theta_{w,m}^2 + 2\theta_{w,m}\theta_c \cos \phi)^{1/2} \\ \theta_{cn} &= \frac{\frac{1}{2}f_c(U \sin \phi)^2}{(s-1)gd_{50}} \end{aligned} \quad (1.56)$$

where f_c is the current-related friction factor, ϕ the angle between wave and current directions, θ_c , $\theta_{w,m}$, and θ_w are respectively the current, mean wave, and maximum wave Shields number, and $\theta_{w,m} = \theta_w/2$ for a sinusoidal wave profile.

The suspended load transport rate is computed as :

$$q_{sw} = U_{c,net} c_R \frac{\epsilon}{W_s} \left[1 - \exp\left(-\frac{W_s h}{\epsilon}\right) \right] \quad (1.57)$$

$$q_{sn} = U \sin \phi c_R \frac{\epsilon}{W_s} \left[1 - \exp\left(-\frac{W_s h}{\epsilon}\right) \right] \quad (1.58)$$

where $U_{c,net}$ is the net mean current after a wave period (taking into account phase-lag effect), c_R the dimensionless reference concentration at the bottom, W_s the sediment fall speed, and ϵ the sediment diffusivity. A description of the swash zone transport formula can be found in *Larson et al.* [2011].

1.4.2 Bed-slope effect

Most of the transport formula described in the previous paragraphs were derived for flat bed conditions. However, bathymetric gradients play an important role in sediment transport. They increase (resp. decrease) the critical shear stress when sediments are moving up-slope (resp. down-slope), acting as a diffusion-like term that stabilizes the numerical simulations. Physically, this effect is considered by modifying the critical shear stress according to the bottom slope along the flow direction [*Carmo*, 1995] or by applying empirical factors to the sediment transport rates in the flow direction and in the transverse direction. This second method has been adopted in this morphodynamic modelling system.

Lesser et al. [2004] proposed a method based on the work of *Bagnold* [1966] and *Ikeda* [1982]. The influence of the bed-slope is included in two parts. In the first part, a longitudinal slope in the direction of the bed-load transport modifies the magnitude of the bed-load vector. This adjustment is computed following a modified form of the expression suggested by *Bagnold* [1966]:

$$(q_{b,x}^*, q_{b,y}^*) = \alpha_s(q_{b,x}, q_{b,y}) \quad (1.59)$$

where

$$\alpha_s = 1 + \alpha_{bs} \left[\frac{\tan \phi}{\cos \beta_s (\tan \phi - \tan \beta_s)} - 1 \right] \quad (1.60)$$

α_{bs} is a user-specified tuning parameter (default = 1), ϕ is the angle of repose of sediments, and β_s is the bed slope angle in the direction of bedload transport vector. In the second part of the method, the direction of bedload transport is adjusted if a bed slope exists in the direction normal to the bedload transport vector. This modification is based on the work of *Ikeda* [1982] and is computed as follows:

$$q_{b,x}^{**} = q_{b,x}^* - \alpha_n q_{b,y}^* \quad (1.61)$$

$$q_{b,y}^{**} = q_{b,y}^* - \alpha_n q_{b,x}^* \quad (1.62)$$

where

$$\alpha_n = \alpha_{bn} \left(\frac{\tau_{cr}}{\tau_b} \right)^{0.5} \tan \beta_n \quad (1.63)$$

in which α_{bn} is a user-specified coefficient (default = 1.5), τ_{cr} is the threshold bed shear-stress for motion of sediments, τ_b is the bed shear-stress due to currents and waves, and β_n is the bed slope angle normal to the bedload transport vector. This method was implemented and tested in several morphodynamic models, such as DELFT-3D [*Lesser et al.*, 2004] and MORSELEF [*Pinto et al.*, 2012].

1.5 The bottom evolution module

The bottom evolution model solves the sediment continuity equation (or Exner equation) to compute the bed changes resulting from the divergence of the sediment fluxes. The Exner equation is:

$$\frac{\partial d}{\partial t} - \frac{1}{1-\lambda} \nabla \cdot \mathbf{q} = 0 \quad (1.64)$$

where $\mathbf{q}$ represents the depth-integrated volumetric sediment flux (expressed in $m^3.s^{-1}.m^{-1}$), λ is the porosity and d is the depth relative to the mean water level. After integration in time, Eq. 1.64 becomes³:

$$\Delta d^i = \frac{1}{1-\lambda} \nabla \cdot \mathbf{Q}^i \quad (1.65)$$

where $\mathbf{Q}^i$ is the sediment transport rate integrated over the morphological time step i . Then, Eq. 1.65 is integrated over a defined control volume, following:

$$\int_{\Omega} \Delta d \, d\Omega = \frac{1}{1-\lambda} \int_{\Gamma} \mathbf{Q} \cdot \mathbf{n} \, d\Gamma \quad (1.66)$$

where Ω indicates the control volume with boundary Γ and $\mathbf{n}$ is the outward unit normal on Γ ⁴. Eq. 1.66 is solved with a node-centred control volume (Fig. 1.3.a) or an element-centred control volume (Fig. 1.3.b).

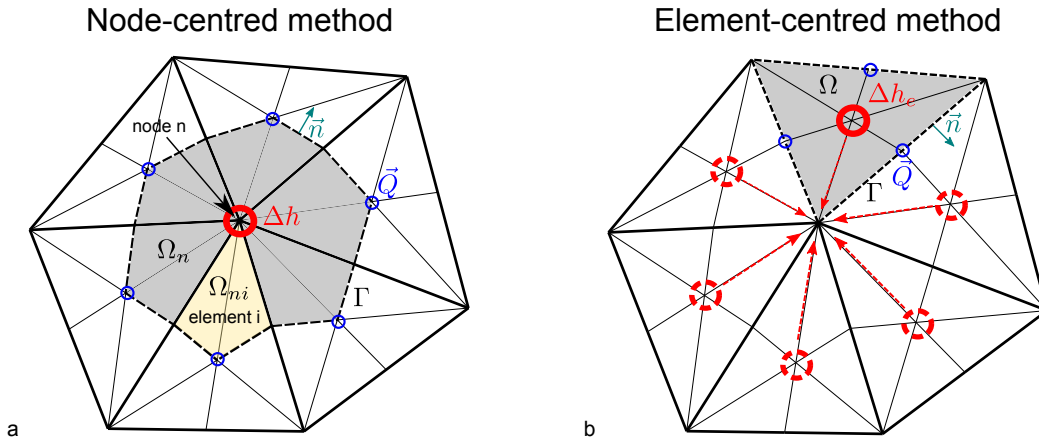


Figure 1.3: Schematic representation of the control volume and associated variable for the node-centred method (a) and the element-centred method (b). The sediment fluxes $\mathbf{Q}$ and the bed changes Δh are computed at the points marked with blue and red circles, respectively.

The former method was initially implemented by *Fortunato and Oliveira* [2004] in the morphodynamic model SAND2D and later in MORSYS2D [*Bertin et al.*, 2009a] and MORSELFÉ [*Pinto et al.*, 2012]. The sediment fluxes are computed at the element centres and are assumed to be constant inside the element while the depth varies linearly within the element. After discretization, the left-hand side term of Eq. 1.66 becomes:

$$\int_{\Omega} \Delta h \, d\Omega = \sum_{i=1}^{nel} \left(\sum_{j=1}^3 \Delta h_{ij} \int_{\Omega} \phi_j d\Omega_{ni} \right) \quad (1.67)$$

³Since the integrated variable is space-independent, the Leibniz rule is invoked to switch the integral and the divergence operators.

⁴Gauss' theorem is used to cast the divergence term into a boundary integral.

where nel is the number of elements surrounding node n , ϕ_j is the element linear shape function that equals 1 at node j and 0 at the two other nodes of the element (Fig. 1.4). The integration of ϕ over the local control volume Ω_{ni} can be expressed as:

$$\int_{\Omega} \phi_j d\Omega_{ni} = C_i A_{ni} \quad (1.68)$$

where A_{ni} is the area of element i and

$$C_i = \begin{cases} 22/108 & \text{if } j = n \\ 7/108 & \text{if } j \neq n \end{cases} \quad (1.69)$$

The right-hand side term of Eq. 1.66 becomes:

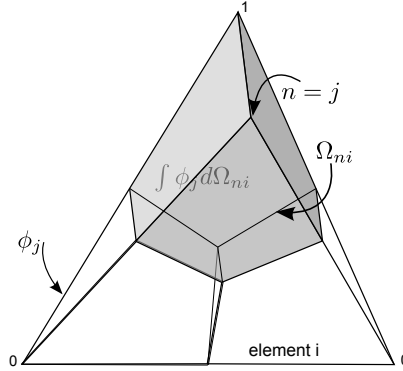


Figure 1.4: Schematic representation of an element i , its associated shape function ϕ_j and the integration volume of ϕ_j over the local control volume Ω_{ni} (shaded volume) used to solve Eq. 1.67

$$\frac{1}{1-\lambda} \int_{\Gamma} \mathbf{Q} \cdot \mathbf{n} d\Gamma = \frac{1}{1-\lambda} \sum_{i=1}^{nel} \left(\sum_{j=1}^2 Q_{ix}(y_{ij} - y_{0i}) - Q_{iy}(x_{ij} - x_{0i}) \right) \quad (1.70)$$

where (Q_{ix}, Q_{iy}) are the x and y components of the sediment flux integrated in time at element i , (x_{0i}, y_{0i}) are the coordinates of the centre of element i and (x_{ij}, y_{ij}) are the coordinates of the centre of side j that belongs to element i and includes node n . The system of equations $Ax = b$ where the right-hand side term of Eq. 1.67 represents Ax and Eq. 1.70 represents b is eventually solved with a Jacobi conjugate gradient method to obtain the bed changes at every node.

In the element-centred method, the sediment fluxes are computed and integrated in time at the sides of the elements (Fig. 1.5) and since h is assumed to vary linearly within the element, the left-hand side term of Eq. 1.66 can be reduced to $\Delta h_e A_e$ where A_e is the area of the element. Hence the bed changes at the centre of the elements h_e are obtained through:

$$\Delta h_e = \frac{1}{A_e(1-\lambda)} \int_{\Gamma} \mathbf{Q} \cdot \mathbf{n} d\Gamma \quad (1.71)$$

Considering the fluxes constant along each side of the element, Eq. 1.71 can finally be expressed as:

$$\Delta h_e = \frac{1}{A_e(1-\lambda)} \sum_{i=1}^3 (Q_{ix}(y_{i+1} - y_i) - Q_{iy}(x_{i+1} - x_i)) \text{ with } x_4 = x_1 \text{ and } y_4 = y_1 \quad (1.72)$$

where (Q_{ix}, Q_{iy}) are the x and y components of the sediment flux integrated in time at side i , (x_i, y_i) are the coordinates of node i (Fig. 1.5). The bed changes at the nodes (Δh) are computed

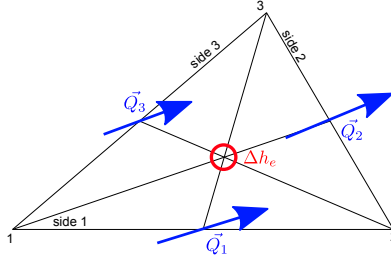


Figure 1.5: Schematic representation of an element with the relative position and names of the nodes, the sides and the associated sediment fluxes.

as the element-area-weighted-averages of the bed changes previously computed for the surrounding elements (Δh_e). While the second method estimates bed changes with a higher resolution⁵, the interpolation of bathymetric changes at the nodes (necessary for the next hydrodynamic computation) is not mass-conservative and may lead to some inaccuracies.

The default morphodynamic time step is equal to the hydrodynamic time step, but it can optionally be set to a multiple of the hydrodynamic time step, in order to speed up the computational time. The time integration of Q is realized with a trapezoidal rule. The Courant-Friedrich-Levy (CFL) condition imposed by the explicit method can be estimated as $Cu \approx \frac{bQ}{h\Delta x}$, where C_u is the Courant number, b is the velocity power in the transport formulae (typically between 3 and 5, depending on the specific formulation), according to *Roelvink* [2006]. Numerical experiments realized with the bottom update model SAND2D, from which the numerical methods presented herein were adapted [*Fortunato and Oliveira*, 2004], indicated that the morphodynamic model was stable for maximum Courant numbers of up to 10 [*Fortunato and Oliveira*, 2007b].

Very often, the results of morphodynamic models are spoiled by undesired spatial oscillations, independently of the numerical methods implemented. These oscillations result from the inherent unstable nature of the non-linear coupling between the sediment transport module and the bed evolution module (de Vriend et al 1987). *Johnson and Zyserman* [2002] showed that the dependence of the bed celerity with the bed levels, due to the non-linear relationship between sediment transport and bed levels, is responsible for high wave number spatial oscillations. They applied a diffusion-like filter to the sediment fluxes in order to mimic some natural smoothing effect (bed slope, random waves) and limit these oscillations. *Callaghan et al.* [2006] provided analytical evidence that an inadequate numerical solver for the Exner equation will lead to dispersion and diffusion in the solution and will result in spatial oscillations. *Kubatko et al.* [2006] implemented a discontinuous Galerkin method to solve the Exner equation on unstructured grids which is combined with a slope limiter to dampen spatial oscillations.

In the bottom update module, several approaches were implemented to prevent the development of spatial oscillations. First, a modified expression [*Fortunato and Oliveira*, 2007b] of the diffusion-like term given by *Horikawa* [1988] was included in the Exner equation:

⁵In unstructured triangular grids the number of element is roughly twice the number of nodes.

$$Q^* = Q + \epsilon(1 - \lambda) \left(|Qx| \frac{\partial h}{\partial x}, |Qy| \frac{\partial h}{\partial y} \right) \quad (1.73)$$

where ϵ is a dimensionless diffusion coefficient, and Q^* replaces Q in Eq. 1.64. Also, a non-linear filter was implemented to eliminate local extrema in the bathymetry after each morphodynamic time step. This filter, implemented by *Fortunato and Oliveira* [2000] is an extension of the one-dimensional filter of *Shyy et al.* [1992] to two-dimensional unstructured and proceeds as follows: a minimum (maximum) occurs for $h_i < h_{min}$ ($h_i > h_{max}$), where $h_{min} = \min(h_j : j = 1, nei)$ ($h_{max} = \max(h_j : j = 1, nei)$), and nei is the number of nodes surrounding node i . For each minimum (maximum), h_i and h_{min} (h_{max}) are made equal while preserving the volume by setting:

$$h_i^f = h_{min}^f = \frac{h_i A_i + h_{imax} A_{imax}}{A_i + A_{imax}} \quad (1.74)$$

$$h_i^f = h_{max}^f = \frac{h_i A_i + h_{imin} A_{imin}}{A_i + A_{imin}} \quad (1.75)$$

where the superscript f indicates a filtered value, and A_i and $A_{imin(imax)}$ are the sum of the areas of the elements surrounding node i and the node where depth is minimum (maximum), respectively. The procedure is repeated until convergence, i.e. until all local extrema have been removed. This filter damps $2\Delta x$ oscillations with minimal introduction of numerical diffusion [*Oliveira and Fortunato*, 2002] (as long as a minimum height threshold under which no extrema can be detected is introduced).

Finally an avalanching method was implemented to reproduce gravity effects on steep bottom surfaces. After each morphodynamic time step, an iterative procedure modifies the bathymetry in a mass-conservative manner to decrease local slopes higher than a certain threshold. Following *Roelvink et al.* [2009] the threshold values of the tangent of the bottom slope for wet and dry sands are set to 0.3 and 1 respectively.

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