Numerical modelling of erosion and accretion of plane sloping beaches at different scales

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A B S T R A C T

The paper focuses on the numerical simulation of erosion of plane sloping beaches by irregular wave attack in three wave flumes of different scales. One of the prime objectives of the tests was to provide a consistent data set for the improvement/validation of numerical beach profile models. A practical application of this research with wave attack on plane sloping beaches is the erosion of the plane beaches after nourishment. Three models (CROSMOR, UNIBEST-TC and DELFT3D) have been used to simulate the flume experimental results focusing on the wave height distribution and the morphological development (erosion and deposition) along the beach profiles. Overall, the model predictions for wave heights show consistent results. Generally, the computed wave heights \( H_{rms} \) and \( H_{1/3} \) are within 10% to 15% of the measured values for all tests (under-prediction of the largest wave heights close to the shore). The three models can simulate the beach erosion of the wave flume tests (erosive tests) reasonably well using default values of the sand transport parameters. The model performance for the accretive tests is less good than that for the erosive tests. A practical application of this research with wave attack on plane sloping beaches is the erosion of nourished beaches, as these beaches generally have rather plane beach slopes immediately after nourishment. Various graphs are given to estimate the beach erosion of nourished beaches.

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1. Introduction

The erosion of the plane sloping beaches by irregular wave attacks in three wave flumes of different scales was studied in the EU-project SANDS (Cáceres et al., 2008). One of the prime objectives was to provide a consistent data set for the improvement/validation of numerical beach profile models, which is the focus of this paper. Another objective was to study the erosional behaviour of the nourished beaches, as these beaches generally are characterized by plane beach slopes immediately after nourishment. At many places in the world beach erosion is counteracted by the regular nourishment of sand. Beach nourishment volumes generally are in the range of 100 to 200 m\(^3\)/m over alongshore length by the regular nourishment of sand. Beach nourishment volumes of different scales was studied in the EU-project SANDS (Cáceres et al., 2008). One of the prime objectives was to provide a consistent data set for the improvement/validation of numerical beach profile models, which is the focus of this paper. Another objective was to study the erosional behaviour of the nourished beaches, as these beaches generally are characterized by plane beach slopes immediately after nourishment. At many places in the world beach erosion is counteracted by the regular nourishment of sand. Beach nourishment volumes generally are in the range of 100 to 200 m\(^3\)/m over alongshore length scales of 1000 to 5000 m (total volume between 0.1 and 1 million m\(^3\)). Immediately after nourishment the beach profile most often consists of two approximately plane sloping sections: a mild sloping upper beach (between 1 to 50 and 1 to 150) and a relatively steep sloping lower beach (between 1 to 10 and 1 to 30), see also Fig. 17. From field experience we know that the lifetime of nourished beaches at exposed locations is relatively short (between 1 and 3 years; see also Yates et al., 2009; Van Rijn, 2010), but so far we have not been able to verify this by predictive numerical models. This latter topic will also be addressed in this paper.

In the past much research has been done on beach and dune erosion due to major storm waves (Birkemeier et al., 1998; Vellinga, 1986; Hallermeier, 1987; Dette and Ulicska, 1987; Dette et al., 2002; Van Gent et al., 2008). Most of the work was, however, focused on dune erosion rather than on beach erosion. Various researchers have performed small-scale laboratory experiments focusing on process measurements of waves, orbital velocities and sand concentrations (Homma and Horikawa, 1962; Bosman, 1982; Shibayama et al., 1986; Bosboom et al., 1999). Before 1980 most of the work was done with regular waves, while irregular waves were mostly used after that date. Dette et al. (2002) have given an overview of work on beach erosion in the large-scale laboratory facilities. A striking result of their work is the strong effect of the beach slope on the erosion volumes (see their Fig. 13). The major limitation of flume experiments is their two-dimensional nature, excluding the longshore dimension. This dimension can be included by numerical models. However, first it has to be shown that these models can really simulate the two-dimensional beach erosion of the flume experiments. More precisely: can advanced beach profile models simulate wave-induced erosion and accretion along a plane sloping beach at small and large scales, as observed in the laboratory experiments. After that, the models will be used to estimate beach erosion of nourished beaches in field conditions including the effect of longshore currents under oblique wave attack. Based on this, estimates can be given of the erosion rates and lifetimes of the nourished beaches.
2. Experimental results

Experiments with plane sloping beaches have been performed at three different scales using identical wave conditions (SANDS EU-Project 2005–2009; Cáceres et al., 2008; Grüné et al., 2008; Deltares/Delft Hydraulics, 2008). The main sediment and hydrodynamic parameters are presented in Table 1. The experiments have been done on three different scales: small-scale Delft wave flume experiments (beach slopes of 1 to 10, 15 and 20), medium-scale Barcelona wave flume experiment (beach slope of 1 to 15) and large-scale Hannover wave flume experiment (beach slope of 1 to 15) using an identical train of irregular waves (single peaked Jonsswap-type of spectrum with gamma = 3.3 and 500 waves). The initial experimental set-up consisted of a horizontal bed followed by a plane sloping beach. The dune geometry above SWL was slightly different in the three experiments. As only the beach erosion with runup below the dune toe was considered, this has no effect on the observed beach erosion profiles. In each flume the erosive test was performed first with a relatively large, constant wave height (see Table 1) during a period of about 24 to 48 h. After that, the accretive wave test was performed using the last profile of the erosive tests as the initial profile of the accretive test. During the tests, various hydrodynamic processes were measured (wave heights, instantaneous velocities, sand concentrations, etc.). The tests were regularly stopped to measure the beach profiles. Measured, only the beach profiles at the start and at the end of the tests and the wave height measurements at initial time (first half hour) will be used. At all three scales the erosive test data with relatively large waves and relatively small wave periods show the erosion of the upper beach by swash motions and the generation of a submerged breaker bar at the lower end of the sloping section. In one test with the mildest slope (1 to 20) of the Delft experiments a small swash bar was formed at the upper end of the beach. The accretive test data with relatively small wave heights and relatively large wave periods show onshore movement of the breaker bar.

Details of the wave spectra are given by the first paper of this special issue (Sánchez-Arcilla, 2011-this issue). A detailed analysis and comparison of the beach profiles from the different physical scale models applying available scaling laws is presented in another paper of this special issue (see Van Rijn et al., 2011-this issue). Herein, the attention is focused on the performance of various numerical models at different scales and, in particular, the modelling of the beach morphology.

3. Modelling results

3.1. Models

Three models have been used to compute the initial wave height distribution and the beach profile development over time: CROS MOR-model, UNIBEST-TC-model and DELFT3D-model. These models have been used because they differ in various ways, as explained below.

The CROS MOR2008-model is an updated version of the CROS MOR2004-model (Van Rijn, 1997, 2006 and Van Rijn et al., 2007). The model has been extensively validated by Van Rijn et al. (2003). The propagation and transformation of individual waves (wave by wave approach) along the cross-shore profile are described by a probabilistic model (Van Rijn and Wijnberg, 1994, 1996) solving the wave energy equation for each individual wave. The individual waves shoal until an empirical criterion for breaking is satisfied. The maximum wave height is given by $H_{max} = \gamma h$ with $\gamma h$ = breaking coefficient and $h$ = local water depth. The default wave breaking coefficient is represented as a function of local wave steepness and bottom slope. The default breaking coefficient varies between 0.4 for a horizontal bottom and 0.8 for a very steep sloping bottom. The model can also be run with a constant breaking coefficient (input value). Wave height decay after breaking is modelled by using an energy dissipation method. Wave-induced set-up and set-down and breaking-associated longshore currents are also modelled.

The application of a numerical cross-shore profile model to compute the erosion of the upper beach poses a fundamental problem which is related to the continuous decrease of the water depth to zero at the runup point on the beach face. The numerical modelling of the (highly non-linear) wave-related processes in the swash zone with decreasing water depths is extremely complicated and is in an early stage of development (Van Thiell de Vries et al., 2008). In the CROS MOR-model the numerical method is applied up to a point (last wet grid point) just seaward of the downrush point, where the mean water depth is of the order of 0.1 to 0.2 m. The complicated wave mechanics in the swash zone is not explicitly modelled, but taken into account in a schematized way. The limiting water depth of the last wet grid point is set by the user of the model (input parameter; typical values of 0.1 to 0.2 m). Based on the input value, the model determines the last wet grid point by interpolation after each time step (variable number of grid points).

The beach erosion zone is defined as the zone between the predicted run-up level and the last wet grid point. The total erosion area ($A_e$) over the length of the beach erosion zone is defined as: $A_e = q_i \Delta t / (1 - p)\rho_s$ with: $q_i = cross-shore transport computed at last wet grid point at the toe of the beach erosion zone, \Delta t = time step, p = porosity factor of bed material, $\rho_s = sediment density. The cross-shore transport generally is offshore directed during high energy (storm) conditions and onshore directed during low energy conditions. The profile of the beach erosion zone is assumed to have a triangular shape, yielding $A_e = 0.5L_e \rho_s$, with $e =$ maximum erosion depth and $L_e =$ length of beach erosion zone between last wet grid point and runup point. The maximum beach erosion depth is: $e = 2q_i \Delta t / (1 - p)\rho_s$.

In the case of the onshore-directed transport ($q_i$) at the last wet grid point, the same procedure is followed resulting in accretion with a triangular shape (swash bar generation). This may occur during low-energy events (post storm conditions). More details are given by Van Rijn (2009).

Two methods are available to compute the cross-shore wave velocity asymmetry under shoaling and breaking waves: 1) the semi-empirical method of Isole and Horikawa (1982; I-H method) with modified coefficients for skewed waves only (Grasmeijer and Van Rijn, 1998; Grasmeijer, 2002) and 2) the semi-analytical method of Ruessink and Van Rijn (R-R method) which can be used for skewed and asymmetric waves. This latter method is new and is briefly explained below.

The velocity and acceleration skewness of the near-bed wave motion in the inner surf and swash zone can to certain extent be represented by:

$$\tilde{U} = U_1 \cos(\alpha t) + \tilde{U}_2 \cos(2\omega t - \beta)$$

(1)

with: $U_1 =$ amplitude of the first harmonic, $\tilde{U}_2 =$ amplitude of the second harmonic, $\beta =$ phase difference.
The skewness of this wave signal (velocity as a function of time) represents the wave asymmetry with respect to the onshore and offshore velocities (high and narrow peaks; wide and shallow troughs) and is defined as $S_k = -U_{rms}/(\sigma_U)^3$ with $(\sigma_U)_{rms} = \langle U^2 \rangle^{1/2} = 0.5 \langle \tilde{U}_1^2 + \tilde{U}_2^2 \rangle$ and $\langle \rangle$ = time-averaging.

The asymmetry with respect to time within the wave cycle (forward leaning waves) is defined as $S_{0.5}$, but replacing $U$ by its Hilbert transform. Symmetric waves (in time) yield a value of $A_{0.5} = 0$.

Using these definitions, it follows that:

$$S_k = \frac{0.75}{\langle \sigma_U \rangle} \left( \tilde{U}_1 \right)^2 \left( \tilde{U}_2 \right) \cos(\beta)$$

and

$$A_{0.5} = \frac{0.75}{\langle \sigma_U \rangle^3} \left( \tilde{U}_1 \right)^2 \left( \tilde{U}_2 \right) \sin(\beta)$$

(2)

$$\tan(\beta) = A_{0.5} / S_k \quad \text{or} \quad \beta = \arctan(A_{0.5} / S_k)$$

(3)

$$\left( \tilde{U}_1 \right)^2 - 2(\sigma_U)^2 \tilde{U}_2^2 + (4 / 3) \sigma_U^3 S_k \cos(\beta) = 0$$

(4)

$$\left( \tilde{U}_1 \right)^2 + \left( \tilde{U}_2 \right)^2 = 2(\sigma_U)^2$$

(5)

Eq. (4) can be solved analytically using a complex function approach (Van Rijn, 1990, 2011), yielding:

$$\tilde{U}_1 = 2P^{0.5} \cos(\psi)$$

(9)

with $P = 2(\sigma_U)^2 / 3$, and $\psi = 1/3 \arcsin\left(-0.5Q^{P^{-1.5}} + n(2\pi), Q = (4/3) (\sigma_U)^3 S_k \cos(\beta); n = 0, 1, 2$ yields three roots; the smallest positive root is the solution $\tilde{U}_1$ follows from Eq. (5).

Using the linear wave theory, the variance of the velocity signal is defined as $\sigma_U = h \sigma_{U,rms} / (1.41 T \sin(\theta))$.

Based on the analysis of a large field data set of measured velocity time series, the parameters $S_k$, $A_{0.5}$ and $\beta$ are found to be:

$$S_k = B \cos(\beta)$$

and $A_{0.5} = B \sin(\beta)$,

$$\beta = -90 + 90 \arctan(0.637 / U_{rms})$$

$$B = 0.79391 + \exp(K)^{-1}$$

$$K = 2.8256 - 0.6065 - \log(\gamma)$$

$$U_{rms} = 0.75(0.5 \gamma h)$$

where: $U_{rms}$ = Ursell number, $k = (2\pi / L)$ = wave number, $h =$ water depth, $\gamma$ = $h_{rms} / 1.41 \gamma h_{rms}$ as input values.

The field data were collected during a series of subtidal and intertidal field campaigns at the barred beaches of Egmond aan Zee and Terschelling (both in the Netherlands) and of Truc Vert (France). The Dutch experiments comprised the Terschelling Nourtec campaign in 1994 (Ruessink et al., 1998) with relatively large onshore-directed velocities and relatively small offshore-directed velocities just outside the zone with breaking waves. Near-bed streaming effects are represented in the CROS-MOR model by semi-empirical expressions based on the work of Davies and Villaret (1997, 1998, 1999). The depth-averaged return current ($u_r$) under the trough of each individual wave (summation over wave classes) is derived from linear mass transport and the water depth ($h_r$) under the trough. The mass transport is given by $0.125 g H^2/c$ with $c = (g h)^{1/3}$ = phase velocity in shallow water. The contribution of the rollers of broken waves to the mass transport and to the generation of longshore currents (Svendsen, 1984; Daily and Oostereek, 1994) is taken into account. The vertical distribution of the undertow velocity is modelled by schematizing the water depth into three layers with a logarithmic distribution in the lower two layers and a third power distribution in the upper layer, yielding velocities which approach to zero at the water surface. The mass transport is gradually reduced to zero very close to the water line. Low-frequency waves are generated in the surf zone due to spatial and temporal variations of the wave breaking point resulting in spatial and temporal variations of the wave-induced set-up creating low-frequency waves (surf beat). This involves a transfer of energy in the frequency domain: from the high frequency to the low frequency waves. The total velocity variance (total wave energy) consists of high-frequency and low-frequency contributions ($U_{rms}^2 = U_{lf, rms}^2 + U_{hf, rms}^2$). Basically, accurate modelling of low-frequency waves requires the application of a long-wave model on the wave group time scale (Reniers et al., 2004a, b; Roelvink et al., 2009). Such an approach is beyond the present scope of work. Herein, a more pragmatic approach is introduced to approximately represent the low-frequency effects. The low-frequency significant wave height is related to the high-frequency significant wave height (Van Rijn, 2009). The long wave velocity is computed from long wave theory. Using this approach, long wave motion (surf beat) is generated under strongly breaking waves (plunging waves) in the surf zone. This engineering method shows rather good results compared with measured data of Delta flume experiments (Van Rijn, 2009).

The sand transport of the CROS-MOR2008-model is based on the TR2004 sand transport formulations (Van Rijn, 2006, 2007a, b, c and Van Rijn et al., 2007). It is noted that these formulations represent the sediment transport capacity (equilibrium transport). The temporal and spatial adjustments of the transport rates, which require an advection–diffusion scheme, are neglected. The effect of the local cross-shore bed slope on the transport rate is taken into account (see Van Rijn, 1993, 2006).

The sand transport rate is determined for each wave (or wave class), based on the computed wave height, depth-averaged cross-shore and longshore velocities, orbital velocities, friction factors and sediment parameters and then summed over all waves. The net (averaged over the wave period) total sediment transport is obtained as the sum of the net bed load ($q_u$) and net suspended load ($q_s$) transport rates. The net bed-load transport rate is obtained by time-averaging (over the wave period) of the instantaneous transport rate (intra-wave) using a formula-type of approach.

The net suspended load transport is obtained as the sum ($q_s = q_u + q_s$) of the current-related and the wave-related suspended transport components (Van Rijn, 1993, 2006, 2007a, b, c). The current-related suspended load transport ($q_{uc}$) is defined as the transport of sediment particles by the time-averaged (mean) current velocities (longshore currents, rip currents, and undertow currents). The wave-related
suspended sediment transport \( q_{w} \) is defined as the transport of suspended sediment particles by the oscillating fluid components (cross-shore orbital motion). The oscillatory or wave-related suspended load transport \( q_{w(o)} \) has been implemented in the model, using the approach given by Hounman and Ruessink (1996). The method is described by Van Rijn (2006, 2007a, b, c). The computation of the wave-related and current-related suspended load transport components requires information of the time-averaged current velocity profile and sediment concentration profile. The vertical advection-diffusion equation is applied to compute the time-averaged sediment concentration profile based on current-related and wave-related mixing. The bed-boundary condition is applied as a prescribed reference concentration based on the time-averaged bed-shear stress due to current and wave conditions.

An additional input factor (srf-factor = suspension enhancement factor) acting on the time-averaged bed-shear stress and hence on the reference concentration in the shallow swash zone at the upper end of the beach has to be selected by the model user; \( s_{rf} = 1 \) is the default reference concentration in the shallow swash zone at the upper end of the beach. The magnitude of the offshore-directed bound-infragravity transport can be scaled with the parameter \( c_{r} \), which changes the phase between the wave group and the infragravity motion from \(-180^\circ\) (maximum offshore transport, \( c_{r} = 1 \)) to \( 180^\circ \) (no transport, \( c_{r} = 0 \)). In the present application the long wave effect was ignored (\( c_{r} = 0 \)).

The near-bed velocity signal \( u_{b,t} \) is constructed to have the same characteristics of short-wave velocity skewness, amplitude modulation \( u_{b(t)} \), and mean flow \( u_{b(mean)} \) as a natural random wave field. To this end, the near-bed velocity \( u_{b(t)} \) is made up of the following three components: \( u_{b(t)} = u_{b(t)}^{\alpha} + u_{b(o)}^{\gamma} + u_{b(mean)} \).

Three options (Rienecker and Fenton, 1981; Isobe and Horikawa, 1982, or Ruessink and Van Rijn) are available to represent the time series of non-linear near-bed short-wave orbital motion \( u_{b(o)}^{\gamma} \) to take the non-linearity of the waves into account (i.e. non-zero skewness but zero acceleration skewness). Herein, the new method of Ruessink–Van Rijn (as described earlier) has been used. As input the local root-mean square wave height, peak period, and water depth are used. The computation of the bound-infragravity series \( u_{b(o)}^{\gamma} \) is based on the method of Sand (1982) (see Roelvink and Stive, 1989).

The vertical advection-diffusion equation is applied to compute the time-averaged sediment concentration profile based on current-related and wave-related mixing. The bed-boundary condition is applied as a prescribed reference concentration based on the time-averaged bed-shear stress due to current and wave conditions.

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The transport formulations are according to Van Rijn (2007a, b) and are identical to the formulations used in CROS MOR (see earlier). It is noted that these formulations represent the sediment transport capacity (equilibrium transport). In the bed-load transport formulation the Bagnold approach is used to account for bed slope-induced transport. The tangent of the angle of repose was set to 0.4 in all cases. The most landward wet computational grid point at each time step is taken as the grid point where the non-dimensional wave period \( T_{p} / (gH)^{1/2} \) exceeds 25 for the first time. For the range of wave periods considered in the present erosion cases, this implies that no hydrodynamic and transport computations are carried out in depths less than about 0.1 m (small-scale test) and 0.5 m (large-scale test) respectively. The sediment transport rate \( S_{w} \) at the last wet grid point is translated into an offshore (negative) or onshore (positive) sediment transport. The horizontal extent of the extrapolation is based on the local run-up: \( z_{ru} = 0.5 T_{p} (gH)^{1/2} \) tan \( \alpha \) in which \( z_{ru} \) is the local run-up height, \( H \) is the offshore wave height and \( \alpha \) is the slope of the dune face.

The sediment transport is extrapolated across the dry part of the beach profile to the run-up level based on the relative profile height: \( S_{w} = f(z_{ru} - z_{ru,le})/(z_{ru,le} - z_{ru,we})(w)S_{w,le} \) in which: \( z_{ru,le} \) is the profile height at the maximum run-up point, \( z_{ru,we} \) is the profile height at the last wet point, \( S_{w,le} \) is the transport at the last wet point and \( w \) is an additional weighting function related to the relative volume of water which passes a certain level (see Walstra and Steetzel, 2003). In the last wet point \( w = 1 \) and at the maximum run-up level it is \( w = 0 \), if \( w \) would be ignored the shape of the dry beach remains unaltered.

Finally the bed evolution is determined based on the divergence of the total transport using a 4-point Preismann implicit scheme. A detailed

![Fig. 1. Velocity time series for Hrms = 0.3 m, T = 7 s, h = 0.3 m.](image)
description of the Unibest-TC-model can be found in Ruessink et al. (2007).

The DELFT3D modelling system applied in this study is described in detail, by Lesser (2009), Lesser et al. (2004) and Van Rijn et al. (2007). Herein, the model has been used in the profile mode (two-dimensional vertical) referred to as DELFT3D and in full three-dimensional mode referred to as DELFT3D. The main components are DELFT3D-WAVE, which is based on the spectral wave model SWAN (Holthuijsen et al., 1993), and DELFT3D-FLOW, which in the form used is a non-stationary, 1DH, 2DV, 2DH or 3D, hydrostatic flow model with a built-in advection-diffusion solver that is used in the computation of the suspended sediment. A crucial extension to the FLOW model, for coastal applications, is the capability to allow for alongshore water level gradients to be applied at lateral model boundaries, which is known as the Neumann-boundary condition (Roelvink and Walstra, 2004). This method eliminates the need for nesting within a larger model to avoid boundary disturbances. In tidal conditions (or if a storm surge travels along a coast) these alongshore gradients vary in time, but for models with a limited cross-shore extent (e.g. approximately 10–15 m water depth), the alongshore gradient of the water levels can be assumed to be constant in the cross-shore direction. To make the solution well posed, a water level boundary is required at the seaward model boundary. Roelvink and Walstra (2004) have demonstrated that applying Neumann boundary conditions also yields reliable predictions when using only one grid cell in the alongshore direction (i.e. effectively reducing a 2DH/3D model to a 1DH/2DV model; referred to as a profile evolution model) under the combined forcing of (breaking) waves, wind and tide.

The numerical scheme applies a curvilinear, orthogonal, staggered grid, where the water-level points and depth points are co-located in the cell centers and the u- and v-velocity points are located in the middle of the cell walls. Wave effects are accounted for through additional driving terms near the surface and the bed, and through enhanced bed shear stress, mass flux and increased turbulence (Walstra et al., 2000). For nearshore applications, the wave driving terms are not directly derived from the wave properties as predicted by the wave model. To achieve a better cross-shore distribution of the wave forces, the wave energy balance equation is solved together with the roller energy balance (Nairn et al., 1990) within the FLOW module during each flow time step. These equations only require the predicted wave directions from the WAVE model (Reniers et al., 2004a). In the profile mode, Snell’s Law (implemented in Flow module) provides the wave direction across the profile.

In the model, bed load transport is computed separately and bottom changes due to the combined effect of the suspended sediment entrainment and deposition and bed load transport gradients are computed every flow time step (Lesser et al., 2004). The implementation of the TR2004 formulations (Van Rijn, 2007a, b, c) in DELFT3D including an extensive validation are described in Van Rijn et al. (2007d). Lesser et al. (2004) and Roelvink (2006) have introduced the concept of morphological acceleration (morfac) to coastal morphodynamic modelling. The so-called morfac concept potentially enables coastal engineers to simulate morphological evolution due to waves and currents in coastal areas at time scales of seasons (Ruggiero et al., 2009) and decades (Lesser et al., 2004; Tonnon et al., 2007). The morfac concept essentially multiplies the bed levels computed by a morphodynamic model after each hydrodynamic time step by a factor (morfac = input factor) to enable faster computation. To allow for the erosion of the dry beach and dune profile a scheme has been implemented that allows the (partial) redistribution of an erosion flux from a wet cell to the adjacent dry cells. The distribution is governed by a user-defined factor, which determines the fraction of the erosion to assign (equally) to the adjacent cells. If this factor equals zero the standard scheme is used, i.e. all erosions occur at the wet cell. If this factor equals 1 all erosions that would occur in the wet cell are assigned to the adjacent dry cells. In the present simulations this factor is set to 1 implying maximum erosion of the adjacent dry cells.

### 3.2. Modelling results of wave breaking

Measured wave heights at approximately the initial time along the plane sloping sand beaches of the laboratory experiments (Delft, Barcelona and Hannover) are shown in Tables 2 and 3 and Figs. 2–6. In all experiments the same wave train has been used based on Froude scaling. The data used herein refer to the initial situation (within 30 min after start of the test) before substantial beach erosion occurred. Computed wave heights ($H_{1/3}$) of the CROSMOR-model are shown in Figs. 2–6. All three models have been applied to the wave data, but only the results of the CROSOM-model are shown in detail, as the other models produce very similar results for $H_{rms}$ and $H_{1/3}$. The CROSMOR wave computations are based on 10 wave classes of equal probability. The wave height and period of each wave class were obtained from the measured offshore wave spectra of the various tests.

The plots show similar results with a pronounced wave shoaling zone with increasing wave heights followed by a wave breaking zone with reduced wave heights at the upper part of the beach slope. The maximum wave height ($H_{1/3}$) in the shoaling zone is about 10% larger than the incoming wave height in the small-scale Delft experiments and about 20% larger in the Barcelona and Hannover experiments. The relative wave heights ($H_{1/3}/h$) in the wave breaking zone vary in the range of 0.4 to 2. The maximum relative wave height is about 2 for a slope of 1 to 10 (Delft experiment).

The probabilistic wave model (CROSMOR wave by wave model) has been used with the default wave breaking parameter $\gamma$ which yields values in the range of 0.4 to 0.8, depending on the local wave steepness and bottom slope. A constant $\gamma$-parameter ($\gamma=0.8$) has also been used. The offshore wave input data are given in Table 1.

Figs. 2–4 show measured and computed wave heights for the three beach slopes of the Delft tests. The sand bed was covered with small-
scale ripples and the effective roughness of these ripples is estimated to be in the range of 0.01 to 0.03 m. Computed wave heights \( H_{1/3} \) are shown for \( k_s = 0.01 \) m and \( k_s = 0.03 \) m. The best agreement with the measured data is obtained for \( k_s = 0.01 \) m. The wave shoaling effect resulting in a pronounced increase of the wave height just before breaking is rather well represented by the wave model. The model also yields the fraction of breaking waves (in range of 0 to 1) showing minor wave breaking at the beginning of the horizontal section of the sand bed and major wave breaking at the sloping section. The computed wave heights \( H_{1/3} \) based on the default \( \gamma \)-parameter are somewhat smaller (10% to 15%) than the measured maximum wave heights just before wave breaking. The model results based on a constant \( \gamma \)-parameter \( (\gamma = 0.8) \) show somewhat better agreement with the measured data for all three slopes. Using \( \gamma = 1 \), the results are similar (very minor improvement; not shown).

Figs. 5 and 6 show measured and computed wave heights for the beach slope of 1 to 15 of the medium-scale Barcelona test and the large-scale Hannover test. The ripple heights in the surf zone of the Barcelona and Hannover tests were in the range of 0.01 to 0.1 m and the ripple lengths were in the range of 0.1 to 1 m. The bed was almost flat (sheet flow) in the swash zone close to the water line in the large-scale Hannover flume. Given the larger scale of the ripples in the Barcelona and Hannover tests, the roughness range is extended to \( k_s = 0.05 \) m over the full length of the flume. This value is somewhat too large for the horizontal floor of the wave flume. In all runs the input wave height at \( x = 0 \) of the erosive Hannover test is the same \( (H_{1/3} = 0.97 \) m). A bed roughness of 0.05 m yields a relatively large reduction of the computed wave heights for the Hannover test. The computed wave height reduces to about 0.93 m at the beginning of the sand bed using a bed roughness of \( k_s = 0.05 \) m (and to 0.95 m for \( k_s = 0.01 \) m). The measured wave height in the lower beach zone between \( x = 180 \) and 200 m where the ripple dimensions are relatively large, is best represented by the larger bed roughness values \( k_s = 0.05 \) m. The measured wave heights close to the water line (flat sheetflow type of bed) in the Hannover flume are slightly better represented by a smaller \( k_s \)-value of 0.01 m. The best agreement (within 10%) is obtained for the constant \( \gamma \)-parameter \( = 0.8 \) and \( k_s = 0.01 \) m. The data of Hannover show that both \( H_{1/3} \) and \( H_{1/10} \) can be very well simulated by the probabilistic wave model.

Overall, the probabilistic model shows very consistent results for all three scales. The breaking of irregular waves at relatively steep slopes (between 1 to 20 and 1 to 10) can be represented very well by a constant wave breaking parameter of \( \gamma = 0.8 \). The Hannover data show that \( H_{1/10} \) can also be represented with relatively high accuracy. The results of the parametric wave models (Battjes–Janssen approach, 1978) of UNIBEST-TC and DELFT2DV are very similar (not shown). Generally, the computed wave heights \( (h_{max} \) and \( H_{1/3} \)) are within 10% to 15% of the measured values for all tests (under-prediction of the largest wave heights close to the shore). The effect of these errors on the computed morphological results will be discussed later.
3.3. Modelling of beach erosion; erosive tests

The three cross-shore profile models have all been applied to 8 test results (5 erosive tests and 3 accretive tests).

The experiments are the three small-scale Delft tests (slopes 1 to 10, 1 to 15 and 1 to 20), one Barcelona test (slope 1 to 15) and one Hannover test (slope 1 to 15). The basic settings of the models are shown in Table 4.

Figs. 7–9 show measured and computed beach profiles for the small-scale Delft test results. The CROSMOR-model is used with default transport and wave breaking settings. The effective roughness of the sediment bed profile is predicted at each x-location. The shear stresses in the swash zone have been increased by 10% (\(sef = 1.1\)) to represent the relatively steep beach slope in the swash zone. The default setting of the sef-parameter varies in the range of 1 to 2 depending on the slope of the beach or dune front. The sef-parameter is 1 for a very flat beach of 1 to 100 and 2 for a very steep dune front of 1 to 3 (Van Rijn, 2009).

The sef-value is 1.1 for beaches with slopes of 1 to 10 to 1 to 20. The sediment transport formulations are applied in the default mode without any calibration. The predicted beach erosion is of the right order of magnitude. The suspended sediments eroded from the beach are deposited in the breaker bar at the toe of the beach slope. The predicted breaker bar is somewhat farther away from the beach than the measured breaker bar. Figs. 10 and 11 show measured and computed beach profiles for the medium-scale Barcelona test and the large-scale Hannover test using the same model settings. A similar pattern of beach erosion at the upper beach and deposition at the lower beach can be observed. The position of the breaker bar at the toe of the slope is rather well predicted for both experiments. Sensitivity runs with the CROSMOR-model using the I–H method and the R–R method to assess the effect of the acceleration skewness have been made for the Hannover test (Fig. 11). The erosion of the upper beach is significantly larger using the new R–R method including acceleration skewness, which is caused by the larger peak onshore and peak offshore velocities of the R–R method (see also Fig. 1). The I–H method leads to significant under-prediction of the beach erosion.

Both other models (UNIBEST-TC and DELFT2DV) are also used with default parameter settings of the sediment transport model. It is noted that the DELFT2DV-model is based on the time-dependent solution of the advection–diffusion sediment equation, while the other two models are based on the equilibrium transport values. The UNIBEST-TC predictions for the Delft tests with initial slopes of 1 to 15 and 1 to 20 show that the erosion at the lower beach is somewhat under-predicted. The prediction for the Delft test with the steepest slope of 1 to 10 is rather good, as are the predictions for the Barcelona test and the Hannover test. The results of the DELFT2DV-model show systematic over-prediction of the erosion at the upper beach, which is related to the relatively simple dry-bed procedure. The erosion at the lower beach is very similar to that of the other two models. The main differences between the UNIBEST-model and the DELFT2DV-model are the better mean flow description and the inclusion of the advection–diffusion model and hence the spatial lag effects of the suspended sediment concentrations by the DELFT2DV-model. The comparison of the results of both models shows that the computed morphology is almost the same in the middle section of the beach, but the upper beach erosion of the DELFT2DV-model is significantly...
larger. This latter effect is, however, fully caused by the relatively simple dry-bed procedure of the DELFT2DV-model. It seems that the inclusion of better physics of the DELFT2DV-model is not really necessary in the high-energy surf zone where the vertical mixing processes proceed relatively fast by the breaking waves. The use of models based on ‘equilibrium’ sediment concentrations seems to be sufficiently adequate in the surf zone. The CROSMOR-model was also run with a constant wave breaking coefficient of 0.8 yielding wave heights which are slightly different (about 10% to 15%). The computed erosion in the upper beach zone was about 10% to 15% larger (not shown). Similarly, the computed deposition at the lower beach zone was about 10% to 15% larger (not shown). Hence, small errors in the computed wave heights do not lead to the serious upscaling of errors due to nonlinear effects between wave heights and sediment transport rates.

The performance of the models relative to a baseline prediction can be judged by calculating the Brier Skill Score (BSS), as discussed by Van Rijn et al. (2003). The BSS compares the mean square difference between the prediction and observation with the mean square difference between the baseline prediction and observation. Perfect agreement gives a BSS of 1 whereas modelling the baseline condition gives a BSS of 0. If the model prediction is further away from the final measured condition than the baseline prediction, the BSS is negative. The BSS is very suitable for the prediction of bed evolution. The baseline prediction for morphodynamic modelling will usually be that the initial bed remains unaltered. In other words the initial bathymetry is used as the baseline prediction for the final bathymetry. A limitation of the BSS is that it cannot account for the migration direction of a bar; it just evaluates whether the computed bed level (at time $t$) is closer to the measured bed level (at time $t$) than the initial bed level. If the computed bar migration is in the wrong direction, but relatively small; this may result in a higher BSS compared to the situation with bar migration in the right direction, but much too large. The BSS will even be negative, if the bed profile in the latter situation is farther away from the measured profile than the initial profile. The BSS-values of the beach zone based on the present model results are...
given in Table 5. The qualification of the model performance is similar to that used by Van Rijn et al. (2003): excellent for BSS in the range of 0.8 to 1, good 0.6 to 0.8, fair 0.3 to 0.6, poor 0 to 0.3 and bad to 0.

The model performance for the erosive tests is rather good with most values in the range of good and some in the range of excellent. This is a very encouraging result as all models have been applied using default values for the sediment transport parameters.

Summarizing, it is noted that all models can simulate the beach erosion of the three wave flume tests at very different scales surprisingly well using default values. Surprisingly, the simulation results of the small-scale test with fine sediments (0.13 mm) are of the same quality as those of the medium-scale and large-scale test with considerably coarser sediments (0.25 to 0.3 mm). The simulation of the small-scale tests suffers from the problem that the waves and the velocity and the bed shear stresses are not much greater than the critical bed shear stresses. Furthermore, fine sediments have a larger critical mobility value than coarser sediments as imposed by the Shields’ curve (rising left limb of the curve). Small errors in the computed bed shear stresses may then easily lead to sediment transport errors and hence to bed evolution errors. This makes the modelling of small-scale tests more difficult.

The modelling of the spatial suspension lag effects (included in the DELFT-model) does not seem to be of major importance for the high-energy upper beach zone. The inclusion of the acceleration skewness (R–R method) produces larger erosion values at the upper beach in both profile models (CROSMOR and UNIBEST). The inclusion of this method (future research) in the DELFT-method will lead to an even larger over-prediction of the upper beach erosion, but the over-prediction is strongly related to the crude dry-bed procedure rather than to the hydrodynamic parameters (acceleration skewness). The dry-bed procedure of the DELFT-model is much too simple and has to be improved significantly (future research). The dry-bed procedure of the UNIBEST-model is fairly simple but seems to work fine for all tests.

Table 4
Parameters and settings of models (C = CROSMOR; U = UNIBEST-TC; D = DELFT2DV).

<table>
<thead>
<tr>
<th>Model parameters</th>
<th>Delft</th>
<th>Barcelona</th>
<th>Hannover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave data $H_{rms}$, $T_p$</td>
<td>0.122 m; 2.3 s</td>
<td>0.38 m; 3.9 s</td>
<td>0.688 m; 5.6 s</td>
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<td>Wave model</td>
<td>C: 10 classes (wave by wave)</td>
<td>C: 10 (wave by wave)</td>
<td>C: 10 (wave by wave)</td>
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<tr>
<td></td>
<td>U: 1 (parametric; Battjes-Janssen)</td>
<td>U: 1 (parametric)</td>
<td>U: 1 (parametric)</td>
</tr>
<tr>
<td></td>
<td>D: 1 (parametric; Battjes-Janssen)</td>
<td>D: 1 (parametric)</td>
<td>D: 1 (parametric)</td>
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<tr>
<td>Wave breaking</td>
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<td>Default</td>
<td>Default</td>
</tr>
<tr>
<td>Roller coefficients</td>
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<td>Default</td>
<td>Default</td>
</tr>
<tr>
<td>Wave skewness and asymmetry</td>
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<td>C: Ruessink–Van Rijn</td>
<td>C: Ruessink–Van Rijn</td>
</tr>
<tr>
<td></td>
<td>D: Isobe–Horikawa</td>
<td>D: Isobe–Horikawa</td>
<td>D: Isobe–Horikawa</td>
</tr>
<tr>
<td>Surf beat effect (low frequency waves)</td>
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<td>C: default</td>
<td>C: default</td>
</tr>
<tr>
<td></td>
<td>U: not used</td>
<td>U: not used</td>
<td>U: not used</td>
</tr>
<tr>
<td></td>
<td>D: not used</td>
<td>D: not used</td>
<td>D: not used</td>
</tr>
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<td>Minimum depth near shore</td>
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<td>C: 0.25 m</td>
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<td>U: default (0.1 to 0.5 m)</td>
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<tr>
<td></td>
<td>D: 0.01 m</td>
<td>D: 0.03 m</td>
<td>D: 0.05 m</td>
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<tr>
<td>Swash effect coefficient ($c_f$)</td>
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<td>C: $c_f = 1.1$</td>
<td>C: $c_f = 1.1$</td>
</tr>
<tr>
<td></td>
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</tr>
<tr>
<td></td>
<td>D: not included</td>
<td>D: not included</td>
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</tr>
<tr>
<td>Sediment $d_{50}$</td>
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<td>0.00025 m ($d_{90} = 2d_{50}$)</td>
<td>0.00027 m ($d_{90} = 2d_{50}$)</td>
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<td>Maximum beach slope</td>
<td>C: 50°</td>
<td>C: 50°</td>
<td>C: 50°</td>
</tr>
<tr>
<td></td>
<td>U: not included</td>
<td>U: not included</td>
<td>U: not included</td>
</tr>
<tr>
<td>Bed roughness</td>
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<td>C: variable (predicted)</td>
<td>C: variable (predicted)</td>
</tr>
<tr>
<td></td>
<td>U: variable (predicted)</td>
<td>U: variable (predicted)</td>
<td>U: variable (predicted)</td>
</tr>
<tr>
<td></td>
<td>D: variable (predicted)</td>
<td>D: variable (predicted)</td>
<td>D: variable (predicted)</td>
</tr>
<tr>
<td>Coefficients transport formulations</td>
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<td>C: default (= 1)</td>
<td>C: default (= 1)</td>
</tr>
<tr>
<td></td>
<td>U: default; $s_{us} = 0.3$</td>
<td>U: default; $s_{us} = 0.3$</td>
<td>U: default; $s_{us} = 0.3$</td>
</tr>
<tr>
<td></td>
<td>D: default; $sw_{us} = 0.2$</td>
<td>D: default; $sw_{us} = 0.2$</td>
<td>D: default; $sw_{us} = 0.2$</td>
</tr>
</tbody>
</table>

Fig. 7. Measured and computed beach profiles after 24 h; Delft (initial slope = 1 to 20).
involved. The dry-bed procedure of the CROSMOR-models is rather detailed but involves an input parameter (sef-value in the range of 1 to 2 depending on the beach and dune slope). Future research is required to relate the sef-parameter to the upper beach and dune slopes so that the model is fully predictive on this.

3.4. Modelling of beach erosion; accretive tests

The three cross-shore profile models have also been applied to the available accretive tests: the small-scale Delft test (slope 1 to 15), the medium-scale Barcelona test (slope 1 to 15) and the large-scale Hannover test (slope 1 to 15). The initial beach profile of each accretive test is the final profile of the previous erosive test. The offshore incoming wave train consisted of relatively low waves and relatively large wave periods. The basic settings of the models are shown in Table 4.

Fig. 12 shows the measured and computed results for the small-scale Delft test. The BSS-values are given in Table 5. Based on default parameter settings, the CROSMOR-model predicts slight onshore movement of the breaker bar in qualitative agreement with observed values, but it is much too small quantitatively. The BSS-value is only 0.22 (poor). Two methods have been used to compute the near-bed orbital velocities: Isobe–Horikawa which only yields the velocity skewness and the new Ruessink–Van Rijn which yields both the skewness and asymmetry (forward leaning waves). The method of Isobe–Horikawa (I–H) produces larger onshore movement of the breaker bar because the velocity skewness is larger than that produced by the method of Ruessink and Van Rijn (R–R) resulting in somewhat larger net onshore transport rates, see also Fig. 1. For example, the peak onshore and peak offshore near-bed velocities at $x = 37$ m of the Delft test are $U_{\text{max, on}} = 0.384$ m/s and $U_{\text{max, off}} = 0.20$ m/s for the I–H-method and $U_{\text{max, on}} = 0.383$ m/s and $U_{\text{max, off}} = 0.327$ m/s for the R–R-method. Due to the relatively large velocity skewness of I–H, the CROSMOR-model produces a relatively large swash bar formed at the upper beach (which is not observed). Various other model settings have been applied including the shear stress enhancement method of Nielsen (2002, 2006), but this did only marginally improve the results (not shown). Most likely scale errors in the sediment transport model in the shallow swash zone are the cause for the discrepancies of the model results. UNIBEST also produces a low BSS-value of 0.14 (poor), while DELFT2DV produces a negative BSS-value (bad) due to the erosion at the upper beach.

Figs. 13 and 14 show the measured and computed results for the medium scale Barcelona test and the large-scale Hannover test. Based on the default parameter settings, the CROSMOR-model predicts onshore movement of the breaker bar in qualitative agreement with observed values for the Hannover test (BSS = 0.44; fair). The erosion of the breaker bar is under-predicted for the Barcelona test, but is almost perfect for the Hannover test. The onshore movement of the bar is even slightly over-predicted in the latter case. The UNIBEST-TC-model produces BSS-values of 0.14 and 0.58 for the Delft test and the Hannover test. The latter result is quite good. The DELFT2DV-model produces negative BSS-values for all accretive tests due to the ongoing erosion at the upper beach. Sensitivity runs with the CROSMOR-model using the I–H
method and the R–R method to assess the effect of the acceleration skewness have also been made for the Barcelona and Hannover tests. The results of both methods are very similar (not shown). The onshore bar movement using the I–H method was slightly larger, as the net onshore transport was slightly larger (similar to that for the Delft case, see above). A swash bar was not generated.

The model performance for the accretive tests is less good than that for the erosive tests. CROSMOR and UNIBEST-TC produce two BSS-values in the range of poor to fair and one in the range of bad; DELFT2DV only produces values in the range of bad. The BSS-value of the latter model is heavily dominated by the strong over-prediction of the erosion at the upper beach. Further research is required to improve all three models with respect to accretive conditions.

4. Field application: erosion of plane sloping beaches after nourishment

4.1. Practical experiences

One of the objectives of the flume experiments was to study the erosional behaviour of the plane sloping beaches. These types of beaches are often present immediately after the execution of beach nourishment works. Beach nourishments are regularly used to mitigate the structural (long term) erosion problems along the Central Holland coast. A detailed analysis of the Dutch beach nourishment data (Van Rijn, 2010) shows that beach nourishments have extremely low life times of 1 to 2 years along the Holland coast. Similar values are reported by the Dutch coastal consultant Witteveen and Bos (2006).

The experimental results of the Delft flume can be used to estimate the beach erosion losses after nourishment and based on that to estimate the recurrence interval of beach nourishment. This can be done by the upscaling of the experimental results, but it can also be done by using the validated numerical models. Both approaches have been applied.

First, the temporal behaviour of the beach erosion volumes based on the available flume experiments are analyzed and discussed. As the numerical models show reasonably successful simulation results for the erosive tests at three different scales, these models can be applied with some confidence to the field scale. The CROSMOR-model using the same settings as those applied in Sections 3.3 and 3.4, has been used to estimate the seasonal storm wave effects on the erosion volumes for various types of sediment (0.2, 0.3 and 0.4 mm sand) for waves normal to the coast. The DELFT3D-model has been used to estimate the three-dimensional erosional behaviour of plane sloping beaches in field conditions.

4.2. Wave flume results

The erosional behaviour of the plane sloping beaches has been studied extensively by performing small-scale and large-scale tests in wave tanks/flumes. Fig. 15 shows beach profiles for an initial slope of 1 to 10, 1 to 20 of the Delft tests and an earlier Delft test with 1 to 40.

The results refer to small-scale tests with $H_{m0} = 0.18 \text{ m}$ and sand of 0.13 mm (Deltares, 2008 and Bosboom et al., 1999). The small-scale test results have been upscaled to prototype conditions with sediment of 0.27 mm and waves of about 1 m at the toe of the beach (length scale = depth scale = 5.5; sediment scale = morphological time scale = 0.5).

Fig. 10. Measured and computed beach profiles after 22.9 h; Barcelona (initial slope = 1 to 15).

Fig. 11. Measured and computed beach profiles after 32.8 h; Hannover (initial slope = 1 to 15).
scale = 2; see paper on scaling laws by Van Rijn et al., 2011-this issue). Based on this upscaling approach, Fig. 16 shows the beach erosion volumes as a function of time. The upscaled results represent prototype erosion by fair weather waves of about 1 m at the toe of the beach. The steepest initial slope of 1 to 10 yields an erosion volume after 1 day of about 12 m$^3$/m. The beach erosion is of the order of 6 to 9 m$^3$/m/day for milder slopes between 1 to 20 and 1 to 40. These results suggest that the erosion of beachfills can be substantially reduced by using relatively gentle slopes at the lower beach (not steeper than 1 to 20). Beach erosion of about 5 m$^3$/m per day by fair weather waves would mean that a beach nourishment of about 200 m$^3$/m can be removed in a few winter months.

### 4.3. Profile model results

To determine the overall efficiency of beach nourishments, the CROSMOR-profile model has been used to compute beach erosion volumes for a schematized case (see Fig. 17) along the Dutch coast (North Sea wave climate). The depth is given with respect to the mean sea level (MSL). The initial beach nourishment volume is about 230 m$^3$/m. The slope of the upper beach is set to 1 to 150; the initial slope of the lower beach is 1 to 20. The North Sea wave climate along the Dutch coast can be represented by six blocks of 2 h; each with a constant water level based on the tidal curve (input data). So, the tide is just causing a change of the sea level over time. An additional storm set-up of 0.5 m has been used during offshore waves of $H_o = 3$ m. Three types of beach material have been used: $d_{50} = 0.2$, 0.3 and 0.4 mm. The offshore boundary conditions are applied at a depth of 15 m to MSL.

Fig. 18 shows computed results for the wave height of $H_o = 3$ m over 10 days and three types of beach materials (0.2, 0.3 and 0.4 mm). Erosion mainly occurs in the beach nourishment section above the water line (0 to + 1.5 m). The eroded sand is deposited at the toe of the beach nourishment between the 0 and −1.5 m depth contours. The deposition layer in front of the beach nourishment slows down the erosion in time by reducing the wave height.

The cumulative erosion volumes (in m$^3$/m/day) for the three wave classes (0.6, 1.5 and 3 m) are shown in Figs. 19-21. Runs without tide show similar values (5% to 10% smaller). The tidal range of 1 m has almost no effect on the overall cumulative erosion volumes, but the cross-shore profile is slightly different in the upper beach zone. The tide merely causes a redistribution of the upper cross-shore profile. The initial erosion volumes in the upper beach zone (after 1 day) for waves <1.5 m are about 6 m$^3$/m for sand of 0.4 mm to 10 m$^3$/m for sand of 0.2 mm. These values are in line with the initial erosion volumes of the upscaled laboratory data (Fig. 16) yielding values of about 6 to 8 m$^3$/m after 1 day for sand of 0.27 mm (slopes between 1 to 15 and 1 to 20).

Over a month these values are significantly smaller. Monthly-averaged erosion volumes (cumulative erosion divided by total duration) are in the range of 1 m$^3$/m/day for sand of 0.4 mm to 2.5 m$^3$/m/day for sand of 0.2 mm and waves with $H_o < 1.5$ m (based on Fig. 20). For waves of $H_o = 3$ m these values increase to 4 m$^3$/m/day to 10 m$^3$/m/day (Fig. 21).

The computed bed profile of a run with $H_o = 3$ m and an offshore wave angle of 30° has also been plotted in Fig. 18, showing an increase of the total erosion volume by about 20%. The cumulative erosion is plotted in Fig. 21. The wave-induced longshore current at the initial time is also shown in Fig. 18. The maximum longshore current is of the order of 1 m/s just in front to the beach nourishment, which enhances the transport capacity and hence the erosion power of the system. The eroded sediments are deposited at the seaward edge of the inner breaker bar. This wave angle effect is a typical storm feature, as it is hardly noticeable for an offshore wave height of 1.5 m (see Fig. 20).

Fig. 22 shows the computed bed level after 1 winter period with a sequence of waves, as follows: 100 days with $H_o = 1$ m, 30 days with $H_o = 1.5$ m and 10 days with $H_o = 3$ m for three sediment diameters.
(d₅₀ = 0.2, 0.3 and 0.4 mm). As can be seen, the beach erosion after 140 days with waves is approximately 150 m³/m for d₅₀ = 0.2 mm; 100 m³/m for d₅₀ = 0.3 mm and 90 m³/m for d₅₀ = 0.4 mm. The beach nourishment volume of 0.2 mm sand is almost completely removed after 1 winter season. At the landward end of the beach a typical scarp-type erosion front is present, which is often observed in nature. The eroded beach sediment is deposited as a new breaker bar beyond the −4 m depth line.

Using the data of Figs. 19–21 and adding the results of each wave class (H = 0.6, 1.5 and 3 m) linearly, yields beach erosion volumes of the order of 100 to 200 m³/m may be easily eroded away in one to two winter seasons in line with observations at the Dutch coast where beach fills have, on average, to be repeated at two year intervals. The trough seaward of the inner breaker bar acts as a sink to the erosion of the beach sediments.

Relatively coarse beach material of 0.3 or 0.4 mm has a lifetime significantly larger (50%) than that of 0.2 mm. The initial beach slope at the toe of the beach has not much effect on the longer term. The beach nourishment is most effective when it is made landward of the inner bar crest, which can act as a terrace (or reef) reducing the wave height. The presence of a trough in front of the beach nourishment should be avoided (trough between inner bar and beach should always be filled with sand).

4.4. DELFT3D results

To estimate the 3D effects of the beachfill erosion, two runs using the DELFT3D model system (in full 3D-mode) have been made. The beach profile and the wave conditions are exactly the same as those used for the CROSMOR runs, but only one bed material diameter (0.2 mm) has been used. Two wave directions have been used: all waves perpendicular to the coast and all waves oblique to the coast (30° with coast normal at deep water). Fig. 23 left shows the sedimentation/erosion plots after 140 days for waves perpendicular to the coast; these plots are obtained by subtracting the initial bed levels from the computed bed levels after 140 days. The outer breaker bar shows a meandering pattern due to the generation of the second order circulating flows as a result of small variations in set-up values. Rip channel erosion spots can be observed in the nearshore zone (close to the mean water line; initial water line indicated by black line in Fig. 23). These oscillating patterns are absent for oblique waves, which generate relatively large longshore currents (Fig. 23 right). In both cases the total beachfill volume is completely eroded away after 140 days. Most of the erosion takes place during conditions with the higher waves of 1.5 and 3 m.

Fig. 24 shows the computed profiles in the central section of the beachfill for waves perpendicular to the coast. A new breaker bar is generated at a more offshore location. Fig. 25 shows similar results for the case with oblique waves, but now two new and more peaky

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**Fig. 13.** Measured and computed bed evolution for accretive test Barcelona.

**Fig. 14.** Measured and computed bed evolution for accretive test Hannover.
breaker bars are generated. The erosion of the beach zone is also much larger than that for the case with perpendicular waves. Fig. 26 left shows the time development of the total beach fill volume, while Fig. 26 right shows the beachfill volume development of the central (middle) section. The total initial volume is about 300,000 m$^3$ and the initial volume per m length of coast is about 230 m$^3$/m. Both plots clearly show that the first period of 100 days with low waves of 1 m does not yield any significant erosion. The erosion starts after 100 days when waves of 1.5 m are generated during a period of 30 days. Most of the beachfill has disappeared after this period. The erosion proceeds much faster when the waves are oblique to the coast due to the generation of strong longshore currents (of the order of 1 m/s close to the shore) which enhance the sediment transport capacity considerably. The erosion volume in the central section computed by DELFT3D for the case with waves perpendicular to the coast is slightly larger than that computed by the CROSMOR-model (compare Figs. 22 and 24 for $d_{50} = 0.2$ mm). Both models have the same sand transport formulations, but the DELFT3D-model includes an advection–diffusion scheme to represent the spatial lag effects. As the results of both models are very similar, it can be concluded that spatial lag effects of the suspended transport rates are not very important for the erosion in the high-energy upper beach zone. The models are very different in the wave modelling (parametric versus spectral). As both models produce similar erosion rates, the obvious
conclusion is that the spectral wave by wave model of CROSMOR does not yield much added value. It seems that beach fill erosion under storm conditions can very well be estimated using a deterministic approach based on the significant wave height. Based on the DELFT3D results, the beach fill erosion is found to be considerably larger (about 20% to 30%) for oblique wave conditions. This also is in line with the CROSMOR-results, see Figs. 18, 20 and 21. The data of Figs. 19–21 should roughly be increased by about 20% to 30% for conditions with oblique wave attack.

5. Conclusions

The erosion of the plane sloping beaches by irregular wave attacks in three wave flumes of different scales was studied in the EU-project SANDS.

The experiments have been done on three different scales: small-scale Delft wave flume experiments (beach slopes of 1 to 10, 15 and 20), medium scale Barcelona wave flume experiment (beach slope of 1 to 15) and large-scale Hannover wave flume experiment (beach
The slope of 1 to 15) using an identical wave train of irregular waves (single-peaked spectrum). The initial experimental set-up consisted of a horizontal bed followed by a plane sloping beach.

In this paper the attention is focused on the numerical modelling of a beach erosion. Three different numerical models (CROSMOR, UNIBEST-TC and DELFT3D) have been used to simulate the flume experimental results focusing on the wave height distribution and the morphological development along the beach profiles. The following conclusions with respect to the numerical modelling of the erosion experiments are given:

• The model predictions of wave heights show very consistent results for all three scales. The breaking of irregular waves at relatively steep slopes (between 1 to 20 and 1 to 10) can be represented very well by a constant wave breaking parameter of $\gamma = 0.8$ in the CROSMOR model which uses a wave-by-wave approach. For UNIBEST-TC and DELFT3D the cross-shore varying $\gamma$ of Ruessink et al. (2003) was used. Generally, the computed wave heights ($H_{rms}$ and $H_{1/3}$) are within 10% to 15% of the measured values for all tests (under-prediction of the largest wave heights close to the shore).

• The three models can simulate the beach erosion of the three erosive wave flume tests at very different scales reasonably well using default values for the sand transport parameters. Most Brier skill scores (BSS) are in the range of good to even excellent. The DELFT-model yields systematic over-prediction of the erosion of the upper beach, which is related to the dry-bed procedure applied, which is much too crude.

• The model performance for the accretive tests is less good than that for the erosive tests. CROSMOR and UNIBEST-TC produce two BSS-values in the range of poor to fair and one in the range of bad;
DELFT2DV only produces values in the range of bad, which is mainly caused by the relatively large over-prediction of the erosion at the upper beach.

- The modelling of the spatial suspension lag effects (included in the DELFT-model) does not seem to be of major importance for the high-energy upper beach zone. Both other models produce similar cross-shore profiles.
- The inclusion of the acceleration skewness (R-R method) leads to larger erosion values at the upper beach in both profile models (CROSMOR and UNIBEST). The inclusion of this method in the DELFT-method will lead to an even larger over-prediction of the upper beach erosion.
- The over-prediction of the erosion at the upper beach by the DELFT-model is strongly related to the crude dry-bed procedure rather than to the hydrodynamic parameters involved. The dry-bed procedure of the DELFT-model is much too simple and has to be improved significantly (future research). The dry-bed procedure of the UNIBEST-model is fairly simple but seems to work well for all tests involved. The dry-bed procedure of the CROSMOR-models is rather detailed but involves an input parameter (sel-value in the range of 1 to 2 depending on the beach and dune slopes). Future research is required to relate the sel-parameter to the upper beach and dune slopes so that the model is fully predictive on this.

A practical field application is the erosion of the plane sloping beaches after nourishment which is characterized by the plane sloping sand surfaces. Immediately after nourishment the beach profile most often consists of two approximately plane sloping sections: a mild sloping upper beach (between 1 to 50 and 1 to 150) and a relatively steep sloping lower beach (between 1 to 10 and 1 to 30). From field experience it is known that the lifetime of these nourished beaches at exposed locations is relatively short (between 1 and 3 years), but so far this has not been verified by the predictive models.

The present computational results simulating a North Sea wave climate (winter season) at the Holland coast show that a beach nourishment volume of the order of 250 m$^3$/m (0.2 mm sand) may be easily eroded away in one to two winter seasons in line with the observations at the Dutch coast where beach fills have, on average, to be repeated at two year intervals. Relatively coarse beach material of 0.3 or 0.4 mm has a lifetime which is significantly larger (50%) than that of 0.2 mm. The beach nourishment is most effective when it is made landward of the inner bar crest, which can act as a terrace (or reef) reducing the incoming wave height. The presence of a trough in front of the beach nourishment should be avoided (trough between inner bar and beach should always be filled with sand).

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Fig. 26. Time development of beach nourishment volume computed by DELT3D-model. Left: total volume (in m³). Right: volume in central section (in m³/m³).

References


