Keywords

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Summary

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References

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| 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. |

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# 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



#### 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.

## Nonhydrostatic pressure correction

Robert

## Groundwater flow

Kees/Robert

### Introduction

The groundwater module in XBeach utilizes the principle of Darcy flow and is therefore limited to laminar flow conditions. In situations in which the groundwater flow may become turbulent, the full momentum equations (e.g. van Gent, 1995) should be applied. 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.

## 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 a velocity magnitude *vmg* and the orbital velocity urms. This section elaborates the method XBeach applies.

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

## Nonhydrostatic pressure correction

Robert

## Advection-diffusion equation for sediment

## Bottom updating schemes

## Avalanching

## Bed composition

Bas

# Boundary conditions

## Waves

### Time series

Kees, Ap review

### Spectra

Kees, Ap review

### Lateral boundary conditions

Dano

## Shallow water equations

### Absorbing-generating

Ap met appendix

### River and point discharge

Bas

### Ship motion

Dano

### Lateral boundaries

Kees

### Tide and surge

Kees

## 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.