Keywords

Place keywords here

Summary

Place summary here

References

Place references here

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| Version | Date | Author | Initials | Review | Initials | Approval | Initials |
|  | jan. 2015 |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |

|  |
| --- |
| State  draft  This is a draft report, intended for discussion purposes only. No part of this report may be relied upon by either principals or third parties. |

Contents

1 Introduction 1

2 Processes and model formulation 2

2.1 Domain and definitions 2

2.2 Hydrodynamics options 2

2.2.1 Stationary mode 2

2.2.2 Non-stationary (surfbeat) mode 2

2.2.3 Wave resolving mode 2

2.3 Short wave propagation 3

2.3.1 Wave action balance 3

2.3.2 Dissipation 4

2.3.3 Roller energy balance 6

2.4 Shallow water equations 7

2.5 Nonhydrostatic pressure correction 8

2.6 Groundwater flow 10

2.6.1 Hydrostatic 10

2.6.2 Non-hydrostatic 12

2.7 Sediment transport 16

2.7.1 Advection-diffusion 16

2.7.2 General parameters: velocity magnitude and orbital velocity 16

2.7.3 Transport formulations 17

2.7.4 Wave asymmetry 18

2.8 Bottom updating 20

2.8.1 Due to sediment fluxes 20

2.8.2 Avalanching 20

2.8.3 Bed composition 21

3 Numerical implementation 22

3.1 Grid types 22

3.1.1 1D 22

3.1.2 Rectilinear 22

3.1.3 Curvilinear 22

3.2 Wave action balance 22

3.2.1 Stationary solver 22

3.2.2 Nonstationary solver 22

3.3 Shallow water equations 22

3.4 Groundwater flow 23

3.4.1 Hydrostatic 23

3.4.2 Non-hydrostatic 23

3.5 Advection-diffusion equation for sediment 27

3.6 Bottom updating schemes 27

3.7 Avalanching 27

3.8 Bed composition 27

4 Boundary conditions 28

4.1 Waves 28

4.1.1 Spectra 28

4.1.2 Non-spectra 29

4.1.3 Lateral boundary conditions 29

4.2 Shallow water equations 29

4.2.1 Absorbing-generating 29

4.2.2 River and point discharge 29

4.2.3 Ship motion 29

4.2.4 Lateral boundaries 30

4.2.5 Tide and surge 30

4.3 Sediment transport 31

5 Input description 32

5.1 General 32

5.2 Grid and bathymetry 32

5.3 Wave input 32

5.4 Tide and surge input 32

5.5 Water level (dam break) 32

5.6 Wind input 32

5.7 Sediment input 32

5.8 Output selection 32

5.9 Time parameters 32

5.10 Model coefficients 32

6 Bibliography 33

7 Tutorial 35

7.1 1-D profile model 35

7.2 2-D area model 35

7.3 Langsgetij + riveroutflow 35

8 Appendices 36

8.1 Numerical implementation non-hydrostatic module 36

8.1.1 Global continuity equation 36

8.1.2 Local continuity equation 37

8.1.3 Horizontal Momentum 37

8.1.4 Vertical momentum equations 40

# Introduction

Dano, Ad, Ap

# Processes and model formulation

## Domain and definitions

Dano - overnemen en nieuw plaatje curvi

## Hydrodynamics options

Dano

### Stationary mode

### Non-stationary (surfbeat) mode

### Wave resolving mode

## Short wave propagation

### Wave action balance

The wave forcing in the shallow water momentum equation is obtained from a time dependent version of the wave action balance equation. Similar to Delft University’s (stationary) HISWA model (Holthuijsen et al., 1989) the directional distribution of the action density is taken into account whereas the frequency spectrum is represented by a frequency, best represented by the spectral parameter *fm-1,0*.The wave action balance is then given by:



In which the wave action *A* is calculated as:



In *θ* represents the angle of incidence with respect to the x-axis, *Sw* represents the wave energy density in each directional bin and *σ* the intrinsic wave frequency. The wave action propagation speeds in x- and y-direction are given by:



With *uL* and *vL* the cross-shore and alongshore depth-averaged Lagrangian velocities respectively (defined below), and the group velocity cg obtained from linear theory. If wave-current interaction is turned off (keyword: *wci=0*) then the last term in either equation is not taken into account. The propagation speed in θ-space is obtained from:



In *h* represents the total water depth and in this formulation bottom refraction (first term) and wave-current interaction (last two terms) are taken into account. If wave-current interaction is turned off (keyword: *wci=0*) then the last two terms are neglected.

The wave number *k* is obtained from the eikonal equations that is described in . In this formulation the subscripts refer to the direction of the wave vector components and *ω* represents the absolute radial frequency.



The wave number is then obtained from .



The absolute radial frequency *ω* is given by . The intrinsic frequency *σ* is obtained from the linear dispersion relation. If wave-current interaction is turned off (*wci=0*) then the last two terms are not taken into account.



### Dissipation

The set of equations of the wave action balance closes with dissipation terms. In XBeach there are three dissipative terms: wave breaking, bottom friction and vegetation. Given the spatial distribution of the wave action (and therefore wave energy) the radiation stresses can be evaluated by using linear wave theory as described in:



#### Breaking

There are in four different wave breaking formulations implemented in XBeach. The formulations are coded with the keyword *break*.

1. Non-stationary waves: formulation of Roelvink (1993a)
2. Stationary waves: formulation of Baldock et al. (1998)
3. Non-stationary waves: adaptation of break=1
4. Non-stationary waves: adaptation of break=1 (Daly et al. ,2010)

For the non-stationary (surf beat) approach the total wave energy dissipation, i.e. directionally integrated, due to wave breaking is modelled according to Roelvink (1993a). This is coded as *break=1*. In *α* is applied as wave dissipation coefficient of O(1), *Qb* is the fraction breaking waves, *p* stands for the water density and *γ* is the breaker index. The total wave energy *Ew* is calculated by integrating over the wave direction per directional bin.



In variation of , one could also use the third wave breaking formulation, presented in . This formulation is somewhat different than the formulation of Roelvink (1993a). This is coded as *break=3.*



On top of that, Daly et al. (2010) developed a formulation presented in , which states that waves are fully breaking if the wave height exceeds a threshold (*γ*) and stop breaking if the wave height fall below another threshold (*γ2*). This is coded as break*=4*.



In the stationary case Baldock et al. (1998) is applied, which is presented in . In this breaking formulation the fraction breaking waves *Qb* and breaking wave height *Hb* is calculated differently compared to the breaking formulations used for the non-stationary situation. In *α* is applied as wave dissipation coefficient, *frep* represents a representative intrinsic frequency and *y* is a calibration factor. The stationary wave breaking formulation is coded with *break=4*.



In either the non-stationary or stationary case the total wave dissipation is distributed proportionally over the wave directions with the formulation in .



#### Bottom friction

The bottom friction dissipation is modelled as



Robert gebruikt Chezy voor turbulente stroming. Hoe verhoudt dit zich met huidige formuleringen?

#### Vegetation

Arnold

### Roller energy balance

Dano

## Shallow water equations

Kees

For the low-frequency and mean flows we use the shallow water equations. To account for the wave induced mass-flux and the subsequent (return) flow these are cast into a depth-averaged Generalized Lagrangian Mean (GLM) formulation (Andrews and McIntyre, 1978, Walstra et al, 2000). In such a framework, the momentum and continuity equations are formulated in terms of the Lagrangian velocity *uL* which is defined as the distance a water particle travels in one wave period, divided by that period. This velocity is related to the Eulerian velocity (the short-wave-averaged velocity observed at a fixed point) by:



In uS and vS represents the Stokes drift in x- and y-direction respectively (Phillips, 1977). The Strokes drift is calculated with in which the wave-group varying short wave energy *Ew* and direction are obtained from the wave-action balance.



The resulting GLM-momentum equations are given by:



In *τbx*and *τby*are the bed shear stresses, *η* is the water level, *F* are the wave-induced stresses, *v* is the horizontal viscosity and *f* is the Coriolis coefficient. The bottom shear stress terms (*τsx*and *τsy*) are calculated with the Eulerian velocities as experienced by the bed and not with the GLM velocities. Also, the boundary conditions for the flow computations are expressed in function of Lagrangian and not Eulerian velocities.

The horizontal viscosity (*vh*) is by default computed using the Smagorinsky (1963) model to account for the exchange of horizontal momentum at spatial scales smaller than the computational grid size, which is given as:



In cS is the Smagorinsky constant, set at 0.1 in all model simulations. It is also possible to use a user-defined value for the horizontal viscosity (keyword *smag = 0*).

## Nonhydrostatic pressure correction

Kees / Robert

For non-hydrostatic XBeach calculations (keyword *nonh=1*) depth-averaged flow due to waves and currents are computed using the non-linear shallow water equations, including a non-hydrostatic pressure. The depth-averaged normalized dynamic pressure (*q*) is derived in a method similar to a one-layer version of the SWASH model (Zijlema et al. 2011). The depth averaged dynamic pressure is computed from the mean of the dynamic pressure at the surface and at the bed by assuming the dynamic pressure at the surface to be zero and a linear change over depth. In order to compute the normalized dynamic pressure at the bed, the contributions of advective and diffusive terms to the vertical momentum balance are assumed to be negligible.



In *w* is the vertical velocity and *z* is the vertical coordinate. The vertical velocity at the bed is set by the kinematic boundary condition, in which :



Combining the Keller-box method (Lam and Simpson 1976), as applied by Stelling and Zijlema (2003) for the description of the pressure gradient in the vertical, the dynamic pressure at the bed can be described by:



Substituting in allows the vertical momentum balance at the surface to be described by:



In the subscript *s* refers to the location at the surface. The dynamic pressure at the bed is subsequently solved by combining and the local continuity equation:



Smit et al. (2010) have shown that the inclusion of the dynamic pressure described above reduces the relative dispersion and celerity errors in the non-linear shallow water equations of XBeach to less than 5% for values of kh<2.5 and allows for accurate modelling over wave transformation on dissipative beaches. In order to improve the computed location and magnitude of wave breaking, we apply the hydrostatic front approximation (HFA) of Smit et al. (2013), in which the pressure distribution under breaking bores is assumed to be hydrostatic. Following the recommendations of Smit et al. (2013), we consider hydrostatic bores if  and reform if .

Although this method greatly oversimplifies the complex hydrodynamics of plunging waves on, McCall et al. (2014) shows that the application of this method provides sufficient skill to describe dominant characteristics of the flow, without requiring computationally-expensive high-resolution discretisation of the vertical and surface tracking of overturning waves.

Discussion - extend derivation to 2D?

## Groundwater flow

Kees/Robert

### Hydrostatic

The hydrostatic groundwater module in XBeach utilizes the principle of Darcy flow and is therefore limited to laminar flow conditions. The module includes a vertical interaction flow between the surface water and groundwater. This flow is assumed to be a magnitude smaller than the horizontal flow and is not incorporated in the momentum balance. Darcy flow is described by the following relationship between the groundwater head gradient *dpgw/dx* and *dpgw/dy*, the permeability *k*, and the horizontal velocity, as can be seen in .



#### Determining groundwater head

The driving force behind groundwater flow according to Darcy is the groundwater head gradient. In the XBeach module, the groundwater head *pgw* is expressed in meter and basically there are two possibilities in the model:

1. There is no surface water, than the groundwater head is equal to the groundwater surface level *ηgw*
2. There is surface water and the groundwater surface level is just below the surface of the bed *zb*. This means the groundwater head is affected by the surface water head *zs*
   * If the groundwater surface level is equal to the bed level, the groundwater head is equal to the surface water head.
   * If the groundwater surface level is more than *dwetlayer* below the surface of the bed, the groundwater head is unaffected by the surface water head and is equal to the groundwater surface level.
   * At intermediate depths a linear interpolation takes place, using the relative groundwater level *fac*.

#### Determining vertical flow

In order to simulate the interaction between the surface water and groundwater, a vertical flow between the surface water layer and groundwater layer (*w*) is introduced. This flow has the unit of m/s and is defined positive from the surface water to ground water and is given in terms of surface water for the continuity equation (i.e. 100% porosity).

Exfiltration, or flow from the groundwater layer to the surface water layer, takes place if the groundwater surface level exceeds the bed level. The volume of groundwater (including porosity) exceeding the bed level is joins the surface water within the same numerical time step. The vertical velocity can therefore be calculated by:



Surface water running up and down a dry slope will infiltrate into the ground. In order to model this fully, a 3D model must be used. In the XBeach groundwater module, the option is made to model infiltration using a quasi-3D model.

In areas where there is surface water and the groundwater level is not greater than the bed level, infiltration can take place. To a certain degree of truth, infiltration can be calculated using Darcy flow.



In an area that is covered by surface water, the head on the top of the bed can be said to be equal to the surface water head. In the absence of groundwater at the bed level, the head under the bed level is zero. As the distance between the top and bottom of the bed level is zero, the head gradient is infinite. The resulting vertical velocity becomes infinite and the method becomes numerically unstable. In order to circumvent this problem the vertical infiltration is divided into an instantaneous, but finite reaction in the upper ground layer and Darcy flow across a non-zero depth. The proportion of the instantaneous part to the Darcy flow part is governed by the relative groundwater level *fac*. The instantaneous part is handled in the same way as exfiltration. The head gradient for the Darcy flow is found by assuming the head at the bottom of the infiltration layer is zero, and the head on the top of the infiltration layer is equal to the height of water standing on the bed (*zs-zb*).

The thickness of the infiltration layer (*dinfiltration*) is increased at the end of every time step by the infiltrating water. The infiltration speed in the next time step will therefore be less than that in the current time step. Infiltrating water is assumed to immediately become part of the groundwater for the purpose of groundwater level and groundwater head calculations. This approach is therefore not fully 3D and only uses a quasi-3D approximation to limit the infiltration speed.



For numerical stability, the infiltration layer thickness is restricted to a minimum of one third of (*dinfiltration*), corresponding with the centroid of the instantaneous infiltration part. The maximum thickness of the infiltration layer is equal to the depth of the groundwater level below the bed level. Once an area has no surface water, the thickness of the infiltration layer is reset to the minimum value, representing the fact that the infiltrated water has sunk out of the way of subsequent infiltrations.

#### Mass balance

The continuity equation for the groundwater system can be written as:



The effective depths through which horizontal ground water flow takes place (*hugw, hvgw*), are found by taking the mean difference between the groundwater level and bed of the aquifer (*zb,acquifir*) in the two surrounding points. This method is faster, but less momentum conservative than the method used in the surface water flow routine. Since large gradients in the groundwater level are not expected, the scheme is assumed sufficient. Groundwater flux is limited in cells that are empty of groundwater. For such cells, groundwater may enter the cell, but no groundwater may leave until the amount of groundwater exceeds a minimum value (*eps*).

#### Boundary conditions

* Vertical boundary conditions: the groundwater level is bounded by the bottom of the aquifer. In the central domain the groundwater level is adjusted naturally by infiltration and exfiltration. The groundwater level has no bounding maximum in the vertical, except on the offshore, bay side and lateral boundaries. Here the groundwater level is bounded vertically by the bed level on the boundaries. The bed of the aquifer is set equal to or less than the regular bed level.
* At the offshore boundary: the groundwater head is set equal to the offshore surface water head.
* Bay side conditions: for cases in which a bay side water level is given explicitly with a tidal level record, the groundwater head on the bay side boundary is set equal to the bay side surface water head. In all other cases, the bay side groundwater head is kept at the initial value.
* Lateral boundary conditions: Neumann boundary conditions are applied to the groundwater head on the lateral boundaries:

The initial groundwater level is calculated from the initial groundwater head. The bed of the aquifer and the initial groundwater head must be specified.

### Non-hydrostatic

Groundwater flow in the swash and surf zone has been shown in previous numerical model (e.g., Li and Barry 2000; Lee et al. 2007) studies to be non-hydrostatic. Therefore, a requirement of the groundwater model is that it does not use the Dupuit–Forchheimer assumption of hydrostatic groundwater pressure. Although the requirement for non-hydrostatic pressure has the benefit of being a more accurate representation of reality, resolving the non-hydrostatic pressure field can be very computationally expensive.

In order to allow for a computationally efficient approximation of the non-hydrostatic groundwater pressure field, XBeach applies a quasi 3D-method to predict depth-averaged horizontal groundwater fluxes, vertical distribution of the groundwater pressure and the flow driven by groundwater-surface water pressure gradients (submarine exchange).

#### Equation of motions

Laminar flow of an incompressible fluid through a homogeneous medium can be described using the well-known Law of Darcy (1856). In which *K* is the hydraulic conductivity of the medium and *H* is the hydraulic head. However, in situations in which flow is not laminar, turbulent and inertial terms may become important, this relation is no longer valid. In these cases groundwater flow should be described using the extended Forchheimer equation.

In XBeach, however, a method comparable with the USGS MODFLOW-2005 groundwater model (Harbaugh 2005), in which the turbulent hydraulic conductivity is estimated based on the laminar hydraulic conductivity (*Klam*) and the Reynolds number at the start of turbulence (*Recrit*) (Halford 2000)



In the Reynolds number (*Re*) is calculated using the median grain size (*D5*0), the kinematic viscosity of water (*n*) and the groundwater velocity in the pores. Since the hydraulic conductivity in the turbulent regime is dependent on the local velocity, an iterative approach is taken to find the correct hydraulic conductivity and velocity.

#### Vertical groundwater head approximation

Since XBeach is depth-averaged, the model cannot compute true vertical profiles of the groundwater head and velocity. In order to improve the estimate of the groundwater head variation over the vertical, a quasi-3D modelling approach is applied, which is set by three conditions:

1. There is no exchange of groundwater between the aquifer and the impermeable layer below the aquifer
2. The groundwater head at the upper surface of the groundwater is continuous with the head applied at the surface
3. The vertical velocity is assumed to increase or decrease linearly from the bottom of the aquifer to the upper surface of the groundwater:

The vertical groundwater head approximation can be solved for the three imposed conditions by a parabolic function. The depth-average value of the groundwater head is used to calculate the horizontal groundwater flux and is found by integrating the groundwater head approximation over the vertical:



In the mean vertical ground water head (*H*) is calculated using the groundwater head imposed at the groundwater surface (*Hbc*), the groundwater head parabolic curvature coefficient (*β*) and the height of the groundwater level above the bottom of the aquifer (*hgw*).

#### Exchange with surface water

In the groundwater model there are three mechanisms for the exchange of groundwater and surface water: 1) submarine exchange, 2) infiltration and 3) exfiltration. The rate of exchange between the groundwater and surface water (*S*) is given in terms of surface water volume, and is defined positive when water is exchanged from the surface water to the groundwater. The groundwater and surface water are said to be in a connected state where and when the groundwater level reaches to the top of the bed and surface water exists above the bed. This state is described by a spatially and temporally varying logical *k*, which is true where groundwater and surface water are connected and false in all other situations.

Submarine exchange represents the high and low frequency infiltration and exfiltration through the bed due pressure gradients across the saturated bed. This process only takes place where the groundwater and surface water are connected. The rate of submarine exchange is determined by the vertical specific discharge velocity at the interface between the groundwater and surface water. The value of this velocity can be found using the vertical derivative of the approximated groundwater head at the groundwater-surface water interface.



Infiltration and exfiltration can only occur in locations where the groundwater and surface water are not connected. Infiltration takes place when surface water covers an area in which the groundwater level is lower than the bed level. The flux of surface water into the bed is related to the pressure gradient across the wetting front.



In the surface water-groundwater exchange flow of infiltration (*Sinf*) is calculated using the effective hydraulic conductivity (*K*), the surface water pressure at the bed () and a thickness of the wetting point (**).

Since the groundwater model is depth-averaged and cannot track multiple layers of groundwater infiltrating into the bed, the wetting front thickness is reset to zero when there is no available surface water, the groundwater exceeds the surface of the bed, or the groundwater and the surface water become connected. In addition, all infiltrating surface water is instantaneously added to the groundwater volume, independent of the distance from the bed to the groundwater table. Since the groundwater model neglects the time lag between infiltration at the beach surface and connection with the groundwater table a phase error may occur in the groundwater response to swash dynamics

Exfiltration occurs where the groundwater and surface water are not connected and the groundwater level exceeds the bed level (Figure 3.4b). The rate of exfiltration is related to the rate of the groundwater level exceeding the bed level.



#### Calculation of groundwater and surface water levels

The curvature coefficient (*β*) in the vertical groundwater head approximation is solved using the coupled equations for continuity and motion, thereby producing the depth-average horizontal groundwater head gradients and vertical head gradients at the groundwater surface, and subsequent depth-average horizontal and vertical specific discharge.

In areas where the groundwater and surface water are not connected, the groundwater level change is related to the vertical specific discharge and the infiltration and exfiltration fluxes:



In these same areas the surface water level is modified to account for infiltration and exfiltration:



In areas where the groundwater and surface water are connected, the groundwater level remains at the level of the bed, since the computed vertical velocity at the surface (w) is exactly equal and opposite to the submarine exchange (Ssub). The surface water level is modified to account for the submarine exchange with the groundwater:



In cases where there is not sufficient surface water to permeate into the bed to ensure the groundwater level remains at the bed level, a fractional time step approach is taken in which the area is considered to be connected while there is sufficient surface water, and considered unconnected once the surface water has drained away. A similar approach is taken when the groundwater level reaches the bed level during an infiltration event.

#### Boundary conditions

Since the groundwater dynamics are described by a parabolic equation, the system of equations requires boundary conditions at all horizontal and vertical boundaries, as well as an initial condition

* At the horizontal boundaries and bottom of the aquifer: a zero flux condition is imposed. This is based on the assumption that the groundwater head is constant
* At the surface of the groundwater
  + If connected: the head is set to the surface water head at the bed
  + If not-connected: the head is equal to the atmospheric pressure head
* The initial condition for the solution is specified by the model user in terms of the initial groundwater head.

## Sediment transport

### Advection-diffusion

Sediment concentrations in the water column are modelled using a depth-averaged advection-difussion scheme with a source-sink term based on an equilibrium sediment concentration (Galappatti and Vreugdenhi, 1985):



In *C* represents the depth-averaged sediment concentration which varies on the wave-group time scale and *Dh* is the sediment diffusion coefficient. The entrainment of the sediment is represented by an adaptation time *Ts*, given by a simple approximation based on the local water depth *h* and sediment fall velocity *ws*. A small value of *Ts* corresponds to nearly instantaneous sediment response.



The entrainment or deposition of sediment is determined by the mismatch between the actual sediment concentration *C* and the equilibrium concentration *Ceq* thus representing the source term in the sediment transport equation.

### General parameters: velocity magnitude and orbital velocity

In transport formulations the equilibrium sediment concentration *Ceq*(for both the bed load and the suspended load) is related to the velocity magnitude (*vmg*) and the orbital velocity (urms). This section elaborates how we calculate both terms.

First of all the velocity magnitude, if long wave stirring is turned on (keyword: *lws=1*), the velocity magnitude *vmg* is equal to the magnitude of the Eulerian velocity, as can be seen in .



If wave stirring is turned off (*keyword: lws=0*), the velocity magnitude will be current-averaged on time scale based on a certain factor *fcats* of the representative wave period *Trep*.



Secondly the root-mean-squared velocity, the urms is obtained from the wave group varying wave energy using linear wave theory. This formulation can be found in .



To take into account for wave breaking induced turbulence due to short waves, the orbital velocity is adjusted (van Thiel de Vries, 2009). In this formulation *kb* is the wave breaking induced turbulence due short waves. The turbulence is approximated with an empirical formulation in XBeach.



### Transport formulations

In the present version of XBeach, two sediment transport formulations are available. The formulae of the two formulations are presented in the following sections. For both methods the total equilibrium sediment concentration is calculated with .



#### Soulsby-Van Rijn

The Soulsby-Van Rijn transport equations are known as (Soulsby, 1997; van Rijn, 1984):



For which the bed-load and suspended load coefficient are calculated with:



The critical velocity defines at which depth averaged velocity sediment motion is initiated:



Finally the drag coefficient is calculated with:





#### Van Thiel-Van Rijn

The Van Thiel-Van Rijn transport equations are known as (van Rijn, 2007; van Thiel de Vries, 2009):



For which the bed-load and suspended load coefficient are calculated with:



The critical velocity is computed as weighted summation of the separate contributions by currents and waves (Van Rijn, 2007).



The critical velocity for currents is based on Shields (1936)



The critical velocity for waves is based on Komer and Miller (1975)



### Wave asymmetry

The wave asymmetry enters the advection-diffusion equation, repeated here:



XBeach considers the wave energy of short waves as averaged over their length, and hence does not simulate the wave shape. A discretization of the wave skewness and asymmetry was introduced by Van Thiel de Vries (2009), to affect the sediment advection velocity. In this equation *ua* is calculated as function of wave skewness (*Sk*), wave asymmetry parameter (*Sk*), root-mean square velocity *urms* and a calibration factor *fua* (keyword: *facua*).



The skewness and asymmetry as parameterized as a function of the Ursell number by Ruessink et al. (2012).



## Bottom updating

### Due to sediment fluxes

Kees

Based on the gradients in the sediment transport the bed level changes according to:



In *ρ* is the porosity, *fmor* is a morphological acceleration factor of O(1-10) (Reniers et al., 2004) and *qx* and *qy* represent the sediment transport rates in x- and y-direction respectively. In order to take account for bed-slope effects on sediment transport a bed-slope correction factor *fslope* is introduced.



### Avalanching

Kees + Pieter

To account for the slumping of sandy material during storm-induced dune erosion avalanching is introduced to update the bed evolution. Avalanching is introduced via the use of a critical bed slope for both the dry and wet area (keyword: *wetslp* and *dryslp*). It is considered that inundated areas are much more prone to slumping and therefore two separate critical slopes for dry and wet points are used. The default values are 1 and 0.3 respectively. When this critical slope is exceeded, material is exchanged between the adjacent cells to the amount needed to bring the slope back to the critical slope.



The change of the bed level within one time step is then given by . In this formulation a threshold of 0.05 m/s has been introduced to prevent the generation of large shockwaves.



### Bed composition

Bas

# Numerical implementation

Dano behalve 3.4,3.8

## Grid types

### 1D

### Rectilinear

### Curvilinear

## Wave action balance

### Stationary solver

### Nonstationary solver

## Shallow water equations

## Groundwater flow

Kees / Robert

### Hydrostatic

Not in manual yet, to-do…

### Non-hydrostatic

In order to solve the equations in xx, the spatial and temporal domain of the groundwater system is split into the same spatial grid and time steps as the XBeach surface water model it is coupled to. At each time step in the numerical model, the depth average groundwater head is calculated in the centre of the groundwater cells, and the fluxes (specific discharge, submarine exchange, infiltration and exfiltration) are calculated on the cell interfaces

#### Infiltration and exfiltration

At the start of the time step, every cell is evaluated whether the groundwater and surface water are connected:



In *ε* is a numerical smoothing constant used to deal with numerical round off errors near the bed, and *i* and *j* represent cross-shore and longshore coordinates in the numerical solution grid, respectively. Infiltration is calculated in cells where the groundwater and surface water are not connected and there exists surface water. As shown in the infiltration rate is a function of the thickness of the wetting front, which is zero at the start of infiltration, and increases as a function of the infiltration rate. The equations for the infiltration rate and the thickness of the wetting front are approximated by first-order schemes, in which the wetting front is updated using a backward-Euler scheme, which ensures numerical stability:



In the superscript *n* corresponds to the time step number and *Δt* is the size of the time step. The infiltration rate in the coupled relationship can be solved through substitution:



At the end of infiltration, i.e. when the groundwater and surface water become connected or there is no surface water left, the wetting front thickness is reset to zero. If the infiltration rate exceeds the Reynolds number for the start of turbulence, the local hydraulic conductivity is updated using the local Reynolds number:



Xbeach iterates until a minimum threshold difference between iterations is found for and . Infiltration in one time step is limited to the amount of surface water available in the cell and to the amount of water required to raise the groundwater level to the level of the bed:



If during infiltration the groundwater level reaches the bed level, the fraction of the time step required to do so is estimated (x) and the remaining fraction is used in the submarine exchange.



Exfiltration is calculated in cells where the groundwater and surface water are not connected and the groundwater level exceeds the bed level:



After infiltration and exfiltration have been calculated, the groundwater level and surface water level are updated:



All updated cells are subsequently re-evaluated on whether the surface water and groundwater are connected or unconnected

#### Horizontal flow and submarine exchange

The horizontal specific discharge on each cell interface can be found through an approximation of the groundwater head gradient:



In the superscripts *x* and *y* refer to the components of the variable in the crossshore and longshore direction, respectively, the subscripts *u* and *v* refer to variables approximated at the horizontal cell interfaces in the cross-shore and longshore direction, respectively, and the subscript *H* refers to variables approximated at the cell centers. The hydraulic conductivity may be different at each cell interface and is therefore computed at every interface where every K is calculated separately. The cell height at the centre of the groundwater cells (*ΔzH,i,j*) is calculated from the groundwater level and the bottom of the aquifer in the centre of the cell, whereas the cell heights at the horizontal cell interfaces are calculated using an upwind procedure:



As described in Section 3.3.6, the head applied on the top boundary of the groundwater domain (*Hbc*) depends on whether the groundwater and surface water are connected or unconnected:



The vertical submarine exchange at the top of the numerical groundwater cell, is found with



In the superscript *z* refers to the vertical component of the variable, the subscript *w* refers to a numerical approximation at the vertical cell interfaces.

Continuity in the groundwater cell is found following



All variables in contain an unknown value for the groundwater pressure head, described in terms of a known head at the surface of the groundwater (*Hbc*) and the unknown curvature of the vertical groundwater head function (*β*). Since water is incompressible, the groundwater pressure must be solved for all cells simultaneously using matrix algebra:



In A is a matrix containing coefficients for the horizontal and vertical specific discharge, x is a vector containing the unknown groundwater head curvature, and b contains the known forcing terms. For a one dimensional cross-shore case, A is reduced to a tridiagonal matrix. The vector of known forcing consists of the numerical gradients in the contribution of the head applied on the top boundary of the groundwater domain to the horizontal specific discharge.

In the one dimensional case, the solution to the tridiagonal matrix A can be computed using the efficient Thomas algorithm (Thomas 1949). In the two dimensional case, matrix A contains two additional diagonals that are not placed along the main diagonal, and vector b contains additional forcing terms from the alongshore contribution. The solution to the two dimensional case requires a more complex and less computationally efficient matrix solver. In this case the Strongly Implicit Procedure (Stone 1968) is used in a manner similar to Zijlema et al. (2011).

The horizontal and vertical groundwater fluxes are calculated using the solution of *x* plus and . Since some local velocities may exceed the critical Reynolds number for the start of turbulence (*Recrit*), the turbulent hydraulic conductivity (*K*) is updated using the local Reynolds number. The solution to and the update of the turbulent hydraulic conductivity are iterated until a minimum threshold difference between iterations is found.

The iterated solution for the specific vertical discharge is used to update the groundwater level and surface water level:



If the groundwater and surface water are connected, and the submarine exchange from the surface water to the groundwater estimated in is greater than the amount of surface water available in the cell, continuity is enforced by lowering the groundwater level to compensate for the lack of permeating water:



## Advection-diffusion equation for sediment

## Bottom updating schemes

## Avalanching

## Bed composition

Bas

# Boundary conditions

## Waves

XBeach allows users to include two different options for wave boundary conditions in the model. First of all, in **Error! Reference source not found.** the method to specify spectra is discussed. Secondly, in 4.1.1 the method to apply non-spectra is elaborated. In 4.1.3 the lateral boundaries for the waves are discussed.

### Spectra

Kees, Ap review

XBeach allows the user to define a wave spectrum. There are 3 possibilities:

1. Spectral parameter input: The user needs to provide JONSWAP parameters in order to let XBeach determine a spectrum and thus wave energy (keyword *instat=jons*).
2. SWAN spectrum input: Wave energy is determined directly from standard .sp2 files (keyword *instat=swan*)
3. Formatted variance density spectrum: other than 1) and 2) (keyword *instat=vardens*)

In general XBeach will then attempt to read the parameters / entire spectrum from a separate file specified (keyword *bcfile:'file.txt'*). The user must also state in params.txt the required record length for the boundary condition file (keyword *rt:<number>*) and the boundary condition file time step (keyword *dtbc:<number>*). If the record length is less than the total simulation time, XBeach will reuse the boundary condition file until the simulation is completed. The boundary condition file time step should be small enough to accurately represent the bound long wave, but need not be as small as the time step used in XBeach.

XBeach assumes the output of the SWAN file is in nautical terms. If the file is in Cartesian angles, the user must specify the angle in degrees to rotate the x-axis in SWAN to the x-axis in XBeach (in Cartesian terms). This value need to be specified (keyword *dthetaS\_XB:<number>*).

For the user-defined spectrum it is important to note that the angles in the input file must be in the calculation coordinate system of XBeach, i.e. 0° is in the direction of the x-axis, 90° is in the direction of the y-axis. Also, the angles must be increasing.

On top of that XBeach offers the possibility of both a time-varying (*filelist*) and space-varying spectra (*loclist*). This is a possibility which can be applied for JONSWAP, SWAN and user-formatted variance density spectra. More information about the input description can be found in 5.3.

If the user does not wish to recalculate boundary condition files or specifically wants to reuse the boundary condition files of another XBeach simulation (keyword: *instat=reuse*). No further wave boundary condition data need be given. Obviously, the calculation grid should remain the same between runs, as the angles and number of grid points are embedded in the boundary condition files.

Discussion:

* Exclude jons\_table option in manual, since one can do so via a filelist?
* Which time step for dtbc is recommended?

### Non-spectra

Kees, Ap review

XBeach also allows the user to define non-spectral wave boundary conditions. There are 4 possibilities to do so:

1. Stationary wave boundary condition. This means that a uniform and constant wave energy distribution is set, based on the given values of Hrms, Tm01, direction and power of the directional distribution function.
   1. One sea state without wave groups (keyword *instat=stat*)
   2. Specify time series of sea states without wave groups (keyword *instat=stat\_table*)
   3. Bichromatic (two wave component) waves (keyword *instat=bichrom*). XBeach will be forced with regular wave groups. The user needs to specify on top of that variables of the stationary situation also a wave period for the long wave. This wave period will be used to calculate the long wave based on the theory of Longuet-Higgins and Stewart’s (1964).
2. Time series of waves. The user can specify the variation in time of the wave energy.
   1. First-order time series of waves (keyword *instat=ts\_1*). XBeach will calculate the bound long wave based on the theory of Longuet-Higgins and Stewart’s (1964).
   2. Second-order time series of waves (keyword *instat=ts\_2*). The bound long wave is specified by the user via a long wave elevation.
3. Boundary conditions for non-hydrostatic (keyword *instat=ts\_nonh*)

Specify the variation in time of the horizontal velocity, vertical velocity and the free surface elevation. Last two terms are optional.

1. No wave boundary conditions (keyword *instat=off)*

This is a simple no wave action boundary condition. It still allows for a tidal record to be specified, however this trough the zs0file parameter.

If the user does not wish to recalculate boundary condition files or specifically wants to reuse the boundary condition files of another XBeach simulation (keyword: *instat=reuse*). No further wave boundary condition data need be given. Obviously, the calculation grid should remain the same between runs, as the angles and number of grid points are embedded in the boundary condition files.

### Lateral boundary conditions

Dano

## Shallow water equations

### Absorbing-generating

Ap met appendix

### River and point discharge

Bas

### Ship motion

Dano

### Lateral boundaries

Kees

Lateral boundaries are the boundaries perpendicular to the coastline. Usually these are artificial, because the model domain is limited but the physical coast will continue. At these boundaries we need to prescribe information about the area beyond the numerical model domain in such a way that the boundary condition does not influence the results in an adverse way. The best way (XBeach default) to do this is to prescribe so-called “no-gradient” or Neumann boundaries, which state that there is locally no change in surface elevation and velocity.

These boundary conditions are activated where the longshore water level gradient is prescribed. The alongshore gradient is prescribed by the difference in specified water levels at the offshore corner points, divided by the alongshore length of the domain. This type of Neumann boundary condition has been shown to work quite well with (quasi-) stationary situations, where the coast can be assumed to be uniform alongshore outside the model domain. So far we have found that also in case of obliquely incident wave groups this kind of boundary conditions appears to give reasonable results when a shadow zone is taken into account. This means that regions where the boundary conditions are not fully enforced the results are not taken into account.

Neumann boundaries can be individually defined (left=0 and right=0). Simple no-flux boundary conditions (walls) can also be applied (left=1 and right=1).

Remarks Kees:

* Is this still true? ‘Wall boundary conditions are preferred over Neumann boundary conditions in 1D (cross-shore) models.’
* Development to-do: cyclic lateral boundaries

### Tide and surge

Kees

XBeach can take in up to four time-vary tidal signals to be applied to the four boundaries (offshore-left, backshore-left, backshore-right, offshore-right). A time-varying water level signal is read into XBeach by reading the specified file in zs0file. The input signal will be interpolated to the local time step of the simulation; therefore the signals only need to be long enough and temporally-fine enough to resolve the water level phenomenon of interest (i.e. tide variations, surge event).

There are now four options for handling the tidal and/or surge contribution to the boundaries:

* Uniform water level (keyword: *tideloc=0*)
* One time-varying water level signal (keyword: *tideloc=1*)
* Two time-varying water level signals, which requires point of application indication. (keyword: *tideloc=2*)
* Four time-varying water level signals (keyword: *tideloc=4*)

For the option with a uniform water level the value specified in the params.txt is applied in the complete model domain (keyword: *zs0=’value’*). For the option with one time-varying water level signal the specified water level is applied (keyword: *zs0file = name\_of\_your\_time\_serie*) to the offshore boundary and a fixed value is applied at the backshore boundary (keyword: *zs0=value*). For the option with two time-varying water level signals two water level signals are read from the zs0file. Note: one tidal record is applied to both sea corners and one tidal record to both land corners. This means there is no alongshore variation. An alongshore variation can be applied when applying four time-varying water level signals.

Orientation must be addressed in the input description!

## Sediment transport

Dano

# Input description

Bas - params en attribute files

## General

## Grid and bathymetry

## Wave input

## Tide and surge input

## Water level (dam break)

## Wind input

## Sediment input

## Output selection

## Time parameters

## Model coefficients

# Bibliography

Holthuijsen, L., Booij, N., & Herbers, T. (1989). A prediction model for stationary, short-crested waves in shallow water with ambient currents. Coastal Engineering, 13(1):23-54.

Roelvink, J.A. (1993a) Dissipation in random wave groups incident on a beach. Coastal Engineering, pp. 127-150.

Roelvink, J.A. (1993b) Surf beat and its effect on cross-shore profiles. Ph.D. Thesis, Delft University of Technology.

Daly, C., Roelvink, J. A., Van Dongeren, A., Van Thiel de Vries, J. S. M., & McCall, R. (2010). Short wave breaking effects on low frequency waves. Proceedings of 32nd International Conference on Coastal Engineering, (1), 1–13.

Walstra, D. J. R., Roelvink, J., and Groeneweg, J. (2000). 3D calculation of wave-driven cross-shore currents. In Proceedings 27th International Conference on Coastal Engineering, pages 1050-1063

Phillips, O. (1977). The dynamics of the upper ocean. Cambridge University Press, page 366

Andrews, D. G.; McIntyre, M. E. (1978a), "An exact theory of nonlinear waves on a Lagrangian-mean flow", Journal of Fluid Mechanics 89 (4): 609-646

Reniers, A.J.H.M., J.A. Roelvink and E.B. Thornton. (2004). Morphodynamic modelling of an embayed beach under wave group forcing. J. of Geophysical Res. , VOL. 109, C01030, doi:10.1029/2002JC001586, 2004

Reniers, A.J.H.M., E.B. Thornton, T. Stanton and J.A. Roelvink. (2004b) Vertical flow structure during Sandy Duck: Observations and Modeling. Coastal Engineering, Volume 51, Issue 3, May 2004, Pages 237-260

Ruessink, B.G., Ramaekers, G. and van Rijn, L.C., 2012. On the parameterization of the free-stream non-linear wave orbital motion in nearshore morphodynamic models. Coastal Engineering, 65, 56-63.

Van Rijn, L. (2007). Unified View of Sediment Transport by Currents and Waves . part I, II, III, IV.Journal of Hydraulic Engineering, (June):649–667.

Van Thiel de Vries, J. S. M. (2009). Dune erosion during storm surges. PhD thesis, Delft University of Technology, Delft.

Soulsby, R. (1997). Dynamics of Marine Sands. Thomas Telford Publications, London. ISBN 0 7277 2584 x.

van Rijn, L.C. (1984). Sediment transport, part iii: Bed forms and alluvial roughness. Journal of Hydraulic Engineering, 110(12):1733-1754

Shields, A. (1936). Anwendung der ahnlichkeits-Mechanik und der Turbulenz-forschung auf die Geschiebebewegung. Preussische Versuchanstalt fur Wasserbrau und Schiffbau, 26:524-526

Komar, P.D. and Miller, M.C. (1975). On the comparison between the threshold of sediment motion under waves and unidirectional currents with a discussion of the practical evaluation of the threshold; reply. Journal of Sedimentary Research, 45(1):362-367

van Gent, M.R.A., 1995. Wave Interaction with Permeable Coastal Structures, Delft University of Technology, Delft.

# Tutorial

Nog niet verdeeld. Later nog in te vullen.

## 1-D profile model

Delfland Deltagoot

## 2-D area model

Ocean bay park: getij+surge, baai, duin, nonerodible, overwash, collision,

## Langsgetij + riveroutflow

getijmodel + rivier + stationair.

# Appendices

## Reflective boundary

Ap

## Numerical implementation non-hydrostatic module

### Global continuity equation

As was outlined in the previous chapter the global continuity equation, which describes the relation between the free surface and the depth averaged discharge, is given by



A simple semi-discretisation of using central differences for the space derivative and using the Hansen scheme for the coupling between velocity and free surface results in



With , and the water depth is defined by a first order accurate upwind interpolation



The resulting scheme is only first order accurate by virtue of the upwind interpolations and mass conservative. When first order computations are considered accurate enough  is set to . For higher order accuracy the first order prediction is corrected using a limited version of the McCormack scheme. The corrector step reads



With  and  is given for positive flow as



Here  denotes the minmod limiter. Similar expression can be constructed for negative flow. The expression for  and  are obtained in a similar manner. Note that the total flux at the cell boundaries thus reads



The predictor-corrector set is second order accurate in regions where the solution is smooth, and reduces locally to first order accuracy near discontinuities. Furthermore, the method remains mass conservative. Note that other flux limiters can be used instead of the minmod limiter. However, as the minmod limiter performed adequately, this has not been investigated. ( For an overview of flux limiters see Hirsch, 2007)

### Local continuity equation

The depth averaged local continuity equation is given by



This equation is discretized using central differences



Missing grid variables are approximated with upwind interpolation. Because there is no separate time evolution equation for the pressure the local continuity equation will be used to setup a discrete set of poison type equations in which the pressures are the only unknown quantities.

### Horizontal Momentum

To obtain a conservative discretisation of the momentum equation the approach from Stelling and Duinmeijer (2003) is followed. However, to improve the accuracy of the method the combined space-time discretisation of the advection is done using a variant of the MacCormack (1969) is used. This scheme consists of a first order predictor step and a flux limited corrector step. The hydrostatic pressure is integrated using the midpoint rule and central differences, while the source terms and the turbulent stresses are integrated using an explicit Euler time integration. Formally the time integration is therefore first order accurate, but in regions where the turbulent stresses are negligible the scheme is of almost second order accuracy.

#### Predictor step

The depth averaged horizontal momentum equation for is given by



A first order accurate predictor step in time and space is then given as



Here Pr represents a discretisation of the dynamic pressure; T the effect of (turbulent) viscosity and S includes all other source terms. The discretisation of the (turbulent) viscous terms is given by central differences:



Here  and  are obtained from the surrounding points by simple linear interpolation.

Due to the incompressible flow assumption the dynamic pressure does not have a separate time evolution equation, but instead it satisfies an elliptical equation in space. As such its effect cannot be calculated explicitly using values at the previous time level. However to improve the accuracy of the predictor step the effect of the dynamic pressure is included explicitly. To do this first the unknown pressure is decomposed as:



where the difference in pressure is generally small. In the predictor step the effect of the pressure is included explicitly using. In the corrector step the full Poisson equation is then solved for . The pressure term in the predictor step is thus given as



Here represents the average pressure over the vertical which is approximated with, in which  is the pressure at the bottom. Furthermore  is given as.

Currently is formulated with the depth integrated momentum as the primitive variable, and not the depth averaged velocity. To reformulate in terms of we use the method by Stelling and Duinmeijer (2003). First note that  and  are approximated as  and . Now using  is equivalent to:





Substituting into the full expressions (including those for ) become:



Where we again use a first order upwind interpolation for and. This is exactly the approximation used by Stelling and Duinmeijer (2003) and is fully momentum conservative.

#### Corrector step

The predictor step is first order accurate in both space and time due to the use of upwind approximations for and Euler explicit time integration for the advective terms, and first order time integration for the source/viscous terms. This level of accuracy is acceptable near shore, where strong non-linearity (wave breaking, flooding and drying) will force the use of small steps in space and time anyway. However, in the region where waves only slowly change (e.g. shoaling/refraction on mild slopes), the first order approximations suffer from significant numerical damping. To improve the accuracy of the numerical model in these regions a corrector step is implemented after the predictor step.

The corrector step is given by:



Or, when formulated in terms of the depth averaged velocity



The values of  are obtained from slope limited expressions. For positive flow these read:



Where  again denotes the minmod limiter. Similar expressions can be constructed for, and.

The predictor-corrector set is second order accurate in regions where the solution is smooth, and reduces to first order accuracy near sharp gradients in the solutions to avoid unwanted oscillations. Furthermore, the method remains momentum conservative.

### Vertical momentum equations

The vertical momentum equation is discretized in a similar manner to the horizontal momentum equations using the McCormack scheme. In terms of the depth averaged vertical velocity the predictor step is:



The pressures are defined on the cell faces and therefore do not have to be interpolated. Furthermore, we can exactly set the dynamic pressure at the free surface  to zero. The vertical velocities are defined on the cell faces and therefore the depth averaged velocity  needs to be expressed in terms of the bottom and surface velocities. Using a simple central approximation gives



At the bottom the kinematic boundary condition is used for the vertical velocity:



Horizontal interpolation of  and  is done using first order upwind similar to . The turbulent stresses are again approximated using a central scheme as



Thus combining, and explicit expressions for  and  are obtained.

#### Corrector

The predicted values are again corrected using a variant of the McCormack scheme and including the pressure difference implicitly gives the corrector step:



Where  and  are obtained using relations similar to . Note that similar to and again the kinematic boundary conditions is substituted for .

The discrete vertical momentum balance of and looks very different from the relations found in Zijlema and Stelling (2005), Zijlema and Stelling (2008) and Smit (2008). This is mainly due to the application of the McCormack scheme for the advection. The discretisation of the pressure term is numerically fully equivalent to either the Keller box scheme as used in Zijlema and Stelling (2005), Zijlema and Stelling (2008) or the Hermetian relation used in Smit (2008).